TRUNK

U.S.Department
of Transportation
Federal Highway
Administration

PAVEMENT DESIGN
Principles and Practices

A TRAINING COURSE

Participant Notebook

Prepared by
ERES Consultants, Inc.

National Highway
Institute

September, 1987

PREFACE AND ACKNOWLEDGMENTS

This course in "Pavement Design, Principles and Practices" was developed for the Federal Highway Administration, U. S. Department of Transportation by ERES Consultants, Inc. of Champaign, Illinois. The initial version of this course was prepared in September 1987.

Samuel H. Carpenter and Michael I. Darter served as Principal Investigators for the development of this course. The following individuals provided a tremendous amount of assistance in the development of the course which could not have been prepared without their effort, their contribution is appreciated: Amy L. Mueller, Mark B. Snyder, Kurt D. Smith, and Kathleen T. Hall. The editing done by Bertie Carpenter is appreciated, as is the effort of the Secretarial and drafting staff of ERES who provided excellent support to the engineering staff.

The Federal Highway Administration is gratefully acknowledged for their review of the technical material, assistance and guidance, and comments during the development of the course.

Extensive use was made of published literature in the compilation of the course. Gratitude is expressed to all who furnished information. The detailed integration of micro computer usage into the course required the use of many micro computer programs. Several of these were made available by ERES Consultants, Inc. Many others were made available through the McTrans micro computer center, and individuals are encouraged to contact the McTrans center to obtain documentation and copies of the software for these programs. The center can be contacted at:

McTrans
The Center for Microcomputers in Transportation
The University of Florida
512 Weil Hall
Gainesville, FL 32611

(904) 392 - 0378 Electronic Bulletin Board (305) 554-2145

The DNPS86 AASHTO design program was provided for demonstration in the course by the American Association of Transportation and Highway Officials. Interested users should consult their agency to determine the licensing policy in effect at that agency regarding usage of the DNPS86 program and availability of the documentation.

The assistance of all individuals contacted is deeply appreciated as they have made the development of this course possible.

Any questions regarding this course should be forwarded to:

ERES Consultants, Inc.
P.O. Box 1003
Champaign, IL 61820

(217) 356 - 4500

INSTRUCTORS:

PAVEMENT DESIGN
Principles and Practices

Prepared for: U.S. Department of Transportation
Federal Highway Administration/National Highway Institute
Presented by: **ERES Consultants, Inc.**

MONDAY

12:30 - 12:45		Introduction and Comments
12:45 - 1:00	**Module 1-1**	Course Description and Training Objectives
1:00 - 1:20	**Module 1-2**	Development of Pavement Design
1:20 - 1:50	**Module 1-3**	Design Considerations in Overall Pavement Management
1:50 - 2:50	**Module 2-1**	Subgrade Soils
2:50 - 3:00	**BREAK**	
3:00 - 4:00	**Module 2-2**	Resilient Modulus
4:00 - 4:30		Computer Introduction

TUESDAY

8:00 - 9:15	**Module 2-3**	Paving Materials
9:15 - 11:00	**Module 2-4**	Drainage Design with **BREAK**
11:00 - 12:00	**Module 2-5**	Design for Performance
12:00 - 1:00	**LUNCH**	
1:00 - 3:00	**Module 2-6**	Vehicle and Traffic Considerations
3:00 - 3:15	**BREAK**	
3:15 - 4:30	**Module 2-7**	Variability and Reliability

WEDNESDAY

8:00 - 10:00	**Module 3-1**	Basic Principles of Flexible Pavement Design
10:00 - 10:15	**BREAK**	
10:15 - 12:00	**Module 3-2**	AASHTO Method of Flexible Pavement Design
12:00 - 1:00	**LUNCH**	
1:00 - 2:30	**WORKSHOP**	
2:30 - 3:45	**Module 3-3**	Other Pavement Design Methods for Flexible Pavement Design with **BREAK**
3:45 - 4:30	**Module 3-4**	Flexible Pavement Shoulder Design

THURSDAY

8:00 - 11:30	**Module 4-1**	Basic Principles of Rigid Pavement Design with **BREAK**
11:30 - 12:30	**LUNCH**	
12:30 - 2:00	**Module 4-2**	AASHTO Method for Rigid Pavement Design
2:00 - 3:15	**WORKSHOP**	
3:15 - 4:15	**Module 4-3**	Other Design Methods for Rigid Pavement Design
4:15 - 4:45	**Module 4-4**	Rigid Pavement Shoulder Design

FRIDAY

8:00 - 10:00	**BLOCK 7**	OVERLAY DESIGNS
10:00 - 10:10	**BREAK**	
10:10 - 11:45	**Module 6-1**	Design Alternatives and Life-Cycle Costs
11:45 - 12:00		Course Evaluations

PAVEMENT DESIGN
PRINCIPLES AND PRACTICES

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BLOCK 1

Introduction

BLOCK 1

INTRODUCTION

Modules

1-1 COURSE DESCRIPTION AND TRAINING OBJECTIVES
1-2 DEVELOPMENT OF PAVEMENT DESIGN
1-3 DESIGN CONSIDERATIONS IN OVERALL PAVEMENT MANAGEMENT

This block conveys an appreciation for the scope and content of the course as a whole. It introduces the topics which the participant should have a working knowledge of when they complete the course. Upon completion of this block, the participant should be able to answer the objectives listed for each module, specifically:

1. Describe the historical development of pavement design procedures and how they influence the procedures used today.

2. Describe the content of the 4 day course and discuss what they as pavement design engineers should take away from the course when completed.

3. Discuss the general objectives of pavement management and discuss the detailed interaction pavement design plays in developing and implementing a pavement management program.

Upon completion of this block the participant will be able to complete the instructional objectives listed for the individual modules.

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MODULE 1-1

COURSE DESCRIPTION AND TRAINING OBJECTIVES

1.0 INSTRUCTIONAL OBJECTIVES

This module presents the contents of the course, its organization, and scope of material. It tells the participants what they will be expected to obtain from the course and how the information can be used in their work.

Upon completion of this module, the participants will be able to accomplish the following:

1. Obtain general knowledge about the course content and organization.

2. List the course objectives.

3. Be able to follow the flow chart of the pavement design process and relate it to the course content during the presentations.

2.0 COURSE DESCRIPTION

This pavement design course presents design concepts to the practicing engineer who is currently involved in designing pavements, or who desires the detailed background to be able to design pavements more economically. It is intended to familiarize engineers who are knowledgeable about one or more design methods (for example, AASHTO) with the philosophy of other methods. With improved testing and design methodology there has been increased emphasis on overlay design using deflection and mechanistic approaches which are described in this course. Other design considerations such as shoulders, drainage, pavement type selection, and pavement management are also discussed.

The course is organized into 7 major blocks of instruction. These blocks are contained in the schematic of the pavement design process outlined in Figure 1-1.1. While each block of instruction is fully developed to include the complete technical background of each topic in this notebook, the instructors will adjust their class presentations to fit the experience and desires of the class. Members of the class are encouraged to discuss design methods used by their agencies and to subjectively assess their experiences with these methods and to share their experiences with the class.

The course follows the process detailed in Figure 1-1.1. Each topic is a building block for the sections which follow. The major blocks of instruction are:

1. Introduction.
   - Introduction and Course Objectives
   - Development of Pavement Design
   - Pavement Design in Overall Pavement Management

2. Initial Considerations in the Pavement Design Process.
   - Roadbed Soils
   - Resilient Modulus

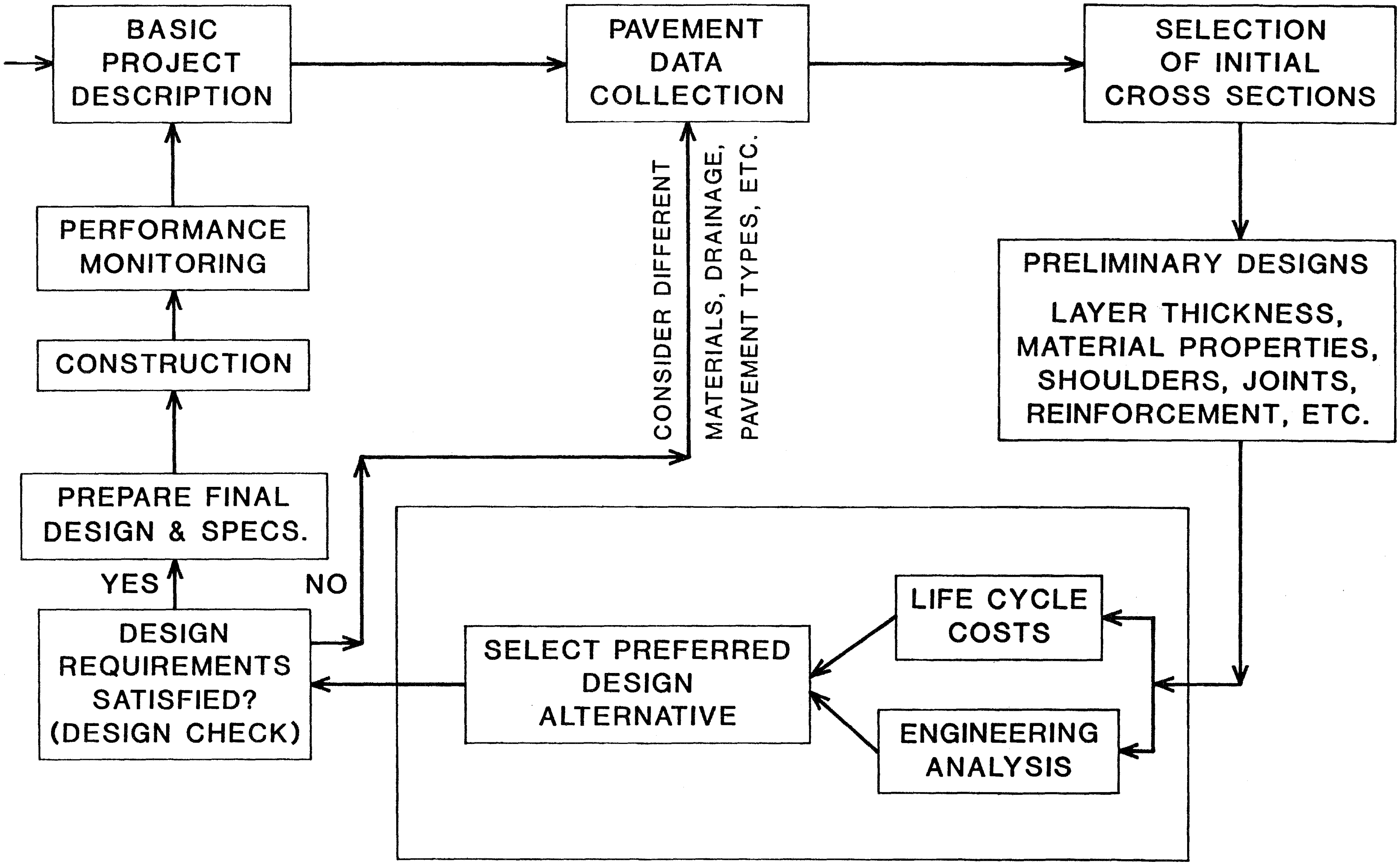

Figure 1-1.1. Flow-Diagram for Pavement Design.

Paving Materials
Drainage Design
Pavement Performance
Vehicle Characteristics and Traffic
Variability and Reliability

3. Flexible Pavement Design.
   Basic Principles of Flexible Pavement Design
   AASHTO Method of Flexible Pavement Design
   Other Methods for the Design of Flexible Pavements
   Design Considerations for Flexible Shoulders

4. Rigid Pavement Design.
   Basic Principles of Rigid Pavement Design
   AASHTO Method of Rigid Pavement Design
   Other Methods for the Design of Flexible Pavements
   Design Considerations for Rigid Shoulders

5. Overlay Design.
   Selection of Rehabilitation Alternatives
   Types and Functions of Overlays
   Overlay Design for Flexible Pavements
   Overlay Design for Rigid Pavements

6. Evaluation of Design Alternatives, Life-Cycle Costs.

7. Workshop.
   Flexible
   Rigid

The following paragraphs give a brief description of the content of each block of instruction.

Block 1. This block shows how pavement design has evolved over time, and how the design process interacts with the pavement management.

Block 2. This block introduces the engineer to the basic information that serves as background material to assist the engineer in understanding that the factors of Subgrades, Materials, Drainage, Pavement Performance, Traffic and Variability are more than just input values to the pavement design process. These quantities are more correctly considered as data elements that indicate potential structural sections that should be investigated as capable of performing satisfactorily in service.

Blocks 3 and 4. These blocks contain the structural evaluation and pavement design procedures for flexible and rigid pavements respectively. After the initial cross sections have been developed through consideration of input parameters, the determination of a final thickness for each layer can be accomplished using the procedures in these blocks for flexible and rigid pavements respectively. The application of existing design procedures to shoulder design are also presented. While emphasis is placed on the AASHTO Design Guide, suitable material is presented in order to develop an understanding of the basic assumptions of several design procedures. This understanding is critical to allow the engineer to compare different design procedures on a consistent basis.

Block 5. This block presents information about the increased emphasis on rehabilitation and the selection of overlay as the most appropriate rehabilitation scheme. The overlay design procedures use many of the same concepts as the pavement design procedures with suitable adjustments for application over an existing pavement.

Block 6. This block relates pavement design and the resulting pavement performance to pavement management. The design and resulting performance of both new pavements and overlays are crucial components of a pavement management system which blends original pavement design with planned rehabilitation at specified intervals. A thorough understanding of pavement design principles is needed to design overlay strategies and calculate life cycle costs for comparisons of the different pavement structural designs.

Block 7. Within each module of the course there are sample problems for the participants to work which reinforce the material presented in that module. Comprehensive pavement design problems for flexible and rigid pavements in Block 7 summarize the entire pavement design process developed during the course.

3.0 TRAINING OBJECTIVES

The course provides the participants with an understanding of the following items:

1. Engineering fundamentals of pavements and the theory of stress distribution in all types of pavements induced by both vehicular and environmental factors.

2. Objectives, theory, strengths, and limitations of empirical (AASHTO) and theoretical rigid design methods such as stress ratio and fatigue. This includes plain, reinforced, and continuously reinforced concrete pavements and their respective design details.

3. Objectives, theory, strengths, and limitations of empirical (AASHTO) and theoretical flexible design methods such as fatigue and rutting.

4. Effects of design input factors on pavement performance. These design input factors include loading, materials, construction practices, soils, environment, etc.

5. Objectives, theory, strengths, and limitations of testing procedures for soils and materials. This includes relating test values to design and performance.

6. Objectives, underlying principles, and methods of providing skid resistant surfaces.

7. Overlay design using the AASHTO and other methods.

8. The steps required in rehabilitation selection and design.

4.0 SUMMARY

This course is prepared for the engineer with experience in pavement design who is currently designing pavements. The material in this course will also provide background information for pavement design engineers who want to broaden their knowledge of available design procedures for both new pavements and overlays. Application of the new AASHTO Design Procedure (1986) is highlighted in the instruction.

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MODULE 1-2

DEVELOPMENT OF PAVEMENT DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module presents a brief historical review of pavement design procedures and concepts. Pavement design has evolved over the years into the design procedures which are in use today. The principles in these procedures lay the foundation for design procedures to be developed in the future. Pavement types are introduced and the differences discussed as a preview to how pavements perform, which is a major design consideration.

Upon completion of this module the participants will be able to accomplish the following:

1. List the advantages and disadvantages of conducting a road test.

2. Describe the AASHO Road Test and contrast the results obtained with this empirical approach with a general mechanistic design approach.

3. Describe the differences between a flexible and a rigid pavement and list the important pavement types.

4. List the material layers which may be found in various pavement types and describe their impact on pavement performance.

2.0 HISTORICAL DEVELOPMENT OF PAVEMENT DESIGN

2.1 Pre-Road Test Design Methods

The historical development of pavement design dates back to the 1920's when design was accomplished primarily by rule-of-thumb, or by "precedent" based on experience. Even though many satisfactory roads were designed using rule-of-thumb procedures, this approach has limitations. For example, the design factors appropriate for one set of soil, traffic, and environmental conditions are not necessarily applicable under different conditions. The rule-of-thumb design cannot adapt itself to these different conditions. This approach is quite often uneconomic because pavements are being designed for the worst conditions and are often overdesigned. These pre-road test design approaches were often developed based upon geotechnical and/or soil mechanics principles. For the most part, these procedures rely on protecting the subgrade by placing sufficient thickness of material above the roadbed.

2.1.1 Group Index Method

The group index method uses the AASHTO method of soil classification to calculate a numerical indicator for the quality of the roadbed soil based on traditional soil properties (1). The group index will be described in Module 2-1, but it is a soil property indicating quality of soil. The higher the group index, the lower the quality of the soil to carry traffic. The higher group index requires a thicker pavement to carry the design traffic. This design scheme does not allow much sophistication in material selection or thickness design.

2.1.2 Pedological Methods

The underlying principle in pedological or soil forming classification methods is that soils produced in a like manner, i.e., those from the same parent rock, similar climate, age, weathering conditions, and topography possess similar engineering properties (2). Various highway departments, including those in Michigan and Wisconsin, have utilized geographic, topographic, and agricultural soil maps to develop classification systems for predicting soil conditions encountered in their given areas. The design philosophy is similar to the group index method. Soils identified as having lower quality properties require thicker pavements.

2.1.3 Strength-Based Methods

Strength-based design methods use shear strength or load-deformation characteristics of the roadbed soil. The test procedures used in these design approaches will be explained in Module 2-1,2-2, and 2-3. The strength tests indicate the relative quality of the roadbed materials.

California Bearing Ratio (CBR)

The California Bearing Ratio method of pavement design (3) uses the load-deformation characteristics of the roadbed soils, aggregate subbase, and base materials, and an empirical design chart to determine the thickness of pavement, base, and other layers. The CBR value is an estimate of the quality of the material as compared to that of an excellent base material, for which the CBR is assumed to equal 100 percent.

Hveem

The Hveem stabilometer was developed in California and is predominately used by Western states as a replacement for the CBR method (4, 5). It measures the horizontal pressures developed as a result of vertically applied load. The greater the resistance to vertical pressure, the better the load carrying capability of the material. The thickness of pavement structure is related to the R (resistance) value of the roadbed material.

Load-Deformation

This design methodology recognizes that pavement performance is highly dependent on the load-deformation characteristics of roadbed soils, and not just the ultimate strength. The plate bearing test is one such procedure where the subgrade is loaded through a rigid plate 12 to 30 inches in diameter and the vertical deflection of the plate is recorded. The thickness design is based on the correlation of plate deflection with pavement performance related to allowable load repetitions for a measured load-deflection relationship.

Triaxial

The triaxial is a strength test that uses confining pressure in a cell to simulate confinement conditions which exist in a pavement structure. Triaxial tests have been used to provide strength comparisons of roadbed soils that might be encountered in the field (3). These strength relationships are correlated with a required thicknesses of base and pavement to protect the roadbed soil through empirical equations or charts. The Kansas (6) and Texas (7, 8) Highway Departments have used the open triaxial systems for thickness design.

2.2 Road Test Designs

The period from the mid 1940s to the 1960s can be described as the period of Road Test Design Methodology. During this time, highway engineers sought to better understand the significance of the vehicular effects on pavement performance with the goal of developing data showing how pavement condition changes over time for different situations. Confronted not only with the effect of changing traffic load regulations and with the need to establish policies on vehicle size and weight, the American Association of State Highway Officials (AASHO) conceived and conducted road tests in Maryland, Idaho, and Illinois during this time.

2.2.1 Maryland Road Test

The 1950 Maryland Road Test was conducted on a 1.1 mile section of existing U.S. 301 located approximately 9 miles south of Laplata, Maryland (9). The principal objective of this road test was to determine the relative effects of four different axle loadings using two vehicle types on a specific concrete pavement design. The loads employed were 18,000 pounds and 22,400 pounds on single axles, and 32,000 pounds and 44,000 pounds on tandem axles. These loadings were selected to represent conditions expected to be encountered in the foreseeable future on the existing roadway network.

2.2.2 WASHO Road Test

In 1953 to 1954 the Western Association of State Highway Officials (WASHO) sponsored a test road consisting of a mile of specially built flexible pavement in Malad, Idaho (10). The vehicular loadings were similar to those of the Maryland Road Test and the expected results from the test were similar.

2.2.3 AASHO Road Test

The AASHO Road Test was the last of the major road tests in the United States (13), conducted from 1958 to 1960 near Ottawa, Illinois about 80 miles southwest of Chicago. This site was chosen because the soil within the area is uniform and is representative of that found in large areas of the country. The climate is typical of that found in the northern United States and much of the earthwork and pavement construction would be used ultimately as part of Interstate 80.

The actual road test facility consisted of four large loops numbered 3 through 6 and two smaller loops numbered 1 and 2. Each loop was a segment of a four-lane divided highway with tangent parallel roadways which were connected by turnarounds. The tangents were 6,800 feet long in loops 3 through 6, 4,400 feet long in loop 2, and 2,000 feet long in loop 1. All vehicles assigned to any one traffic lane of loops 2 through 6 had the same axle loads and types. No traffic operated over loop 1. In all loops the north tangents were surfaced with asphalt concrete and south tangents with portland cement concrete. All the variables for pavement studies were concerned with pavement thickness design, load magnitude, and environmental effects. Pavement sections varied in length with a minimum of 100 feet and were separated by a short transition section in order to separate the design effects for statistical analysis. The actual designs used in Loop 4 are shown in Figure 1-2.1.

The AASHO road test introduced the concept of serviceability into the thickness design process. During the two years traffic was on each loop, a survey panel rode over each pavement section every other week to rate its ride comfort on a scale of zero for failed to five for perfectly smooth. A present serviceability rating (PSR) curve was developed for each pavement section as shown in Figure 1-2.2. With this curve the number of loadings to reduce the ride comfort (PSR) to a failure level (terminal serviceability level) could be determined for each pavement section. This empirical data is the basis for the structural design equations developed from the road test which will be presented in Block 3 and 4 for flexible and rigid pavements respectively.

2.2.4 Extensions

The AASHO road test was the most comprehensive of the road tests, yet it was still limited to the influence of only the environment of Central Illinois, the roadbed soil of Central Illinois, and the materials of Central Illinois which were used to construct the pavement sections. One immediate concern was to develop expanded criteria which would allow different conditions and materials to be considered in the design process. Components of the design procedure requiring local verification are:

1. Regional Factor (Climate).
2. Soil Support Value (Roadbed soils).
3. Structural Layer Coefficients (Material Properties).

A series of "satellite studies" were to be conducted throughout the United States to extend the road test findings to the individual states. Unfortunately, this comprehensive effort was never fully implemented which left the design guide with limitations in:

1. Verification.
2. Inadequate statistical data base.
3. No true definitions of failure.

The basic principles established and validated by the road test still serve as the basis for a large number of performance-based design procedures being used in the United State today. The AASHTO Interim Guide design for rigid and flexible pavement, Corps of Engineers, Louisiana, Utah and Kentucky designs are among a large family of pavement design techniques which were primarily developed on the basis of field performance taken from the Road Test. Their popularity indicates the usefulness of the data collected on the road test.

Loop 4					
Axle Load					
Lane 1			Lane 2		
18,000-S			32,000-T		
Main Factorial Design Design 1					
Surface Thickness	Base Thickness	Subbase Thickness	Factorial Block	Test Section No.	
				Lane 1	Lane 2
3	0	4	1	633	634
		8	2	607	608
		12	3	571	572
			3	569	570
	3	4	2	599	600
		8	3	573	574
		12	1	617	618
	6	4	3	585	586
		8	1	623	624
		12	2	601	602
4	0	4	3	583	584
		8	1	619	620
		12	2	603	604
	3	4	1	627	628
		8	2	589	590
			2	597	598
		12	3	575	576
	6	4	2	595	596
		8	3	577	578
		12	1	625	626
5	0	4	2	605	606
		8	3	587	588
		12	1	621	622
	3	4	3	579	580
		8	1	631	632
		12	2	593	594
	6	4	1	629	630
			1	615	616
		8	2	591	592
		12	3	581	582

Figure 1-2.1. Pavement Sections Constructed on Loop 4 of the AASHO Road Test.

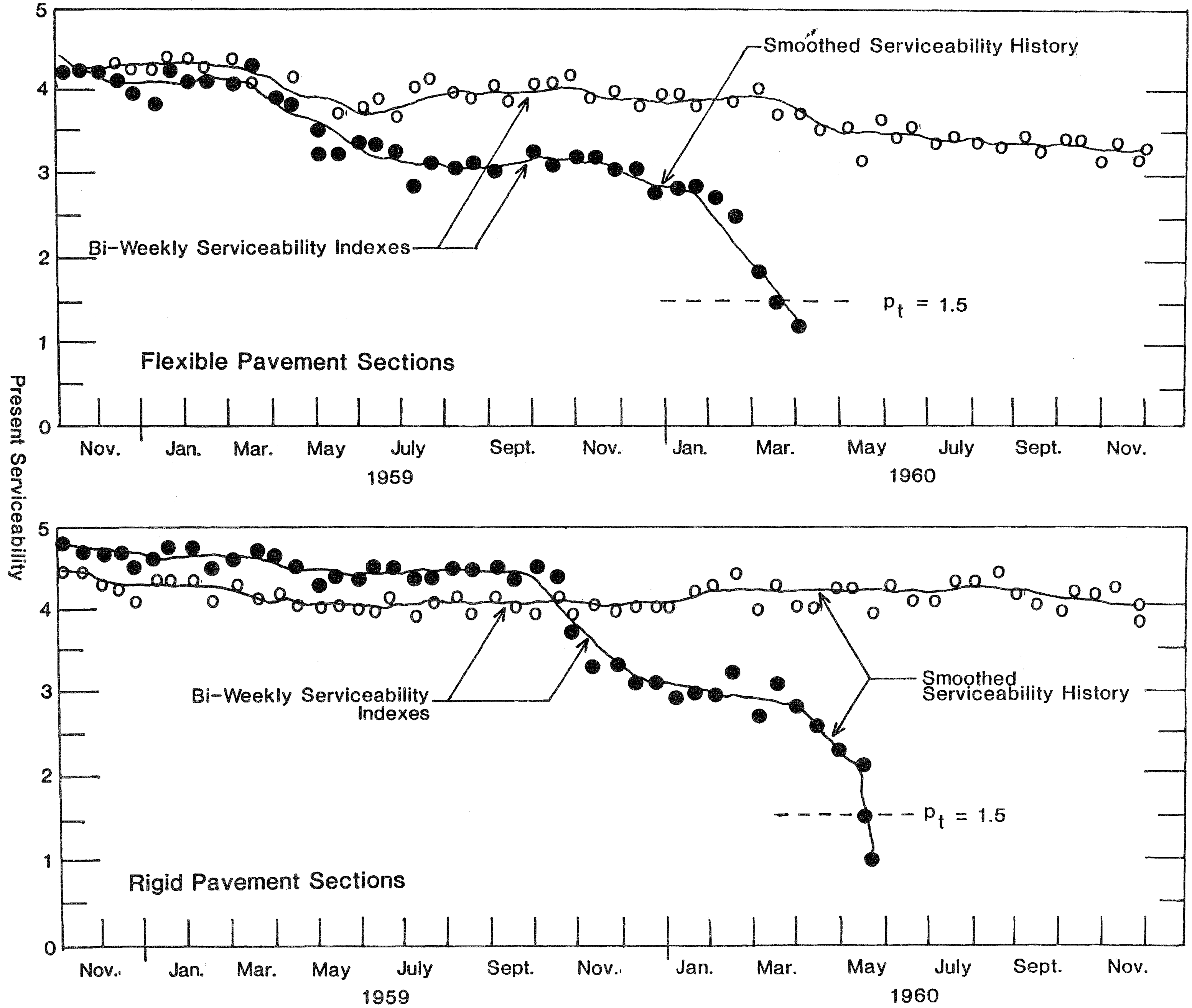

Figure 1-2.2. Typical Serviceability Histories from the AASHO Road Test.

2.3 Mechanistic-Empirical Methods

Mechanistic pavement design procedures are based on mechanics of materials equations that relate an input such as a wheel load to an output or pavement response such as stress, strain or deformation. Laboratory testing is often included to provide a relationship between loadings and failure. Empirical design methods typically relate observed field performance to design variables, such as a road test. Mechanistic-empirical design approaches combine the theory and physical testing with the observed performance to design the pavement structure.

The basis of a mechanistic design procedure is an analytical program to calculate the stress or strain. The pavement response values calculated by these programs are input for a second program using transfer functions to predict distress resulting from the response. Transfer functions can be developed from laboratory test data or they can be based on observed performance data collected in the field. Dependence on observed performance is the empirical nature of this design approach. VESYS is an example of a program that uses empirical transfer functions which are based on the AASHO road test performance data.

As more distress survey data becomes available theoretical models may be more accurately calibrated to represent observed performance models. Calibration with field performance is a necessity for accurate designs as theory alone has not proven sufficient to design pavements realistically.

2.4 AASHTO Design Guide - 1986

Given the limitations of the 1972 Interim Design Guide, extensive revisions have been made to include more fundamental concepts (some recommended in mechanistic approaches) and extend the applicability of the design procedure. These revisions include:

1. Replacement of Soil Support Value and the modulus of subgrade reaction with the Modulus of Resilience for both flexible and rigid pavements.

2. The inclusion of design reliability.

3. The use of resilient Modulus testing to select layer coefficients for flexible pavements.

4. Drainage has been included through recognition of the impact of drainage on performance and suitable adjustments to material properties.

5. Improved environmental design has been included for frost heave, swelling soils, and thaw weakening.

6. Subbase erosion can be accounted for in rigid pavement designs.

7. Load Transfer can be designed for in rigid pavements.

8. Life-cycle cost information has been included for use in evaluating alternate designs.

Other items in the design guide which have been included or expanded include rehabilitation, pavement management, load equivalency factors, traffic considerations, and low volume road design. Guidance is even provided on mechanistic-empirical design. These items will be expanded upon in the appropriate section of the course.

3.0 GENERAL PAVEMENT TYPES

Pavement design is generally concerned with the design of flexible or rigid pavements. These two pavement types are characterized by the different mechanics of distributing loads to the subgrade and resulting construction details required for acceptable performance. The typical structural layer arrangement for each pavement type is shown in Figure 1-2.3. The rigid pavement, by virtue of the stiff slab action of the portland cement concrete, spreads the wheel load over a large area producing a low stress on the subgrade. The flexible pavement, by virtue of the flexibility of the asphalt concrete, deforms more than the portland cement concrete, and produces a higher stress distribution on the subgrade. These different stresses and how they are produced are the reason pavements perform differently and require different design procedures. These differences will be detailed in later sections of the course.

Rigid pavements can be subdivided into three separate designs:

1. Jointed Plain (JPCP).

2. Jointed Reinforced (JRCP).

3. Continuously Reinforced (CRCP).

Jointed plain pavements are characterized by short joint spacings of 13 to 30 feet with no reinforcing steel in the slab. The joints are typically dowelled for load transfer. Jointed reinforced pavements are characterized by long joint spacings of 27 to 120 feet with reinforcing steel in the slab to hold the shrinkage cracks tight. Continuously reinforced pavements have no joints and contain a greater percentage steel to control cracking as the concrete shrinks during curing.

Flexible pavement types are differentiated by the material used in the individual layers. The most significant difference is when the granular base is replaced with asphalt stabilized material to make a full-depth asphalt pavement. This is still a flexible pavement because the stress distribution resulting in the pavement is not at all similar to that in a rigid pavement.

The differences between these two pavement types necessitates very different design considerations which will be developed in the remainder of this course.

4.0 SUMMARY

The historical development of modern day pavement design methodologies can be divided roughly into four periods representing awareness or application of different design methodologies. These periods are:

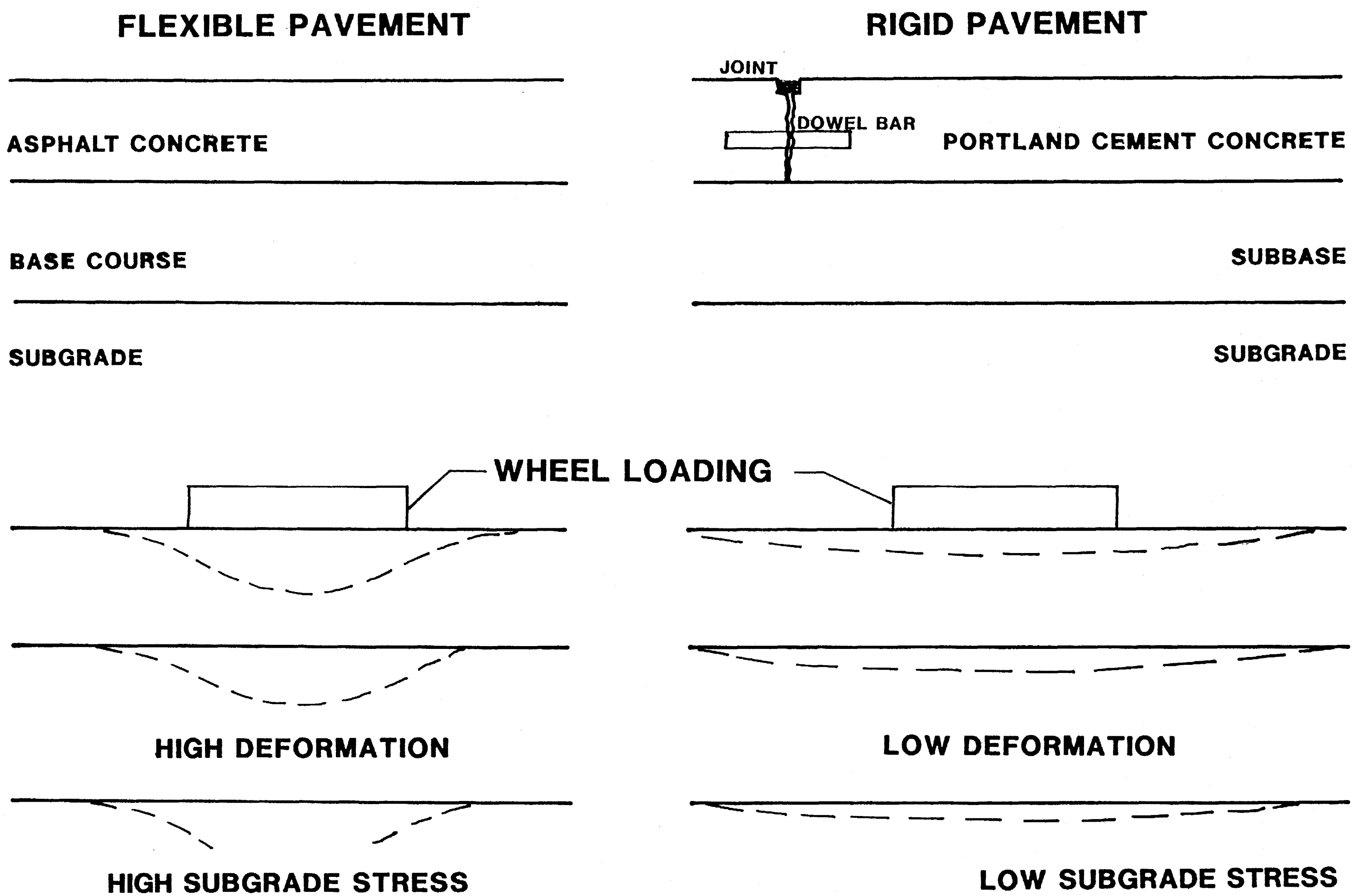

Figure 1-2.3. Differences Between Flexible and Rigid Pavement Under Load.

1. Pre-Road Test Design Procedures.

2. Interim Road Test Design Procedures.

3. Mechanistic-Empirical Procedures.

4. AASHTO Design Guide 1986.

This course is designed to develop an awareness of the different design methodologies, their advantages, or drawbacks, and the critical items which should be present in any design procedure.

5.0 REFERENCES

1. Yoder, E. J., and M. W. Witczak, Principles of Pavement Design, John Wiley & Sons, Inc., New York, 1975

2. Michigan State Highway Department, "Field Manual of Soils Engineering,"

3. U.S. Waterways Experiment Station, Technical Report, No. 3-582, 1961.

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5. Hveem, F. N., and G. B. Sherman, Highway Research Board, Bulletin No. 8, 1963.

6. Lacey, D. L., Highway Research Board, Research Report 16-B.

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8. McDowell, C., Highway Research Board, Bulletin 114.

9. Highway Research Board, "Final Report on Road Test one MD," HRB Special Report 4, 1952.

10 Highway Research Board, "The WASHO Road Test," HRB Special Reports 18 and 22, 1955.

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MODULE 1-3

DESIGN CONSIDERATIONS IN OVERALL PAVEMENT MANAGEMENT

1.0 INSTRUCTIONAL OBJECTIVES

This module provides an introduction to the role pavement design should play in overall pavement management activities. Pavement design is the most critical activity in pavement management in terms of affecting both initial and life cycle costs.

Upon completion of this module the participants will be able to accomplish the following:

1. Briefly describe overall pavement management activities and goals.

2. List the major changes that have taken place in the national highway system in the past decade that have caused an increased concern for improved pavement management.

3. List the reasons why pavement design is a critical aspect of pavement management in terms of life-cycle costs.

4. Describe how the information gained from monitoring pavement performance can be used to improve pavement design.

2.0 OVERALL PAVEMENT MANAGEMENT PROCESS

Pavement management has existed in some form ever since the first highway departments were organized in the early 1900's. This section presents an overall view of pavement management as it has evolved today.

2.1 Large Pavement Investment

Massive highway construction over the past 30 years means that pavements now represent one of the most costly of all public investments. There are 3.9 million centerline miles of roadway in the U.S. Approximately two million of these miles are paved and over one million miles are either a high type flexible or rigid pavement (5).

This large paved mileage can be studied by breaking it down into individual projects. If a typical length of project is three miles there are nearly 700,000 individual "design projects" that require continual planning, design, construction, maintenance and rehabilitation. The adequacy of each of these pavement management activities will result in a stream of costs that will go on for as long as the roadways exist. This stream of costs is greatly affected by the adequacy of the planning, design, construction, evaluation, maintenance and rehabilitation for each project. These are the major activities which should be improved by overall pavement management. Cost-effective management of such a vast pavement network can only be accomplished through an integrated and comprehensive management approach.

2.2 Major Changes In Highways

The following major changes have taken place in the national highway system during the past few years:

1. Rising costs.
2. Reduced revenues.
3. Increased utilization (particularly trucks).
4. A changing emphasis from highway expansion to modification and preservation (2).

Such changes have resulted in highway agencies reassessing their management practices, particularly with regard to pavements.

It must be recognized that highways are the overall product of highway agencies, and pavements represent the end product of the highway system that daily affects the lives of millions of users (2). Therefore, the goal of pavement management must be to improve the product in a cost-effective manner.

2.3 Pavement Needs

Pavements represent the major cost activity in providing highways. The following funding levels have been estimated as necessary to maintain the existing conditions on the United States highway system through the year 2000:

1.	Interstate 4R	-	\$64.3 billion.
2.	Primary	-	\$93.3 billion.
3.	Secondary	-	\$46.6 billion.
4.	Urban	-	\$72.7 billion.

Because it is unlikely that all of the needed funding will be available the importance of efficient pavement design becomes even more critical in controlling the flow of costs. To provide for the efficient use of such large amounts of funds, improved management and engineering practices and training of personnel are required in every aspect of the pavement management process.

2.4 Pavement Management Definition

The AASHTO Guidelines on Pavement Management provides the following definition:

> Pavement Management (PM) is the effective and efficient directing of the various activities involved in providing and sustaining pavements in a condition acceptable to the traveling public at the least life-cycle cost (6).

The AASHTO Guide for Design of Pavement Structures 1986 adds that:

> Pavement Management in its broadest sense encompasses all of activities involved in the planning, design, construction, maintenance, evaluation, and rehabilitation of the pavement portion of a public works program (1).

Pavement management activities involve many different interrelated aspects as illustrated in Figure 1-3.1. Recognition of the fact that these activities are interrelated is critical. For example, poor design results in large maintenance costs and rehabilitation costs. Good design with poor maintenance results in shortened pavement life and higher costs. Poor construction causes even an excellent design to fail prematurely resulting in increased maintenance, rehabilitation, and user costs. Thus, all pavement management activities must be considered as a system in order to improve the overall product, i.e. pavements, in a cost-effective manner.

3.0 DESIGN AND REHABILITATION CONSIDERATIONS OVER THE LIFE CYCLE

Pavement design is a critical aspect of pavement management. Poor design practice will reflect itself throughout many years in higher pavement maintenance and rehabilitation requirements. Figure 1-3.2 shows which decisions have the most influence over the expenditure of funds during the life-cycle of a project (7). The preliminary and final design phase have by far the greatest effect on life-cycle costs as they are decisions which must be made early in the life of a pavement project.

Figure 1-3.3 shows an example of the relative costs for various design alternatives for a given pavement. The initial design affects not only the initial construction cost but also has an impact on all other costs such as maintenance, rehabilitation, salvage, and user. This figure illustrates that pavement design can improve the pavement management process through the consideration of different design strategies and all associated costs over the design analysis period. This is commonly referred to as life-cycle costing of different design alternatives.

Recent developments in pavement technology such as improved testing equipment and increased microcomputer availability provide the pavement designer with more "tools" to use in the evaluation of the consequences of pavement design alternatives to estimate their life-cycle costs.

4.0 FEEDBACK OF PERFORMANCE MONITORING DATA TO IMPROVE DESIGN

Improvement of any product comes about essentially through the feedback performance information that relates to design and construction. This feedback concept for pavements is illustrated in Figure 1-3.4. Perhaps the weakest link in the pavement management process has been the lack of feedback of critical performance monitoring information, and its use to improve planning, design, construction, maintenance and rehabilitation procedures.

Until recently, there have been limited pavement monitoring and evaluation activities in highway agencies. A number of agencies are beginning to monitor their pavements for planning and programming purposes, but there is still a general lack of performance monitoring and use of the feedback data to improve other pavement management activities, particularly

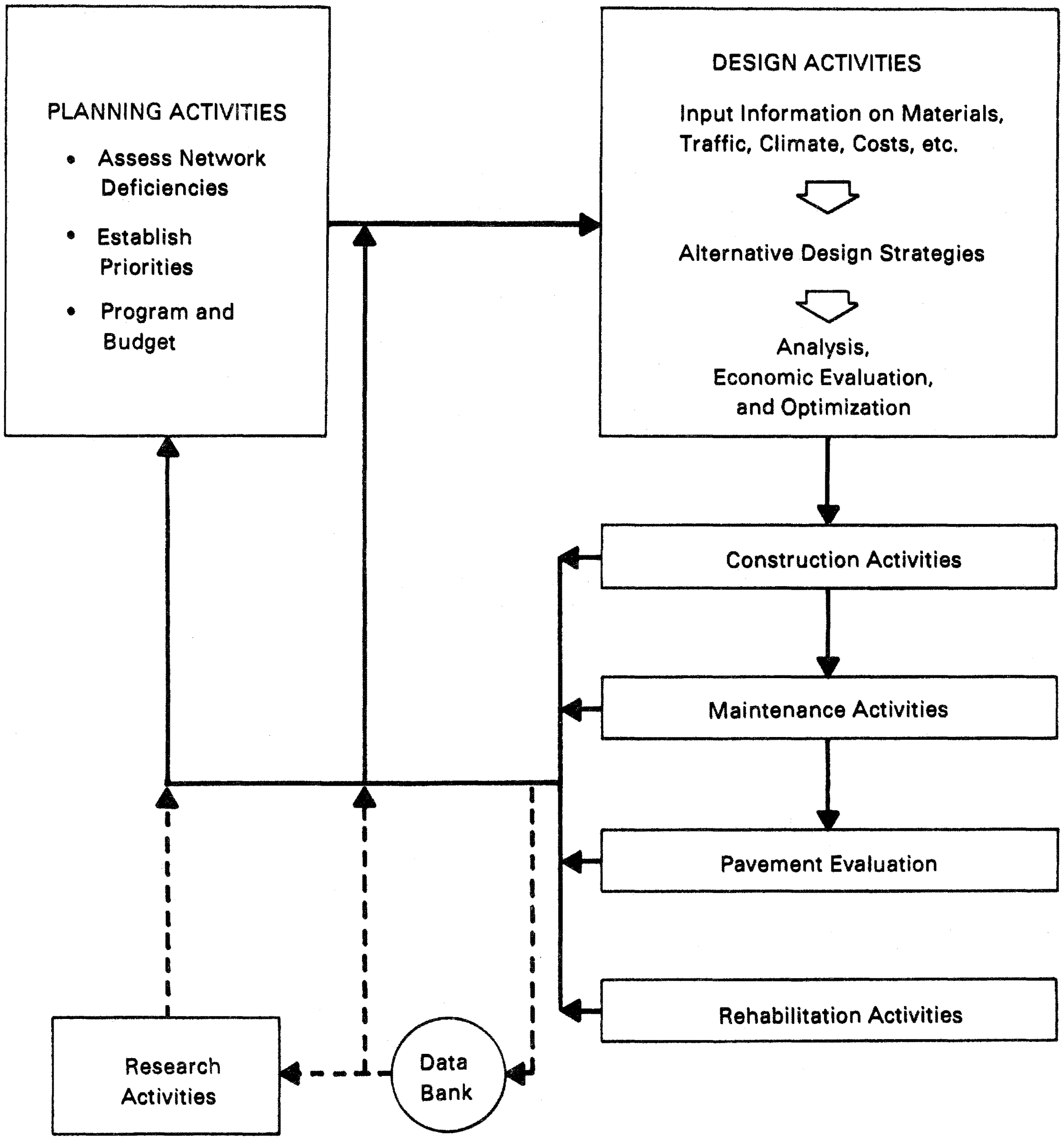


Figure 1-3.1. Major Classes of Activities in a Pavement Management System (1).

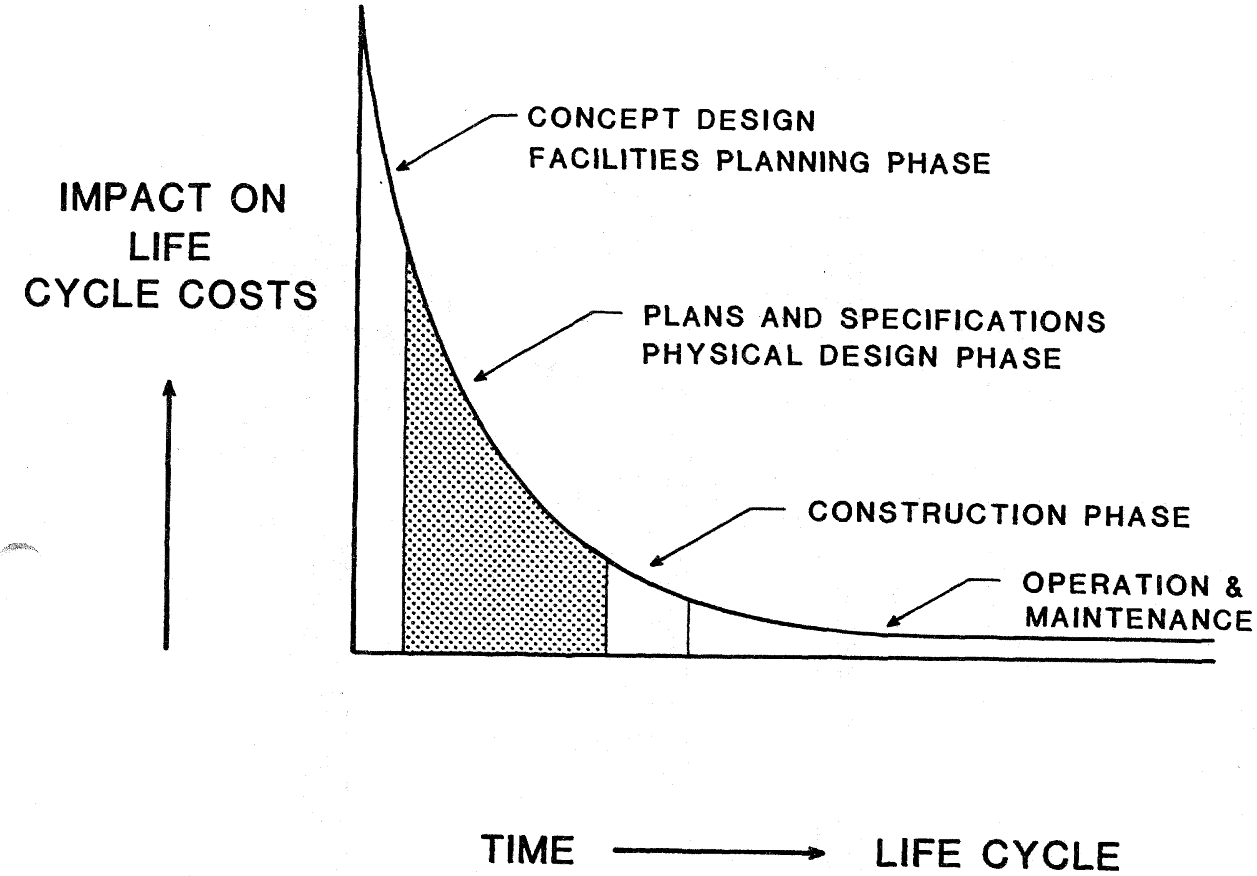


Figure 1-3.2. Decision Makers Influence Costs (7).

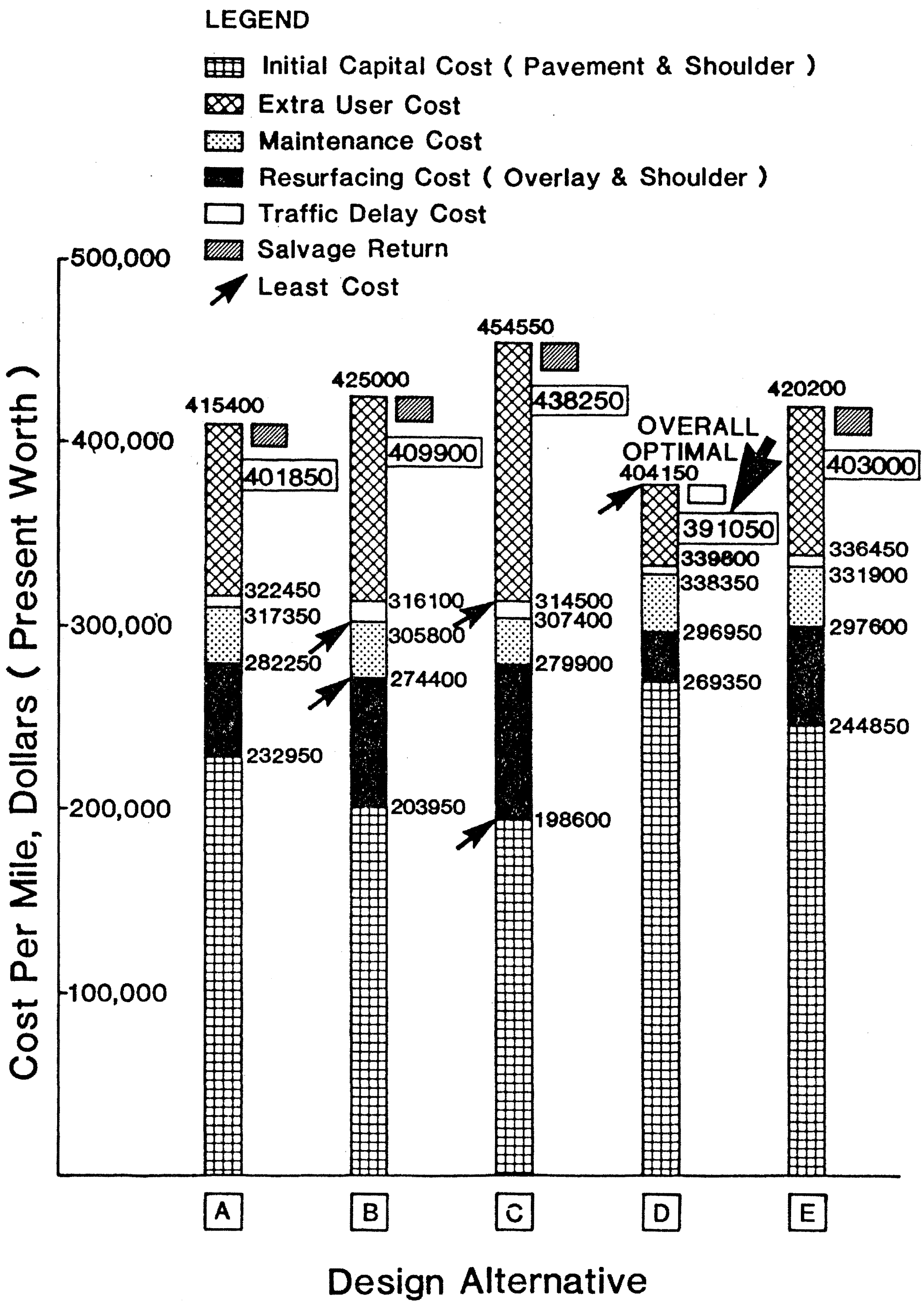
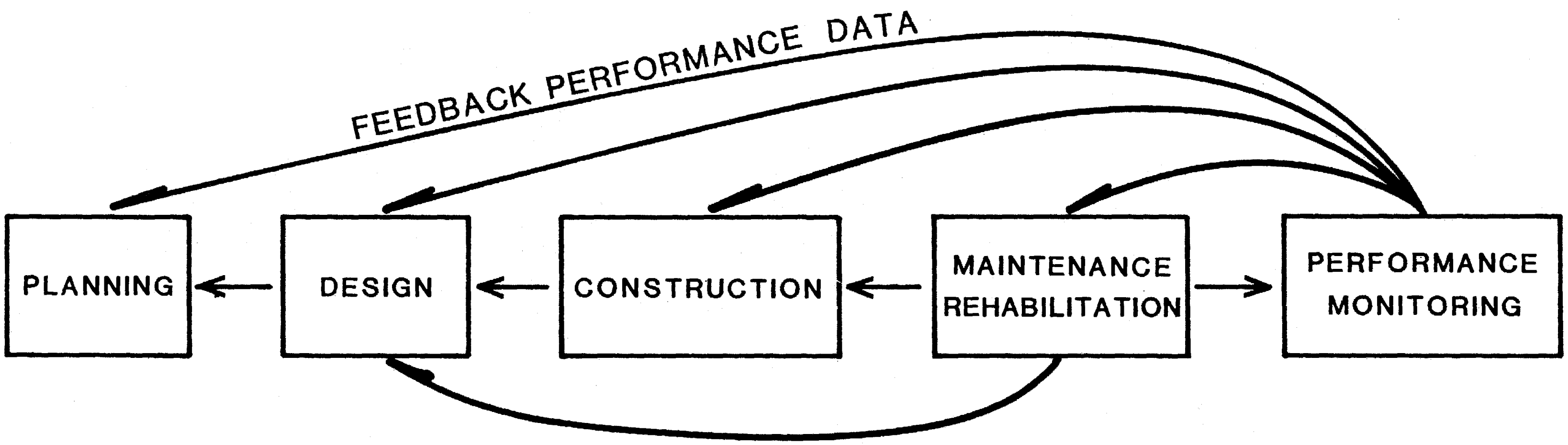


Figure 1-3.3. An Example of the Relative Costs for a Given Pavement Design (4).



IMPROVEMENT DESIGN THROUGH FEEDBACK DATA

Figure 1-3.4. Concept of Improving Pavement Technology Through Feedback of Performance Data.

design. Because design plays such an important role in determining overall life-cycle costs, feedback is essential to improve the design process.

A pavement monitoring program should provide at least the following performance information:

1. Distress (detailed cracking, joint deterioration, rutting, etc.)

2. Roughness.

3. Deflection.

4. Friction characteristics, particularly at high accident locations.

5. Traffic loadings through weigh-in-motion scales.

This information is considered essential to the development of a comprehensive database. Such a computerized database is essential to aid in the management and analysis of such a comprehensive amount of data. The database must also include key design, materials and construction data so that the pavement monitoring data can be related to traffic, design, construction and materials characteristics. This database becomes the "heart" of the overall pavement management system as shown in Figure 1-3.5.

The importance of using feedback performance data to improve pavement technology is shown in the importance being given to the "Long-Term Pavement Performance" (LTPP) program which is part of the Strategic Highway Research Program (SHRP). The LTPP research program proposes to monitor approximately 3,000 in-service pavement sections over a 20-year period to obtain comprehensive performance data. The development of empirical and mechanistic pavement design procedures and performance models from this data is one of the major objectives of the LTPP study.

5.0 SUMMARY

This module has discussed overall pavement management and the importance of pavement design to this management process. Specific items of concern to the pavement designer are as follows:

1. Pavements represent a large public investment, and therefore, comprehensive management systems and tools should be used to insure that all of the pavement management activities are accomplished effectively.

2. Pavement design is a critical aspect of pavement management in terms of affecting a stream of costs over many years.

3. The weakest link in the pavement management process has been the lack of feedback of critical performance monitoring information. this information could be used to improve planning, design, construction, maintenance and rehabilitation. Lack of this data has resulted in many design errors being perpetuated.

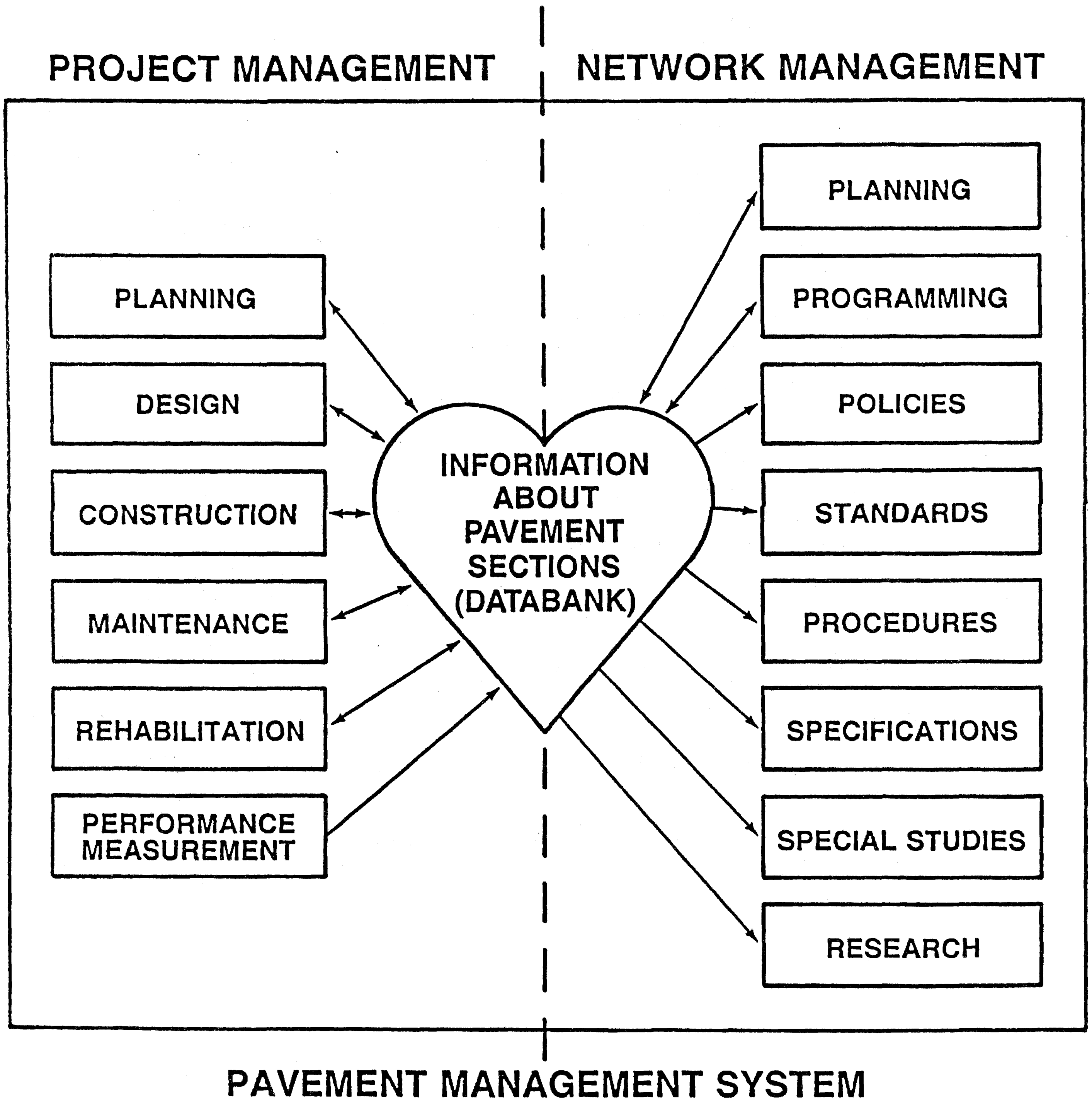


Figure 1-3.5. The Database Becomes the "Heart" of the Overall Pavement Management System (3).

4. Pavement Management includes pavement design as a major element.

5. The importance of pavements in the context of highway transportation can be summed up as follows:

   a. Highways are our product. For those we serve, this means one thing and one thing only -- PAVEMENTS.

   b. Right-of-way, grading, drainage, and subbases are all engineering and procedural requirements to build good pavements. Good signing, good geometrics, effective guardrails, and good delineation are all needed to aid safe use of our pavements. Safe and well designed PAVEMENTS are our end product -- what our consumers want, use, and pay for.

6.0 REFERENCES

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2. Pavement Seminar For Chief Administrative Officers, Participant Notes, Federal Highway Administration, 1986.

3. Roman, R. J., M. B. Snyder, M. I. Darter and M. R. Broten, "An Illinois Pavement Feedback System: Feasibility and System Requirements," Interim Report No. FHWA/IL/UI-209, Illinois Department of Transportation and University of Illinois, 1985.

4. Kher, R. K., W. A. Phang and R. C. G. Hass, "Economic Analysis Elements In Pavement Design," Transportation Research Record No. XXX, 1975.

5. Highway Statistics 1983, Federal Highway Administration, Superintendent of Documents, U. S. Government Printing Office, Washington, D. C., 20402.

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BLOCK 2

Initial Considerations in the Pavement Design Process

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BLOCK 2

INITIAL CONSIDERATIONS IN THE PAVEMENT DESIGN PROCESS

Modules

2-1 SUBGRADE SOILS
2-2 RESILIENT MODULUS
2-3 PAVING MATERIALS
2-4 DRAINAGE DESIGN
2-5 PAVEMENT PERFORMANCE
2-6 VEHICLE AND TRAFFIC CONSIDERATIONS
2-7 VARIABILITY AND RELIABILITY

This block presents the important items required for the overall pavement design process. These items are needed by the engineer in order to begin to select typical cross sections to handle the many variables which affect the performance of a pavement. If these items are not fully considered, the final thicknesses calculated for the proposed cross section may not be economical, or they may fail prematurely. The block is structured to relate these topics to the design process and the selection of typical cross sections capable of performing adequately.

Upon completion of this block the participants will be able to complete the instructional objectives listed for each module.

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MODULE 2-1

ROADBED SOILS

1.0 INSTRUCTIONAL OBJECTIVES

This module presents the physical and engineering properties of roadbed soils. Test procedures and classification systems are introduced to establish an understanding of the composition of a soil. Material property and strength test results are discussed as they relate to pavement design procedures for both rigid and flexible pavements. Special emphasis is placed on interpreting test results and correlating an existing test result with other methods of characterizing materials.

Upon completion of this module the participant will be able to accomplish the following:

1. List the steps required in classifying a roadbed soil and relate the classification to an indication of roadbed soil support.

2. List the common tests performed on roadbed soils and describe the significance of the test result and its relationship to roadbed soil support.

3. Describe the differences in the role of a roadbed soil on the performance of a flexible and rigid pavement.

4. Be able to correlate different roadbed soil tests and relate to resilient modulus.

2.0 INTRODUCTION

Since the early days in the development of pavement design, it has been recognized that the properties of individual pavement components have a great impact on the performance of the pavement. In the 1940's, the pavement design concept was primarily based on the physical and engineering properties of roadbed soil upon which the pavement was constructed. The properties of importance included soil classification, plasticity, shear strength, frost susceptibility and drainage.

Since the late 1950's, more emphasis has been placed on the fundamental properties of roadbed soils, and test methods have been developed for better characterization of these soils. Test methods using static or low strain rate loading conditions such as the California Bearing Ratio (CBR), unconfined compressive strength, etc., have been replaced with dynamic and repeated load tests such as modulus of resilience which more realistically simulate stresses and strains developing in the actual pavement, but these tests are still being conducted and used in pavement design.

The subjects discussed in this module relate to pavement design, and will broaden the pavement designer's appreciation for the requirements of a roadbed. Specifically:

1. Suitability of roadbed materials to be used for construction.

2. Design input requirements for roadbed material.

The material properties to be presented in this module are divided into the following:

1. Physical properties: These properties are most often used for the material selection, construction specifications and quality control.

2. Engineering properties: These properties provide an estimate of the design potential of the pavement materials. The design potential of the roadbed soils can be related to the primary response parameters which are modulus of resilience, Poisson's ratio, soil support value, modulus of subgrade reaction (K) value, etc., some of which will be discussed in Module 2-2, Resilient Modulus.

3.0 PHYSICAL PROPERTIES OF ROADBED SOILS

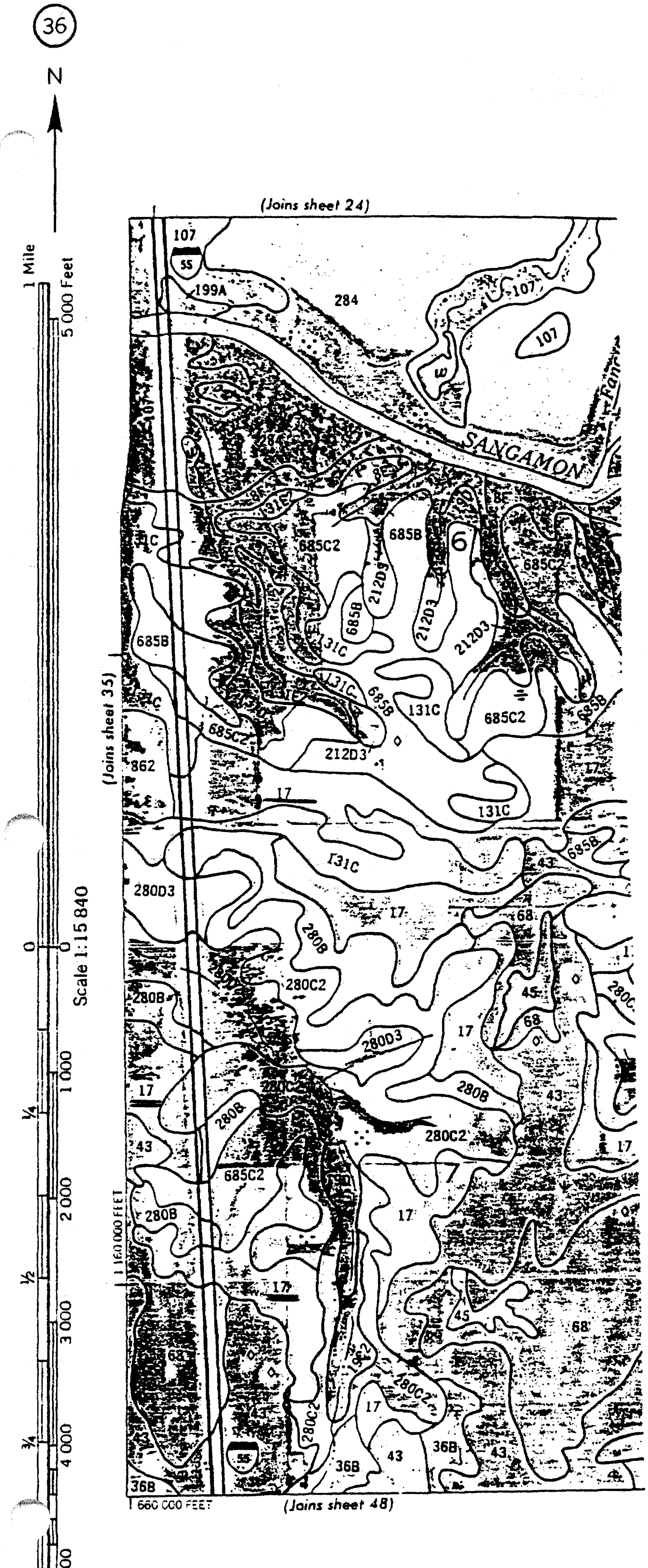
3.1 Initial Soil Properties

The pavement design process requires properties of the roadbed soil for design because the roadbed soil represents a design quantity that cannot be changed in the design, and it must be characterized very thoroughly. This characterization involves extensive sampling of the soils along the right-of-way of the proposed highway. The samples are brought to the laboratory,and the tests described in this module are conducted on the soils collected. The testing program must be designed to provide an indication of the variability of the soils along the length of the project. This variability will have an influence on the final thickness and even on the pavement type selected for construction.

Before extensive field sampling and testing is performed, there are a number of published reports on soil properties which should be consulted. The most comprehensive of these is the county soil map published by the Agricultural Extension Services and the Soil Conservation Service. These maps are prepared from extensive field sampling and testing and the information is plotted on aerial photographs with a scale of 1 inch to 5000 feet. The different soil types are indicated along with the existing appurtenances, as is shown in Figure 2-1.1 which is taken from a county soil map in Illinois. Also indicated on this figure are some of the typical material properties which can be obtained in the publication. Each soil type with its number is identified in the report, and detailed information is supplied. These maps provide a ready source of information on properties and potential variability along the project length. This information can help avoid excessive testing costs by providing initial data on soils in the area.

3.2 Soil Classification

The engineering classification of soils is the most universally accepted indicator of the physical property of a naturally occurring soil. The AASHTO method of soil classification is the method most commonly accepted by engineers in the highway field. This procedure relies on the grain size distribution and the plasticity characteristics of the soil to differentiate between soils based on their potential to perform as a roadbed under a pavement structure.



Example: Data Available for Soil 17,
Sangamon County, Illinois,
Three locations along project.

Hydrologic Group:	C		
Flooding:	None		
High Water Table:	2-4 feet April-June		
Frost Action:	High		
Permeability:	0.2-0.6 ins/hr		
Shrink-Swell:	High		
Organic Matter:	1-2 % A horizon		
Soil Classification:	A	CL-ML	A-4,A-6
	B	CH-CL	A-7
	C	CL	A-7
Liquid Limit:	A	25-35	
	B	45-60	
	C	35-50	
Plasticity Index:	A	5-15	
	B	30-45	
	C	20-30	

Stability as a Construction Material:

Poor Roadfill - low strength
Poor Sand - excess fines
Poor Gravel - excess fines

Stability as Septic Field:

Severe (poor) - percolates water too slowly, remains too wet

Figure 2-1.1. Soil Map for Example Project Illustrating Soil Data Available for Soil 17.

The AASHTO classification system (AASHTO M145) breaks the grain sizes into the following ranges delineated by U.S.A. Standard Sieves:

1. Gravel, Three inch to No. 10 Sieve.

2. Coarse Sand, No. 10 to No. 40 Sieve.

3. Fine Sand, No. 40 to No. 100 Sieve.

4. Silts and Clays are all passing the No. 200 Sieve.

The break-point between coarse and fine-grained soils is the amount of material passing the No. 200 sieve. In the AASHTO procedure a fine-grained soil has more than 35 percent passing the No. 200 sieve. Thus roadbed soils will commonly be either A-4, A-5, A-6, or A-7 soils as shown in Figure 2-1.2. One coarse-grained soil classification with similar characteristics to a fine-grained soil is the A-2 classification because it may have nearly the same amount of material passing the No. 200 sieve as the fine-grained materials.

With similar gradations, the plasticity characteristic of the soil particles smaller than No. 40 sieve have a large influence on the performance of the soil (AASHTO T89 and T90). The plasticity characteristics are determined by the Atterberg Limit tests. These tests are the Liquid Limit, LL, the Plastic Limit, PL, and the Plasticity Index, PI. The PI is equal to the LL minus the PL. These limits are the moisture content at which the soil exhibits a change in behavior from liquid to plastic to elastic respectively. These limits are affected primarily by the clay particles in the soil and relate closely to the potential performance of the soil.

The plasticity limits for each soil classification are indicated in the classification chart given in Figure 2-1.2, and are shown graphically in Figure 2-1.3. Higher limits generally indicate poorer performance when used as a roadbed soil, but even within a particular classification there will be variability in both plasticity and gradation which are not differentiated in the general classification. To provide this differentiation, the "group index" was developed. The group index is an empirical equation based on the service performance of many soils that indicates the relative amount of fine silts and clays and the plasticity characteristics of a soil. It provides a more detailed description of the soil and its performance potential. The equation to calculate the group index was given in Figure 2-1.2

There are other soil classification schemes in use by other agencies, including the Unified Soil Classification System. The Federal Aviation Administration (FAA) Classification System, Pedological Classifications, and the United States Department of Agriculture Classification System are examples other agencies are using.

The classification of a soil is an important input for the pavement design process. Relationships that have been developed which correlate soil classification and structural design properties are an easy method for developing input for the initial selection of pavement structures without extensive testing.

General Classification	*Granular Materials (35% or less passing No. 200)*							*Silt-Clay Materials (more than 35% passing No. 200)*			
	A-1			*A-2*							*A-7*
Group Classification	*A-1-a*	*A-1-b*	*A-3*	*A-2-4*	*A-2-5*	*A-2-6*	*A-2-7*	*A-4*	*A-5*	*A-6*	*A-7-5, A-7-6*
Sieve analysis, % passing:											
No. 10	50 max	......	......	......	......	......	......	......	......	......	
No. 40	30 max	50 max	51 min	......	......	......	......	......	......	......	
No. 200	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40:											
Liquid limit	......		......	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min†
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

* Reprinted by permission of the American Association of State Highway and Transportation Officials from *AASHTO Materials*, 12th edition, 1978.

† Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30.

Group Index = (F-35) [0.2 + 0.005 (LL - 40)] + 0.01 (F-15) (PI-10)

F = percentage passing the No. 200 sieve, expressed as a whole number.

LL = liquid limit (whole number)

PL = plastic limit (whole number)

PI = plasticity index = LL - PL

Notes:

1. If the calculated group index is negative, report the group index as zero (0).
2. If the soil is nonplastic and when the liquid limit cannot be determined, report the group index as zero (0).
3. Report the group index to the nearest whole number.
4. For A-2-6 and A-2-7 subgroups, use only the PI portion of the formula.

Figure 2-1.2. AASHTO Soil Classification.

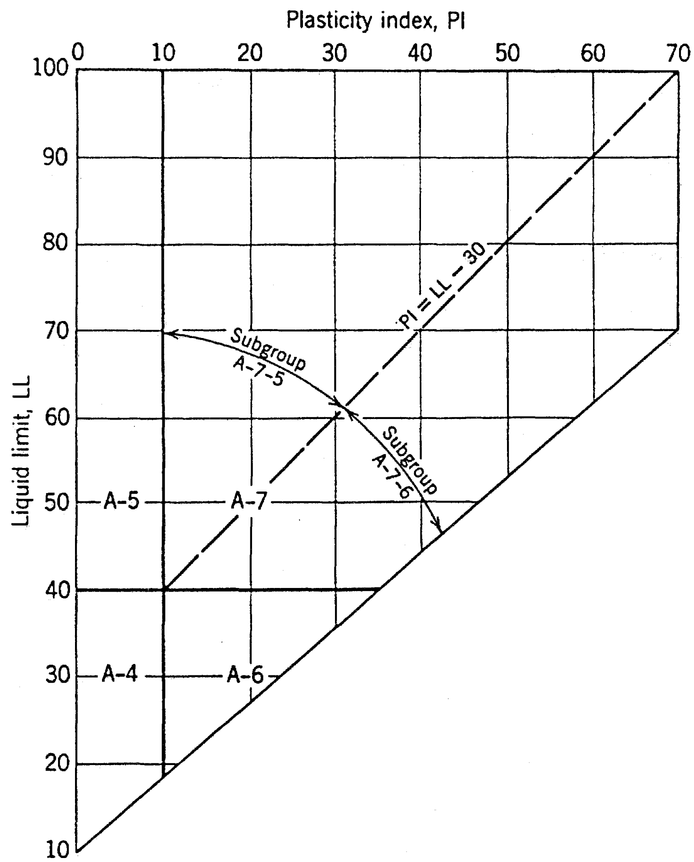


Figure 2-1.3. Plasticity Chart for AASHTO Soil Classification.

3.3 Moisture-Density

The moisture content and density relationship (AASHTO T99 or T180) for a soil is a critical factor affecting the strength and deformation properties of any prepared soil. Careful laboratory testing to establish this relationship is critical for its use in specifications to ensure adequate quality control of the material used in construction to provide the desired structural capacity in the roadbed material for the pavement design. The laboratory data is critical for accurately monitoring a project during construction and for knowing the compacted density at all times. This importance extends beyond design requirements to determining density and moisture values of the roadbed material anytime rehabilitation is planned because these values will exhibit seasonal variation.

A Proctor compaction curve is shown in Figure 2-1.4 indicating the maximum density and optimum moisture content which are the two principal output values from this test. It must always be remembered that each soil has an individual relationship that must be established in the laboratory. Figure 2-1.5 contains typical compaction curves for a variety of soils. It is critical to select a moisture density specification that ensures a specific level of physical performance in the soil and to perform sufficient tests to ensure that these values are obtained in the field. The moisture density state of a compacted soil affects the strength and deformation characteristics of the soil in the pavement which is related to its structural adequacy. Figure 2-1.6 shows the influence of moisture and material type on the CBR values of compacted materials.

With any design procedure, it is important to recognize the relationship between the moisture density relationship of a soil and the resulting design when the moisture or density changes. One drawback of the traditional strength testing is that they require a specific testing program which relies on specific moisture density conditions and does not allow design variability. The resilient modulus provides a means to fully describe a soil's design potential.

With the new design procedures relying on an accurate determination of the resilient modulus, the control of density becomes more important because a statistical selection of a minimum value will not be allowed as they were in the old design procedures. Previously, the average strength value for design minus two standard deviations was used in the design process. The new design guide uses only the average value, and variability must be accounted for separately. The importance of moisture density control will be shown in subsequent sections which describe test procedures and results for various soils.

4.0 SOIL STRENGTH PARAMETERS FOR HISTORICAL DESIGN PROCEDURES

Various tests have been used to measure the ultimate response of roadbed soils and to evaluate the ability of the roadbed to adequately sustain traffic loads. The most common strength tests are:

1. California Bearing Ratio (CBR) (AASHTO T193).

2. Resistance Value (R value) (AASHTO T190).

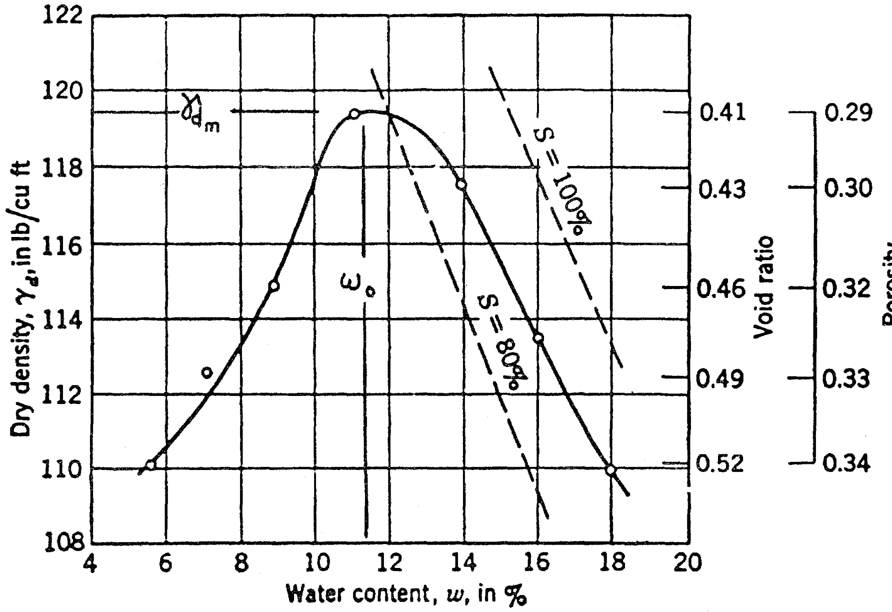
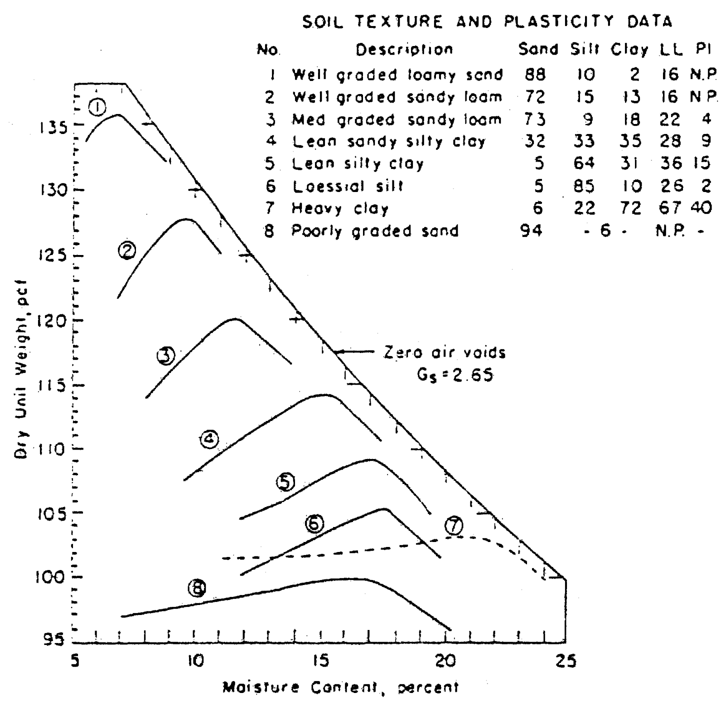


Figure 2-1.4. Typical Proctor Moisture-Density Curve.



SOIL TEXTURE AND PLASTICITY DATA

No.	Description	Sand	Silt	Clay	LL	PI
1	Well graded loamy sand	88	10	2	16	N.P.
2	Well graded sandy loam	72	15	13	16	N.P.
3	Med graded sandy loam	73	9	18	22	4
4	Lean sandy silty clay	32	33	35	28	9
5	Lean silty clay	5	64	31	36	15
6	Loessial silt	5	85	10	26	2
7	Heavy clay	6	22	72	67	40
8	Poorly graded sand	94	- 6 -		N.P.	-

Figure 2-1.5. Typical Compaction Curves for Different Soil Types.

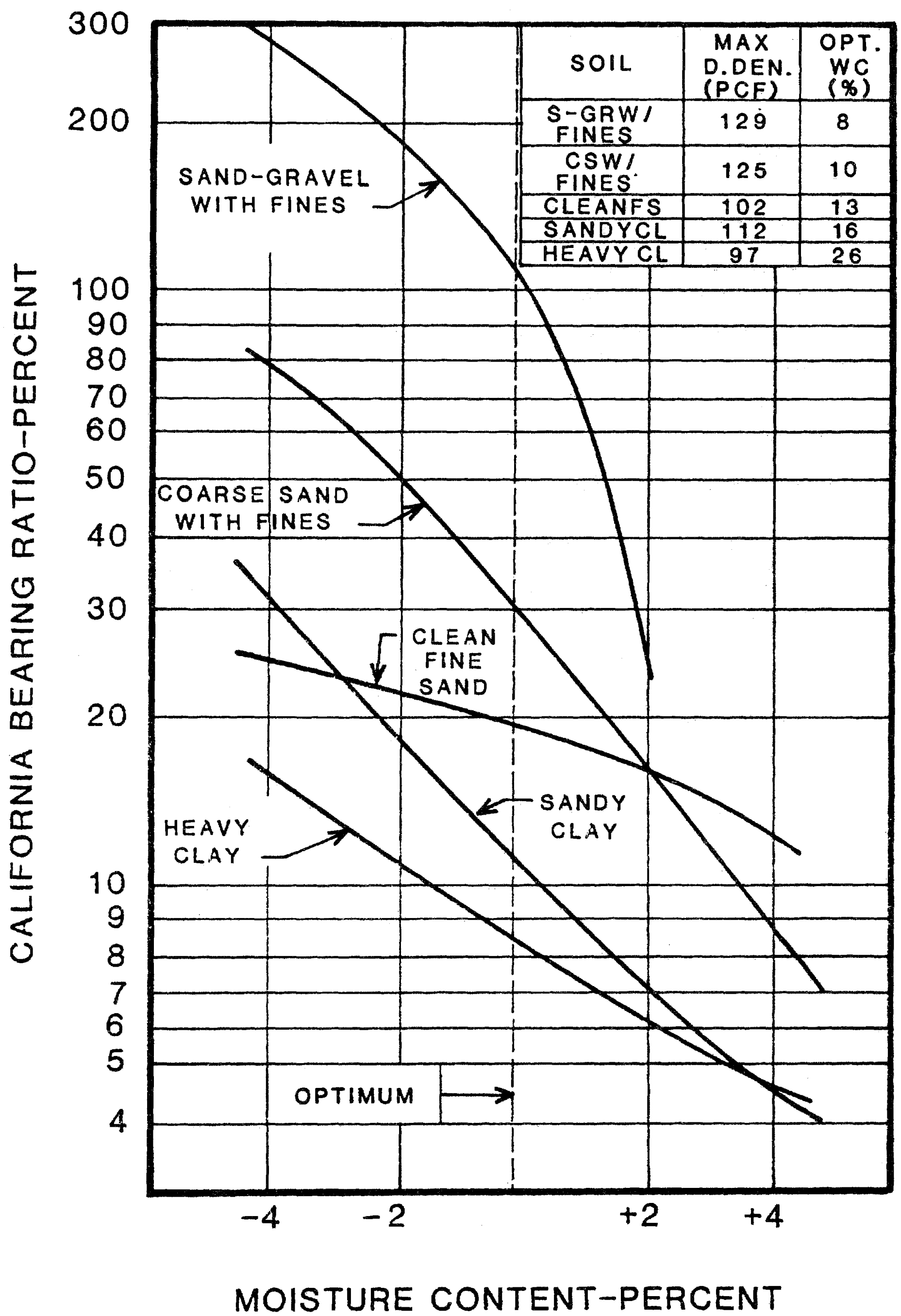


Figure 2-1.6. Influence of Moisture Content and Soil Type on CBR Values.

3. Modulus of Subgrade Reaction (k value).

4. Triaxial.

Each test was developed for a specific purpose and met a need at the time it was developed. Few of these tests provide a rational analysis of basic soil properties such as the resilient modulus (M_R), permanent deformation, or volume stability under traffic loading.

4.1 CBR

The CBR (California Bearing Ratio) test measures the resistance of the soil to penetration. A piston with an end area of 3 square inches is pressed into a six inch diameter, five inch tall soil specimen in a steel compaction mold at a standard rate of 0.05 inches per minute. The load required to force the piston into the soil is measured at given penetration intervals. The resulting penetrations are compared to the penetration recorded for a standard, well-graded crushed stone to get the bearing ratio as a percentage of the standard. CBR curves for a wide range of soils are pictured in Figure 2-1.7. Because this test is arbitrary in nature, it has many limitations. An advantage is the relatively simple equipment needed and the large amount of historical data available for correlating results with field performance. A major disadvantage is that the test method is very sensitive to the method of specimen preparation. There have been significant modifications to the original CBR method to improve its applicability.

In Figure 2-1.8 the interrelationship between strength, as measured by CBR, density and moisture content are shown. The CBR "as molded" will gradually decrease for samples compacted at the higher moisture contents. However, CBR values after a four-day soaking period show a peak similar to that of moisture-density curves. If a swelling soil is compacted below the optimum moisture content, upon soaking, its strength is substantially lost. The relationship between CBR, moisture and density does provide valuable information for the designer. For pavement design purposes, an estimate of soil CBR can be obtained from Figure 2-1.9 and Table 2-1.1.

4.2 R Values

The R value (resistance value) is derived from a test conducted in a stabilometer as shown in Figure 2-1.10. A cylindrical sample (4 inches in diameter, 2-1/2 inches tall) is enclosed in a membrane and loaded vertically over the full face of the sample to a given pressure. The resulting horizontal pressure is measured and used to calculate the resistance (R) value.

The R value method of test developed by F. N. Hveem and R.M. Carmany of the California Division of Highways (1) has been used most frequently in the western states. The procedure actually involves two separate tests:

1. The thickness (weight) of cover required to resist expansion of the soil is determined by the expansion pressure test.

2. The R value test evaluates the soil's relative ability to sustain loads.

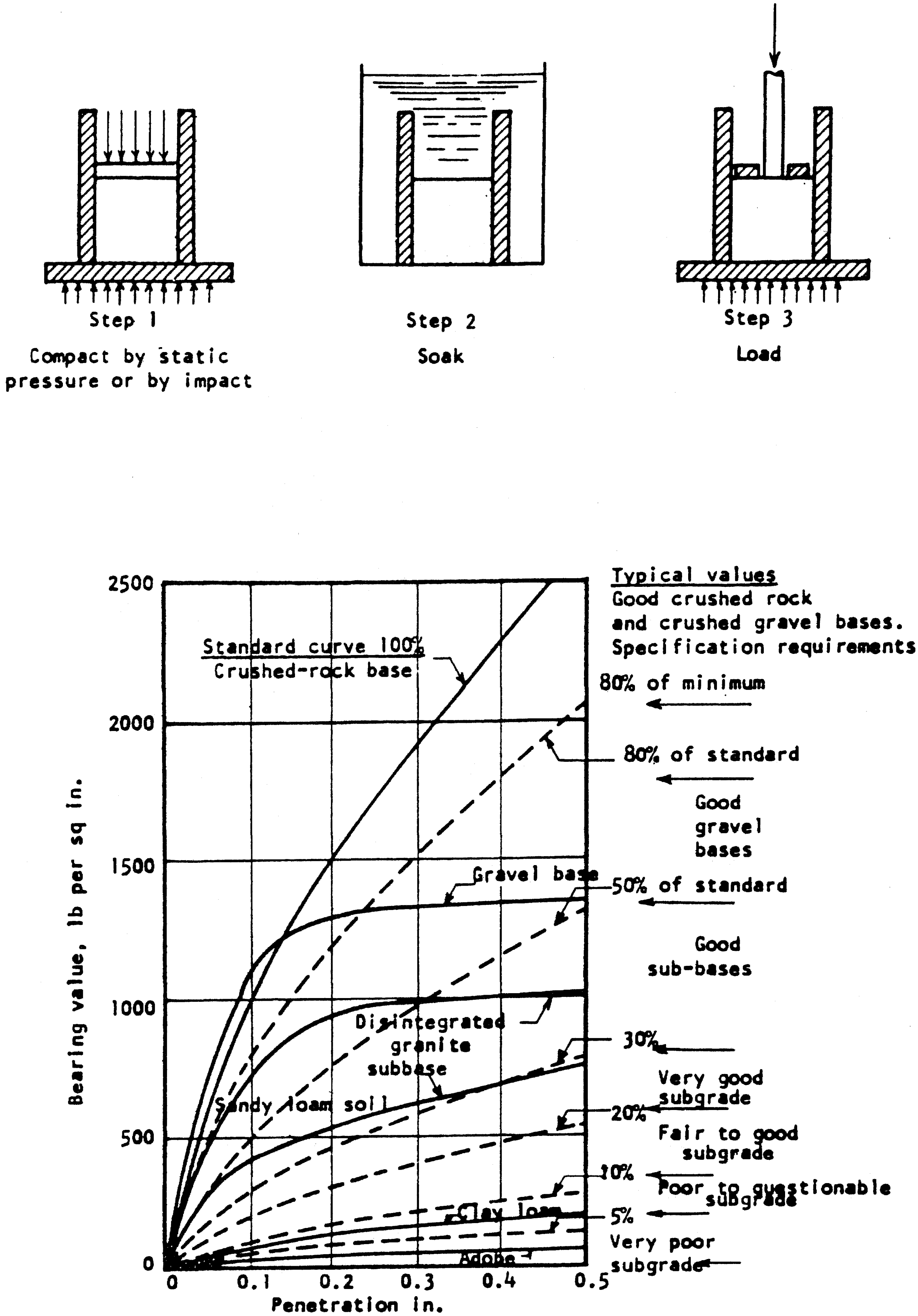


Figure 2-1.7. CBR Testing Procedure and Load-Penetration Curves for Typical Soils.

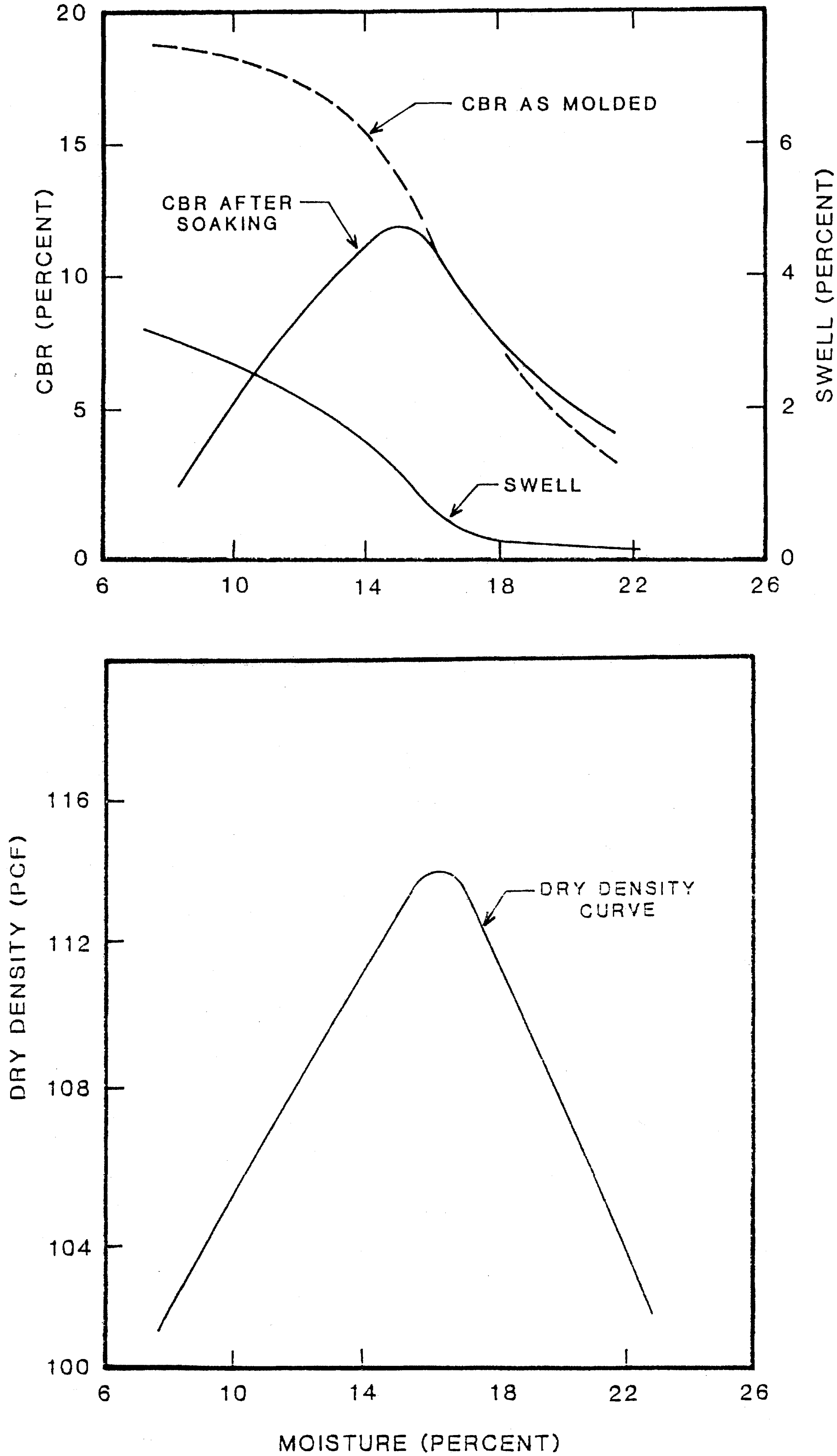


Figure 2-1.8. Impact of Moisture (Soaking) on Strength and Volume Change.

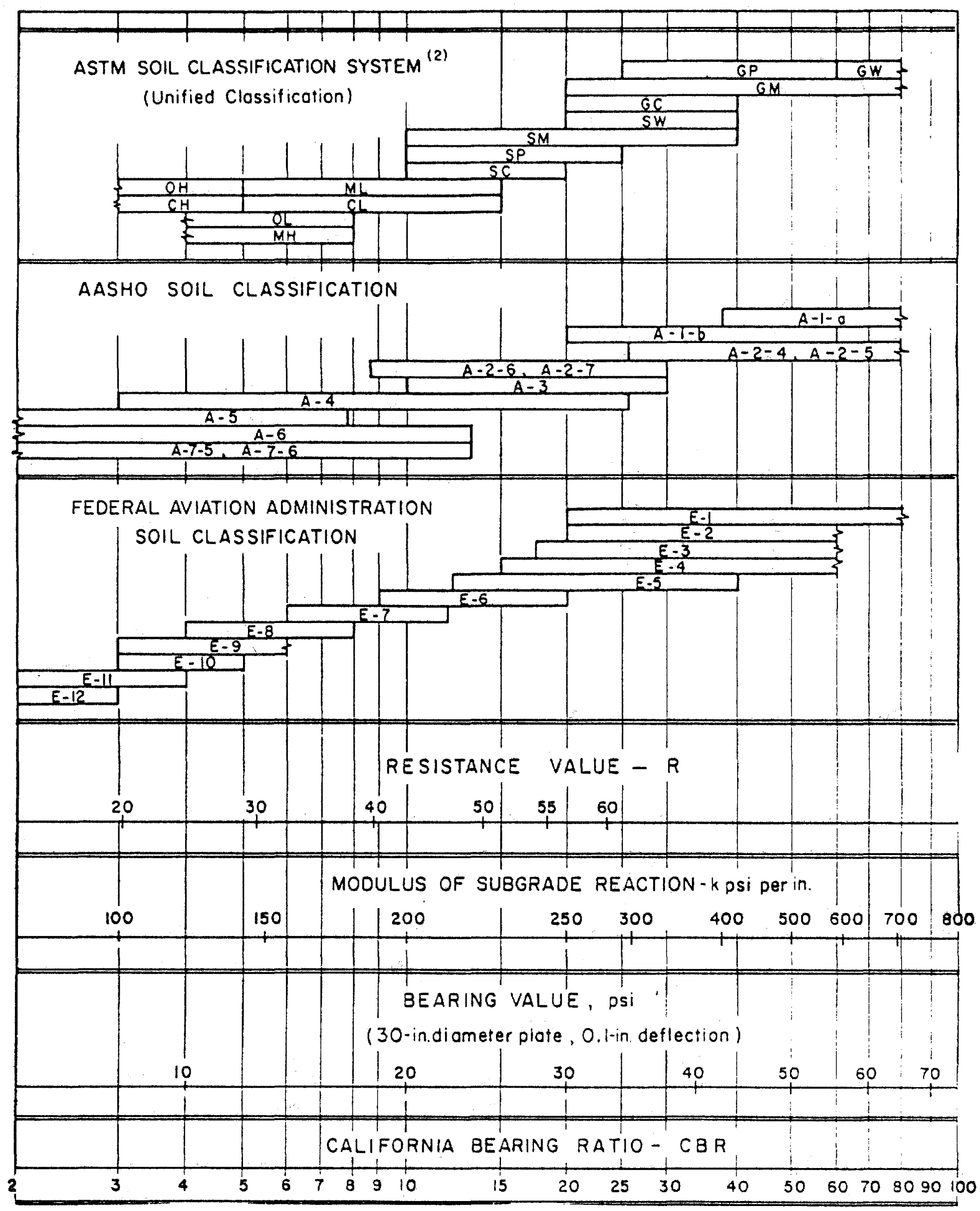


Figure 2-1.9. Soil Classification Related to Strength, From PCA.

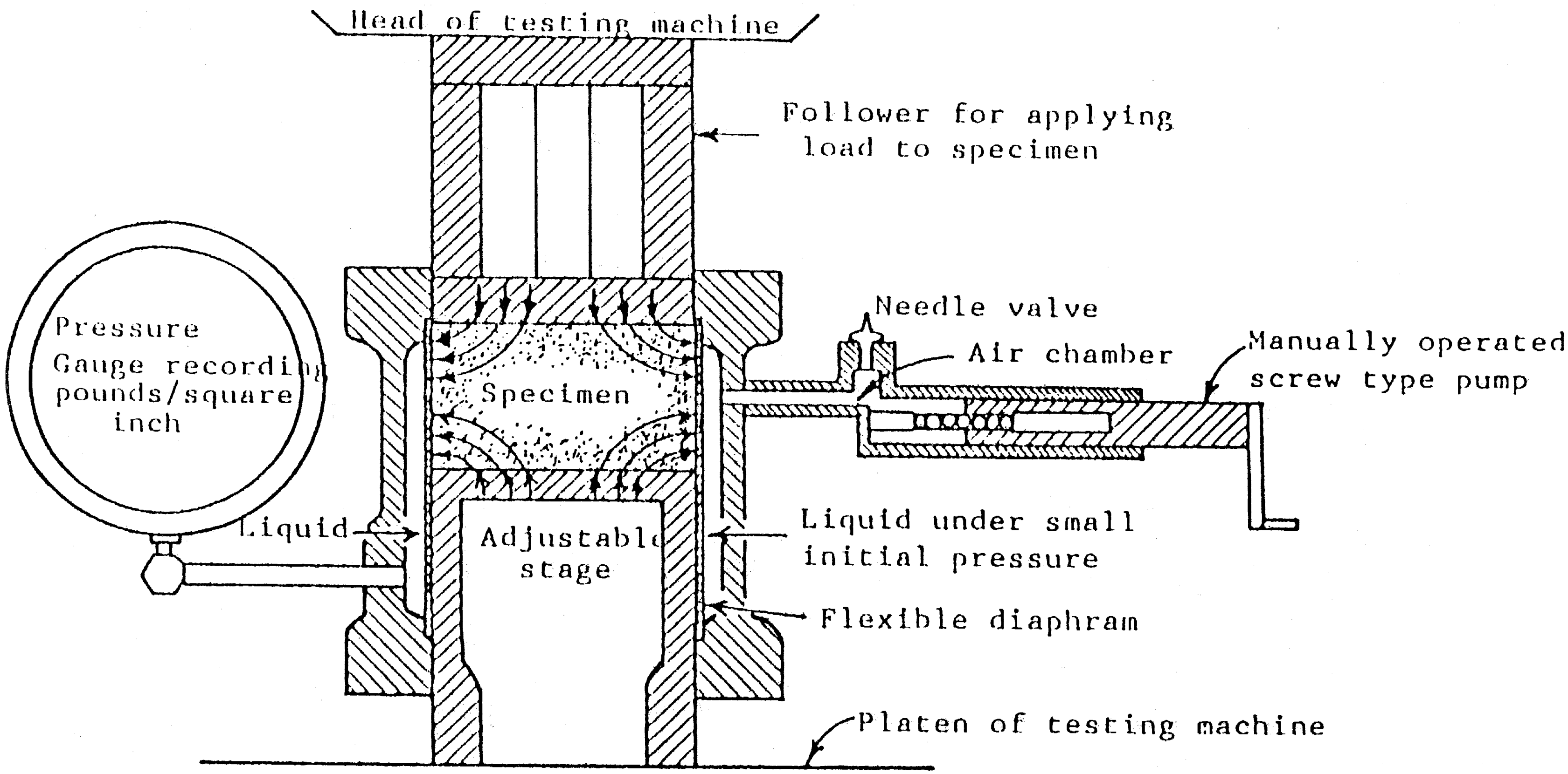


Figure 2-1.10. Diagram of Hveem Stabilometer.

Table 2-1.1 Approximate Range of CBR and K Values for Soil Groups of the Casagrande Soils Classification as Used by Corps of Engineers, Department of Army.

Major Div.	Soil groups and typical description	CBR	Approximate Range of k-values
Gravel and Gravelly Soils	Well-graded gravel and gravel-sand mixtures. Little or no fines	80-100	500-700, greater
	Well-graded gravel-sand-clay mixtures. Excellent binder.	80-100	400-700, greater
	Poorly graded gravel and gravel-sand mixtures. Little or no fines	30-60	300-500
	Gravel with fines, very silty gravel clayey gravel, poorly graded gravel-sand-clay mixtures	30-60	250-500
Sands and Sandy Soils	Well-graded sands and gravelly sands Little or no fines.	25-60	250-575
	Well-graded sand-clay mixtures Excellent binders.	25-60	250-575
	Poorly-graded sand. Little or no fines	15-25	200-325
	Sand with fines, very silty sand, clayey sands, poorly graded sand-clay mixtures	10-25	175-325
Fine grained soils with low to medium compress-ibility	Silts (inorganic) and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	5-25	150-300
	Clays (inorganic) of low to medium plasticity, sandy clays, silty clays, low plasticity clays	5-15	125-225
	Organic silts and organic silt-clays of low plasticity	3-10	100-175
Fine grained soils with high compress-ibility	Micaceous or diatomaceous fine sandy and silty soils, elastic silts	1-5	50-175
	Clays (inorganic) of high plasticity, fat clays	1-3	50-150
	Organic clays of medium to high plasticity	1-3	50-125

For design purposes, the R values for various soil types have been correlated with the Group Index, CBR and other soil properties. R values for different soils were presented previously in Figure 2-1.9.

4.3 Modulus of Subgrade Reaction

The modulus of subgrade reaction, K value, is determined from a plate loading test set up as indicated in Figure 2-1.11 (2). A thirty-inch diameter plate is loaded to a given pressure (usually 10 psi) at a specified rate and the resulting deflection is measured. The K value is calculated as the unit load on the plate divided by the deflection of the plate. The test must be conducted in the field and requires expensive equipment.

The modulus of subgrade reaction, K, is given by

$$K = p/\Delta$$

where p = unit pressure on the plate, typically 10 psi
Δ = vertical deflection of the plate, in

Since determining the K value is a field test procedure, it cannot be conducted at various densities and moisture contents to approximate different service conditions. It is recommended that the field K value be adjusted for the most unfavorable roadbed condition. This correction factor is obtained as a ratio of deformation at 10 psi pressure of unsaturated soil, given by d, to that of saturated soil, given by "ds":

$$K = (d/ds) \times K_{uncorrected}$$

The modulus of subgrade reaction, K, is also dependent on the plate size. Thirty-inch diameter plates are used for rigid pavements; whereas plate sizes for flexible pavements range between 12 and 18 inches.

The modulus of subgrade reaction can be estimated based on soil classification data or using correlations with other soil engineering properties such as CBR and modulus of resilience, as shown previously in Figure 2-1.9 and Table 2-1.1.

4.4 Triaxial Compression Test

Triaxial testing is used by several states to evaluate the shear strength of roadbed soils. The unsaturated, unconsolidated, undrained test is most commonly used (3) although a saturated test is preferred. The soil samples are compacted either statically or dynamically to near optimum moisture and density in a cylindrical mold with a finished size of from 1.4 x 2.8 inches to 6 x 12 inches. The sample is then fitted with a flexible membrane and placed in a triaxial cell capable of sustaining a confining pressure around the sample as shown in Figure 2-1.12.

A confining pressure comparable to the in situ stress is applied to the cell. A controlled rate of strain (usually 0.05 inches per minute) is applied along the vertical axis of the sample, and readings of vertical load (stress) and total strain are made to plot a curve as shown in Figure 2-1.13. A series of stress-strain curves are developed by testing similar samples under various confining pressures. Using the maximum stress from

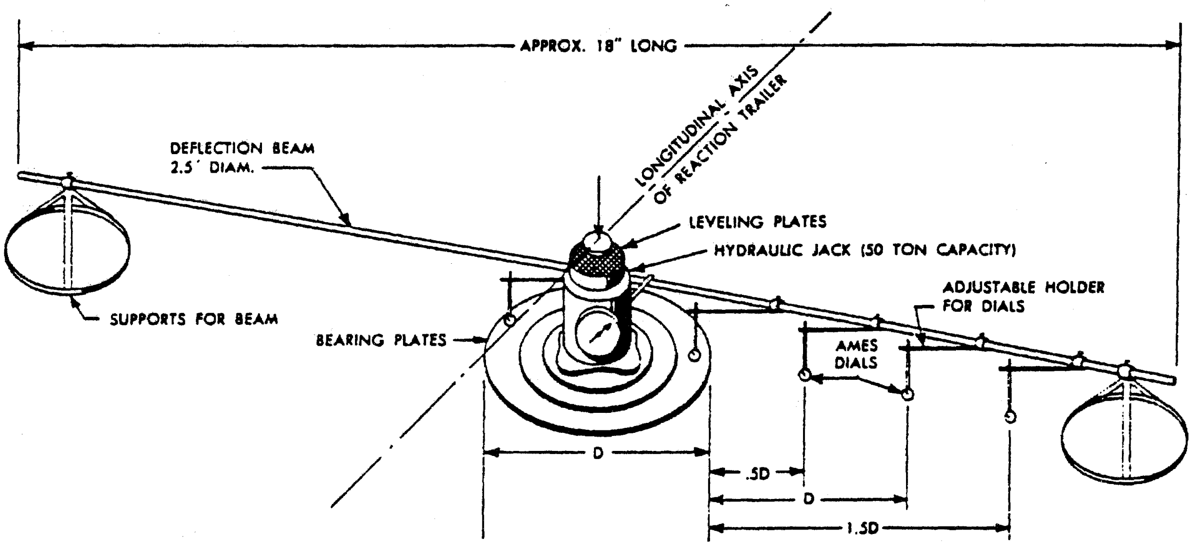


Figure 2-1.11. Arrangement of Equipment for Plate Bearing Test.

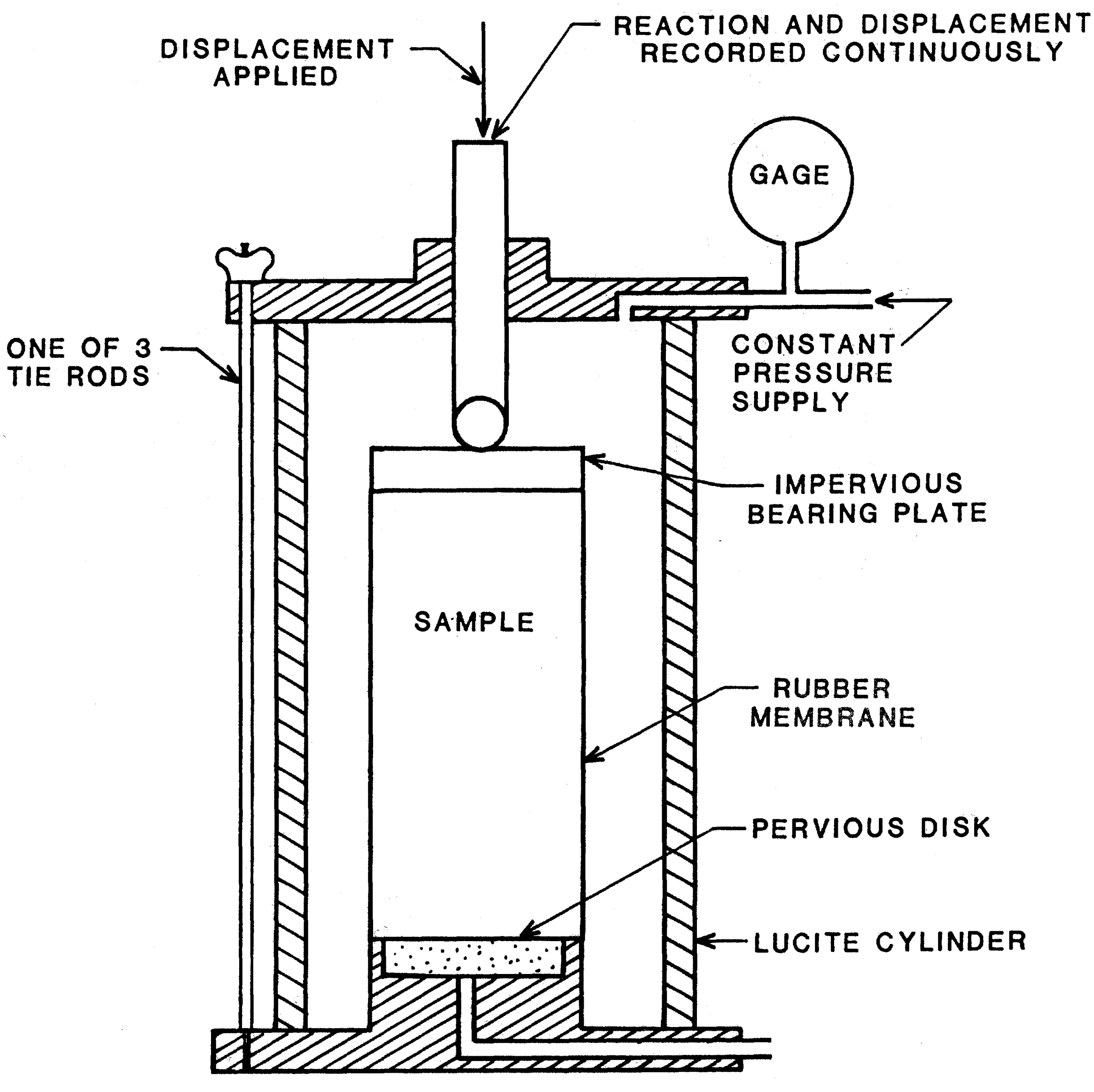


Figure 2-1.12. Triaxial Cell.

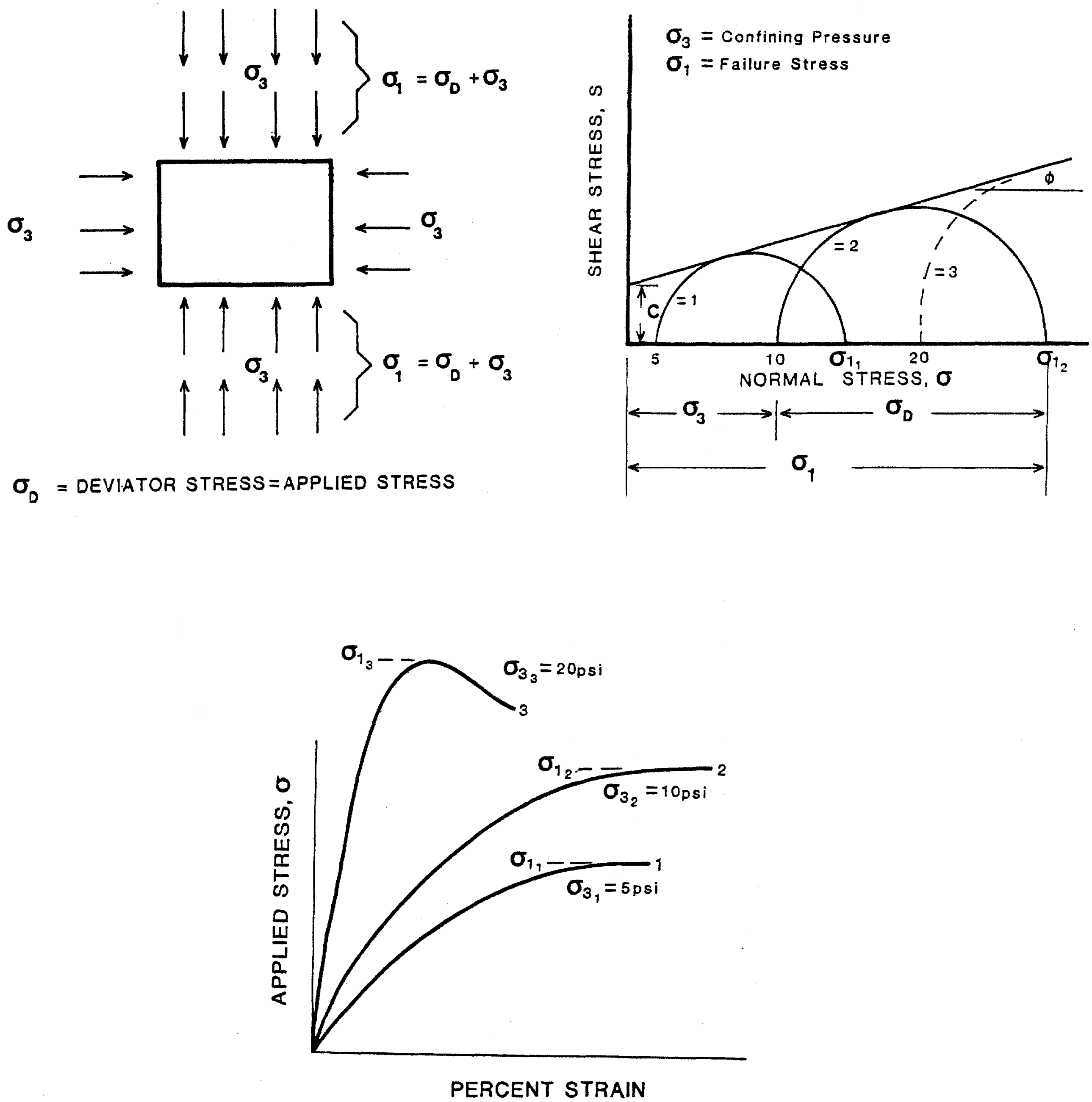


Figure 2-1.13. Development of Mohr Failure Envelope From the Triaxial Test.

each test, a Mohr rupture envelope is constructed, as shown in Figure 2-1.13. From the Mohr's diagram, the cohesion and angle of internal friction may be obtained as indicated. Figure 2-1.14 shows the Texas triaxial design which classifies triaxial envelops into categories requiring differing thicknesses for protection of the subgrade.

As with the other test procedures discussed, the method of compaction, the moisture, and the density of the sample significantly affect the results obtained. Since the shear strength is estimated using the failure stress of each individual specimen, tested under different confining pressures, it is essential that all specimens prepared for this determination have the same initial moisture content and density. This necessitates some judgment in the evaluation of test results.

These strength tests provide relative indications of quality which relate, empirically, to a pavement thickness required to protect the roadbed and provide a measure of performance based on previously observed pavements. These tests have no relationship to theory, which limits their applicability for use in new or innovative pavement designs. This is a major reason behind the adoption of the modulus of resilience as the roadbed soil strength parameter. It has a theoretical relationship which allows mechanistic procedures to be used to evaluate new and innovative designs.

5.0 SOIL STRENGTH PARAMETERS FOR PAVEMENT DESIGN

The new AASHTO design guide replaces the old Soil Support Value (S) with the resilient modulus (M_R) which will be detailed in Module 2-2. The M_R value in the AASHTO design procedure is the average value for the roadbed soil. This average is the average of all tests along the route of the pavement, and points out the importance of testing samples at the density and moisture content they will develop in service. This average value is further adjusted in the design process for seasonal variability.

The traditional pavement design procedures which utilize the parameters discussed in this module all require the use of an average value that is adjusted to account for the statistical variability in the material along the length of the roadway. This adjustment is typically the average value plus or minus two standard deviations. The new AASHTO design procedure has been structured to use only the average value for the materials. The use of extreme conditions to represent minimum expected values must be judiciously avoided in the future, and gives reason for improved testing procedures to very accurately characterize the roadbed soil.

5.1 CBR

The CBR value represents the strength of a saturated soil as compared to an excellent crushed stone base material. The design principle is that the lower the CBR, the thicker the pavement material placed on top of the roadbed to protect the roadbed soil from being overstressed. The design curves using CBR have all been developed from field observation of actual pavements under traffic and failure was related to CBR, pavement thickness, and number of traffic loadings. A design curve utilizing CBR is shown in Figure 2-1.15.

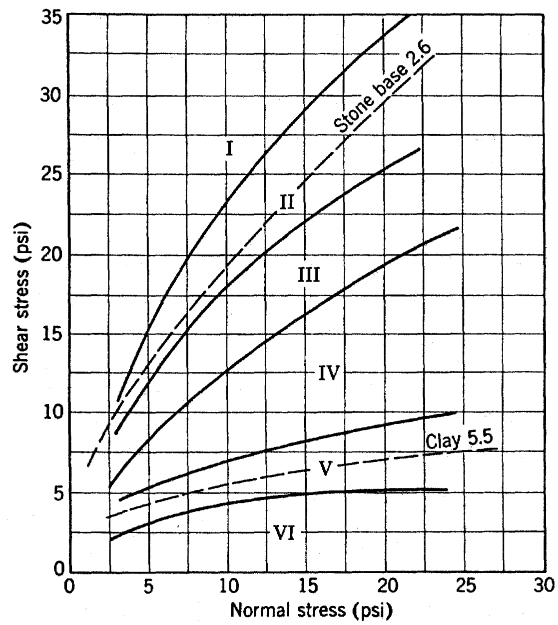


Figure 2-1.14. Texas (McDowell) Triaxial Test Classfication Chart.

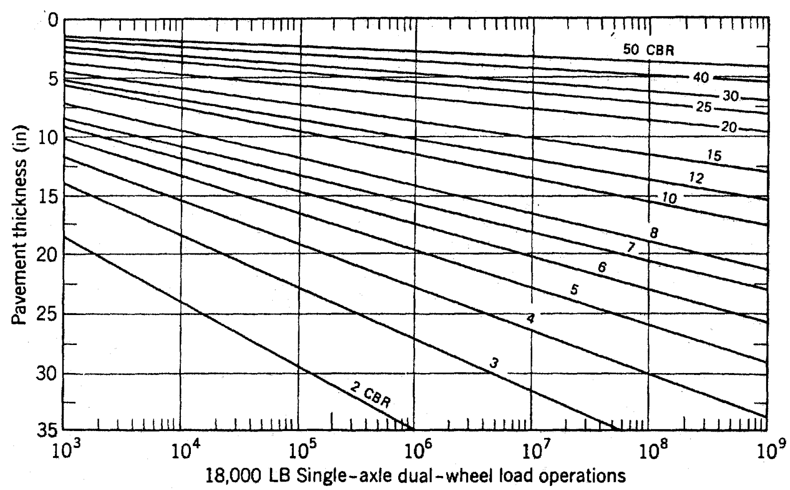


Figure 2-1.15. CBR design Curves for 18,000 EAL. (From Corps of Engineers.)

5.2 Hveem

The Hveem test produces a number with the same relationship to thickness requirements as the CBR test does. The nature of the test does relate more directly to the loading mode of a pavement soil, in that the soil is loaded, and the resistance to lateral deformation is used to calculate the R value. The more resistance to deformation under load, the better the roadbed soil is and the less pavement material that is required to carry the design traffic. A design chart for the Hveem test is shown in Figure 2-1.16.

5.3 Modulus of Subgrade Reaction

The k value is a field test, and typically is conducted for airfield pavements rather than highway pavements, and is limited strictly to design of rigid pavements. The use of this test for highways is traditionally developed from correlations with CBR or Hveem through limited field tests on different soils, and design curves for rigid pavements can be found in the AASHTO design guide and will be discussed in Block 4.

5.4 Triaxial

The failure envelops from the triaxial test shown earlier in Figure 2-1.13 represent levels of material quality. The lower curves indicate weaker materials which could not sustain as high a load as the others. As confining pressure is increased, the roadbed soils will increase in load-carrying capacity in the triaxial mode of testing providing the upward slope to the curves. The design principle with the triaxial test is exactly the same as for CBR with the weaker materials requiring more pavement thickness to protect the roadbed. The Texas triaxial design chart is shown in Figure 2-1.14 with various classifications of material quality indicated.

6.0 CORRELATION CONSIDERATIONS FOR MODULUS OF RESILIENCE

With the introduction of the resilient modulus value into the AASHTO design process it will become necessary for all states to conduct appropriate testing to develop resilient modulus values for the soils in their state. It is realized that not all states will have the necessary equipment to develop the M_R data required for immediate use in design projects. Therefore, some general correlations will have to be used to relate these common tests to the resilient modulus of the roadbed soils before specific data are developed for the state. These correlations will be discussed in Module 2-2.

6.1 California Bearing Ratio

The M_R values for soils must be developed to take full advantage of the AASHTO guide as well as the mechanistic procedures which rely solely on the resilient modulus. In the same manner that the old Soil Support Value was related to different soil tests, until these precise correlations are developed some standard correlations can be used.

An accepted approximate correlation is:

$$M_R = B * CBR$$

For soils with a CBR equal to or less than 10 the value of B is 1500 although the value may vary from 750 to 3000.

Figure 2-1.16. California Gravel Equivalents of Structural Layers in Feet.

Actual Thickness of Layer (ft)	ASPHALT CONCRETE — Traffic Index (TI)										BTB and LTB	Cement-treated Base — Class		Aggre-gate Base	Aggre-gate Sub-base
	5 and below	5.5 6.0	6.5 7.0	7.5 8.0	8.5 9.0	9.5 10.0	10.5 11.0	11.5 12.0	12.5 13.0	13.5 14.0		A	B		
	Gravel Equivalent Factor (G_f)										G_f	G_f		G_f	G_f
	2.50	2.32	2.14	2.01	1.89	1.79	1.71	1.64	1.57	1.52	1.2	1.7	1.2	1.1	1.0
0.10	0.25	0.23	0.21	0.20	0.19	0.18	0.17	0.16	0.16	0.15	0.12	—	—	—	—
0.15	0.38	0.35	0.32	0.30	0.28	0.27	0.26	0.25	0.24	0.23	0.18	—	—	—	—
0.20	0.50	0.46	0.43	0.40	0.38	0.36	0.34	0.33	0.31	0.30	0.24	—	—	—	—
0.25	0.63	0.58	0.54	0.50	0.47	0.45	0.43	0.41	0.39	0.38	0.30	—	—	—	—
0.30	0.75	0.70	0.64	0.60	0.57	0.54	0.51	0.49	0.47	0.46	0.36	—	—	—	—
0.35	0.88	0.81	0.75	0.70	0.66	0.63	0.60	0.57	0.55	0.53	0.42	—	—	0.39	0.35
0.40	1.00	0.93	0.86	0.80	0.76	0.72	0.68	0.66	0.63	0.61	0.48	—	—	0.44	0.40
0.45		1.04	0.96	0.90	0.85	0.81	0.77	0.74	0.71	0.68	0.54	0.77	0.54	0.50	0.45
0.50		1.16	1.07	1.01	0.95	0.90	0.86	0.82	0.79	0.76	0.60	0.85	0.60	0.55	0.50
0.55			1.18	1.11	1.04	0.98	0.94	0.90	0.86	0.84	0.66	0.94	0.66	0.61	0.55
0.60				1.21	1.13	1.07	1.03	0.98	0.94	0.91	0.72	1.02	0.72	0.66	0.60
0.65				1.31	1.23	1.16	1.11	1.07	1.02	0.99	0.78	1.11	0.78	0.72	0.65
0.70					1.32	1.25	1.20	1.15	1.10	1.06	0.84	1.19	0.84	0.77	0.70
0.75						1.34	1.28	1.23	1.18	1.14	0.90	1.28	0.90	0.83	0.75
0.80						1.43	1.37	1.31	1.26	1.22	0.96	1.36	0.96	0.88	0.80
0.85						1.52	1.45	1.39	1.33	1.29	1.02	1.45	1.02	0.94	0.85
0.90							1.54	1.48	1.41	1.37	1.08	1.53	1.08	0.99	0.90
0.95								1.56	1.49	1.44	1.14	1.62	1.14	1.05	0.95
1.00								1.64	1.57	1.52	1.20	1.70	1.20	1.10	1.00
1.05									1.65	1.60	1.26	1.79	1.26	1.16	1.05

Notes:

BTB is bituminous-treated base.
LTB is lime-treated base.
For the design of road-mixed asphalt surfacing, use 0.8 of the gravel equivalent factors (G_f) shown above the asphalt concrete.

$$G_f = 0.0032\ (TI)\ (100-R)$$

TI = Traffic Index

6.2 R Value

The relation for Hveem resistance is:

$$M_R = A + B \text{ (R-Value)}$$

For R-values equal to or less than 20 the recommended A-value is 1000 and the B value is 555. A may vary from 772 to 1155 while B varies over the range of 369 to 555 (1).

Figure 2-1.17 contains some commonly accepted correlations between other tests and the resilient modulus. It is expected that each state will develop individualized correlations specifically for their materials such as that done in Ohio which is shown in Figure 2-1.18.

7.0 EXAMPLE PROBLEMS

7.1 Soil Classification

Classify the following soils by the AASHTO classification procedure:

	Percent passing			Plasticity	
	#10	#40	#200	LL	PI
A -	37.1	21.1	8.6	12.3	NP
B -	100.0	96.3	75.6	33.7	8.9

Solutions:

Sample A - A-1a
Sample B - A-6

7.2 CBR Calculation

Calculate the CBR for this roadbed soil from the following data.

Penetration, inches	Load, Pounds
0.025	112
0.05	200
0.10	300
0.2	380
0.3	435
0.4	460
0.5	470

Solution:

The CBR for this soil is (300/3000)*100 = 10

7.3 Triaxial Test

Plot the Mohr circle data and determine the Angle of internal friction, and cohesion for this soil.

Confining stress (σ_3), psi	Vertical Stress at Failure (σ_1), psi
0	30
15	75

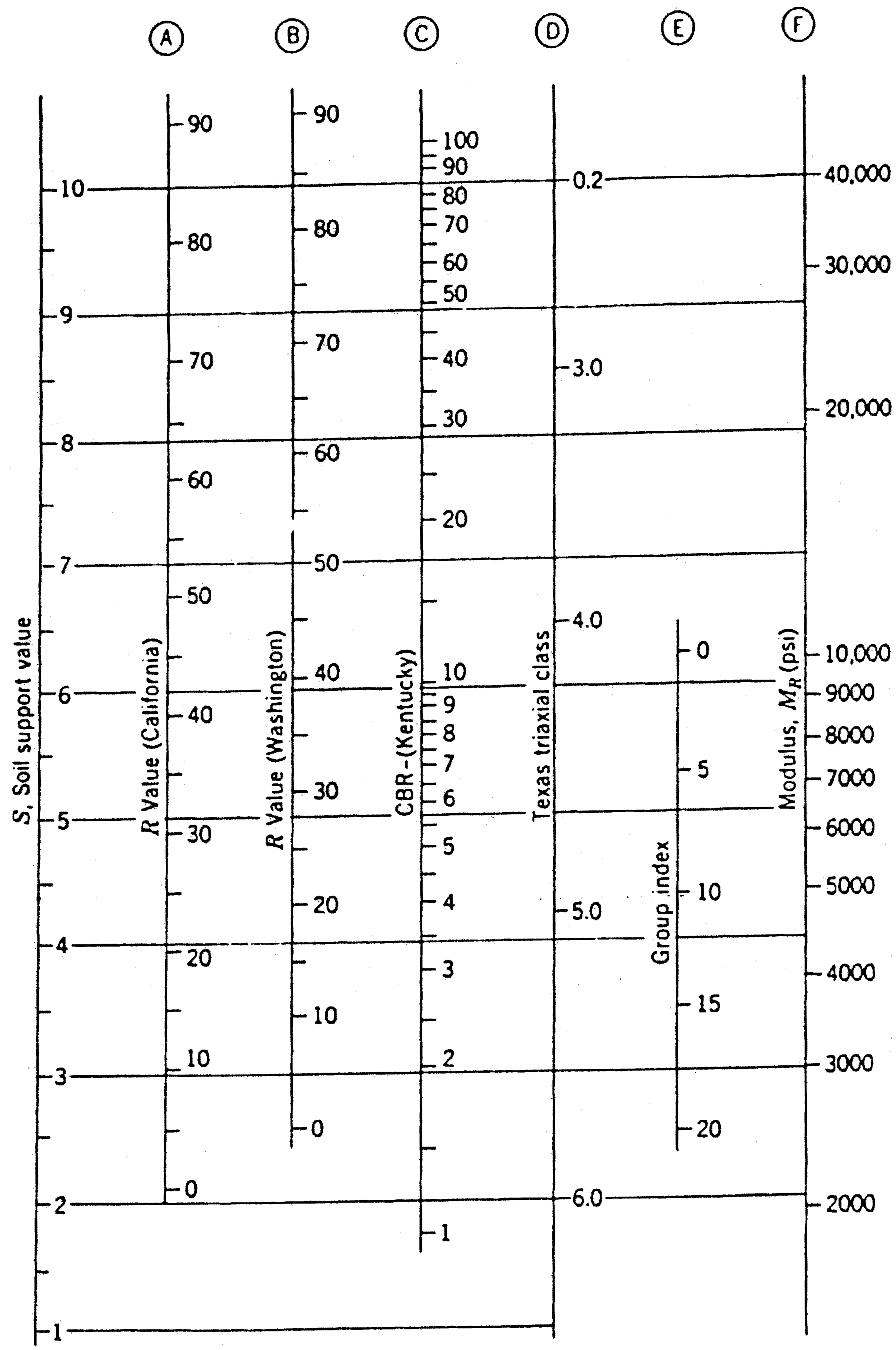
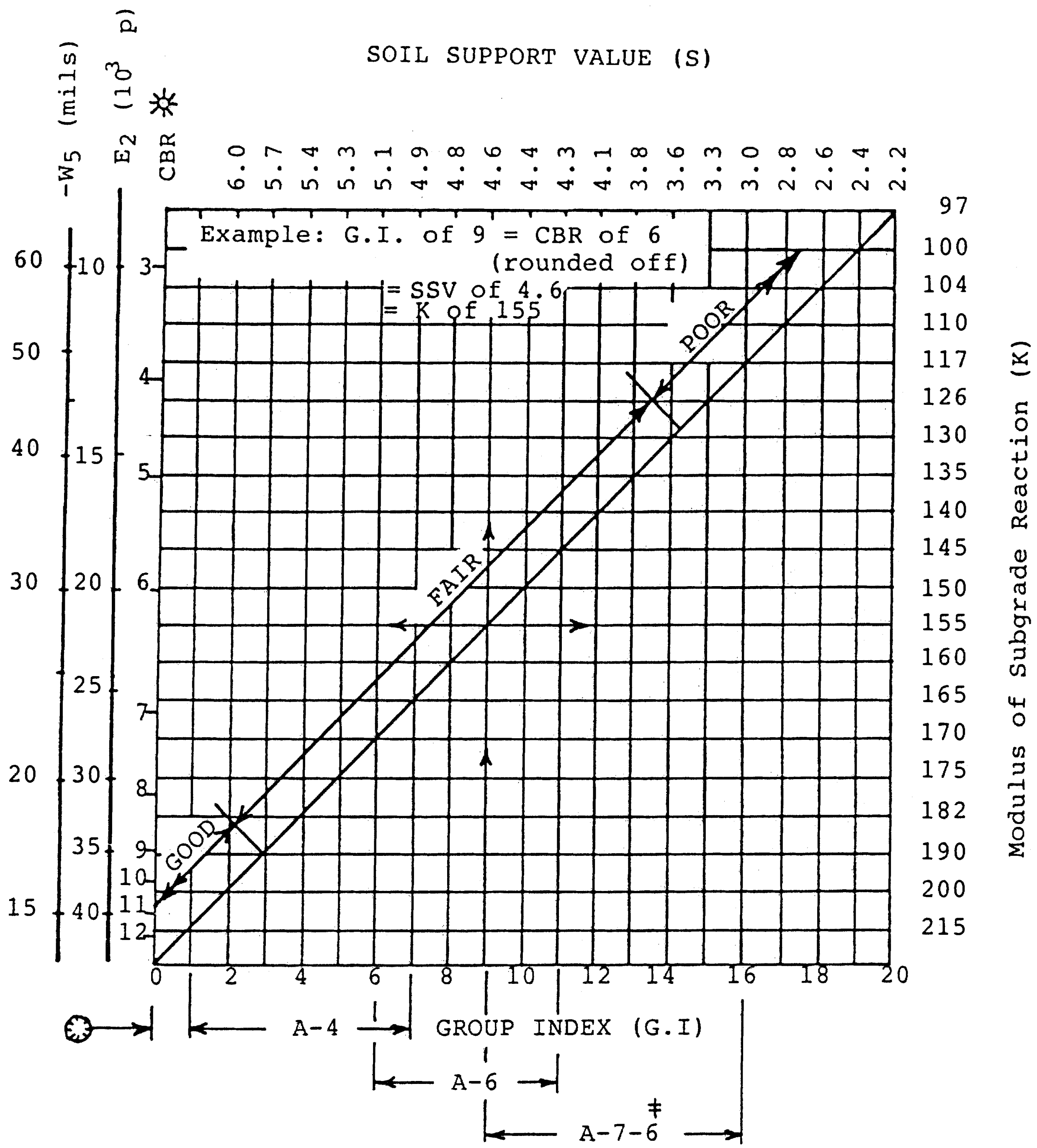


Figure 2-1.17. Graph for Converting Soil Support Values.



AASHTO Classes A-1, A-2 & A-3 lie below O. SSV-6-10; K=200+.

‡ Usual range of AASHTO Classes

5-1/2 lb. hammer, 12" drop, 4 layers, 45 blows per layer, compacted at optimum moisture as determined by AASHTO T-99.

Figure 2-1.18. Correlation of Subgrade Strengths, Ohio Soils.

Solution:

The Cohesion is approximately 9 psi, and the angle of internal friction is approximately 30 degrees.

7.4 Correlations

Find the value in Column A, given the value in Column B.

	A	B
1	SSV	M_R = 5,000 psi
2	M_R	CBR = 3.5
3	M_R	SSV = 5
4	M_R	Group Index = 10

Solution:

The SSV for No. 1 is approximately 6.3. The M_R for No. 2 is approximately 3,300 to 5250 psi depending on the formula or chart used. The M_R for No. 3 is 6,300 psi. The M_R for No. 4 is approximately 5,000 psi.

8.0 SUMMARY

This module presents the general physical and engineering properties of roadbed soils. Classification of soils is discussed to develop an understanding of the composition of a soil that can impact its performance in a pavement.

Correlations have been presented that show relationships between the old strength tests and the newer deformation test, the resilient modulus, M_R. The M_R test is required for use in the AASHTO design guide to represent roadbed soil structural adequacy. The need for accurate characterization of the M_R is demonstrated by the effect density, moisture, and environment have on the M_R value.

9.0 REFERENCES

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5. Robnett, Q. L., and M. R. Thompson, "Interim Report - Resilient Properties of Subgrade Soils - Phase I - Development of Testing Procedure," Civil Engineering Studies Transportation Engineering Series No. 5, Illinois Cooperative Highway Research Program, Series No. 139, University of Illinois at Urbana-Champaign, May, 1973.

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MODULE 2-2

RESILIENT MODULUS

1.0 INSTRUCTIONAL OBJECTIVES

This module presents the physical and engineering properties of roadbed soils as determined by the resilient modulus test for the AASHTO Design Procedure. The test procedure is described in detail to provide an understanding of the complexity of the test procedures and equipment required. Test results are discussed as they relate to pavement design procedures in the AASHTO Procedure. Emphasis is placed on interpreting test results and correlating the more traditional test results with the resilient modulus data for fine-grained soils and granular materials.

Upon completion of this module the participant will be able to accomplish the following:

1. List the steps required to perform the resilient modulus test for fine-grained and coarse grained materials.

2. Understand the concept behind resilient modulus testing and the importance of its inclusion in the AASHTO Design Guide and its relationship to Soil Support Value.

3. Describe the differences in the role of a roadbed soil on the performance of a flexible and rigid pavement.

4. Be able to correlate different tests and relate them to resilient modulus values.

5. Calculate resilient modulus values for use in pavement design from laboratory data.

2.0 INTRODUCTION

The resilient modulus test was developed to provide a material property that more accurately describes the behavior of the soil or other paving material under the effect of a moving wheel. The response to a wheel load is a deformation of the pavement whether the wheel load is moving or stationary. The magnitude of the deformation, however, is very different depending on the speed of the wheel load. The total deformation of the pavement is a summation of the individual deformations of the individual pavement layers. Traditional design procedures have been developed around laboratory tests which were static, and which were merely strength comparison tests which merely rank materials for their suitability of use. Very seldom do the materials in an actual pavement receive loads which approach failure. The performance of materials is very different at low load levels compared to high load levels.

Unlike the tests discussed in Module 2-1, which are static or slow, a moving wheel imparts a dynamic load pulse to all pavement layers and the subgrade as shown in Figure 2-2.1. The moving stress pulse builds from a low value when the wheel load is far from the point being investigated to a peak load over a time interval that is related to the speed of the vehicle. A

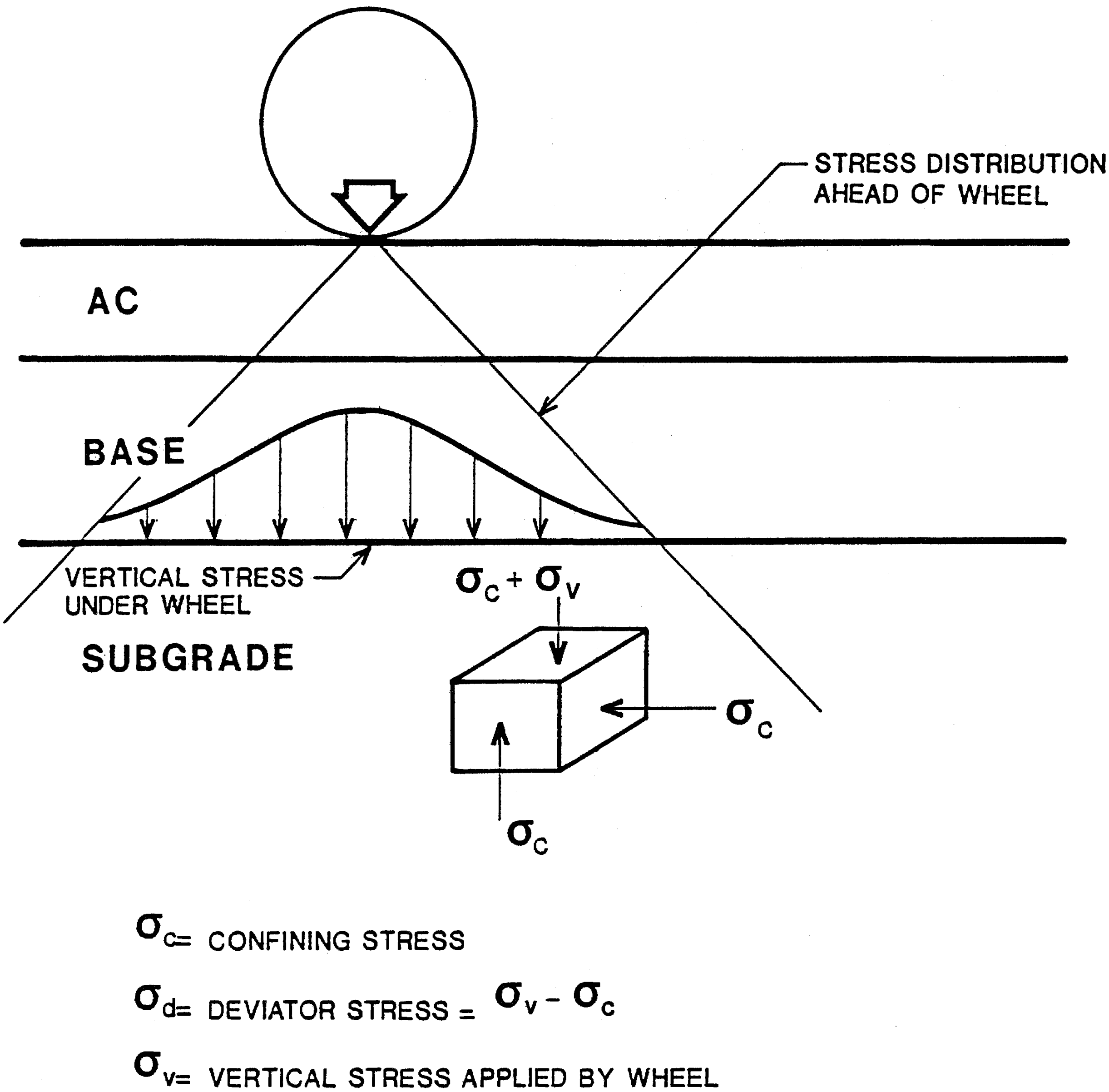


Figure 2-2.1. Stress Distribution in Pavement Layers
Beneath a Moving Wheel Load.

test procedure useful in pavement design should determine the properties of the paving material under the conditions that will actually be experienced in the field. This is the background for the development of the resilient modulus test for paving materials which will be discussed in this module.

3.0 RESILIENT MODULUS TEST

3.1 General

The resilient modulus test as adopted by AASHTO (AASHTO T274-82) is a modification of the triaxial test discussed in Module 2-1. The resilient modulus test is not a strength test, and samples are not failed during performance of the test. A cylindrical sample of soil or granular base is confined in a triaxial cell as shown in Figure 2-2.2 which allows varying confining pressures to be applied to the sample to model in-place characteristics of the pavement. A suitable loading system is used to apply a repeated load pulse of a fixed magnitude and fixed time duration to the cylindrical soil sample. The deformation of the sample is recorded for analysis.

3.2 Recorded Data

3.2.1 Loads

The load applied to the sample must be recorded for each test. The confining pressure can be recorded easily with a suitable pressure gage. The load applied to the sample must be monitored with an electronic load cell. Both values are required to calculate the stress parameters discussed in Module 2-1. For fine-grained roadbed soils, the deviator stress is the stress parameter which must be calculated, while the bulk stress is required for coarse-grained materials. The deviator stress is the numerical difference between the maximum vertical stress on the sample and the confining pressure. The bulk stress is the sum of the stresses and pressures on the sample. These are depicted in Figure 2-2.3. The equipment to record these values will be discussed in a subsequent section.

3.2.2 Deformation

The longitudinal deformation of the cylindrical sample is the response to the dynamic repeated loads, and is analogous to the deformation of the pavement layer under the dynamic wheel load. The general response to a dynamic load is shown in Figure 2-2.4. The response of the sample to a dynamic load is made up of several components as indicated in this figure. Each component provides for calculation of a parameter that can be used in design or performance prediction of the sample. The components are:

1. Total Deformation - Total deformation under the load, ϵ_t.
2. Resilient Deformation - Deformation recovered when the load is removed, ϵ_r.
3. Permanent Deformation - Deformation not recovered when the load is removed, ϵ_p.

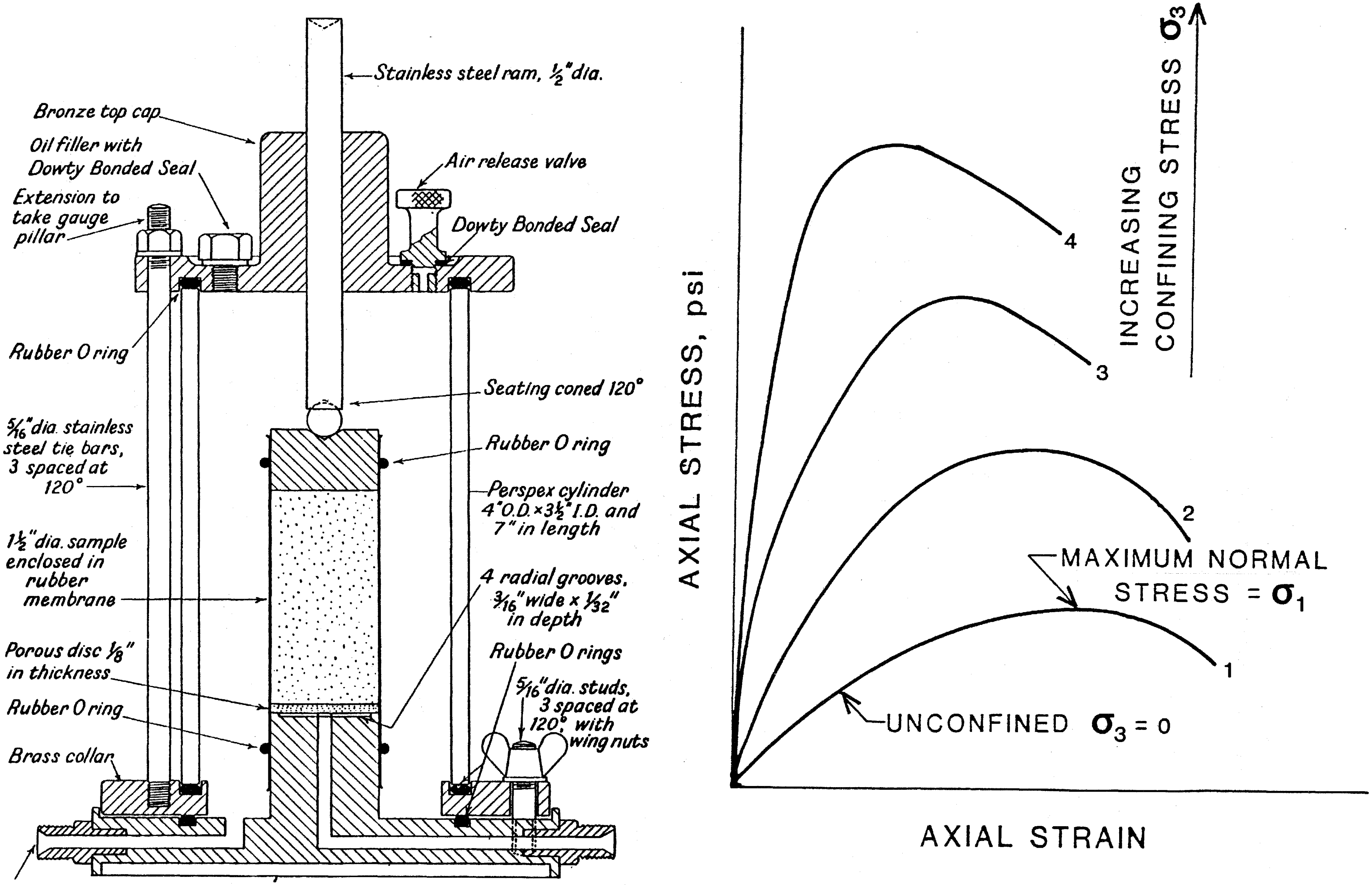


Figure 2-2.2. Triaxial Setup for Mohr Coulomb Development.

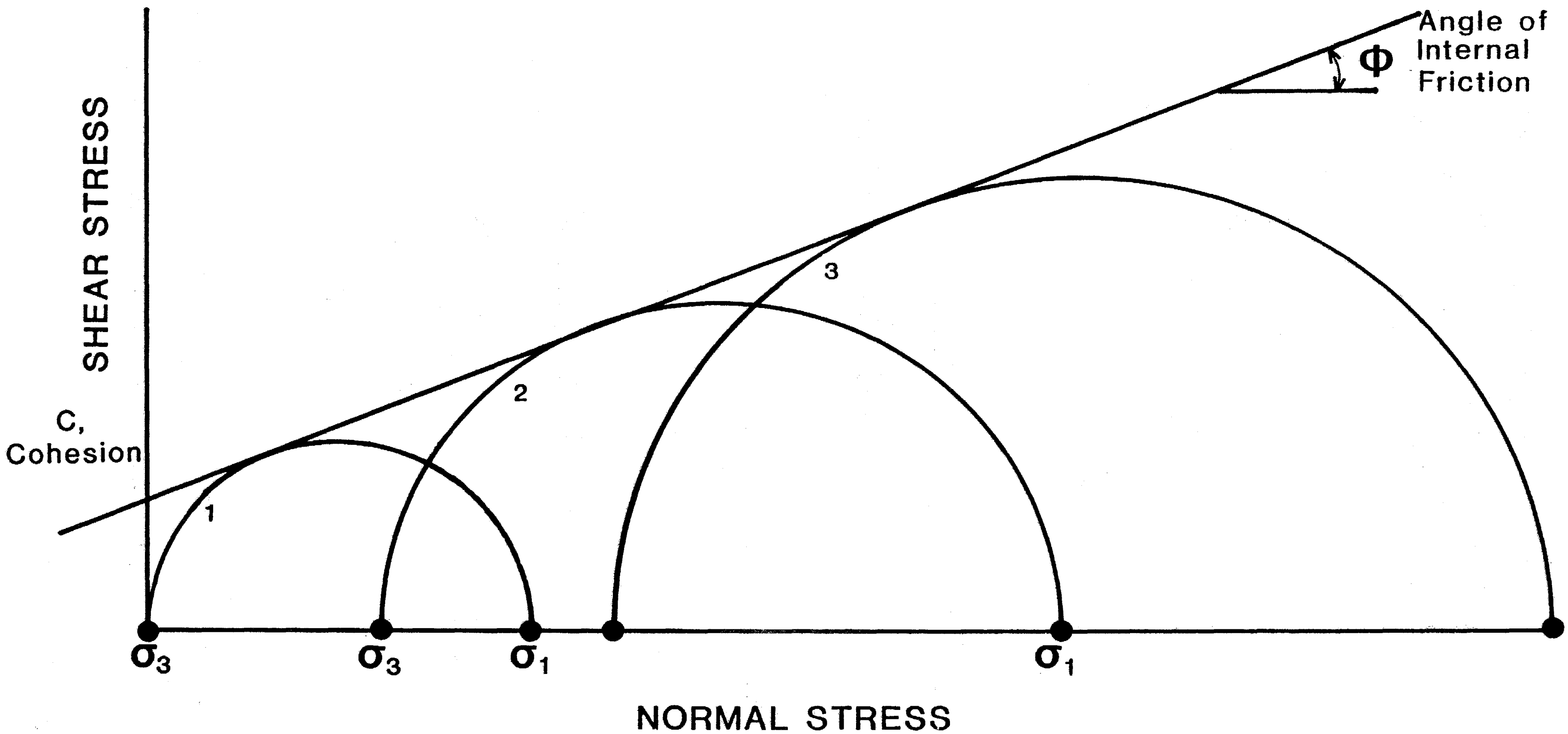
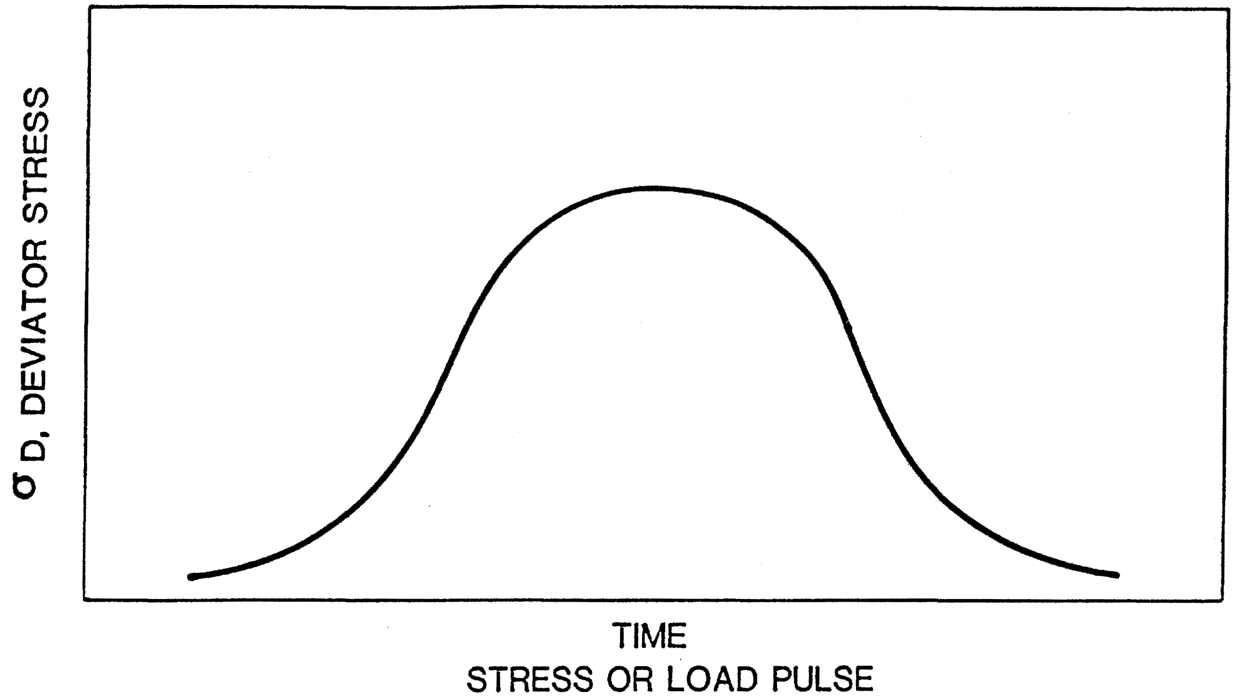


Figure 2-2.3. Mohr Conlomb Representation of Stresses on Samples in Compression Testing, Note: $\sigma_D = \sigma_1 - \sigma_3$.



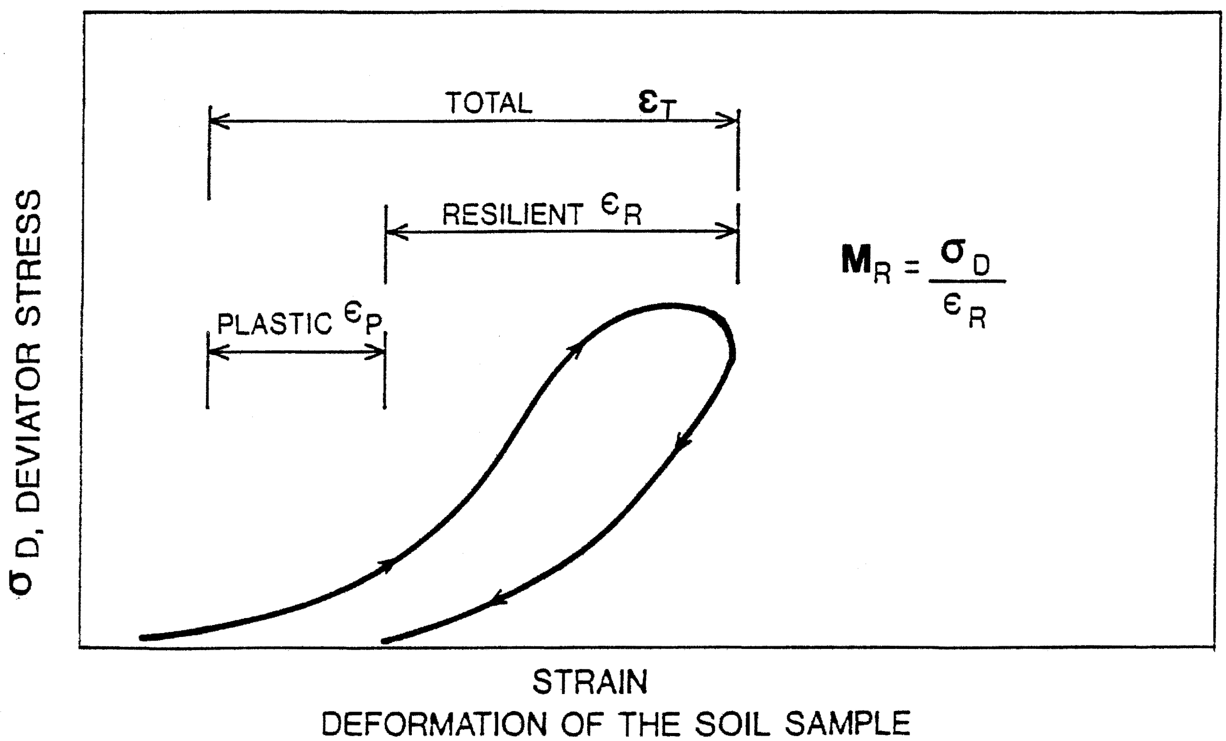


Figure 2-2.4. Response of Soil Sample in Resilient Modulus Test.

The deformations recorded for each test are converted to strains for use in the resilient modulus calculations. Strain is defined as the deformation resulting from a load divided by the original length of the specimen being deformed. For the cylindrical samples used in this test, the original length is the original height of the specimen.

The use of these data will be described in detail in a later section of this module.

3.3 Equipment

The equipment setup recommended in the AASHTO Test Method is shown in Figure 2-2.5. The basic components include:

1. Repetitive Loading device.

2. Triaxial cell and pressure control.

3. Electronics for data recording

These pieces of equipment can be very sophisticated or they can be very simple depending on the data desired from the test and the materials being tested.

3.3.1 Repetitive Load Device

The repetitive loading device can be an air actuated piston assembly with electronic solenoid control, or it can be a sophisticated hydraulic servo motor MTS type arrangement with precise control on the shape of the load pulse being used. It is not known yet whether the stress pulse is significant in modulus determination.

3.3.2 Pressure Control

Because the test will be conducted at several confining pressures for granular materials, the pressure control must be capable of maintaining a constant pressure on the sample. There are no specific requirements for this system.

3.3.3 Data Recording

The determination of a resilient modulus requires that the total axial deformation of the sample be recorded. Generally, this is done after a prescribed number of conditioning cycles. The equipment to record these deformations can be a simple digital meter such as a comparative peak reading voltmeter which holds the peak deformation for the load cycle and the low value after the load is released. More traditional devices are pen recorders which record the deformation during the entire load cycle for later analysis.

The test procedure lends itself to analysis of permanent deformation as well as resilient modulus. The analysis of permanent deformation requires continuous monitoring of the deformations, with complete traces of specified load cycles during the life of the test, which may be for several hundred thousand load cycles. Use of microcomputers for data collection will make

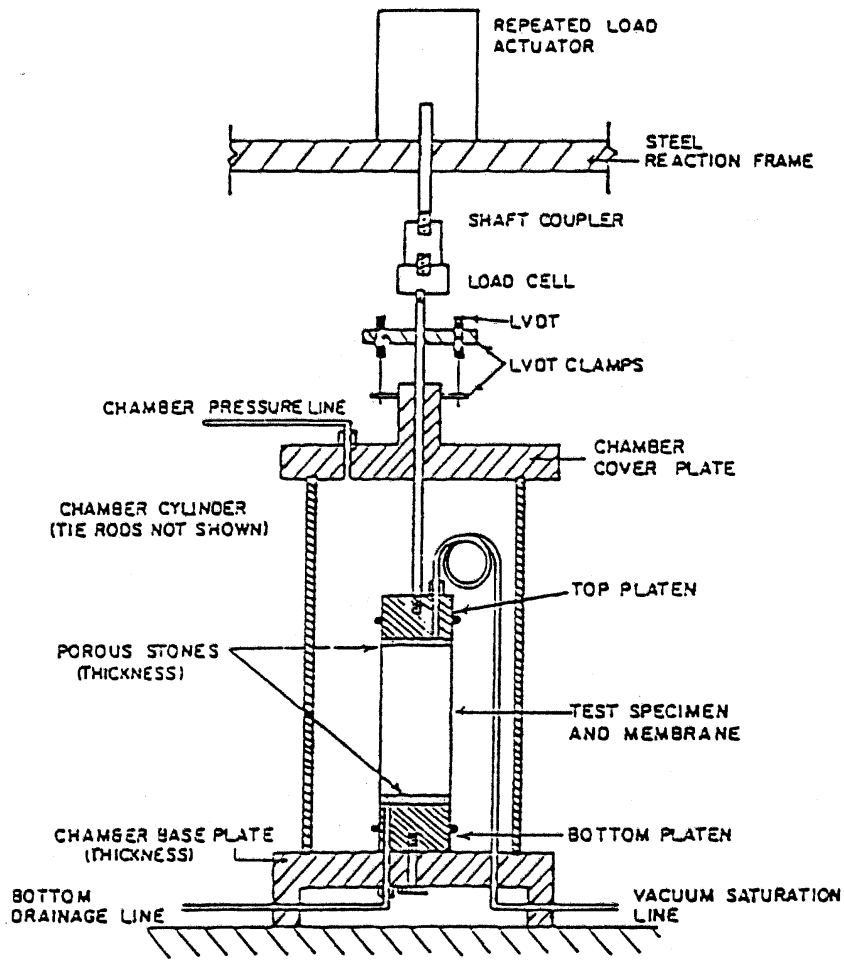


Figure 2-2.5. Subgrade Resilient Modulus Test Apparatus (AASHTO T274).

this process easier in the future by minimizing the different pieces of equipment required.

The deformations of the soil sample are measured by LVDT assemblies which clamp to the soil cylinder or loading ram as shown in Figure 2-2.6. An LVDT is essentially an electric coil which senses the position of a metal rod in the coil. As the soil sample deforms, the rod slides in the LVDT sending an electric signal to the amplifiers and recorders. Calibration of the voltage from the LVDT with known deflections is required to obtain the deformation of the sample.

3.4 Resilient Modulus Test

3.4.1 M_R Calculation

The deformation response to a load pulse was shown previously in Figure 2-2.4. The resilient modulus is calculated using the resilient strain from the test. The resilient strain is the recoverable strain from any load cycle. The resilient modulus is defined as:

$$M_R = \sigma_D / \epsilon_r$$

where:

σ_D = Repeated deviator stress.

ϵ_r = Resilient (recoverable) axial strain.

M_R = Resilient modulus, psi.

The resilient modulus is often termed the stiffness or elastic modulus of the soil. The test procedure for the resilient modulus determination is prescribed in AASHTO T274. The test procedure is designed to determine the "stress dependency" of the soil. For fine-grained cohesive soils, the resilient modulus decreases with increasing stress while granular materials will stiffen with increasing stress.

3.4.2 Fine-Grained Soils

Two basic stress dependent behavior models have been utilized for describing the stress softening behavior of fine-grained soils. One is the linear model, and the other is a semi-logarithmic presentation of the same date. While the arithmetic presentation of the data is most commonly used, different design programs may utilize one model or the other. The schematic variation of resilient modulus with deviator stresses is presented in Figure 2-2.7. The arithmetic model is demonstrated for some actual soils in Figures 2-2.8. The mathematical relationship for this stress dependency is:

$$M_R = K_1 \sigma_D^{-K_2}$$

where:

σ_D = Deviator stress = $\sigma_1 - \sigma_3$, psi
σ_1 = Major principal (vertical) stress, psi
σ_3 = Confining pressure, for unconfined compression test, $\sigma_3 = 0$
K_1, K_2 = Material constants

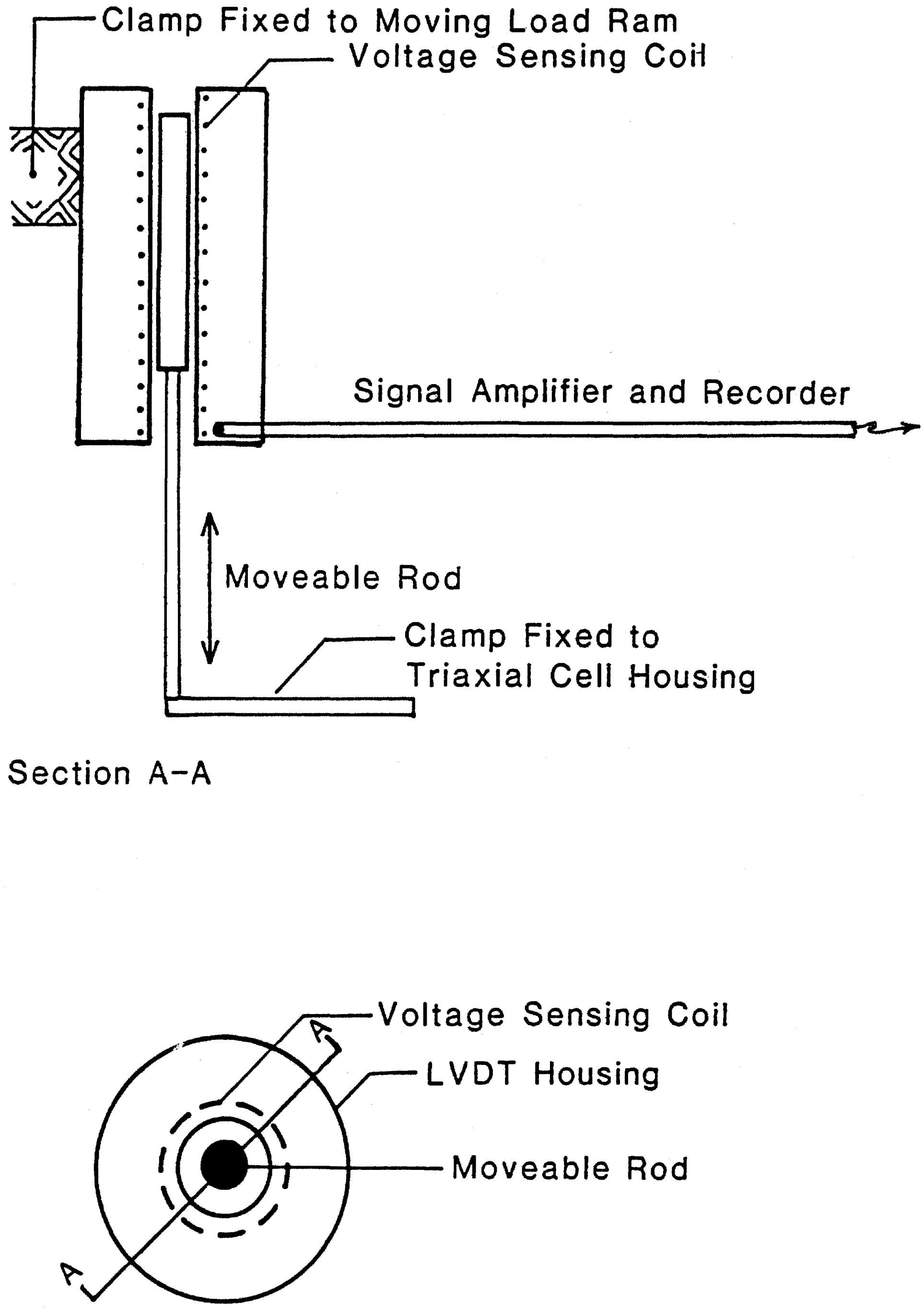


Figure 2-2.6. Schematic of LVDT Used to Measure Deformation in Resilient Modulus Test.

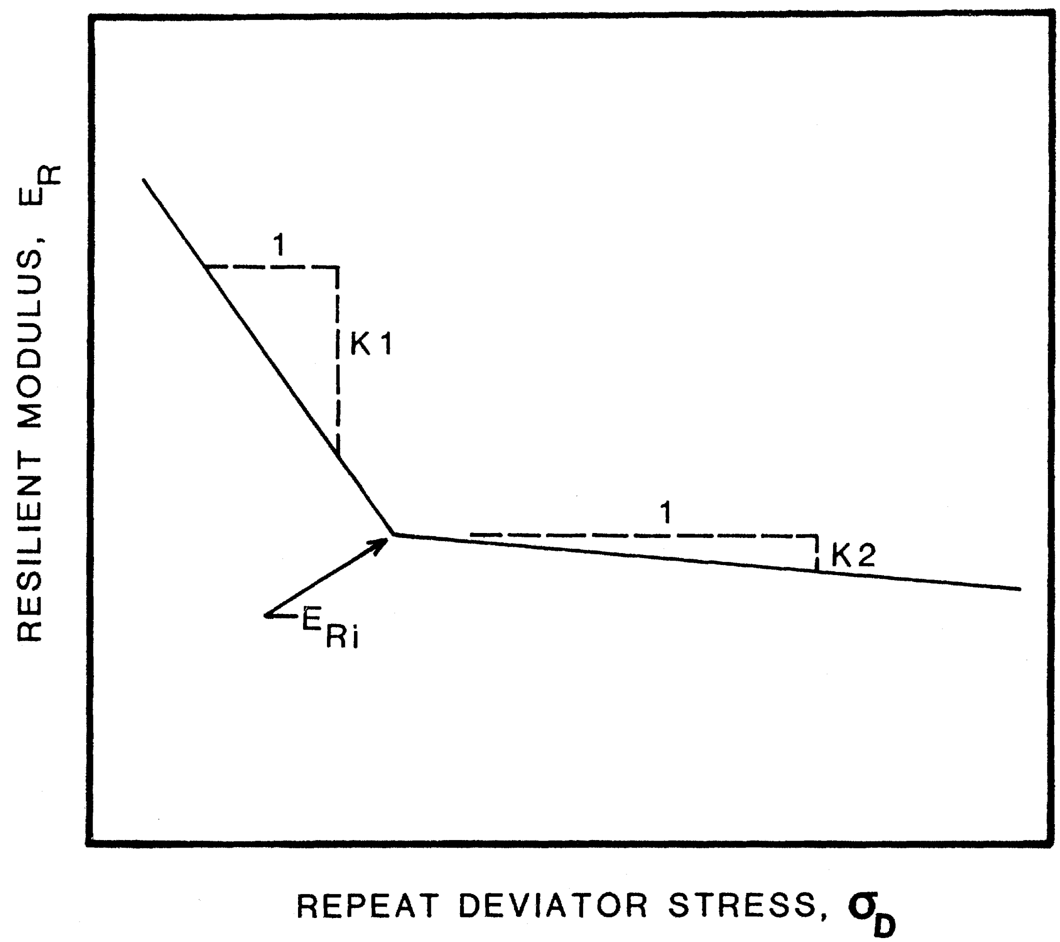


Figure 2-2.7. Idealized Resilient Modulus Curve for a Fine Grained Cohesive Soil.

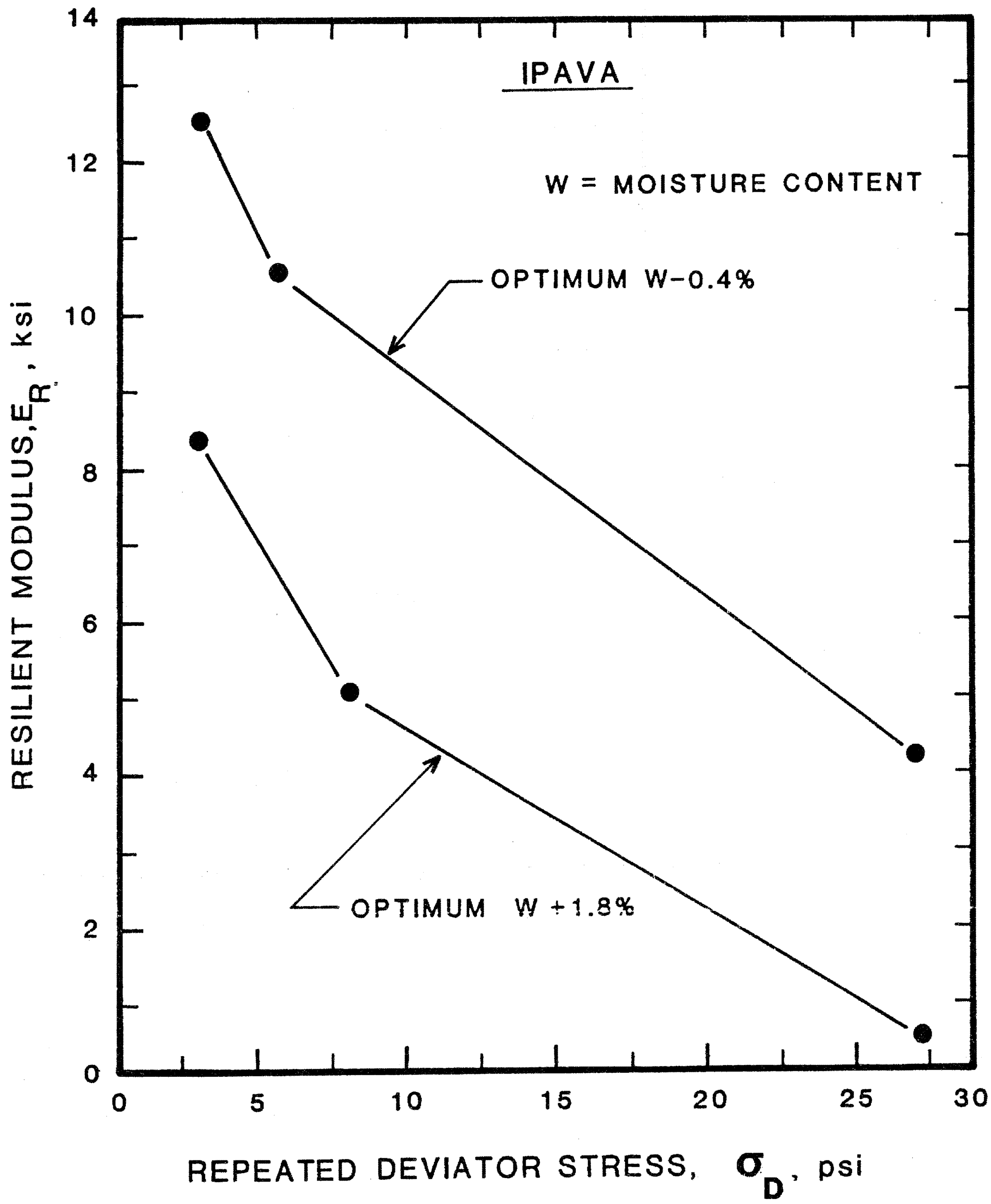


Figure 2-2.8. Reslient Modulus Curve Illustrating Stress Dependency and Impact of Moisture Variation.

In this arithmetic model, the value of the resilient modulus at the breakpoint in the bilinear curve as indicated by E_{Ri} in Figure 2-2.7 is a good indicator of a soil's resilient behavior. The slope values of K_1 and K_2 display less variability and influence pavement structural response to a smaller degree than E_{Ri}. Thompson and Robnett (99) have developed simplified procedures for estimating the resilient behavior of fine-grained soils based on soil classification, soil properties, and moisture contents.

3.4.3 Coarse-Grained Soils

The stress sensitivity of coarse-grained non-cohesive soils is opposite to that exhibited by the fine-grained roadbed materials. A typical resilient modulus curve is shown in Figure 2-2.9 for a coarse grained soil. The resilient modulus increases as the stress conditions increase. The model for this behavior is:

$$M_R = K_1(\theta_3)^{K_2}$$

where:

θ_3 = Bulk stress, or summation of the principal stresses.

K_1, K_2 = Material properties determined from the curve.

3.5 Permanent Deformation using Resilient Modulus Test Data

The test sequence for resilient modulus provides data with more applications than calculating the resilient modulus. Continual monitoring of the resilient modulus test over extended periods provides indications of the permanent deformation potential of the roadbed soil. Figure 2-2.10 shows the accumulation of permanent strain over one million load applications. Future mechanistic design procedures will use these relationships in their programs to predict the rutting in a flexible pavement. This testing will be required in the future.

4.0 SOIL PROPERTY INFLUENCES

The resilient modulus test is highly sensitive to soil properties and construction variables to a much greater extent than the strength tests discussed in Module 2-1. A significant benefit for the resilient modulus test is that it is a non destructive test that can be conducted on the same sample for several stress levels, minimizing preparation of different samples which may induce errors. The samples can be prepared to varying levels of moisture content and compaction allowing the test to model behavior of the roadbed soil in conditions which more closely model those which will be found in the actual pavement.

4.1 Moisture

Figure 2-2.11 shows the relationship between E_{Ri} for Illinois soils and saturation. The modulus of the soils decreases substantially when saturation increases, as would be expected. This points out the adjustments required to modulus when the pavement becomes saturated during certain periods of the year. Figure 2-2.8 showed the effect of varying the water content at time of compaction above and below optimum on resilient modulus,

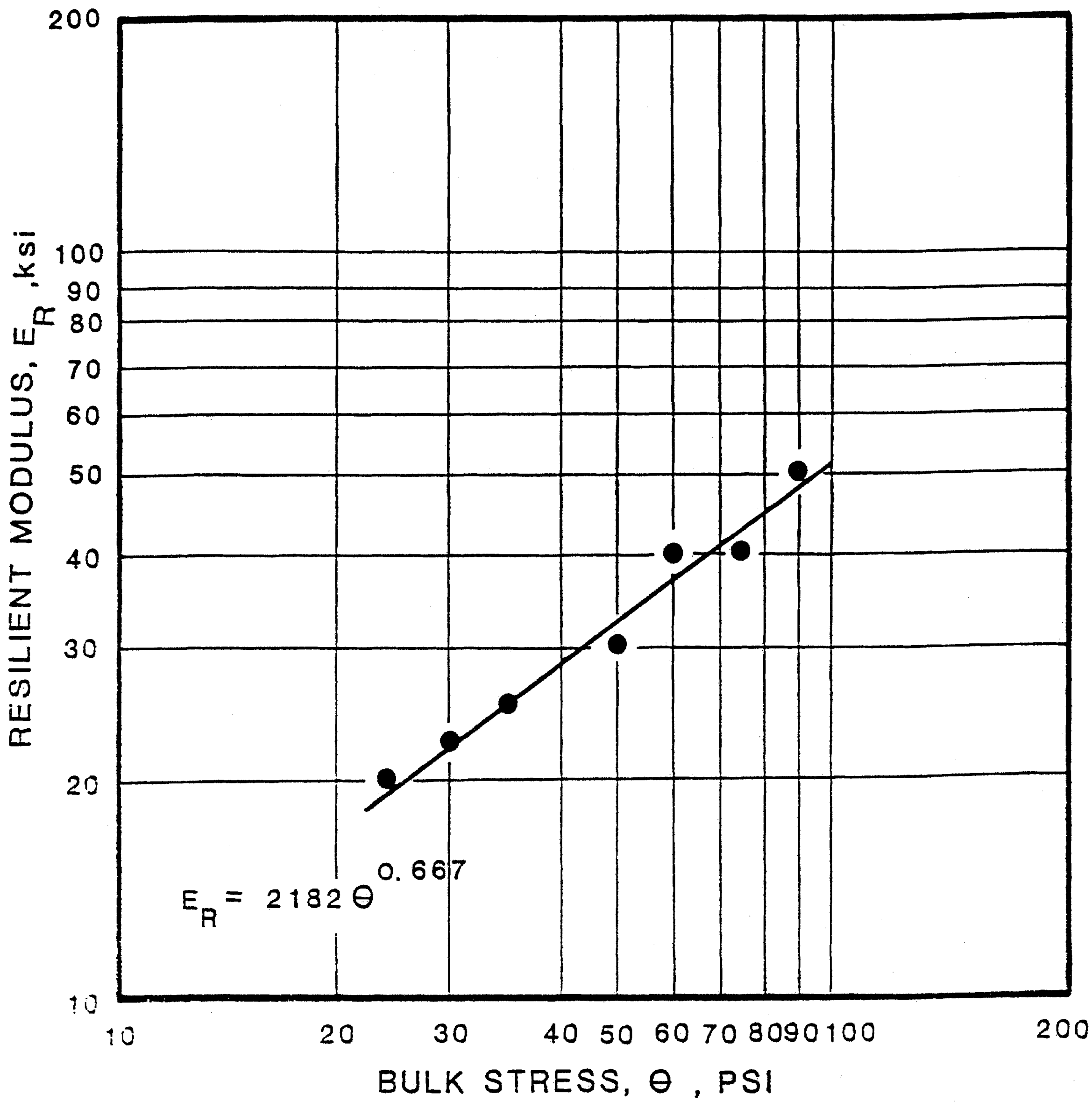


Figure 2-2.9. Resilient Modulus Relation for a Sandy Gravel (AASHTO A-1-b(0)).

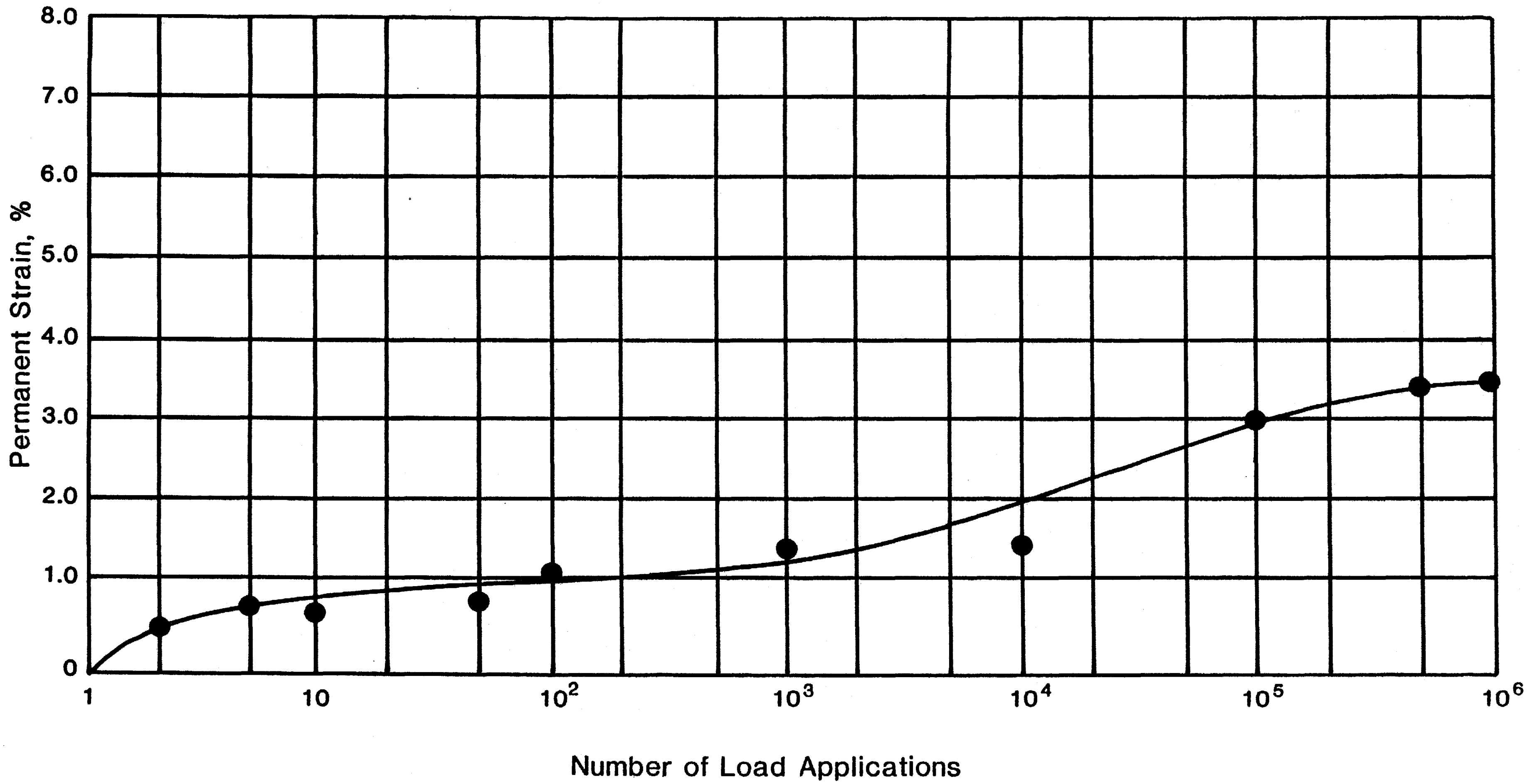


Figure 2-2.10. Accumulation of Permanent Strain in a Resilient Modulus Test on AASHTO A-1-b(o) Soil.

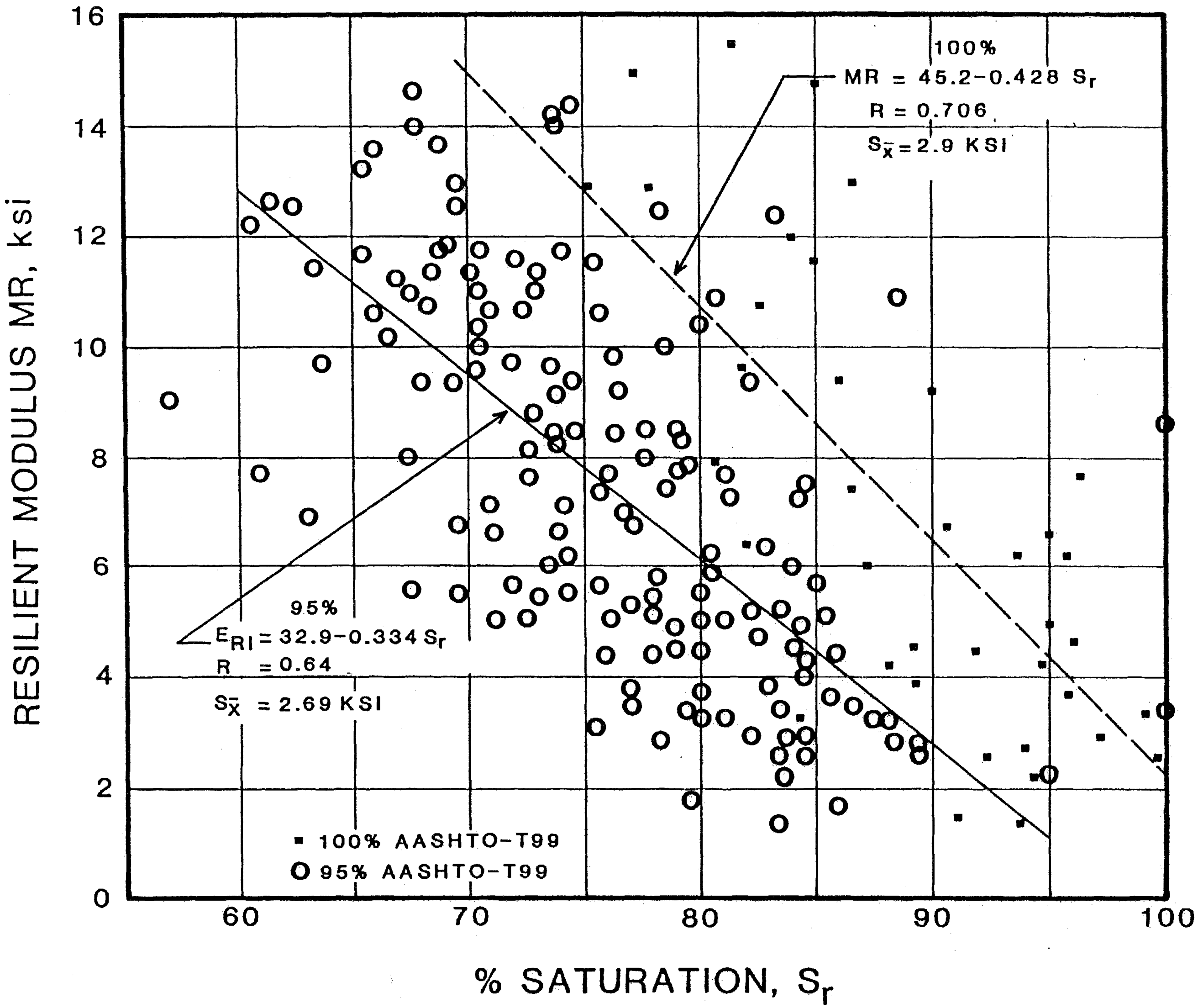


Figure 2-2.11. Influence of Moisture and Density on Resilient Modulus.

indicating the control that should be exercised during construction. The impact of different resilient modulus values on thickness design will be shown more clearly in Blocks 3 and 4.

Figure 2-2.12 shows a similar relationship developed for soils in Louisiana. These curves are for several different soils at different moisture contents. The trend is obvious, and points out the need to establish field conditions to be used in design and to control these field values very closely.

4.2 Density

The level of compaction also influences the resulting resilient modulus as shown previously in Figure 2-2.11. Here, the only difference is in the percent of compaction at the level of AASHTO-T99. A five percent difference can produce a drop in modulus of over 40 percent. This much variability in an actual pavement structure would cause concern in the design reliability. These items will be discussed in a later module.

4.3 Environment

The environment plays an important role in establishing the resilient modulus beyond the moisture influence. Temperature cycling can alter the modulus Freezing produces the changes shown in Figure 2-1.13 (6). The stabilizing effect of lime on the environmental deterioration is also shown in this figure for the same soil. The decrease in modulus caused by even one freeze-thaw cycle can have a tremendous impact on the thickness design for that soil.

4.4 Typical Values

It is very difficult to assign typical values of resilient modulus to roadbed soils. This value is affected not only be construction variables, but also by soil type, fines, clay content, and size of fine particles. Each state must conduct research to validate any relationships for typical values which they will use in design. Figure 2-2.14 contains effective resilient modulus values for low-volume roads. The values vary with estimated drainage quality and climatic region.

Recent studies by Thompson and Robnett developed relationships between the E_{Ri} at a deviator stress of 6 psi and the volumetric moisture content, w, (volume of water/volume of soil). They determined:

1. For dry density less than 100 pcf.
   $$E_{Ri} = 27.06 - 0.526(w)$$

2. For dry density greater than 100 pcf.
   $$E_{Ri} = 18.18 - 0.404(w)$$

In these relationships the volumetric water content, w, is input in percentage form, and the resilient modulus at a repeated deviator stress of 6 psi is calculated in ksi. These equations were developed from extensive testing of roadbed soils throughout the state of Illinois.

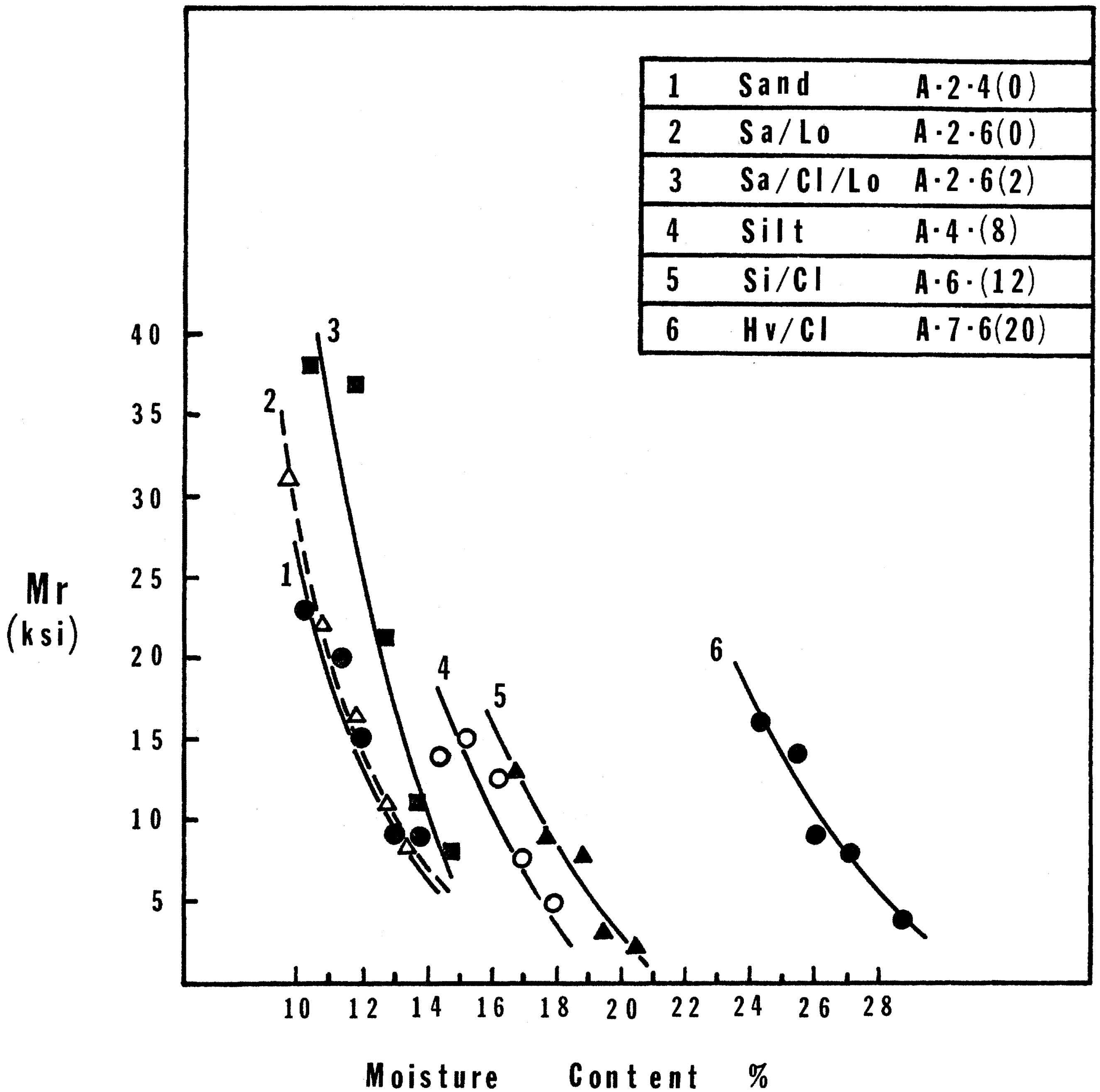


Figure 2-2.12. Variation in Resilient Modulus with Moisture Content for Various Soils.

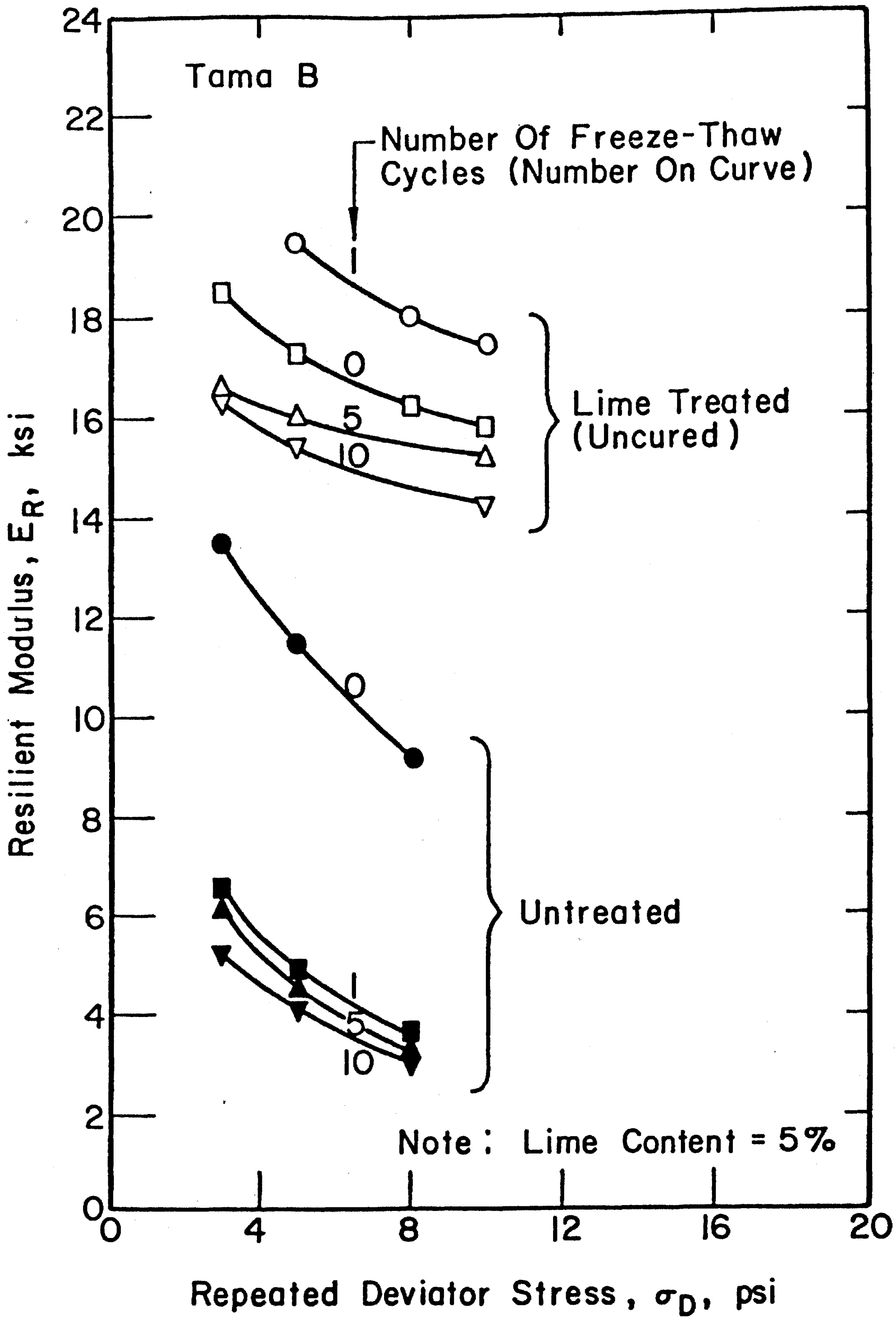


Figure 2-2.13. Effect of Freeze-Thaw Temperature Cycling on Resilient Modulus, (6).

U.S. Climatic Region	Relative Quality of Roadbed Soil				
	Very Poor	Poor	Fair	Good	Very Good
I	2,800*	3,700	5,000	6,800	9,500
II	2,700	3,400	4,500	5,500	7,300
III	2,700	3,000	4,000	4,400	5,700
IV	3,200	4,100	5,600	7,900	11,700
V	3,100	3,700	5,000	6,000	8,200
VI	2,800	3,100	4,100	4,500	5,700

*Effective Resilient Modulus in psi

Figure 2-2.14. Effective Roadbed Soil Resilient Modulus Values, M_R (psi), that may be Used in the Design of Flexible Pavements for Low-Volume Roads. Suggested Values Depend on the U.S. Climatic Region and the Relative Quality of the Roadbed Soil.

4.5 Permanent Deformation

The same factors which affect the resilient modulus also affect the permanent deformation behavior of the soil. Figure 2-2.15 shows the influence of moisture on permanent deformation with a four percent increase above optimum leading to failure Figure 2-2.16 shows the influence of percent compaction on permanent deformation. Figure 2-2.17 shows the impact of load level or stress level on permanent deformation. The influence of moisture content is more pronounced for the permanent deformation than it is for resilient modulus, which may indicate a design consideration.

5.0 RESILIENT MODULUS IN DESIGN

5.1 Use in Design

The resilient modulus of the roadbed soil is a direct input for mechanistic programs which use elastic layer theory as well as for the AASHTO Design Guide, 1986. The modulus of the roadbed soil exerts an extremely strong influence on the structural requirements of layers placed over the roadbed and hence the overall performance of the pavement.

The new AASHTO design guide replaces the old Soil Support Value (S) with the resilient modulus (M_R). The M_R value in the AASHTO design procedure is the average value for the roadbed soil. This average is the average of all tests along the route of the pavement, and points out the importance of testing samples at the density and moisture content they will develop in service. This average value is further adjusted in the design process for seasonal variability. The use of extreme conditions to represent minimum expected values must be judiciously avoided.

5.1.1 Effective Roadbed Soil Resilient Modulus

The AASHTO flexible pavement design procedure requires the input of an effective roadbed soil resilient modulus, which accounts for the combined effect of all seasonal modulus values. The computation of the effective modulus is described below. This method should be used only for estimating the modulus of soils under flexible pavements that are to be designed using serviceability criteria.

Seasonal resilient modulus values must be determined to quantify the relative damage a pavement is subjected to during each season of the year and include this damage in the overall design. These values can be estimated in any of the following ways:

1. Perform laboratory resilient modulus tests (AASHTO T274) on representative soil samples in stress and moisture conditions simulating those of the primary moisture seasons (i.e., those seasons during which a <u>significantly</u> different resilient modulus will be obtained). This will establish a laboratory relationship between resilient modulus and moisture content which can be used with estimates of in-situ moisture content of the soil beneath the slab during various seasons to generate resilient modulus values for those seasons.

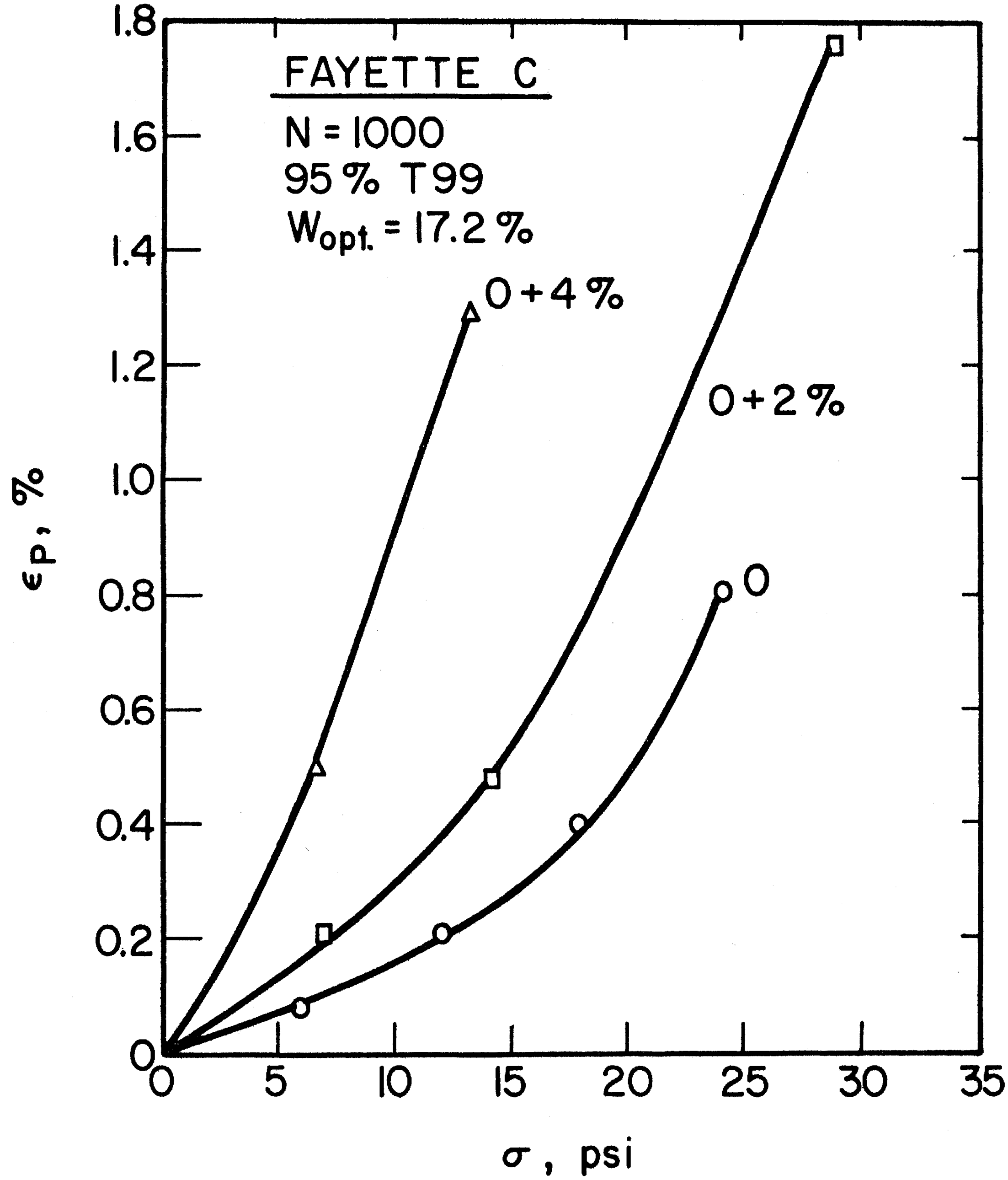


Figure 2-2.15. Change in Permanent Strain Under Repeated Loading With a Change in Compaction Moisture (6).

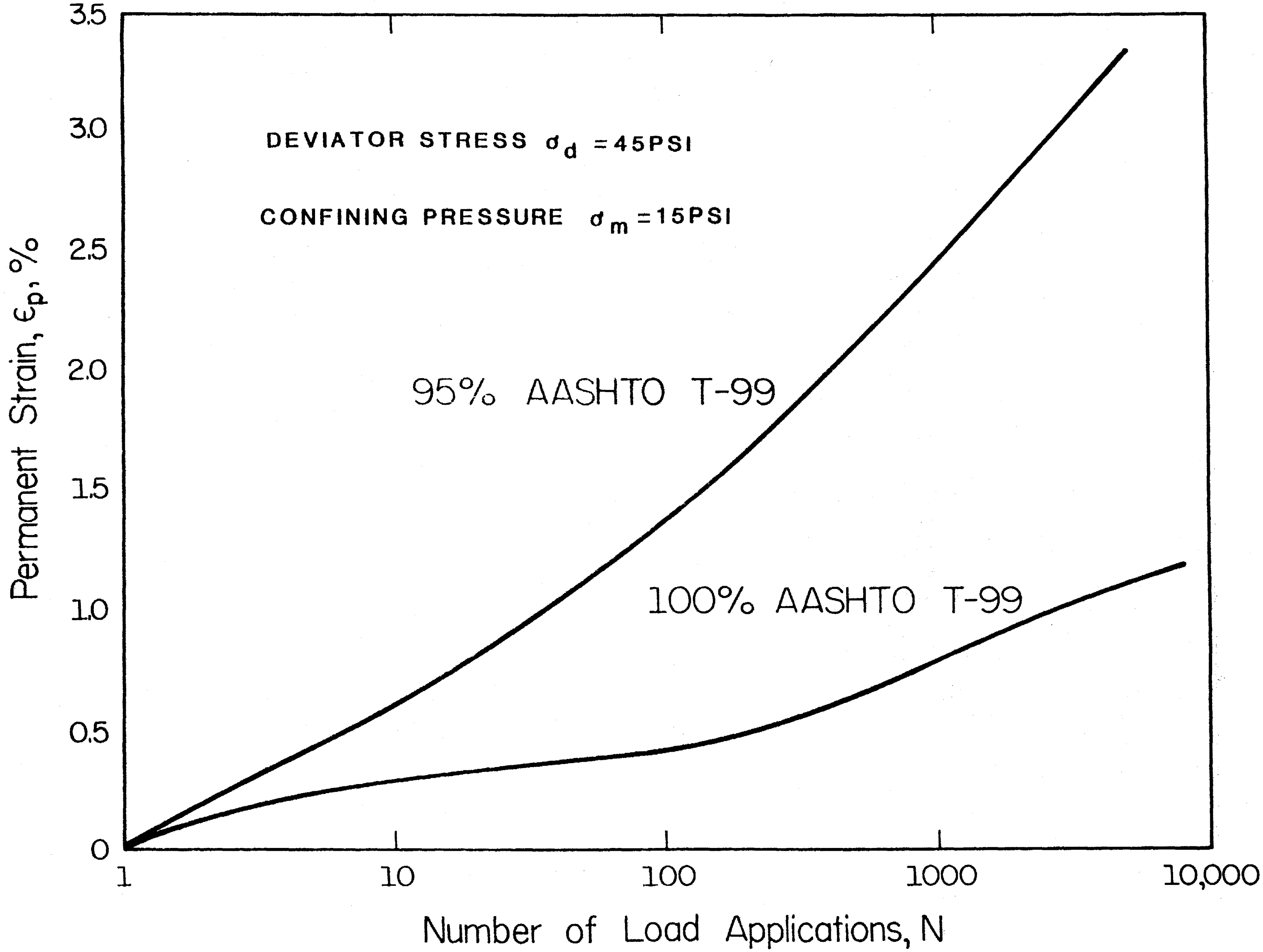


Figure 2-2.16. Permanent Deformation as a Function of Load Applications for Two Compaction Efforts.

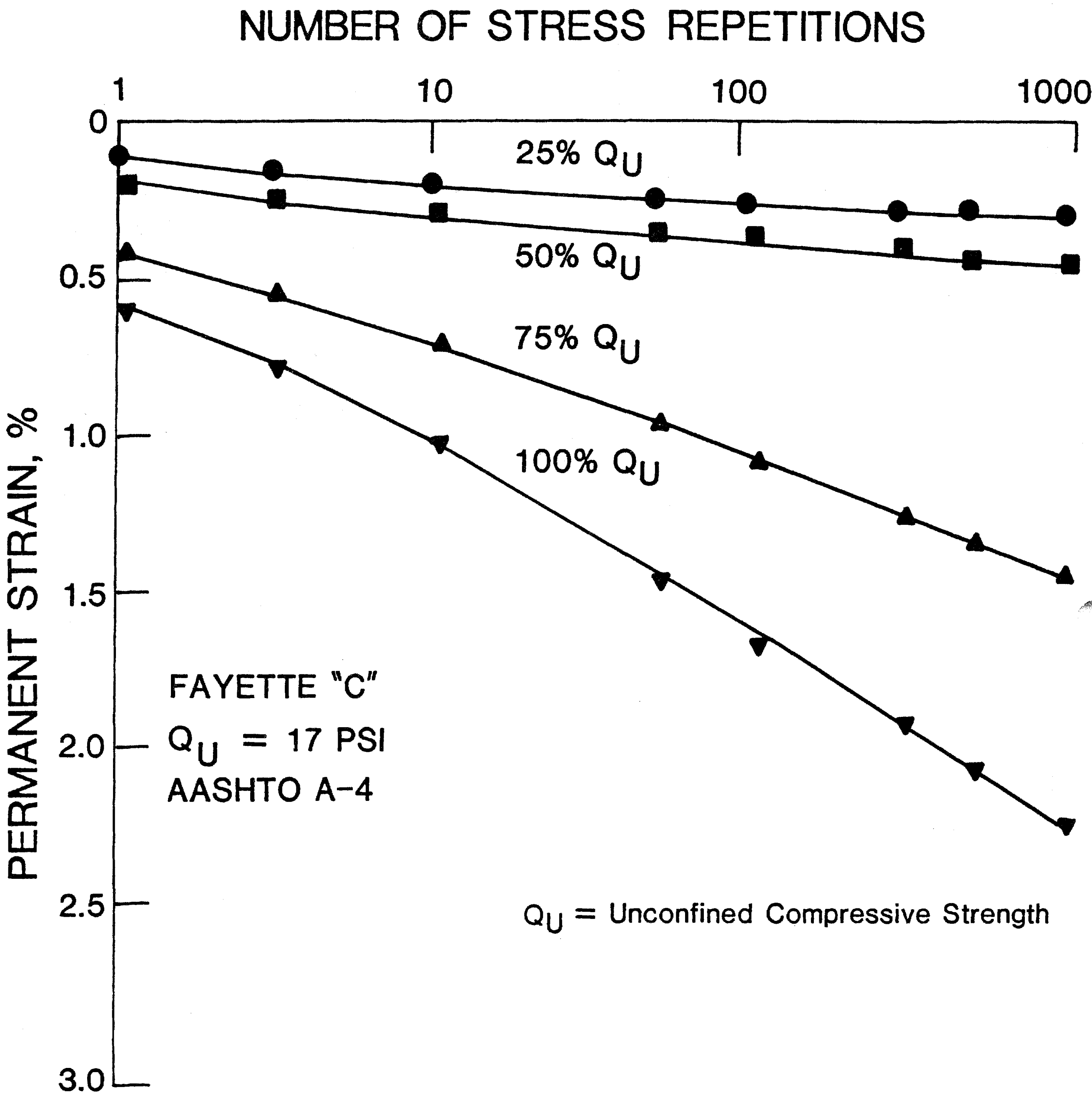


Figure 2-2.17. Accumulation of Permanent Strain at Various Stress Levels in the Resilient Modulus Test on an A-4(9) Soil.

An alternative is to back-calculate the resilient modulus for different seasons using deflections measured on in-service pavements.

Estimate "normal" or summer resilient modulus values from known relationships between resilient modulus and known soil properties (e.g., clay content, plasticity index, etc.) and use empirical relationships to estimate seasonal variations (CMS). The spring thaw modulus is typically 10 to 20 percent of "normal" or summer modulus. The frozen subgrade modulus is typically two orders of magnitude greater than the normal modulus. The "recovery" time for the modulus to increase from the thaw value to 80 percent of the normal value is typically 35 to 65 days.

2. Separate the year into time intervals during which the different seasonal moduli are effective. All of the "seasons" must be definable in terms of the selected time interval. It is suggested that the one-half month should be the shortest time interval used. Figure 2-2.18 presents a chart for estimating effective roadbed soil resilient modulus that provides for entry of seasonal roadbed soil moduli at half-month intervals.

3. The relative damage value (u_f) corresponding to each seasonal modulus must be estimated using the vertical scale or corresponding equation shown at the right of Figure 2-2.18. For example, the relative damage corresponding to a roadbed soil resilient modulus of 4000 psi is 0.51. Each damage value is entered in the appropriate box adjacent to the corresponding resilient modulus.

4. The relative damage values should all be added together and divided by the number of seasonal increments (in this case, 24) to determine the average relative damage.

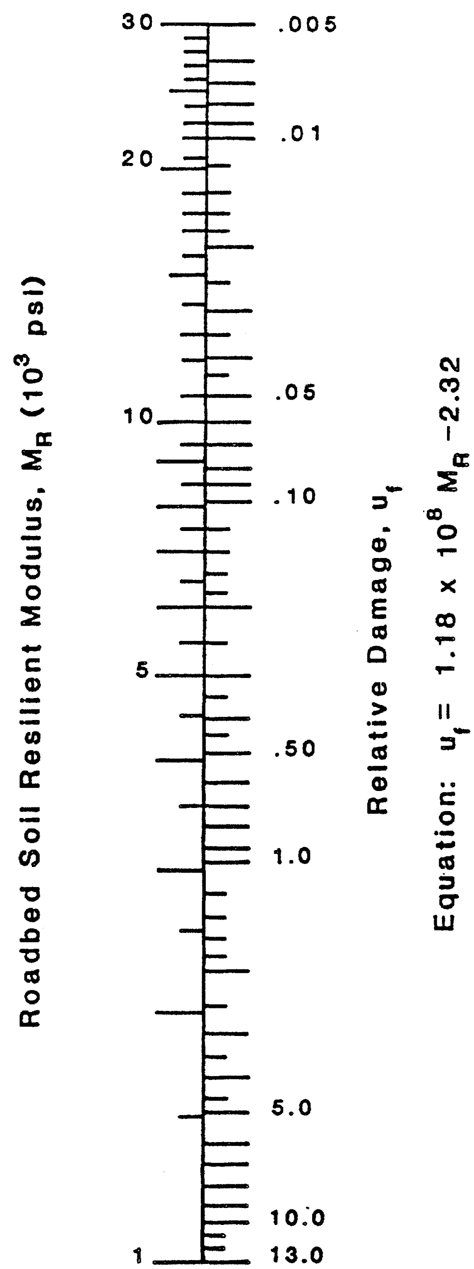
5. The effective roadbed soil resilient modulus (M_R) is estimated as the value corresponding to the average relative damage on the M_R - u_f scale.

If the procedures described above cannot be accomplished, Figure 2-2.14 and Figure 2-2.19 provide guidelines (intended for use on low-volume roads) for assigning effective roadbed soil resilient modulus values based on climate zone and relative quality of subgrade soil.

6.0 CORRELATION CONSIDERATIONS FOR MODULUS OF RESILIENCE

With the introduction of the resilient modulus value into the AASHTO design process it will become necessary for all states to conduct appropriate testing to develop resilient modulus values for the soils in their state. It is realized that not all states will have the necessary equipment to develop the M_R data required for immediate use in design projects. Therefore, some general correlations will have to be used before specific data are developed for the state.

Month	Roadbed Soil Modulus, M_R (psi)	Relative Damage, u_f
Jan.		
Feb.		
Mar.		
Apr.		
May		
June		
July		
Aug.		
Sept.		
Oct.		
Nov.		
Dec.		
Summation: $\sum u_f =$		



Average: $\bar{u}_f = \dfrac{\sum u_f}{n} =$ ______

Effective Roadbed Soil Resilient Modulus,

M_R (psi) = ______ (corresponds to $\bar{u}_f$)

Figure 2-2.18. Chart for Estimating Effective Roadbed Soil Resilient Modulus.

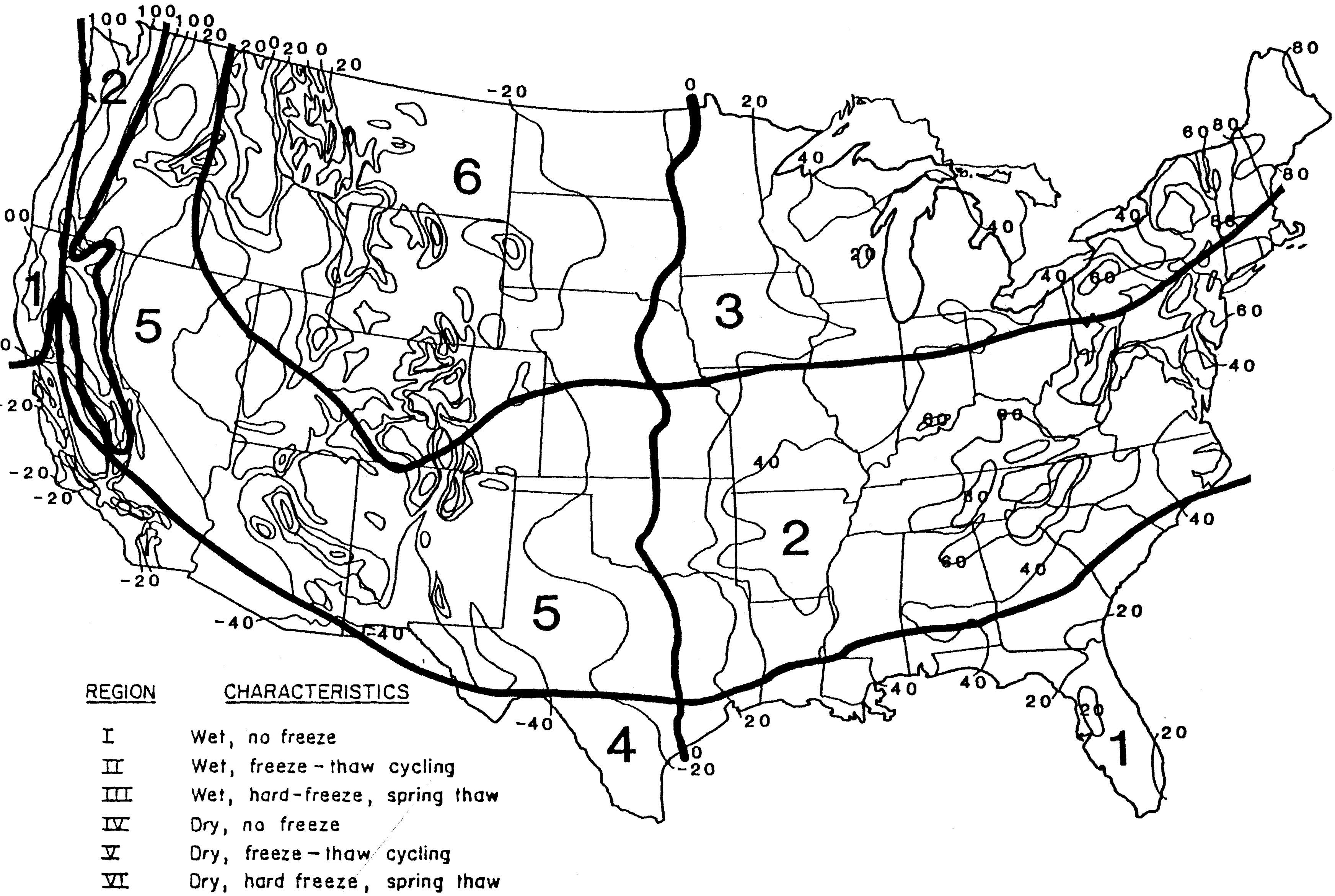


Figure 2-2.19. Six Climatic Regions in the United States for Pavement Design.

6.1 California Bearing Ratio

The M_R values for soils must be developed to take full advantage of the AASHTO guide as well as the mechanistic procedures which rely solely on the resilient modulus. In the same manner that the old Soil Support Value was related to different soil tests, until these precise correlations are developed some standard correlations can be used.

An accepted approximate correlation is:

$$M_R = B * CBR$$

For soils with a CBR equal to or less than 10 the value of B is 1500 although the value may vary from 750 to 3000.

6.2 R Value

The relation for Hveem resistance is:

$$M_R = A + B \text{ (R-Value)}$$

For R-values equal to or less than 20 the recommended A-value is 1000 and the B value is 555. A may vary from 772 to 1155 while B varies over the range of 369 to 555 (1).

Figure 2-2.20 contains some commonly accepted correlations between other tests and the resilient modulus. It is expected that each state will develop individualized correlations specifically for their materials such as that done in Ohio which is shown in Figure 2-2.21. Results assembled from several studies are shown in Figure 2-2.22. These correlations must be applied judicously as the resilient modulus is highly influenced by soil properties, and general correlations suffer from the inability to compensate for these property variations.

7.0 SAMPLE PROBLEMS

7.1 Calculate and plot the resilient modulus, M_R, in Figure 2-2.23 and give the E_{Ri} for the soil from the given data.

Deviator Stress, psi	Strain, in/in
3	0.000231
5	0.000833
8	0.00167
15	0.005

Solution, E_{Ri} is approximately 6,000 psi.

7.2 Figure 2-2.24 contains estimated resilient modulus values for reach of the 12 months. Calculate the effective roadbed resilient modulus which will be used in a design.

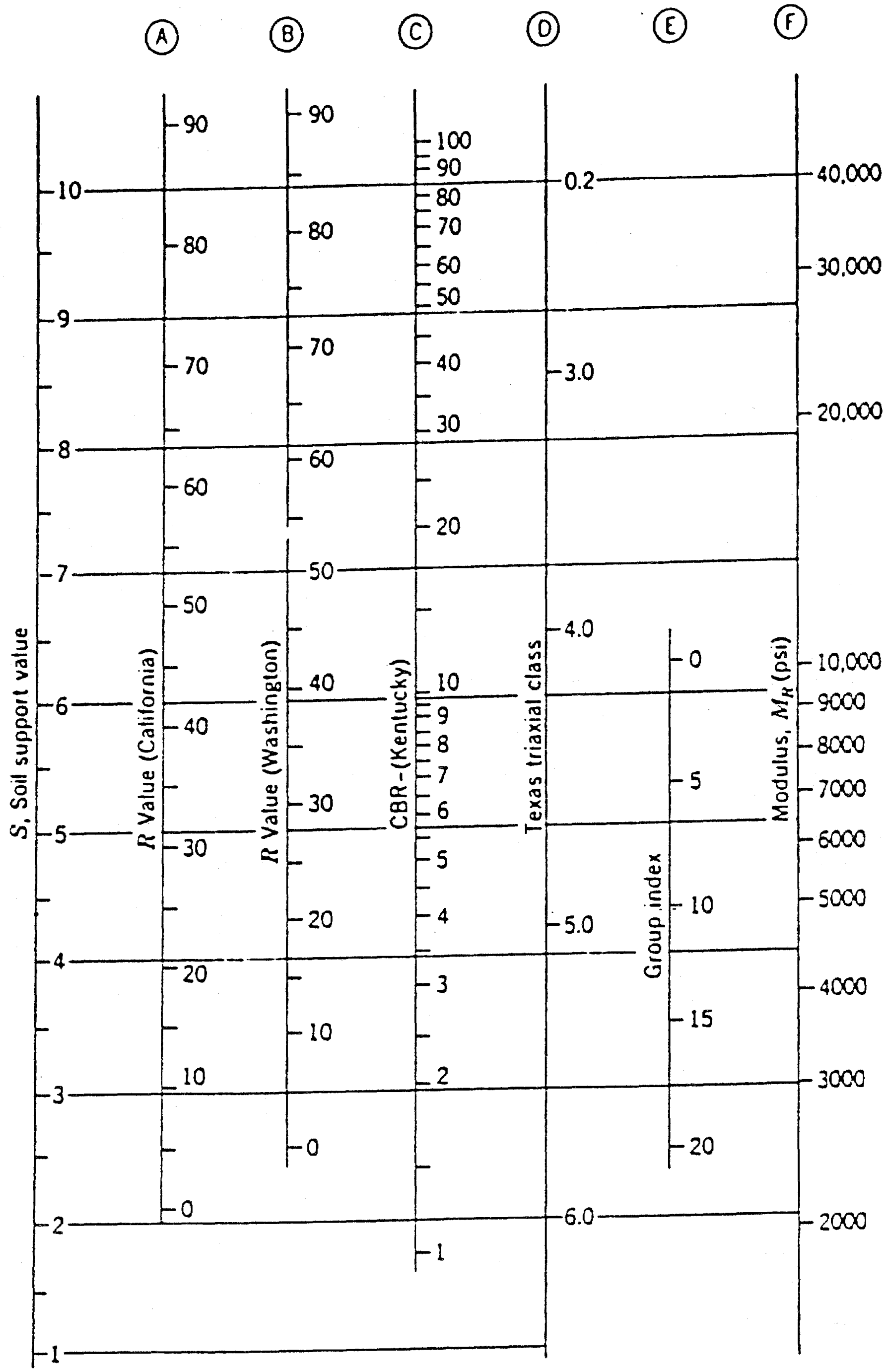
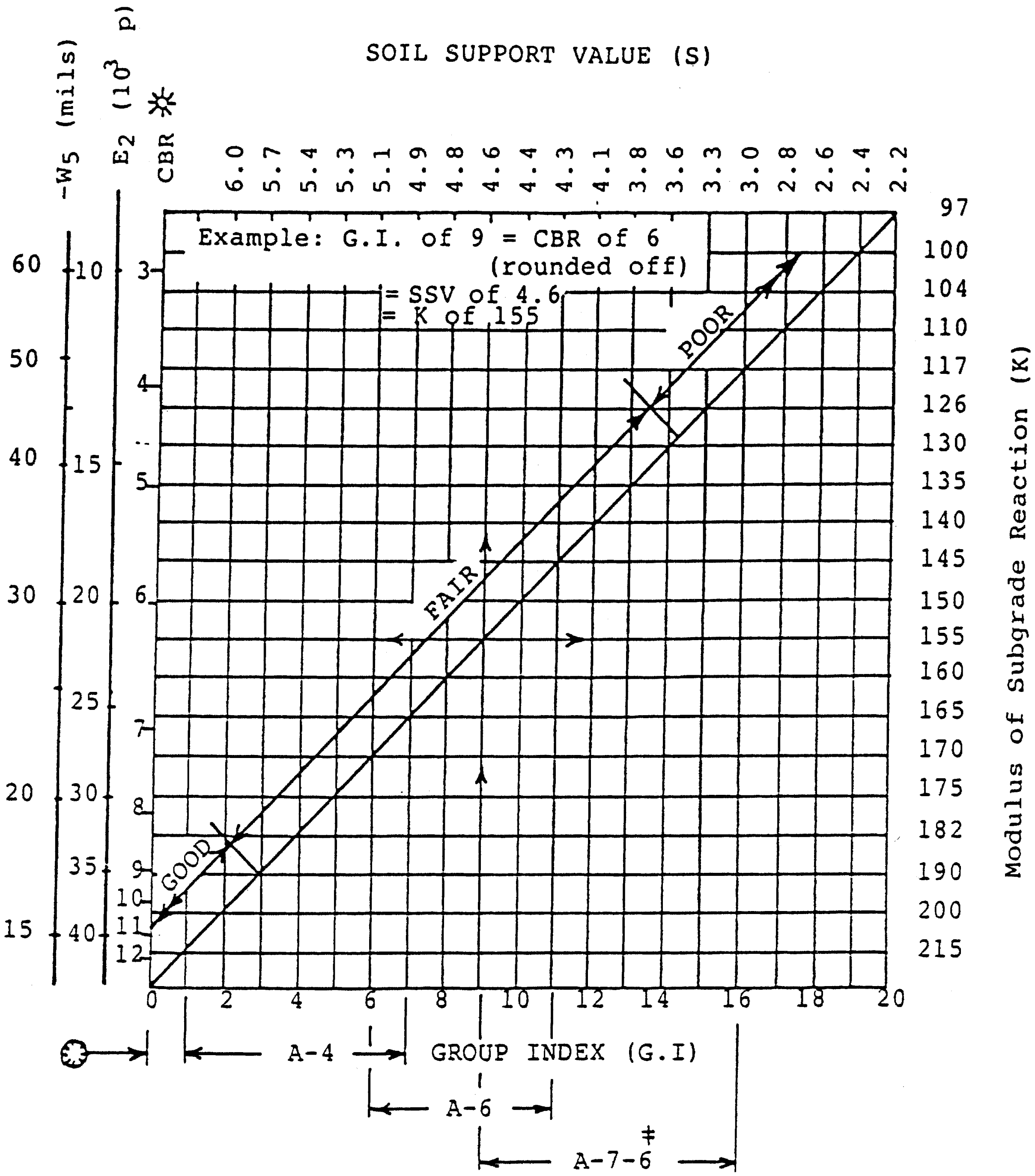


Figure 2-2.20. Graph for Converting Soil Support Values.



AASHTO Classes A-1, A-2 & A-3 lie below O. SSV-6-10; K=200+.

‡ Usual range of AASHTO Classes

5-1/2 lb. hammer, 12" drop, 4 layers, 45 blows per layer, compacted at optimum moisture as determined by AASHTO T-99.

Figure 2-2.21. Correlation of Subgrade Strengths, Ohio Soils.

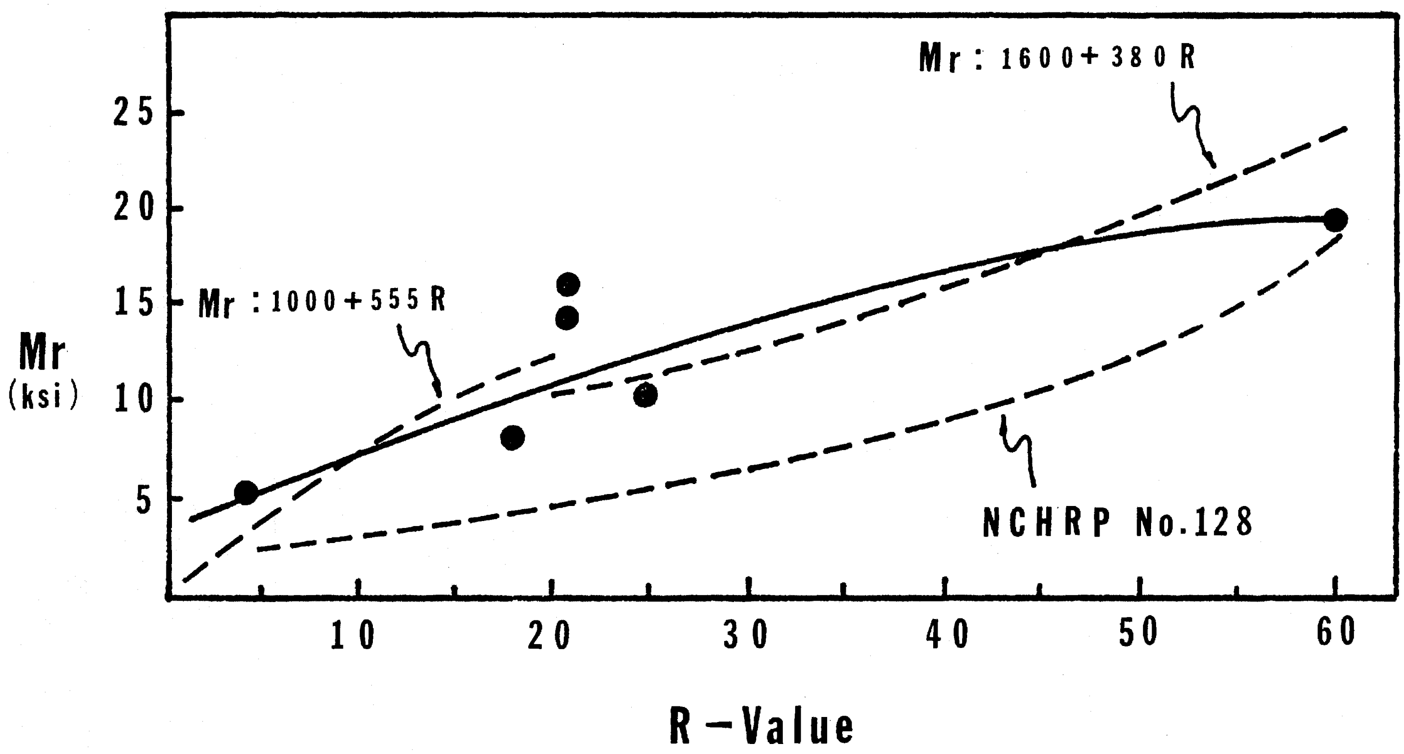


Figure 2-2.22. Laboratory Relationship Between Hveem R Value and Resilient Modulus.

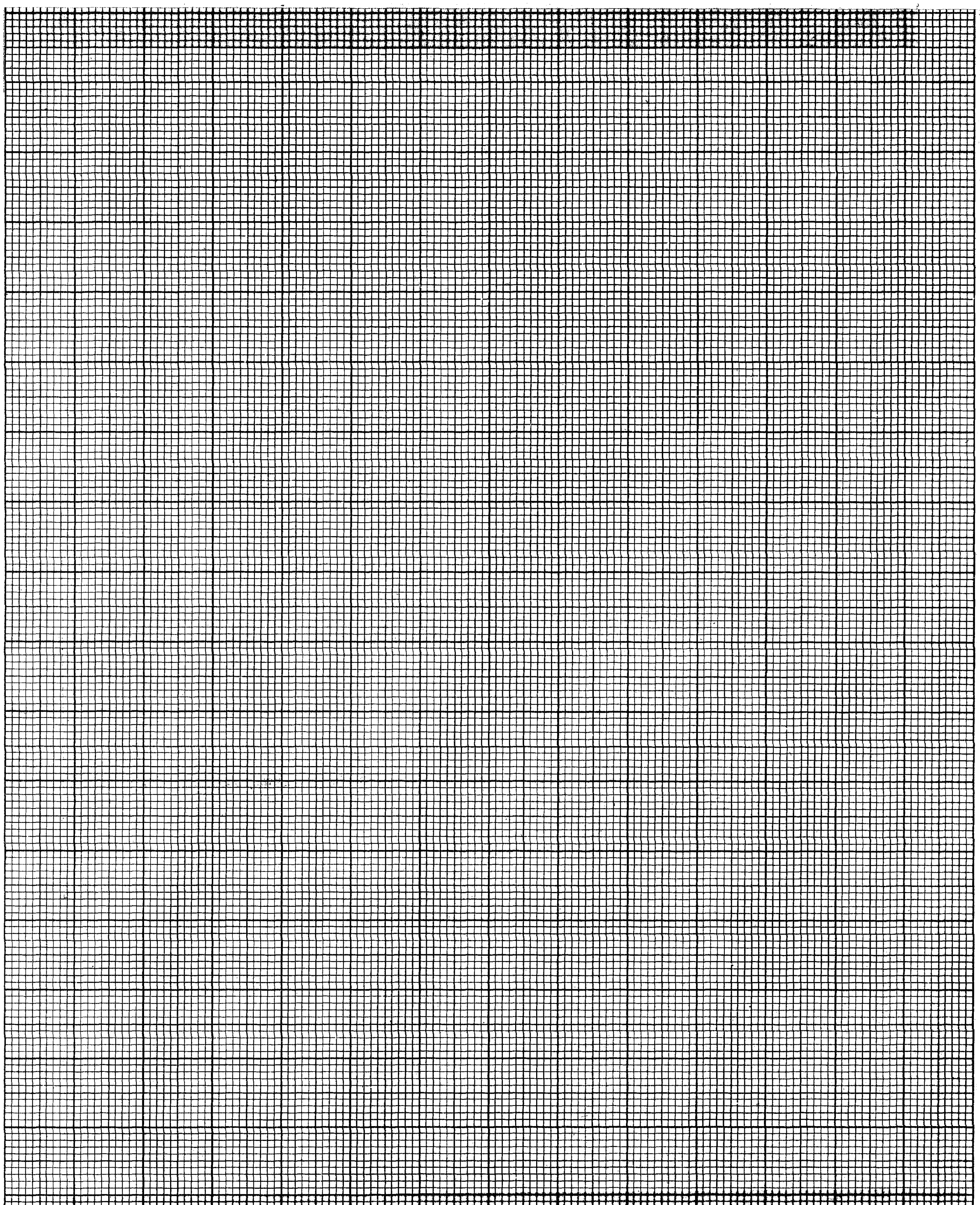
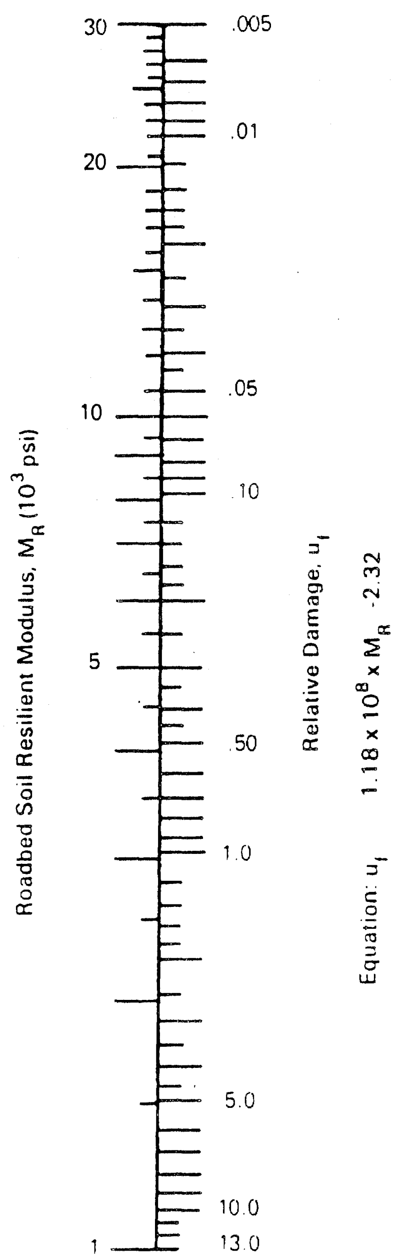


Figure 2-2.23. Graph Paper.

Month	Roadbed Soil Modulus, M_R (psi)	Relative Damage, u_f
Jan.	20,000	0.01
Feb.	20,000	0.01
Mar.	2,500	1.51
Apr.	4,000	0.51
May	4,000	0.51
June	7,000	0.13
July	7,000	0.13
Aug.	7,000	0.13
Sept.	7,000	0.13
Oct.	7,000	0.13
Nov.	4,000	0.51
Dec.	20,000	0.01
	Summation: Σu_f =	3.72



Average: $\bar{u}_f = \frac{\Sigma u_f}{n} = \frac{3.72}{12} = 0.31$

Effective Roadbed Soil Resilient Modulus, M_R (psi) = 5,000 (corresponds to $\bar{u}_f$)

Figure 2-2.24. Chart for Estimating Effective Roadbed Soil Resilient Modulus for Flexible Pavements Designed Using the Serviceability Criteria.

8.0 SUMMARY

The resilient modulus, M_R test has been described in detail to show the relationships between the old strength tests and this new deformation test. The M_R test is required for use in the AASHTO design guide to represent roadbed soil structural adequacy. The need for accurate characterization of the M_R is demonstrated by the effect density, moisture, and environment have on the M_R value.

9.0 REFERENCES

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2. Yoder, E. J., and M. W. Witzcak, Principles of Pavement Design, John Wiley & Sons, 1979.

3. "Test Procedures for Characterizing Dynamic Stress-Strain Properties of Pavement Materials," Special Report 162, Transportation Research Board, 1975.

4. Robnett, Q. L., and M. R. Thompson, "Interim Report - Resilient Properties of Subgrade Soils - Phase I - Development of Testing Procedure," Civil Engineering Studies Transportation Engineering Series No. 5, Illinois Cooperative Highway Research Program, Series No. 139, University of Illinois at Urbana-Champaign, May, 1973.

5. "PCA Soil Primer," Engineering Bulletin, Portland Cement Association, 1973.

6. Thompson, M. R., et. al., "Final Report Subgrade Stability," Report FHWA-IL-UI-169, 1985.

MODULE 2-3

PAVING MATERIALS

1.0 INSTRUCTIONAL OBJECTIVES

This module presents the physical and engineering properties of the various layers of a pavement structure. Test procedures and typical results are presented and discussed as to how they relate to the design of a pavement and its performance. Special emphasis is placed on developing structural layer coefficients from test results. The different requirements for a base course or subbase in flexible and rigid pavements are discussed.

Upon completion of this module the participant will be able to accomplish the following:

1. List the tests typically conducted on paving materials and describe the significance of the test result and its relationship to structural capacity of the material.

2. Determine optimum material property conditions for placement of paving materials and describe the influence that variability in these conditions can have on pavement performance.

3. Describe the different functions a base or subbase must perform in a flexible and rigid pavement.

4. Develop structural layer coefficients for the paving materials used in flexible and rigid pavements and relate these specifically to resilient modulus values.

2.0 INTRODUCTION

The influence of material quality on the performance of a pavement structure has long been recognized. In the AASHO Road Test the quality of the material used in each layer was quantified with the structural layer coefficient, "a," used in the Interim Guide. The coefficients used by each agency have been validated through extensive testing and performance monitoring of constructed pavements.

With the development of mechanistic design procedures the use of the resilient modulus, E, of each material gained importance. With the inclusion of E values as the material quality parameter in the 1986 AASHTO Design Guide the significance of material quality and testing is again an open question. Specific modulus values must be developed for all materials and suitable correlations established.

This module will broaden the pavement designers' appreciation for the following basic material requirements:

1. Suitability for use in construction.

2. Design input requirements.

3. Composition and the relation to quality.

The materials to be discussed include portland cement concrete, asphalt concrete, and granular base materials, both stabilized and unstabilized.

3.0 PORTLAND CEMENT CONCRETE

3.1 Composition

Concrete is a composite material composed of a coarse graded granular material embedded in a hard matrix of material (cement mortar) that fills the space between the particles and glues them together. The aggregates are typically obtained from local sources and must meet strict quality and gradation specifications. The cement binder is a specially manufactured material. Without going into detail, the process consists of heating a mixture of limestone, iron ore and clay to form clinkers which are finely ground with 5 percent gypsum to form the cement. The term portland is a trade name now commonly applied to normally available cements. There are eight recognized types of cements:

1. Type I - Normal.
2. Type Ia - Type I, Air entrained.
3. Type II - Moderate heat of hydration, good strength gain, moderate sulfate resistance.
4. Type IIa - Type II, Air entrained.
5. Type III - High Early Strength.
6. Type IIIa - High Early Strength, air entrained.
7. Type IV - Low Heat of Hydration - Low Strength Gain.
8. Type V - Sulfate Resistant.

Of these, Types I, Ia, III, and IIIa are commonly available. Type II is prevalent in the Western United States. The remaining types are specialty cements requiring special orders.

The quality of a concrete is principally a function of the volumetric composition of the cement, aggregate, water, and air. The most common value related to concrete quality is the water/cement ratio and entrained air content. The ultimate strength can also be altered by changing aggregate size, using special additives, or by altering the curing conditions.

3.2 Admixtures

The most common admixture is used to ensure air entraining. These admixtures ensure a pore structure with microscopic air bubbles. The air is necessary to provide freeze-thaw durability in the hardened concrete.

The next most common group of admixtures includes chemical admixtures which function as water reducers, accelerators, retarders, and combinations.

Water reducers produce high slump at low water/cement ratios which provides good workability with high strength. The newest group of water reducing admixtures are termed "superplasticizers" and they provide for water reductions of 15 to 30 percent. Set accelerators and retarders alter the rate of strength gain during curing. Retarding the initial set may be desirable in very hot climates while accelerating it is desirable in very cold areas or where early opening of the concrete pavement to traffic is desired.

Admixtures should not be used unless approved. Additionally, mixes with multiple admixtures must be fully tested to ensure design strengths are being met and that the admixtures are compatible with each other and the cement.

3.3 Curing

An adequate supply of moisture, a sufficiently high temperature, and an appropriate period of time at that temperature level are required to ensure that the design strength is attained in the pavement.

The cement will not hydrate (gain strength) when the level of moisture drops below a certain value. This relationship is shown in Figure 2-3.1 which shows that the longer the concrete has access to moisture after placement the higher the strength. Proper application of the curing compound ensures moisture remains in the concrete rather than evaporating. Also shown on this figure is the effect of time on the strength. Not only must sufficient time be provided but the temperature during curing must be high enough to provide for strength gain. The temperature influence is shown in Figure 2-3.2. It will generally take longer to gain a specific strength when the temperature is lower.

3.4 Testing

The tests normally run on cured concrete can be grouped into the following categories:

1. Quality control and mix design.
   - Compressive strength.
   - Diametral tensile strength.
   - Slump.
   - Consistency.
   - Air content.

2. Design.
   - Structural strength (Modulus of Rupture).

3. Mechanistic/Empirical.
   - Modulus of Elasticity and Poisson's Ratio.
   - Coefficient of Thermal Expansion.
   - Fatigue Constants.

These test values are interrelated and conversions exist to allow different test values to be approximated.

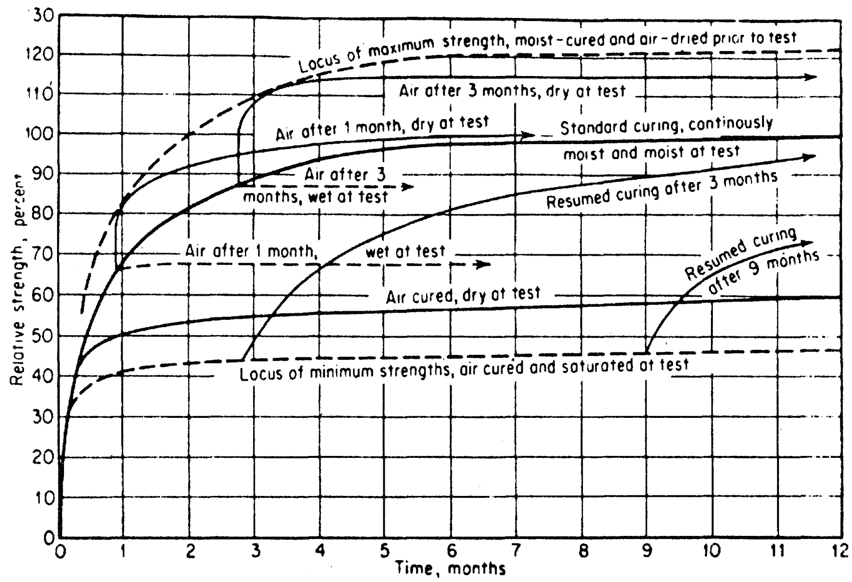

Figure 2-3.1. Effect of Moist-Curing Conditions at 70 F and Moisture Content of Concrete at Time of Test on Compressive Strength of Concrete.

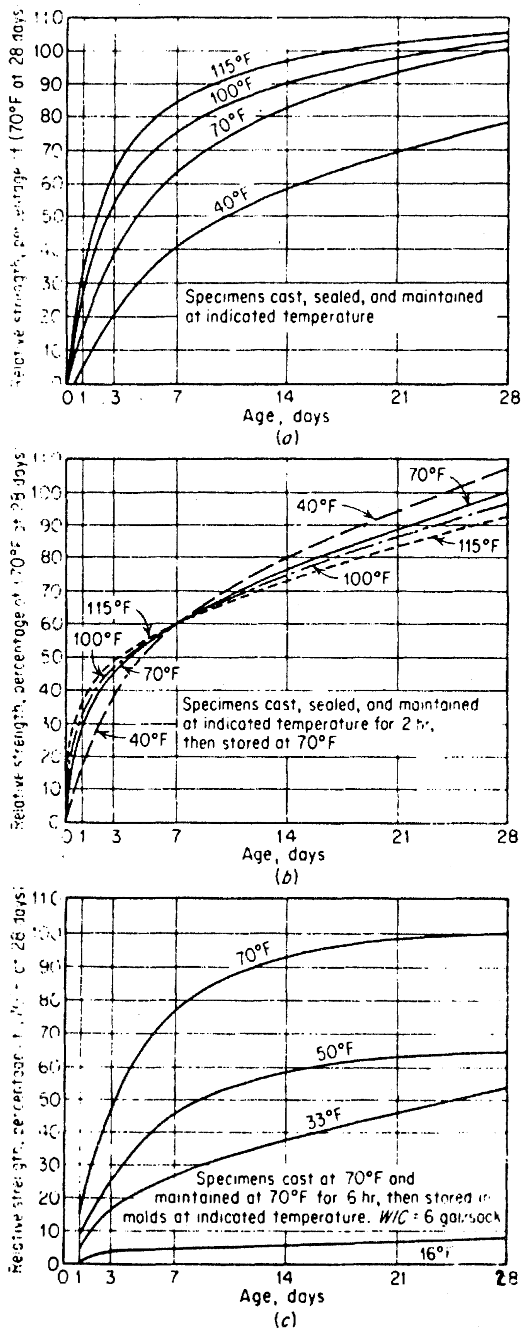

Figure 2-3.2. Effect of Various Temperature Conditions of Casting and Curing on Compressive Strength of Concrete.

3.4.1 Compressive Strength

The compressive strength of concrete is considered a universal measure of concrete quality and durability. That is, a high compressive strength is an indicator of high quality concrete. The concrete compressive strength is a function of aggregate size, aggregate type, coarse aggregate shape, cement composition and additives incorporated in concrete as well as the compositional factors mentioned.

The modulus of rupture (fr), tensile strength (ft') and the modulus of elasticity (Ec) can be related to compressive strength by the following empirical relationships:

$$f_r = 0.60\,(w \times f'_c)^{1/2}$$

$$f'_t = 1/3\,(w \times f'_c)^{1/2}$$

$$E_c = 33\,w^{3/2}(f'_c)^{1/2}$$

where:

w = unit weight of concrete, pcf

f'_c = compressive strength, psi.

Strength is related to a combined effect of time and temperature which can be defined as maturity. Concrete maturity is a summation of the integrals of time-temperature of the concrete above a selected datum temperature. The datum temperature for maturity may be defined as the curing temperature at which the strength of the concrete remains constant regardless of age. Therefore, the maturity is calculated as the time of curing, in hours, multiplied by the temperature, in degrees, above the datum temperature. Experimental data indicates that the datum temperature equals 11 ^{o}F (-11 ^{o}C).

3.4.2 Tensile Strength

Tensile strength is not normally measured directly. A flexure or indirect tension test is normally conducted. The indirect (splitting) tensile test is most often used to determine tensile strength of concrete. The modulus of elasticity can be determined from these tensile tests. The indirect tensile strength is given by:

$$f'_t = 2P/\pi Dt$$

where:

f'_t = Indirect tensile strength, psi

P = Applied load, pounds

D = Diameter, inches

t = Thickness, inches

The indirect tensile strength and unconfined compressive strength have been correlated. It has been shown that for concrete pavement design purposes, the tensile strength can be taken as 0.40 to 0.50 fr, where fr is the modulus of rupture.

3.4.3 Modulus of Rupture (Flexure Strength)

For pavement design purposes, the allowable stress in a rigid pavement is calculated using the modulus of rupture, which is the extreme fiber stress under breaking load. The modulus of rupture is given by a flexural equation:

$$f_r = Mc/I$$

where:

f_r = Modulus of rupture, psi

M = Bending moment at breaking load, lb-in

c = One half beam depth, inches

I = Moment of inertia, inches^4

The test is conducted on a beam in third point loading shown in Figure 2-3.3. The modulus of rupture determined by any other configuration will not be the same as that from the third point loading and suitable correlations must be developed if another test is to be used. Such a correlation would be the relationship between modulus of rupture and indirect tensile strength.

The AASHTO Design Guide, 1986, now requires that the average modulus of rupture be used, not the old "working stress" that was commonly used.

3.4.4 Modulus of Elasticity

The rigidity of the pavement slab and its ability to distribute loads is represented by its modulus of elasticity, E_c. As shown in Module 4-1, the rigid pavement deflections, curvature, stresses and strains are directly influenced by the modulus of elasticity of the concrete layers. The tensile stresses and strains developed in the concrete layer are also functions of the modulus of elasticity.

In continuously reinforced concrete (CRC) pavements, the modulus of elasticity along with the coefficient of thermal expansion, α_s, and the shrinkage coefficient of concrete, α_c, influence the state of stresses in the reinforcement.

The modulus of elasticity will become more important as the mechanistic empirical design procedures gain in popularity. The elastic modulus of the concrete is a major input into the newer finite element programs for accurate stress and strain calculations. The modulus of elasticity can be approximated from the modulus of rupture data as:

$$f_r = 43.5(E/10^6) + 488.5$$

where:

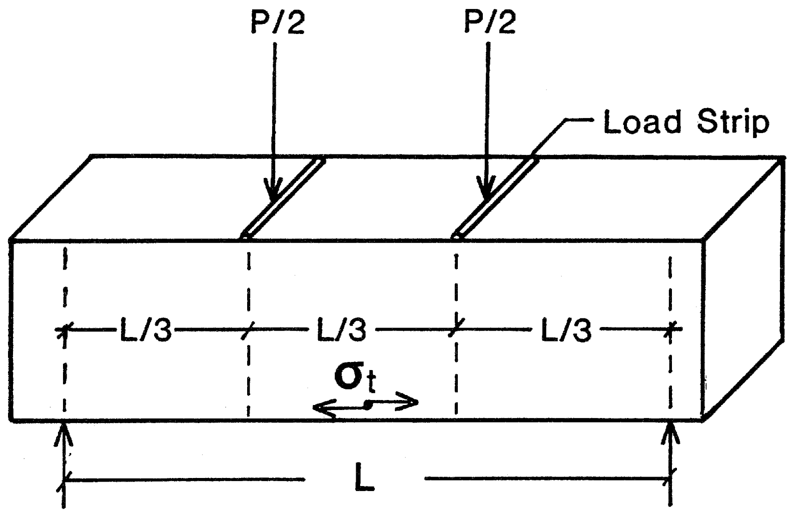

Figure 2-3.3. Illustration of Third Point Loading to Determine the Modulus of Rupture of Concrete Beams.

f_r = Modulus of rupture, psi.

E = Modulus of elasticity of PCC, psi.

3.4.5 Beam Flexural Fatigue Test

Concrete fatigue properties are an important design input consideration. The interrelationship between the flexural stress and the number of load repetitions is shown in Figure 2-3.4 and can be given by an equation of the form:

$$N_f = K_1(\sigma/f_r)^4$$

where:

N_f = Number of load repetitions to failure

σ= applied flexural stress, psi

f_r = modulus of rupture, psi

K_1 = material constant

The test uses a repeated flexural loading on beam specimens 15 inches (37.5 cm) long, 3 inches (7.5 cm) wide and 3 inches (7.5 cm) deep. Loading is generally applied at the rate of 1 to 2 pulses per second, with a load duration of 0.1 second. This third point loading configuration applies a constant bending moment over the middle third of the 15 inch long beam specimen.

The extreme fiber stress in the beam is calculated and plotted against the number of loads at that stress which produce failure, as shown in Figure 2-3.4. In these tests, it is generally recognized that concrete will not fail in fatigue when the ratio of applied stress to modulus of rupture is below approximately 0.5, although no real limit has been shown up to 10-20 million loadings (4, 5).

3.5 Fatigue Models

Factors which affect the modulus of rupture will also alter the fatigue life. This relationship has been used to relate field performance to laboratory data to develop design curves for rigid pavements shown in Figure 2-3.5. Each curve is design procedure specific and have been developed entirely differently and separate from one another. Different curves must not be used in the different procedures as the results will not be predictable.

3.5.1 PCA Model

The PCA design curve can be described with the equation:

$$\log N_f = 11.78 - 12.11(\sigma/f_r) \qquad \text{for } 0.5 < (\sigma/f_r) < 1$$

$$\log N_f > 5.725 \qquad \text{for } (\sigma/f_r) < 0.5$$

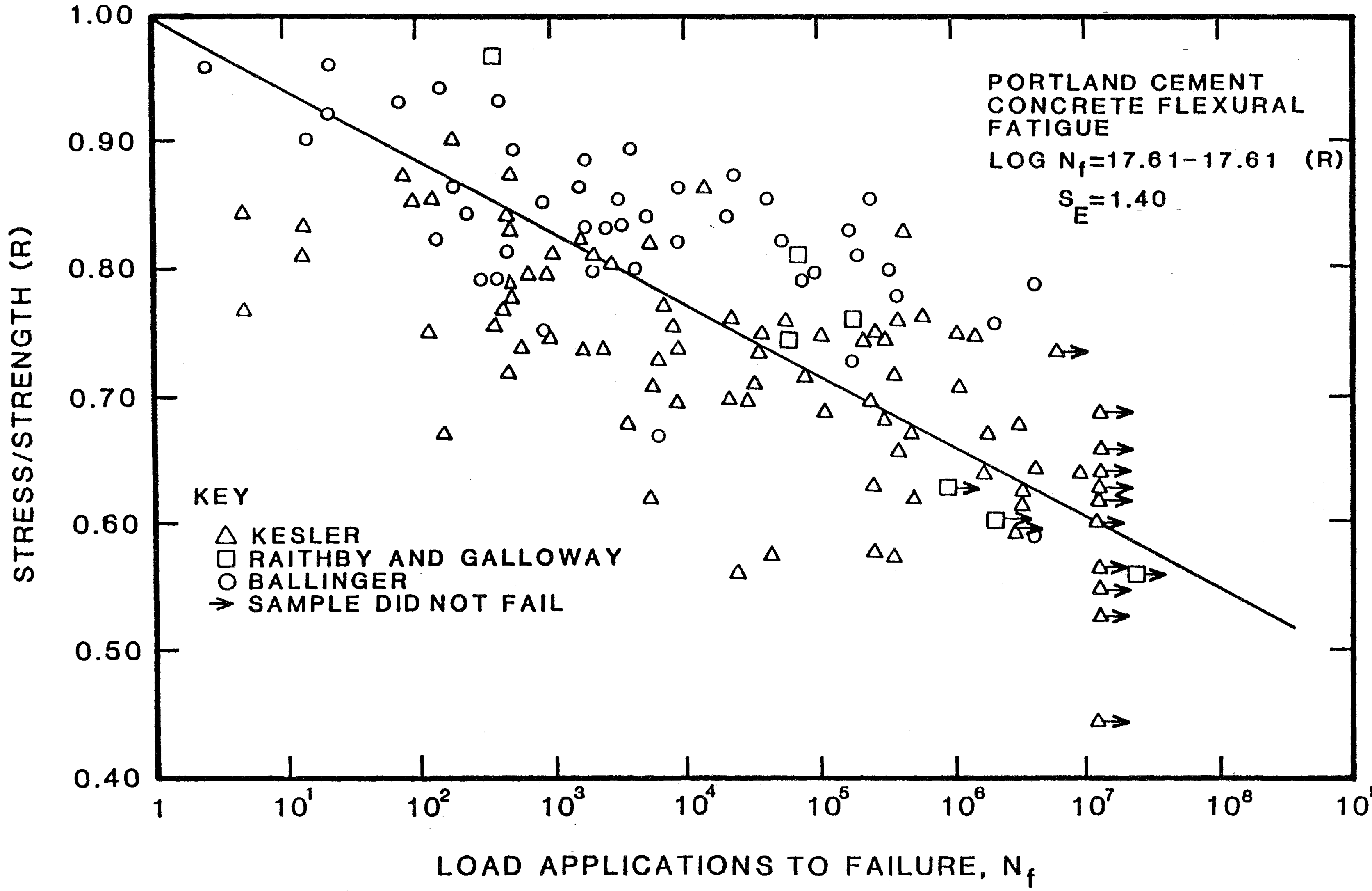

Figure 2-3.4. Typical Figure Curve for Portland Cement Concrete, From PCA.

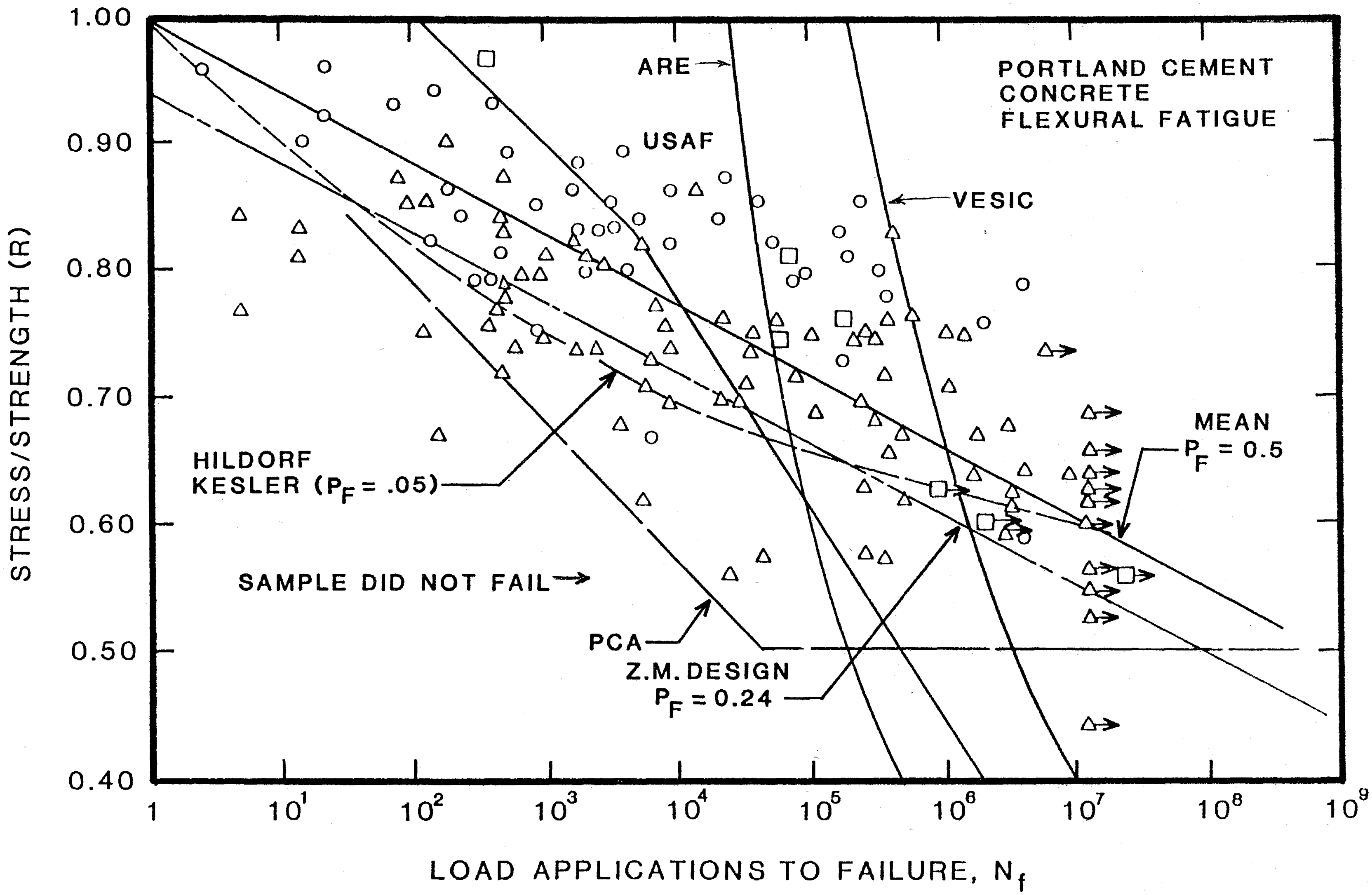

Figure 2-3.5. Summary of PCC Fatigue Data with Various Fatigue Models

where:

N = Number of load applications to failure

σ = Applied stress, psi

f_r = flexural strength (modulus of rupture) at 90 days

Upon examining Figure 2-3.5 it is seen that this model may be over conservative as most of the data points fall above the design line. The limiting design value of 0.5 is not substantiated by laboratory data.

3.5.2 Zero Maintenance Fatigue Model

In the development of a "Zero-Maintenance" design procedure for plain jointed concrete pavements, the following equation was developed to provide for a 24 percent failure rate.

$$\log N_f = 17.61 - 17.61\,(\sigma/f_r)$$

Where the variables are as previously defined.

3.5.3 AASHTO/ARE Fatigue Model

In this model, all AASHO Road Test (8) slabs developing class 3 and 4 cracking were analyzed with elastic layer theory to calculate mid-slab stresses. The equation is:

$$N_f = 23{,}440\,(f_r/\sigma)^{3.21} \quad \text{(R-squared = 0.83)}$$

Where the variables are as previously defined.

Possibly because voids, partial contact, curling, and other factors which increase the actual stresses, the design equation under-predicts at low stress ratios, and over-predicts life at higher ratios.

3.5.4 AASHTO/Vesic Distress Model

Vesic and Saxena (7) analyzed the AASHTO Road Test data (9), and developed another fatigue design equation:

$$\text{Log}\, N_f = 225{,}000\,(f_r/\sigma)^4$$

Where the variables are as previously defined.

This analysis used Westergaard Plate Theory and the tensile stress caused by the average placement of the wheel load. Failure was defined as the number of loadings to produce a terminal serviceability of 2.5. Because of these differences, the two AASHTO based equations are not comparable. Further, they should be used with great care when using stresses calculated from other analytical programs.

3.6 Expansion and Contraction Properties

The expansion and contraction of concrete due to climatic effects, are functions of the thermal properties of concrete. Mechanistic procedures which calculate temperature gradients require Thermal conductivity, K, Thermal diffusivity, θ, Specific heat, C, and the Coefficient of thermal expansion or contraction, α.

The temperature gradient across the depth of the slab is used with the coefficient of thermal expansion, which ranges between 3 to 8 x 10^{6} per degree F. The movement produces curling in the slab which can increase stresses under load and which must be accounted for in the design process.

Concrete shrinkage is related to the water content of the mixture, as well as the general mixture parameters and cement type. The placement of concrete at high temperatures requires more water resulting in more shrinkage. The curing conditions, such as relative humidity, temperature and early concrete protection are also of great significance.

Shrinkage of concrete is a time-dependent process that occurs over an extended period of time. The initial drying shrinkage might be assumed to occur as early as the second day after placement. Prior to this time, plastic shrinkage has taken place. A significant portion of the drying shrinkage normally occurs within the first two weeks of placement. The ultimate amount of shrinkage is typically 415 to 1070 x 10^{-6} in./in. Given a long enough slab, this shrinkage can crack the concrete, which is a reason for reinforcing steel in CRC pavements.

4.0 CHARACTERISTICS OF ASPHALTIC CONCRETE MATERIAL

4.1 Composition

Asphaltic concrete is a composite material composed of a carefully graded aggregate embedded in a matrix of asphalt cement that fills part of the space between the aggregate particles and binds them together. Asphalt cement remains flexible and provides structural integrity through its waterproofing and coating of the aggregate and its cohesive properties. Because asphalt cement is semi-solid at normal pavement temperatures, the quality and gradation of the aggregate are much more important to the performance of asphaltic concrete than for portland cement concrete.

The asphaltic concrete mixture must have a precise amount of asphalt cement to provide the necessary air voids in the mixture. Further, the mixture must be constructed to meet tight specifications on density while maintaining the desired air voids. Density and air void variability can have a significant impact on performance. A pavement may be classified as "failed" even though only the asphalt concrete has deteriorated because of a poor quality mixture.

4.2 Asphalt Cements

Unlike portland cement, different types of asphalt cement are not produced. Different grades are produced based on the fluidity as measured by the viscosity of the material. The current grades and their viscosity ranges are shown in Table 2-3.1.

Table 2-3.1. Viscosity Grading for Asphalt Cements.

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TABLE 1 Requirements for Asphalt Cement, Viscosity Graded at 140°F (60°C)

NOTE—Grading based on original asphalt.

Test	Viscosity Grade				
	AC-2.5	AC-5	AC-10	AC-20	AC-40
Viscosity, 140°F (60°C), P	250 ± 50	500 ± 100	1000 ± 200	2000 ± 400	4000 ± 800
Viscosity, 275°F (135°C), min, cSt	80	110	150	210	300
Penetration, 77°F (25°C), 100 g, 5 s, min	200	120	70	40	20
Flash point, Cleveland open cup, min, °F (°C)	325 (163)	350 (177)	425 (219)	450 (232)	450 (232)
Solubility in trichloroethylene, min, %	99.0	99.0	99.0	99.0	99.0
Tests on residue from thin-film oven test:					
Viscosity, 140°F (60°C), max, P	1250	2500	5000	10 000	20 000
Ductility, 77°F (25°C), 5 cm/min, min, cm	100[A]	100	50	20	10

[A] If ductility is less than 100, material will be accepted if ductility at 60°F (15.5°C) is 100 minimum at a pull rate of 5 cm/min.

TABLE 2 Requirements for Asphalt Cement Viscosity Graded at 140°F (60°C)

NOTE—Grading based on original asphalt.

Test	Viscosity Grade				
	AC-2.5	AC-5	AC-10	AC-20	AC-40
Viscosity, 140°F (60°C), P	250 ± 50	500 ± 100	1000 ± 200	2000 ± 400	4000 ± 800
Viscosity, 275°F (135°C), min, cSt	125	175	250	300	400
Penetration, 77°F (25°C), 100 g, 5 s, min	220	140	80	60	40
Flash point, Cleveland open cup, min, °F (°C)	325 (163)	350 (177)	425 (219)	450 (232)	450 (232)
Solubility in trichloroethylene, min, %	99.0	99.0	99.0	99.0	99.0
Tests on residue from thin-film oven test:					
Viscosity, 140°F (60°C), max, P	1250	2500	5000	10 000	20 000
Ductility 77°F (25°C), 5 cm/min, min, cm	100[A]	100	75	50	25

[A] If ductility is less than 100, material will be accepted if ductility at 60°F (15.5°C) is 100 minimum at a pull rate of 5 cm/min.

TABLE 3 Requirements for Asphalt Cement Viscosity Graded at 140°F (60°C)

NOTE—Grading based on residue from rolling thin-film oven test.

Tests on Residue from Rolling Thin-Film Oven Test:[A]	Viscosity Grade				
	AR-1000	AR-2000	AR-4000	AR-8000	AR-16000
Viscosity, 140°F (60°C), P	1000 ± 250	2000 ± 500	4000 ± 1000	8000 ± 2000	16000 ± 4000
Viscosity, 275°F (135°C), min, cSt	140	200	275	400	550
Penetration, 77°F (25°C), 100 g, 5 s, min	65	40	25	20	20
% of original penetration, 77°F (25°C), min	...	40	45	50	52
Ductility, 77°F (25°C), 5 cm/min, min, cm	100[B]	100[B]	75	75	75
Tests on original asphalt:					
Flash Point, Cleveland Open Cup, min, °F (°C)	400 (205)	425 (219)	440 (227)	450 (232)	460 (238)
Solubility in trichloroethylene, min, %	99.0	99.0	99.0	99.0	99.0

[A] Thin-film oven test may be used but the rolling thin-film oven test shall be the referee method.

[B] If ductility is less than 100, material will be accepted if ductility at 60°F (15.5°C) is 100 minimum at a pull rate of 5 cm/min.

Grading viscosities are all measured at 60 ^{o}C (140 ^{o}F) on the asphalt cement coming directly out of the refinery for the AC grades and on the residue from the Thin Film Oven for the AR grades. The grade selected for use in any area should be determined by the environment. When the average temperature in an area is cold, lower viscosity grades should be chosen to resist low temperature cracking. Likewise, in warm climates a stiffer viscosity grade should be used to resist rutting. Experience will indicate which grade is best suited for a particular area. Grade selection is important in pavement design because it alters the stiffness of the asphaltic concrete which is a very important design parameter in both the mechanistic and the 1986 AASHTO Design Guide.

4.3 Aggregate

The coarse aggregate in all asphaltic concrete should be crushed material. The fine aggregate may be natural sands if desired, but several agencies have found it necessary to require at least some manufactured sand size material in mixes subjected to very heavy traffic. The use of these manufactured sands can produce construction problems, but the stiffer mix that results can be beneficial to the overall design.

The gradation should follow the straight line on the 0.45 power gradation paper as shown in Figure 2-3.6. As indicated on the figure, the gradation used in the mixture should follow a smooth curve either above or below the line. The gradation should never lie directly on the line nor should it be allowed to criss-cross the line, particularly for sizes near the No. 40 sieve, since this can produce mixes with low resistance to deformation under load.

4.4 Testing

The tests normally run on compacted asphaltic concrete samples can be grouped into the following categories:

1. Quality Control and Mix Design.
    - Marshall Test.
        - Stability
        - Flow
        - Air Voids
        - Moisture susceptibility
    - Hveem.
        - Resistance
        - Cohesiometer
        - Air voids
        - Moisture susceptibility

2. Design.
    - Resilient Modulus

3. Mechanistic.
    - Fatigue Constants
    - Rutting Parameters
    - Indirect Tensile Strength

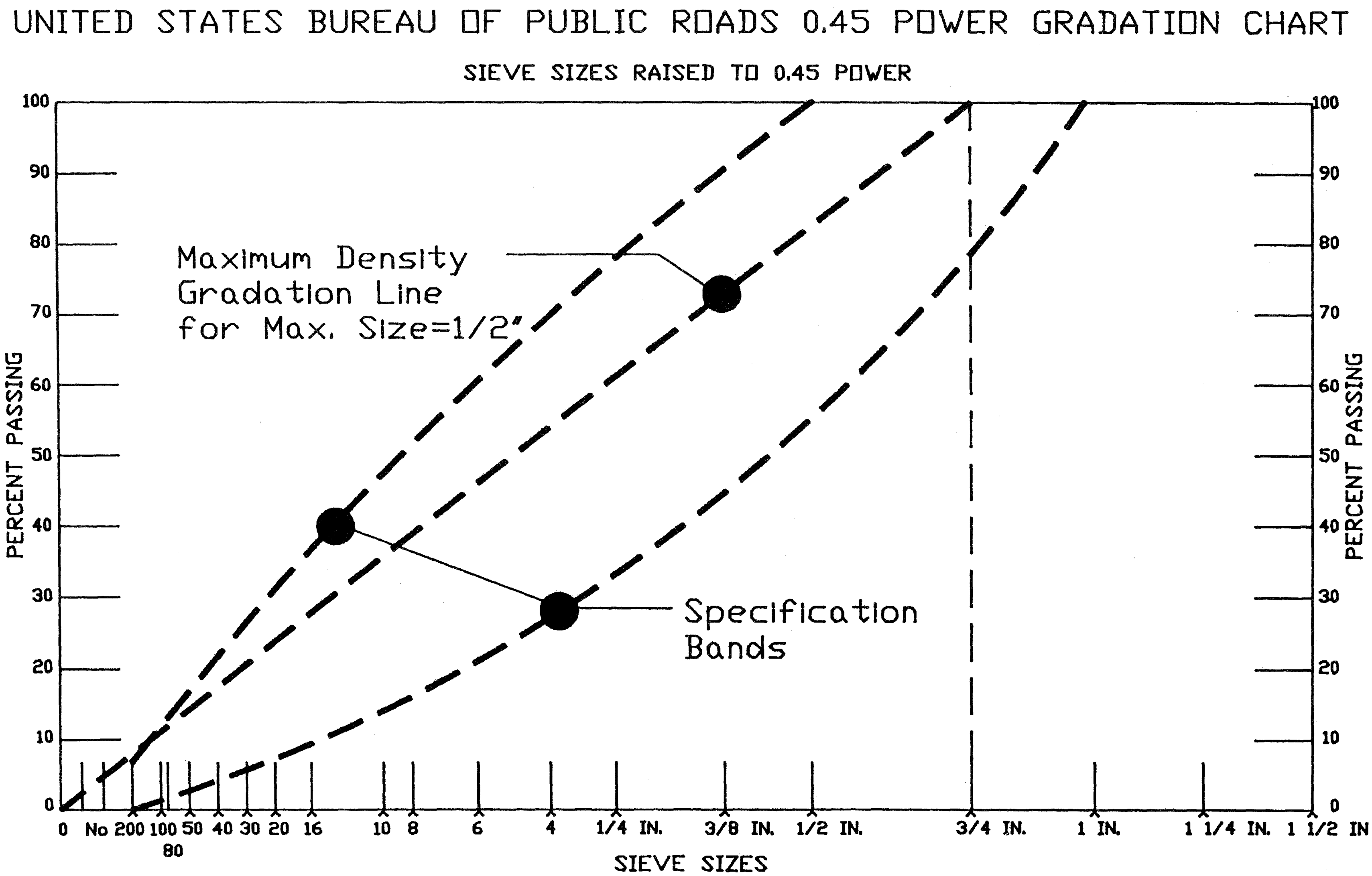


Figure 2-3.6. Asphalt Concrete Aggregate Gradation Chart.

4.3.1 Mix Design Testing

The most common strength tests used in design to describe the engineering characteristics of asphalt concrete are:

1. Marshall.
2. Hveem.
3. Triaxial.

Marshall

This test, which was initially developed by the Corps of Engineers for designing asphaltic concrete for airport pavements, has been adopted by a majority of highway departments. Mixture stability at 140 ^{o}F and the deformation under the maximum load (known as flow) are measured simultaneously. For the design of an asphaltic mixture for a given traffic condition, the initial development indicated that stabilities in excess of 500 pounds would be sufficient. Subsequent evaluation has periodically increased the stability even though there was no indication in the initial data that increased stabilities would be beneficial. Many states developed their own supporting data for mix criteria modifications.

In addition to the strength and deformation characteristics the air voids must fall within a narrow range of three to five percent. When these are satisfied, the final mix must be tested for resistance to moisture (stripping) by a suitable immersion test (ASTM D175).

Hveem

The Hveem test provides the relative stability, S-value, of asphaltic concrete mixtures. A load is applied to the top of a 4 x 2.5 inch specimen and the horizontal load developed in the fluid confining the sides of the specimen is measured. Measuring the frictional resistance of the mixture is carried out at 140 degrees F. Stability values in the range of 30 - 37 produce satisfactory mixes.

Cohesiometer

This test was developed to provide an indication of the tensile strength of asphaltic mixtures. The same briquette tested in the Stabilometer is transferred to the Cohesiometer where the specimen is subjected to tension by bending it around a diameter of the base. The results are expressed on an arbitrary scale; a zero value indicates no tensile strength, and 700 compares to a very good bituminous mixture. Some agencies have dropped this test from their mix design procedure due to the lack of supporting data relating C to performance.

Triaxial Test

Triaxial tests using either open or closed systems have been used on bituminous mixtures. The open system is similar to that used on roadbed soils. In the closed triaxial system, as the applied vertical load increases, the external pressure on the fluid confining the specimen also increases. In this test method one specimen can be used to develop an interrelation between vertical load, V, and confining pressure, σ_c. The results are then used to calculate the cohesion, C, and angle of internal friction, Φ of the mixture.

4.3.2 Diametral Resilient Modulus

The test procedure for determining the diametral modulus of resilience involves a repetitive loading test on disc-shaped specimens (typically Marshall sized specimens). The stress and strain distribution developed within the sample are identical to those developed in the indirect tensile test described earlier (10). A setup is shown in Figure 2-3.7.

In this procedure a dynamic load is applied through a load cell across the vertical diameter of the specimen, which measures 4" (10.16 cm) in diameter and approximately 2.5" (6.35 cm) in height. The specimen is secured in a sample collar and placed on its side beneath the load cell. Two Statham UC-3 transducers are attached to the collar and they are adjusted until the tips just touch the opposite sides of the sample.

The vertical load produces deformation across the horizontal diameter of the specimen which is measured by the transducers. Horizontal movements and vibration effects are cancelled out by the additive coupling of the transducers.

The compressive load is applied at a frequency of 8 to 10 Hz, corresponding to a vehicle speed of about 50 mph (80 kph), and is repeated every three seconds. This gives a load duration that can range from 0.1 to 1.0 second repeated 20 times/minute. This range of loading tests the specimens within their elastic range with a rest interval between loads to allow substantial creep recovery.

The magnitude of the repeated load and the total deformation are recorded during the test procedure, and the resilient modulus is calculated using the following formula:

$$M_R = P\,(\mu + 0.2734)/\Delta t$$

where:

P = magnitude of dynamic load, pounds

μ = Poisson's ratio

Δ = total deformation, inches

t = specimen thickness, inches

Poisson's ratio is generally taken as 0.35. Dynamic load amplitudes of 40, 50 and 60 pounds with a load duration of 0.1 second applied every three seconds are typical. The test is generally conducted at three temperatures, 40, 70 and 100 degrees F to generate design values over the range of temperatures normally encountered for pavement design. The resilient modulus of asphaltic concrete is a temperature dependent parameter. Figure 2-3.8

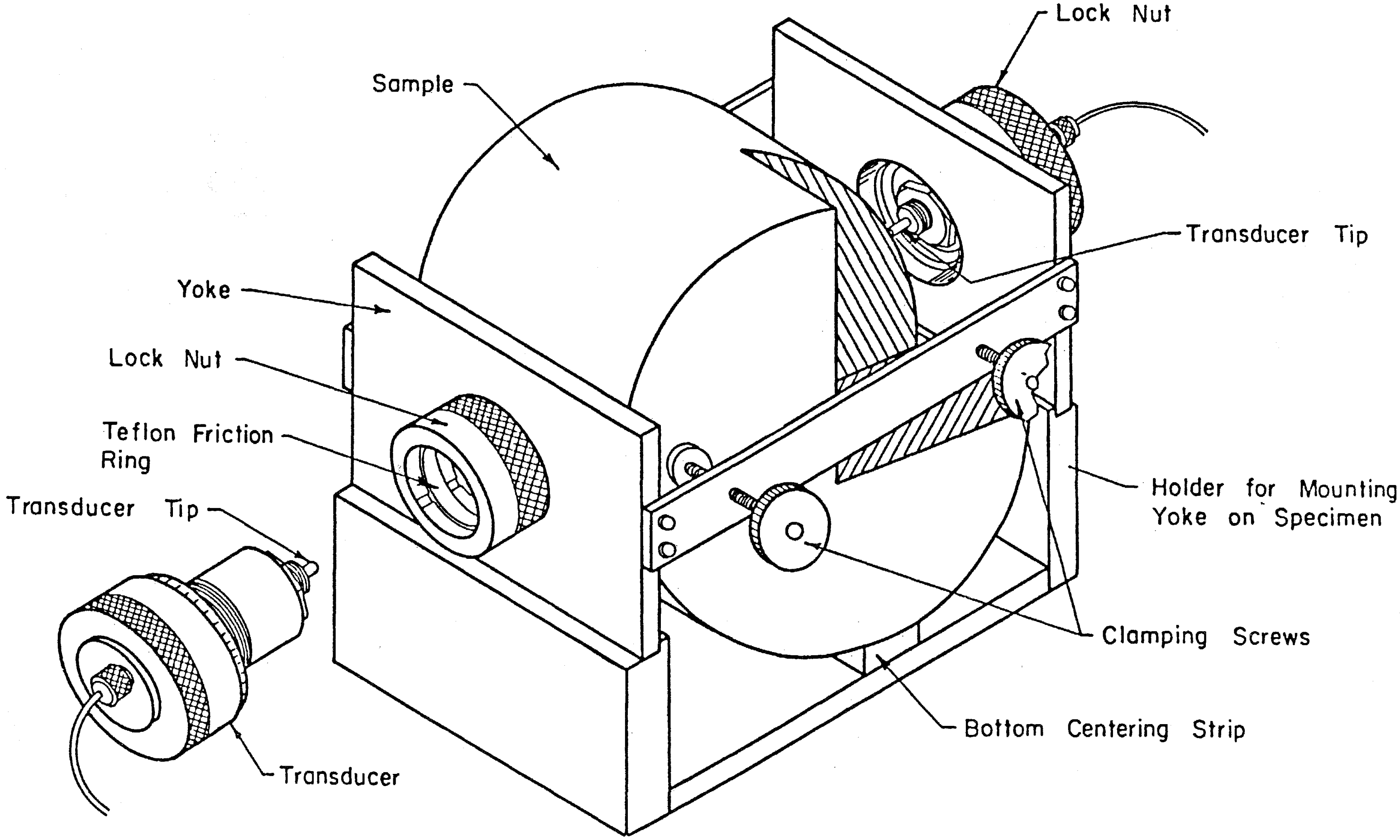


Figure 2-3.7. Transducer Arrangement to Measure Resilient Modulus of Asphaltic Concrete.

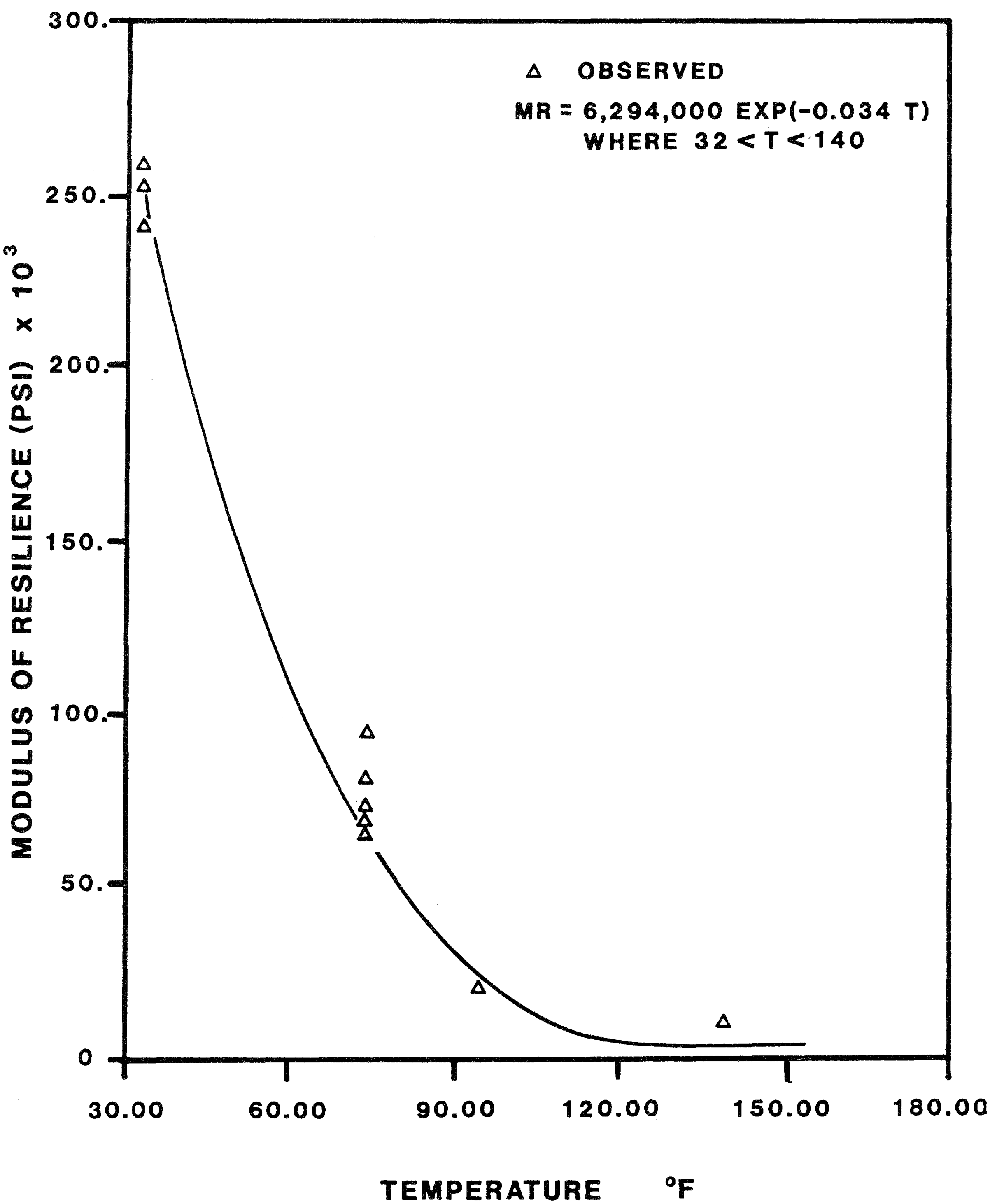


Figure 2-3.8. Modulus of Resilience as a Function of Temperature.

shows the relationship between temperature and the resilient modulus of an asphaltic concrete mixture. This variation in modulus will have an impact on the design of the flexible pavement. This dependence shows the importance of including the seasonal temperature variation in the design process as the modulus of resilience is a major design factor for the asphalt concrete surface.

The resilient modulus test can also be carried out on cylindrical, triaxial specimens. The testing procedures are similar to resilient modulus testing on soils where the modulus is defined at the ratio of axial deviator stress, σ_d, to the recoverable axial strain, ϵ_r.

Estimates of the resilient modulus can be obtained from other sources such as the Heukelom and Klomp nomograph procedures (12) or regression equations such as that in The Asphalt Institute Design procedure (13) can be used:

$$
\begin{aligned}
\mathrm{Log}E^* = {} & 5.55338 + 0.02883(P_{200}/f^{0.17033}) - 0.03476(V_v) + \\
& 0.070377(n_{70\,F,10^6}) + 0.000005[t_p^{(1.3+0.49825\mathrm{Log}(f)p_{ac}^{0.5}}] \\
& - 0.00189[t_p^{(1.3+0.49825\mathrm{Log}(f)(p_{ac}^{0.5}/f^{1.1})}] + \\
& 0.931757(1/f^{0.02774})
\end{aligned}
$$

where:

E = dynamic modulus, psi

P_{200} = - #200 material

f = Frequency, Hz

V_v = Air voids

$n_{70\,F,10^6}$ = Absolute viscosity at 70 F x 10^6

p_{ac} = Asphalt content by weight of mixture

t_p = Temperature, F.

These two procedures, and others, require knowledge of specific asphalt cement properties and mix parameters, which may make them difficult to apply in the design process.

4.3.3 Dynamic Stiffness Modulus

The dynamic stiffness modulus of asphaltic concrete can be obtained from flexural fatigue tests. The flexural stiffness, E_o, is calculated after 200 repetitions, and is given by:

$$E_o = Pa\,(3L^2 - 4a^2)/48I\Delta$$

where:

E_o = Flexural stiffness, psi

P = Dynamic load, pounds

a = (l - 4)/2

L = Reaction span, inches

I = Moment of inertia, in^4

Δ = Dynamic center deflection, inches

f = Frequency

Typical values of the dynamic stiffness modulus of asphaltic concrete at different frequencies are shown in Table 2-3.2.

4.3.4 Indirect Tensile Strength

The ultimate strength of asphaltic mixtures is typically expressed as the tensile strength of the mixture. The tensile strength is most easily determined in the indirect tensile test procedures on disc-shaped specimens with 4" (10.16 cm) diameter and approximately 2.5" (6.35 cm) height using the procedures mentioned earlier for concrete cylinders.

For asphaltic concrete specimens the load is applied at a constant deformation rate of 2.0 inches per minute at a standard 72 F temperature. This mode of loading produces a horizontal tensile stress along the vertical axis as shown in Figure 2-3.9. There is also a static compressive load acting parallel to and along the vertical diameter. The testing equipment is the same as used for other indirect tests with the only difference being the addition of curved one-half inch wide steel strips for load distribution.

For most engineering materials, the initial failure occurs by tensile splitting along the vertical diameter. The indirect tensile strength of material can be calculated from:

$$t = 2P_{max}/\pi Dt$$

where:

P_{max} = Maximum applied load, lb.

D = Specimen diameter, in.

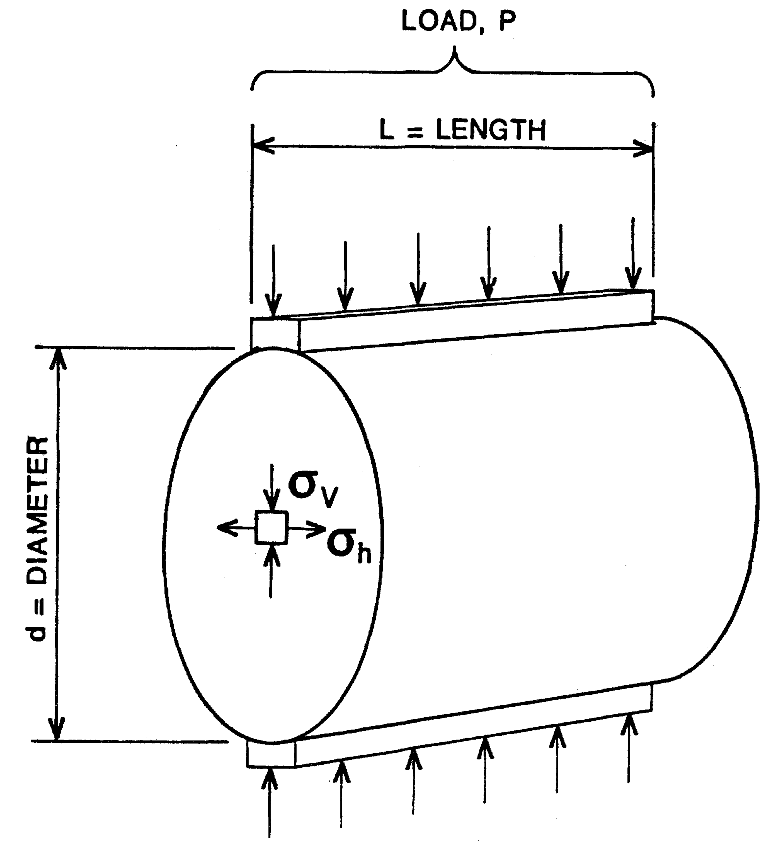
t = Specimen thickness, in.

This test may be supplemented with vertical measurements of the vertical deformation of the loading head and of the horizontal deformation resulting from the load. With these measurements of load and deformation the elastic properties of the mixture under this load may be calculated from the data shown in Figure 2-3.10 (15). This information, particularly the tensile strain at failure, is useful in analyzing the low temperature behavior of the asphaltic concrete. Mixes which are brittle at cold temperatures will fail with very low tensile strains. The tensile strength of mixes at low

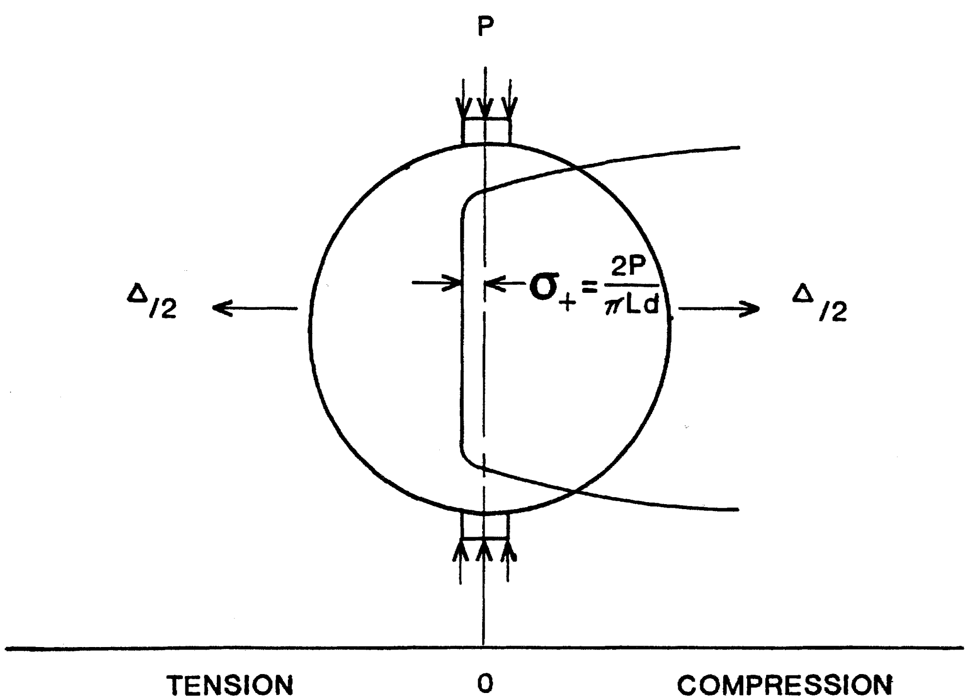
Table 2-3.2. Typical Asphalt Concrete Modulus Values.

Temperature F		Load Frequency (Hz.) f = 1 cps	f = 4 cps	f = 16 cps
40	mean	12.0*	16.0	18.0
	range	(6.0 - 18.0)	(9.0 - 27.0)	(10.0 - 30.0)
70	mean	3.0	5.0	7.0
	range	(2.0 - 6.0)	(4.0 - 9.0)	(5.0 - 11.0)
100	mean	0.07	1.0	1.6
	range	(0.5 - 1.5)	(0.7 - 2.2)	(1.0 - 3.2)

* Value of modulus x 10^6 psi.



SPECIMEN LOADING



STRESS MAGNITUDE ALONG AXIS OF LOADING

Figure 2-3.9. Depiction of Loading and Stress Modes in Resilient Modulus Test.

Static Properties

(1) Tensile strength S_T, psi $= \frac{P_{Fail}}{h} \cdot A_0$

(2) Poisson's ratio ν $= \frac{DR \cdot A_1 + B_1}{DR \cdot A_2 + B_2}$

(3) Modulus of elasticity E, psi $= \frac{S_H}{h} (A_3 - \nu \cdot A_4)$

(4) Tensile strain ϵ_T $= X_T \left[\frac{A_5 - \nu \cdot A_6}{A_1 - \nu \cdot A_2} \right]$

(5) Compressive strain ϵ_C $= Y_T \left[\frac{B_3 - \nu \cdot B_4}{B_1 - \nu \cdot B_2} \right]$

P_{Fail} = total load at failure (maximum load P_{max} or load at first inflection point), pounds

P = applied load or repeated load, pounds

h = height of specimen, inches

DR = deformation ratio $\frac{Y_T}{X_T}$ (the slope of line of best fit* between vertical deformation Y_T and the corresponding horizontal deformation X_T up to failure load)

X_T = total horizontal deformation, inches

Y_T = total vertical deformation, inches

S_H = horizontal tangent modulus $\frac{P}{X_T}$ (the slope of the line of best fit* between load P and horizontal deformation X_T for loads up to failure load)

H_{RI}, V_{RI} = instantaneous resilient horizontal and vertical deformations, respectively

Diameter, inches	A_0	A_1	A_2	A_3	A_4	A_5	A_6	B_1	B_2	B_3	B_4
4.0	.156	.0673	-.2494	.2692	-.9974	.03896	-.1185	-.8954	-.0156	-.1185	.03896

Figure 2-3.10. Calculation Procedures for Indirect Tension Test.

temperatures is also a variable which helps explain performance in cold climates. Typical stress and strain values at low temperatures are shown in Figure 2-3.11 for several asphalt concrete specimens illustrating different low-temperature behavior (16).

Recent research results have indicated that fatigue coefficients can be calculated from the indirect tensile strength test data (17). This work by Maupin clearly shows that dense-graded asphaltic concrete mixtures with paving grade asphalt cements can be characterized for fatigue from the indirect tensile test by the following equations:

CONSTANT STRAIN

$$N_f = K_2(1/e)^n$$

where:

N_f = Number of loadings required to reduce the dynamic stiffness modulus by one-third

e = Radial tensile strain in the asphalt concrete layer

$K_2 = 10^{(7.92 - 0.0122\,S_{it})}$

$n = 0.0374S_{it} - 0.744$

S_{it} = Indirect tensile strength, psi

CONSTANT STRESS

$$N_f = K_1(1/S)^n$$

where:

N_f = Number of loads to collapse of the sample

S = Applied radial stress in the asphalt concrete layer

$n = 11.6 - 0.000396E_{it}$

E_{it} = Stiffness at 3/4 of the failure strain

$K_1 = \exp^{(n[\ln(12.6S_{it} - 558)])}$

S_{it} = Indirect tensile strength, psi

The constant strain representation of fatigue data is most widely used for normal fatigue testing in conjunction with thin pavement sections. The constant stress test is useful for testing and designing for thicker surfacings, due to the stress/strain distributions. In the design process, the constant strain representation is most commonly used owing to the ease of testing as will be discussed in a subsequent section.

Poisson's Ratio

Poisson's ratio is defined as the ratio of the lateral strain to the axial strain. For most pavement materials, the sensitivity of Poisson's ratio to testing variables is relatively small. Poisson's ratio for asphalt concrete varies from 0.3 at low temperatures to 0.4 at high temperatures with 0.35 being a good average value to use.

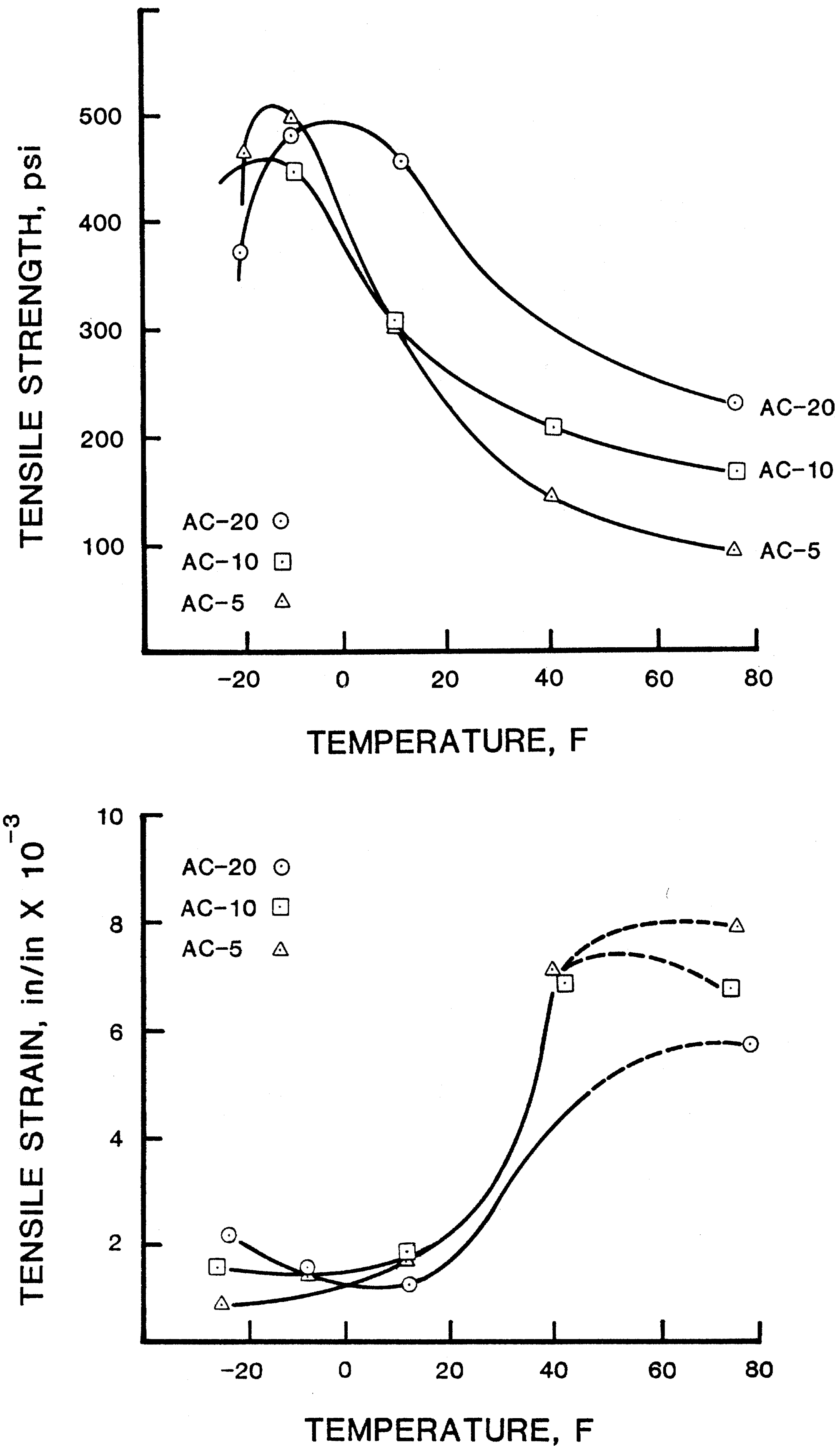


Figure 2-3.11. Tensile Strength and Tensile Strain at Failure as a Function of Temperature.

4.5 Fatigue Testing

Fatigue cracking is one of two major load-associated failure modes for asphaltic pavements. This distress involves the progressive formation of cracks under repetitive loadings. As the number of loads increases, the crack propagates through the pavement layer, producing a crack in the pavement. Failure in fatigue is generally defined as the point when a given percent of the surface area becomes covered with the fatigue cracking.

There are many different test methods for determining the fatigue properties of asphaltic mixtures in the laboratory. The most commonly accepted procedure called for in most rational methods uses third point loading on asphaltic concrete beams (2) as shown in Figure 2-3.12. Other testing methods use indirect tensile loading with repetitive loading, loading of beams resting on elastic (rubber) foundations, testing of diaphragms (slabs) resting on specific foundations, and testing of trapezoidal specimens.

4.5.1 Flexural Fatigue Tests on Asphaltic Concrete Beams

This procedure has been outlined in the VESYS User's Manual (3) and many other rational design methods. It employs third point load testing on 3 x 3 x 15 inch (7.62 x 7.62 x 38.1 cm) simply-supported asphaltic beams. The specimens are first brought to the required testing temperature. Then repeated loadings in the form of haversine loads at a frequency of two cycles per second was applied, with a load duration of 0.1 second and a rest period of 0.4 second between loads.

The applied load is selected such that the extreme fiber stress will produce failure somewhere between 1,000 to 1,000,000 load cycles. The beam center point deflection and the applied dynamic load are measured after approximately 200 load repetitions and are used to calculate the extreme fiber strain from beam bending theory. The test is then continued at the constant load level until the sample is fractured. Eight to 12 tests are run for each temperature. Different loadings are used to vary the number of load cycles over the desired range for a good characterization.

The data from the test is analyzed by plotting the initial strain against the number of cycles producing failure on log-log paper. Typical fatigue curves for several asphalt concretes samples are shown in Figure 2-3.13 (18). The fatigue data are analyzed by determining the least squares equation for the straight line. The fitted relationship is of the form:

$$N_f = K_1(1/e)^{K_2}$$

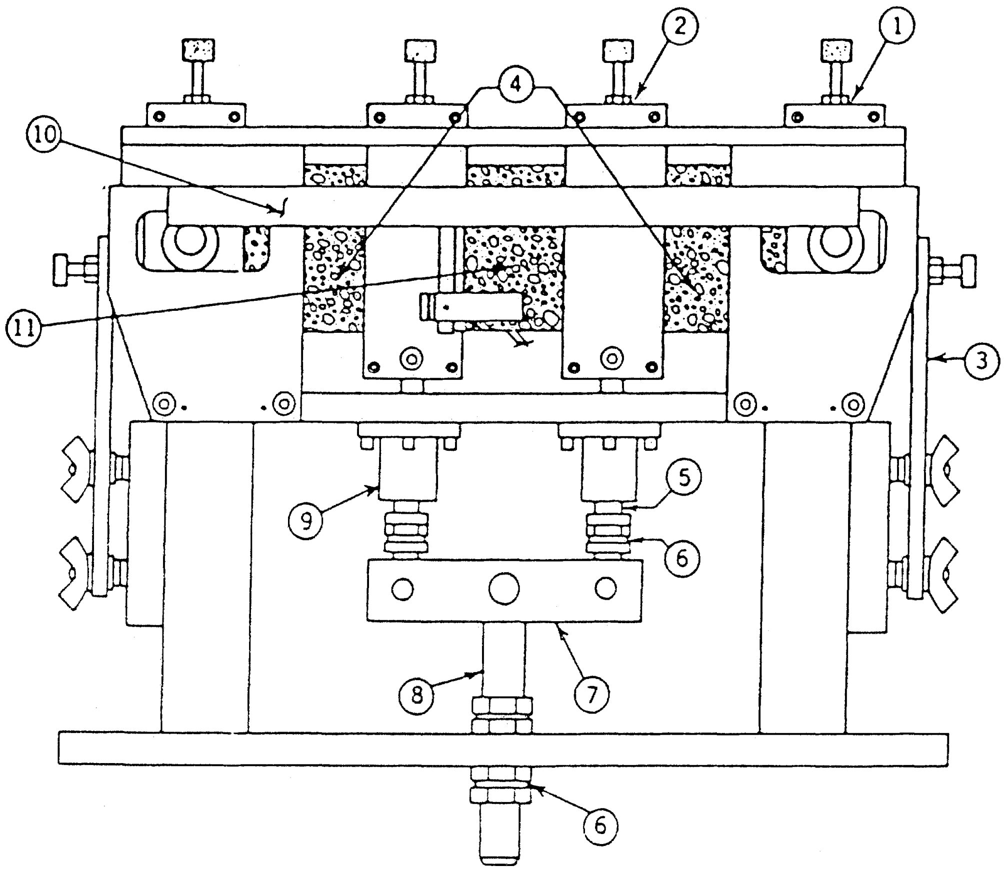
where:

N_f = Number of load repetitions to failure

e = Initial strain at 200th load repetition

K_1, K_2 = Regression coefficients

In Table 2-3.3, typical K_1 and K_2 values for different asphaltic concrete mixtures are presented.



KEY:

1. Reaction Clamp
2. Load Clamp
3. Restrainer
4. Specimen
5. Loading Rod
6. Stop Nuts
7. Load Bar
8. Piston Rod
9. Thompson Ball Busing
10. LVDT Holder
11. LVDT

Figure 2-3.12. Fatigue Test Setup for Asphaltic Concrete.

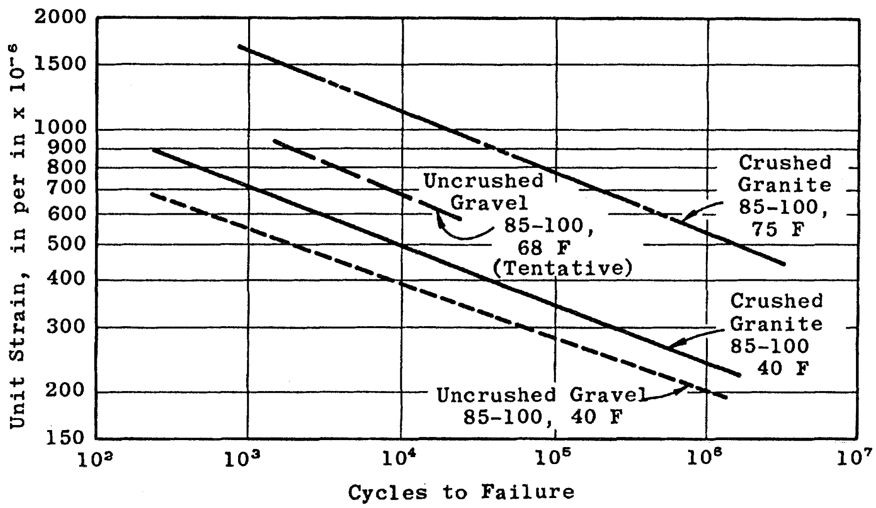


Figure 2-3.13. Fatigue Curves for Typical Asphaltic Concrete Mixes.

Table 2-3.3. Summary of Laboratory Fatigue Data Generated from other Sources

Data Source	Mixture	Asphalt	Test Temp.	K_1	K_2
Texas A&M	Laboratory Standard	AC-10	68°F	6.0×10^{-6}	2.864
Texas A&M	Recycled California Valley	Salvaged AC + Recycling Agent C	68°F	2.5×10^{-6}	3.205
Texas A&M	Recycled California Valley	Salvaged AC + Recycling Agent B	68°F	5.2×10^{-5}	2.682
Texas A&M	Recycled Woodburn, Oregon	Salvaged AC + Recycling Agent C	68°F	1.1×10^{-5}	3.150
Pell	Crushed Rock Rock and Sand	8% 45 pen.	68°F	8.8×10^{-15}	5.10
Monismith	Granite	8% 40-50 pen	68°F	6.1×10^{-6}	3.38
Britain	Coarse Granite	6% 85-100 pen	68°F	3.2×10^{-5}	2.49
California	Fine Granite	6% 85-100 pen	68°F	8.9×10^{-7}	2.95
California	Medium Granite	6% 85-100 pen	68°F	2.9×10^{-6}	2.83
California	Medium Granite	6% 60-70 pen	68°F	1.1×10^{-7}	3.26
California	Medium Granite	6% 40-50 pen	68°F	1.0×10^{-10}	4.01
California	Medium Granite	6% 60-70 pen	68°F	1.3×10^{-7}	3.22
Gonzales Field	Shale	6% 85-100 pen	68°F	2.1×10^{-8}	3.60

4.5.2 Fatigue Tests on Asphaltic Concrete Cores

In this procedure Marshall sized specimens can be made in the laboratory or obtained from existing pavements by coring. In using field cores, specimens can be cut from field cylinders to the required 2.5 inches thickness to produce the desired plain strain condition in the test. The fatigue test set-ups are very similar to the method used for diametral modulus of resilience. The dynamic load is applied using a haversine function at a frequency of two cycles per second, with 0.1 second load duration and 0.4 second rest period. Different stress levels are selected to yield different numbers of fatigue lives (N_f). The fatigue life value is defined as the total number of cycles at which the sample is completely split into two pieces or the number of cycles to produce a decrease in the resilient modulus of 50 percent. The data is presented in the same manner used for the beam fatigue tests.

4.5.3 Fatigue Models for Flexible Pavements

The fatigue cracking of asphaltic concrete has been studied by numerous investigators, both in the laboratory and from field performance data. The resulting test data all confirm the linear relationships shown previously. In this relation, K1 and K2 may or may not be temperature dependent, depending on whether or not the test data shows a temperature modulus dependence; the literature is about equally divided between temperature dependent and temperature independent fatigue equations.

Laboratory determined fatigue relationships generally predict failure much sooner than is observed in field performance studies. To compensate for this discrepancy, a "shift factor" is normally applied to K1 with the justification that laboratory tests predict crack initiation. In a pavement, the cracks start at the bottom, take some time to reach the surface, and exist as a crack for some time before they have deteriorated to the point they are recognized as a distress crack (19).

Most field performance models are based on the AASHTO Road Test data and use elastic layer theory to compute the radial tensile stress/strain at the bottom of the asphalt concrete layer. The actual traffic history is converted to equivalent 18 Kip axle loads using the AASHTO relationships, and a regression equation is developed between Nf and the critical strain. These analyses have assumed that average annual temperature conditions exist throughout the year so that the layer properties can be considered to be constant for the test duration.

The AASHTO/ARE fatigue equation is given by:

$$N_f = 9.73 \times 10^{-15} (e_R)^{-5.16}$$

This model was developed using linear elastic layer theory with two circular loads to represent the two-tire wheel. The layer moduli were determined from laboratory tests on samples with confining pressure corresponding to that expected in a pavement structure but were assumed to be stress-independent in the regression analysis.

There are two areas of concern in the development of this model: the critical strain used in the above equation is not the maximum strain, and the base/roadbed soil moduli are assumed to be stress-independent. The strain used in the ARE equation is the strain parallel to the axle which causes longitudinal crack formation. However, the true maximum strain occurs perpendicular to the axle, producing transverse cracking. Both laboratory and field test data indicate that the moduli of granular materials and cohesive soils depend on the state of stress as was discussed previously.

Majidzadeh and Ilves (19) developed a similar performance-related distress function from the AASHTO Road Test data using the same assumptions as were used by ARE, except that the base and roadbed soil moduli were assumed to be stress dependent. Gravity stresses resulting from the self-weight of layers were included in stress, and the maximum critical strain (in the direction perpendicular to the axle) was used in the regression equation. The resulting equation is:

$$N_f = 7.56 \times 10^{-12}(e_R)^{-4.68}$$

In the mechanistic/empirical design procedures, the radial strain in these equations is the strain calculated in a proposed pavement structure. The equation calculates the number of loads the proposed pavement section is capable of carrying. If the calculated number of loads exceeds the desired traffic loadings, the pavement is sufficient. If the pavement is not sufficient, thicknesses can be increased.

4.6 Rutting Distress

Rutting is the gradual accumulation of permanent deformation in the pavement layers. Rutting models include the Shell Model, VESYS Model, PDMAP Model, Monismith Model, DEVPAV, WATMODE, Herschek Model, and OSU Model. The VESYS and PDMAP Models are probabilistic models that use the statistical variation of material properties. The other models compute rut depth as the sum of permanent deformations in each pavement layer except for the SHELL Model which only examines the asphalt concrete layer. The rutting equation used in PDMAP is the AASHTO Road Test Data, and WATMODE utilizes Brampton and St. Anne Road Test results. All other models utilize permanent deformation properties of pavement materials determined from laboratory tests. The testing required by the Monismith Model is somewhat more complicated in that repeated load triaxial tests are required. With the above mentioned differences, the models are all very similar in that all but VESYS and DEVPAV use elastic layer theory in analyzing stresses and strains in the roadway; DEVPAV utilizes a Finite Element Method (FEM) program developed in Ireland and VESYS uses elastic solutions altered with a superposition principle to provide viscoelastic solutions.

The most common design model for roadbed soil rutting is based on an allowable roadbed soil strain limit, given by:

$$N_f = 1.365 \times 10^{-9}(e_v)^{-4.477}$$

where:

N_f = Allowable number of load repetition

e_v = Maximum vertical strain at the top of roadbed soil, in/in

This procedure limits the vertical strain on the roadbed soil to a value that will not overstress the soil, but it does not provide any design for the upper pavement layers. Therefore, it is necessary that material specifications be closely controlled to insure minimal deformations.

4.8 Thermal and Moisture Characteristics

4.8.1 Low-Temperature Cracking

The temperature related characteristics of asphaltic concrete pavements are not the same as for rigid concrete pavements. The asphalt concrete will not curl under daily temperature gradients. Rather, as the temperature drops during a day or during a season, the pavement contracts which builds up a thermal tensile stress in the asphalt concrete. This thermal stress is responsible for the transverse cracking seen in the Northern climates. This cracking can also develop in more temperate locations when a stiff asphalt cement is used. Numerous studies by McLeod (12) have addressed the problem in the Northern areas. Studies by Shahin (20), Carpenter (21, 16) and Ruth (22) have documented the potential for problems to develop in the Southern states under daily temperature cycles. Asphalt concrete has a thermal coefficient of contraction that is much larger than portland cement concrete, and the stresses that develop are often sufficient to crack the asphalt concrete. A typical contraction coefficient for asphalt concrete is 5×10^{-5} in/in/F. Procedures exist for evaluating the potential for asphaltic concrete mixtures to develop low temperature cracking by calculating the limiting stiffness temperature (23). These references should be consulted if temperature cracking is expected in any area.

4.8.2 Stripping

Asphalt cement aggregate combinations all have varying sensitivity to the stripping phenomenon. Stripping is the separation of the asphalt film from the aggregate surface in the presence of moisture. This separation eliminates the bonding of the asphalt film which reduces the modulus, tensile strength, and load-carrying capacity of the mixture. A pavement design cannot be done with any degree of certainty if the quality of the mixture is not satisfactory. Testing must be conducted to determine if any additives should be considered to reduce the potential for stripping to develop.

The tests available include simple immersion tests, freeze thaw tests on specially prepared samples (24) and complicated vacuum saturation freeze thaw procedures designed specifically to model the development of stripping expected to develop in the field. This last procedure developed by Lottman can be used to determine the gradual decrease in load carrying capacity as determined from the diametral resilient modulus test. If the modulus is expected to decrease over the life of the pavement, the reduced modulus can be used if desired.

There are additives which can be used if the testing indicates a potential for stripping. These additives are commonly an organic compound which alters the surface chemistry of the asphalt/aggregate combination to better resist the penetration of moisture into the interface. Lime can also be used as a filler that will increase the resistance to stripping (25).
None of these additives should be used without laboratory testing with the materials to be used in the mixture.

4.9 Structural Layer Coefficients

The structural layer coefficient for asphalt concrete was used in the Interim Design Guide to select asphalt concrete thicknesses. Figure 2-3.14 shows the relationships for asphaltic concrete, various tests, and the layer coefficient. With the resilient modulus value being used more prevalently now, the relationship shown in Figure 2-3.15 can be used to relate modulus to coefficient. Different mixes will have different modulus values, and hence a different effectiveness in the design process. Every mix should be investigated to determine the relationship to be used in any particular state.

5.0 CHARACTERISTICS OF BASE COURSE MATERIALS

5.1 Base Course

The characteristics of base and subbase materials vary depending upon whether these materials are unbound, such as granular bases, or bound materials, such as cement treated or asphalt treated materials. In this section the test properties of the granular materials influencing the design process are briefly reviewed for the different types of base materials which could be used in construction of different pavements.

5.2 Soil Aggregate Mixtures

The design parameters for soil-aggregate mixtures are strength, modulus of resilience, and permeability requirements.

5.2.1 Strength

The stability of a soil aggregate mixture depends upon the particle size distribution, relative density, internal friction, and cohesion. The granular base or subbase is designed for maximum stability and high internal friction. The particle size distribution and grain to grain contact provide the necessary shearing resistance. The strength of unbound base and subbase materials is most often presented by CBR, triaxial tests or R values. The CBR requirements for subbase and base course materials are presented in Table 2-3.4. The test procedures used on base materials are the same as described earlier.

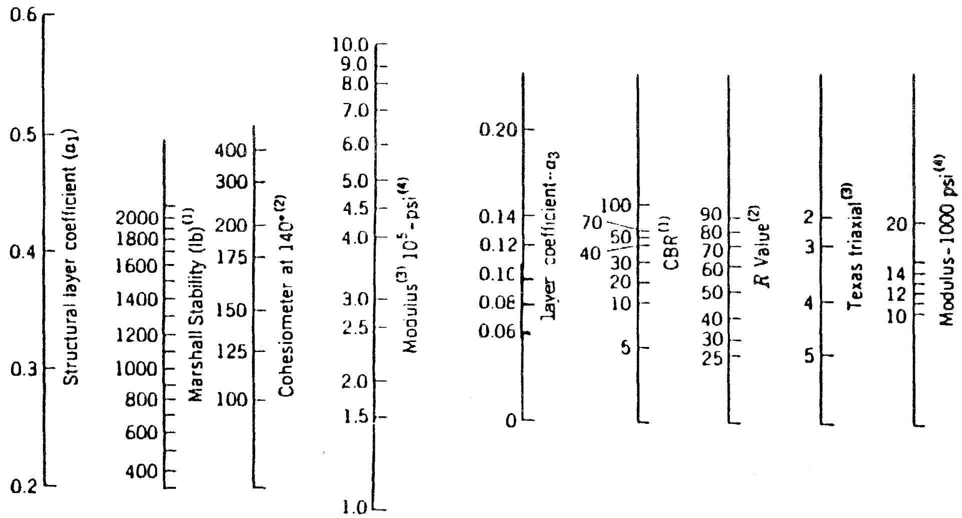
In the design process, the CBR of granular base is related to the CBR values of the underlying roadbed soils as given by:

$$CBR_{\text{base}} = F \times CBR_{\text{roadbed soil}}$$

The relationship between F and roadbed soil CBR is shown in Figure 2-3.16.

5.2.2 Modulus of Resilience

The modulus of resilience of the granular material is highly dependent on the state of stress. Just as the fine-grained cohesive soils of the roadbed were altered by stress, the modulus of granular base materials is stress sensitive but in the same manner as the coarse-grained roadbed soil. The typical resilient modulus data for granular bases is given by:



(1) Scale derived by averaging correlations obtained from Asphalt Institute, IL; LA, NM, & WY
(2) Scale derived by averaging correlations from CA & Texas
(3) Scale derived on this project
(4) Modulus at $68^{O}F$

(a)

(1) Scale derived from correlations from IL.
(2) Scale derived from correlations from the Asphalt Institute, CA, NM, & WY
(3) Scale derived from correlations from Texas
(4) Scale derived on this project

(b)

Figure 2-3.14. AASHTO Structural Layer Coefficient related to Other Asphaltic Concrete Tests.

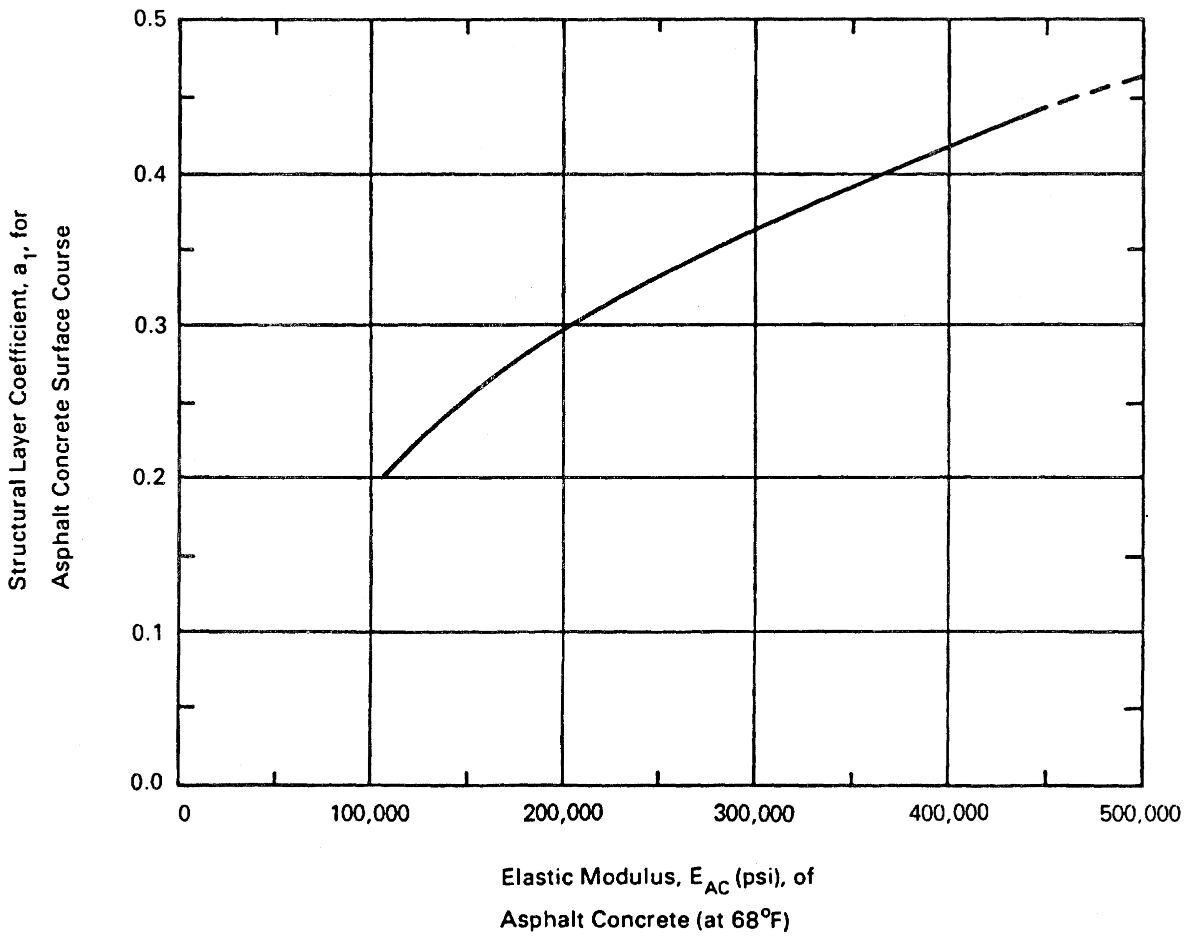


Figure 2-3.15. Resilient Modulus of Asphaltic Concrete Related to Structural Layer Coefficient.

Table 2-3.4. CBR and Design Requirements for Base and Subbase

			Maximum Permissible Values			
			Gradation Requirements Percent Passing			
Material	Max. Design CBR	Size	#10	#200	LL	PI
Subbase	50	3"	50	15	25	5
Subbase	40	3"	80	15	25	5
Subbase	30	3"	100	15	25	5
Select Matls	20	3"	-	25	35	12

Base Type	Design CBR
Graded Crushed Aggregate	100
Water-Bound Macadam	100
Dry-Bound Macadam	100
Bituminousu intermediate and surface courses, central plant hot-mix	100
Limerock	80-100
Stabilized Aggregate	80

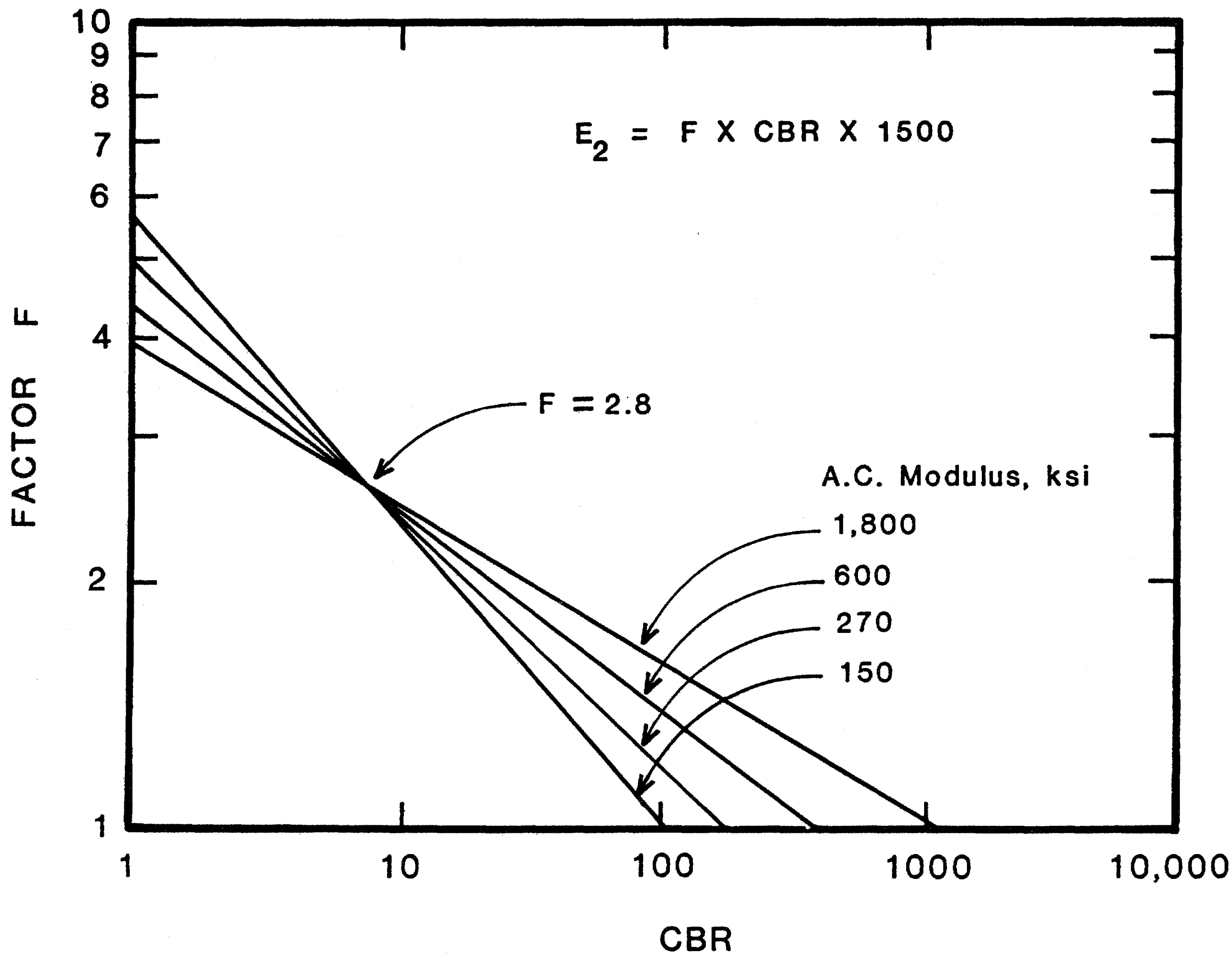


Figure 2-3.16. Assumed Relationship Between Subgrade and Base Modulus. (10)

$$M_R = K_1(\theta_3)^{K_2}$$

where:

θ_3 = confining pressure, psi (often $\sigma_1+\sigma_2+\sigma_3$ is used)

K1, K2 = regression constants

A typical resilient modulus curve for a granular soil is shown in Figure 2-3.17. Typical confining pressure values range from low stress levels of 5 psi to high stress levels of 50 psi depending on the loading and layer thicknesses. This figure has the added feature of showing the effect of base contamination by soil fines which can reduce the modulus of the base, shortening the life of the pavement. Average material coefficients are: K_1 = 9600 and K_2 = 0.55. Using these coefficients, the modulus for the low stress condition is 23,265 psi, and 82550 psi for high stress levels.

The modulus of the base is dependent on the support provided by the roadbed soil, and an average modulus can be selected using:

$$E_{base} = K \times E_{roadbed\ soil}$$

where K values are as follows:

K	E (roadbed),psi
3.5 - 4.8	3,000
2.4 - 2.7	6,000
1.8 - 1.9	12,000
1.6 - 1.8	20,000
1.5 - 1.7	30,000

5.3 Cement-Treated Bases

Cement-treated bases are used under both asphaltic concrete and rigid portland cement concrete pavements. The design of cement-treated bases are based on minimum strength requirements and resistance to freeze and thaw.

The resistance to freeze and thaw is measured by the percent loss of the sample after subjecting the specimen to 12 cycles of freezing at 18 degrees F and thawing for one day. The strength criterion is expressed as a minimum 7 day compressive strength, as shown in Figure 2-3.18. The compressive strength of cement-treated bases is influenced by the dust ratio, which is defined as a ratio of the percent passing Number 200 sieve to the percent passing Number 30 sieve. Such a relationship is shown in Figure 2-3.19.

5.3.1 Strength

The unconfined compression test has been adopted by many agencies to ensure a durable cement-treated mixture. The minimum 7-day compressive strength for these mixes can be estimated using the nomograph illustrated in Figure 2-3.18

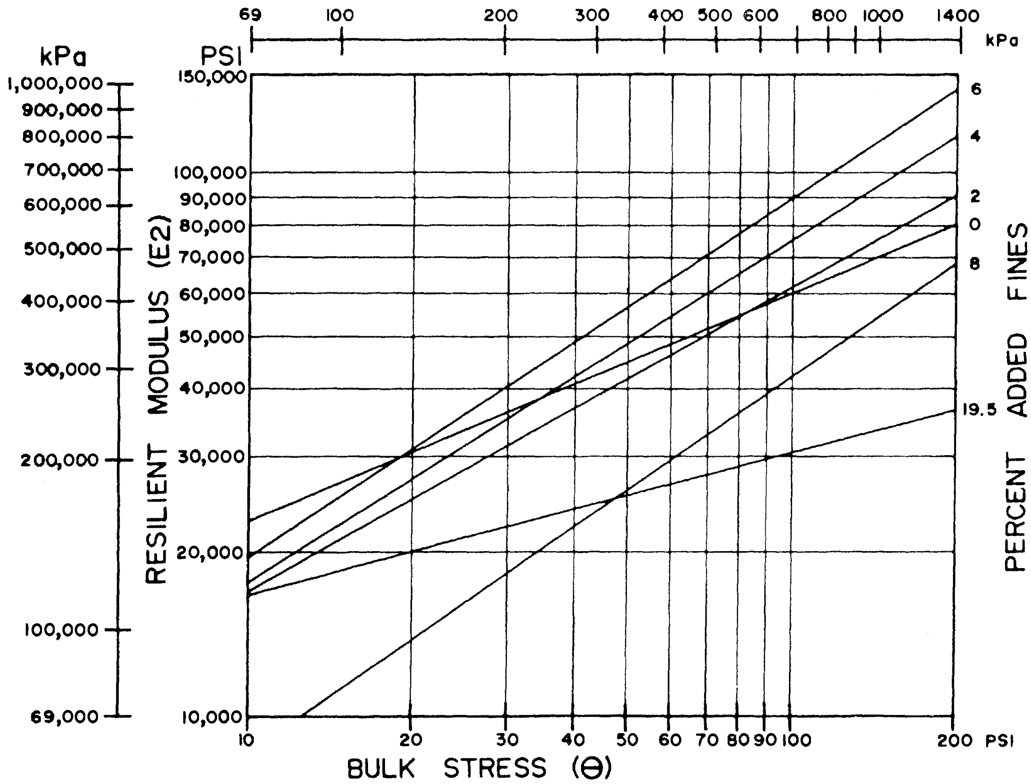


Figure 2-3.17. Resilient Modulus for Aggregate Base Course With Added Fines, Above Original 5.5 Percent (26).

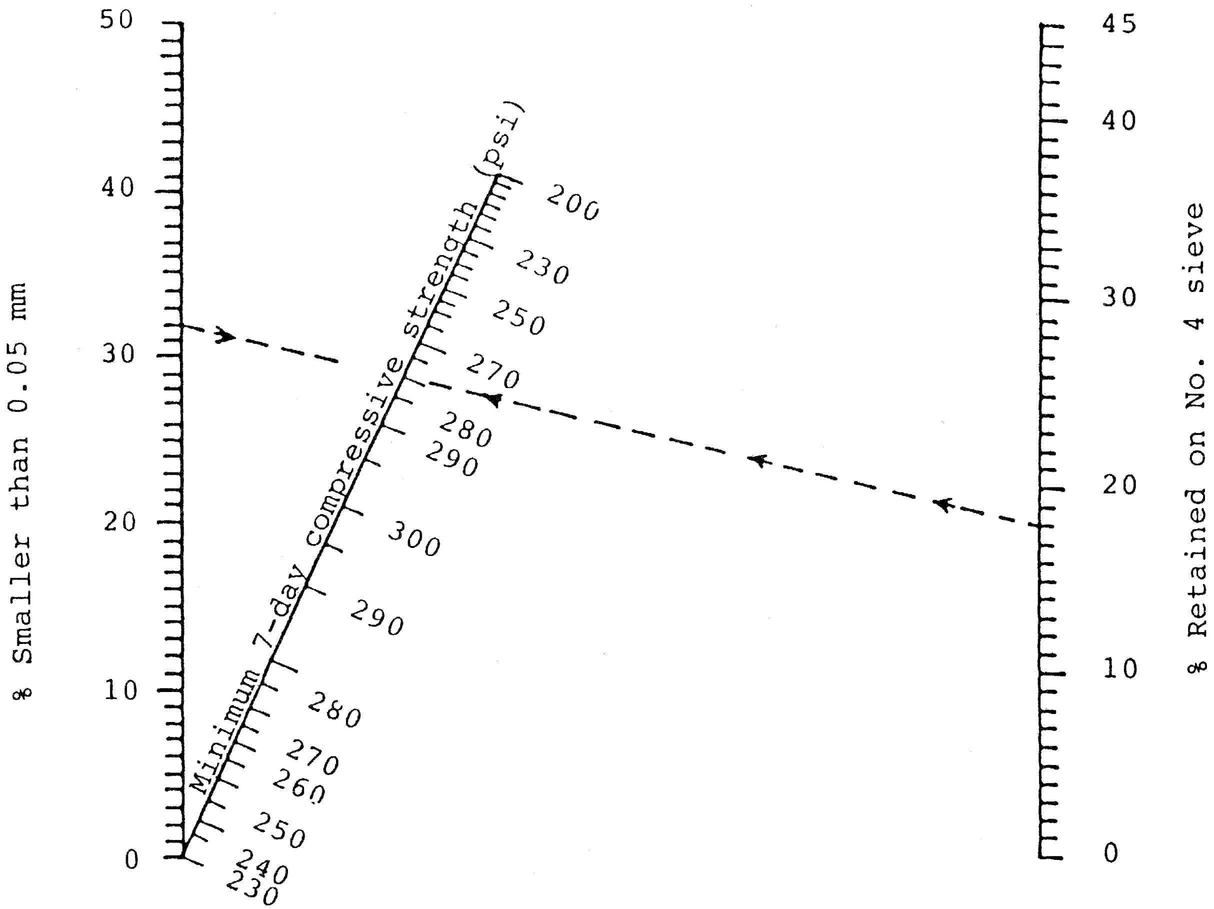


Figure 2-3.18. Minimum 7-day Compressive Strengths Required for Soil Cement Mixtures Containing Material Retained on the No. 4 Sieve (From Portland Cement Assoc.)

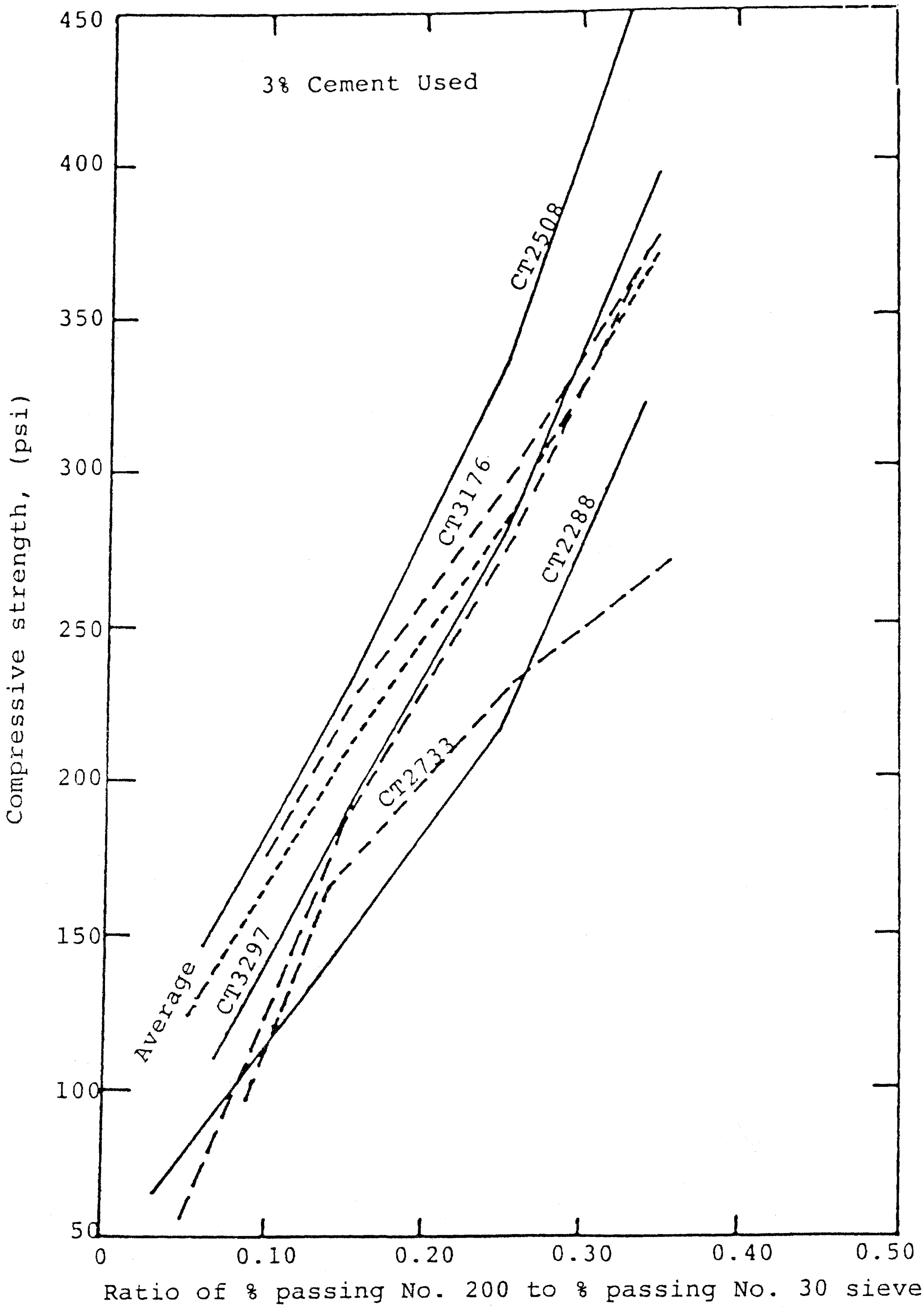


Figure 2-3.19. Effect of the Dust Ratio on Compressive Strength of Several Cement-Treated Bases (After Hveem and Zube)

5.3.2 Modulus

The modulus values for cement-treated bases are dependent upon the soil type, properties, and cement content. The modulus of these mixtures is independent of applied stress as these materials are linear elastic. The modulus of elasticity will, over time, increase due to the pozzolanic reaction. The modulus of elasticity of soil cement materials ranges from 50,000 psi to 2,000,000 psi, while for cement-treated bases it ranges between 1,000,000 and 3,000,000 psi.

While these strengths can be determined from nomographs as shown, it is recommended that laboratory testing be conducted with the actual soils and additives to determine the average strength or modulus to be used in the design.

5.4 Asphalt-Treated Bases

Asphalt-treated bases have been extensively used in all pavement types (10). These materials are designed with consideration to increase structural strength, resistance to pumping, and to provide drainage capabilities. The principles underlying the design of these mixtures are the same as those underlying the design of asphaltic concrete.

Asphalt-treated bases can also be constructed using emulsified asphalts. The modulus of resilience of these emulsion aggregate mixtures falls between granular and asphaltic concrete mixtures, ranging from 80,000 to 500,000 psi. The following equation can be used as an estimate for the resilient modulus of these mixtures:

$$\ln(M_R x 10^{-3}) = 0.4\gamma + 2.46(SF) - 0.015(Pen) - 1.13$$

where:

γ = Density, pound per cubic foot

SF = Sand fraction, percent

Pen = Asphalt penetration at $77^{o}F$

For a mixture with 4 percent sand at 140 pounds/cubic foot density, the M_R estimated by this equation is 200,000 psi.

These asphalt-treated mixes used as base courses must be carefully evaluated for moisture resistance because they typically must function as a drainage layer and be continually exposed to moisture.

5.5 Structural Layer Coefficients

The strength tests, and modulus values must be converted to structural layer coefficients to be used in the AASHTO Design procedure. The modulus values can be used directly in the mechanistic empirical design procedures. The charts shown in Figure 2-3.20 and Figure 2-3.21 can be used to translate any test value for these granular materials into a structural layer coefficient.

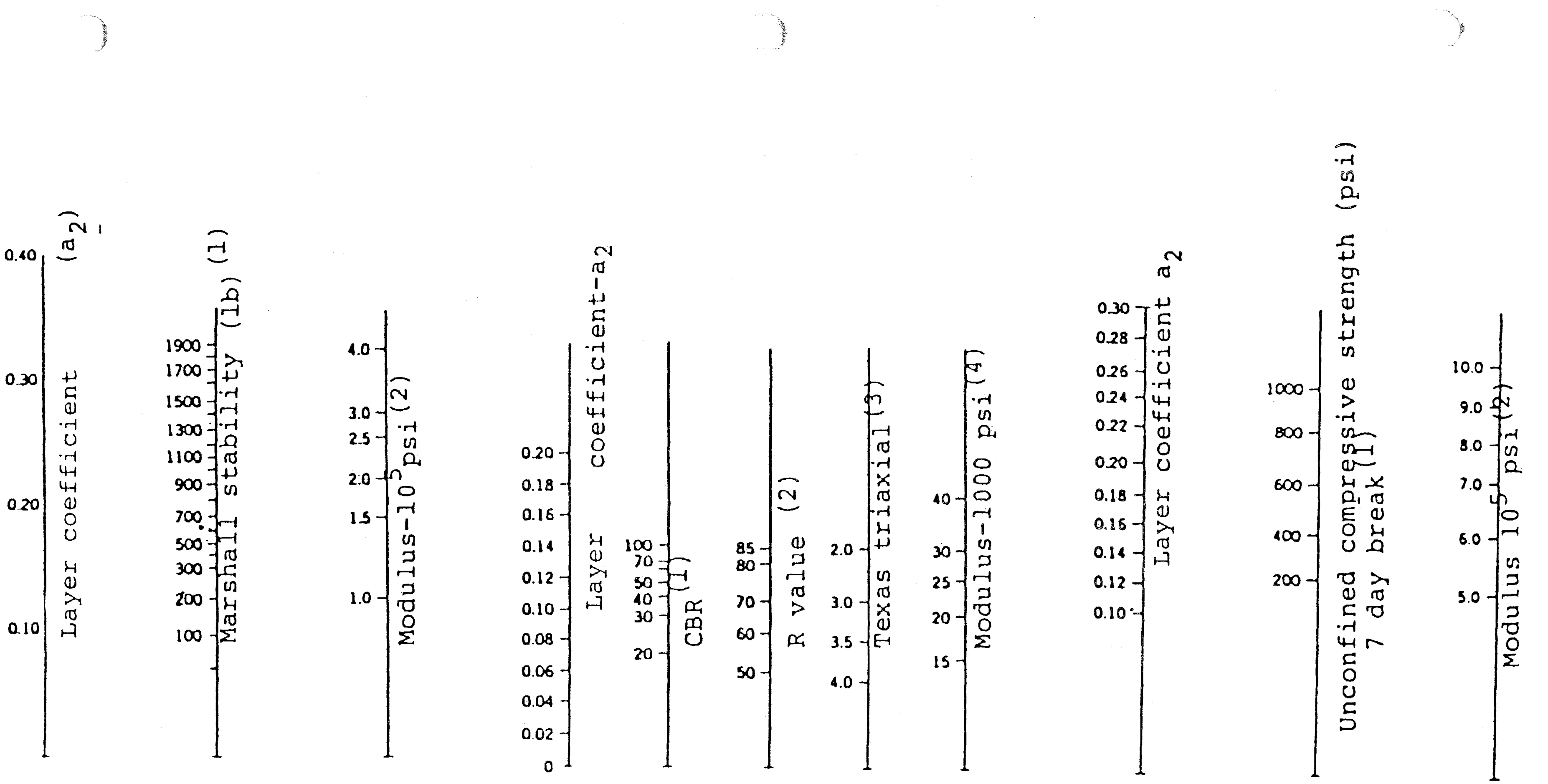
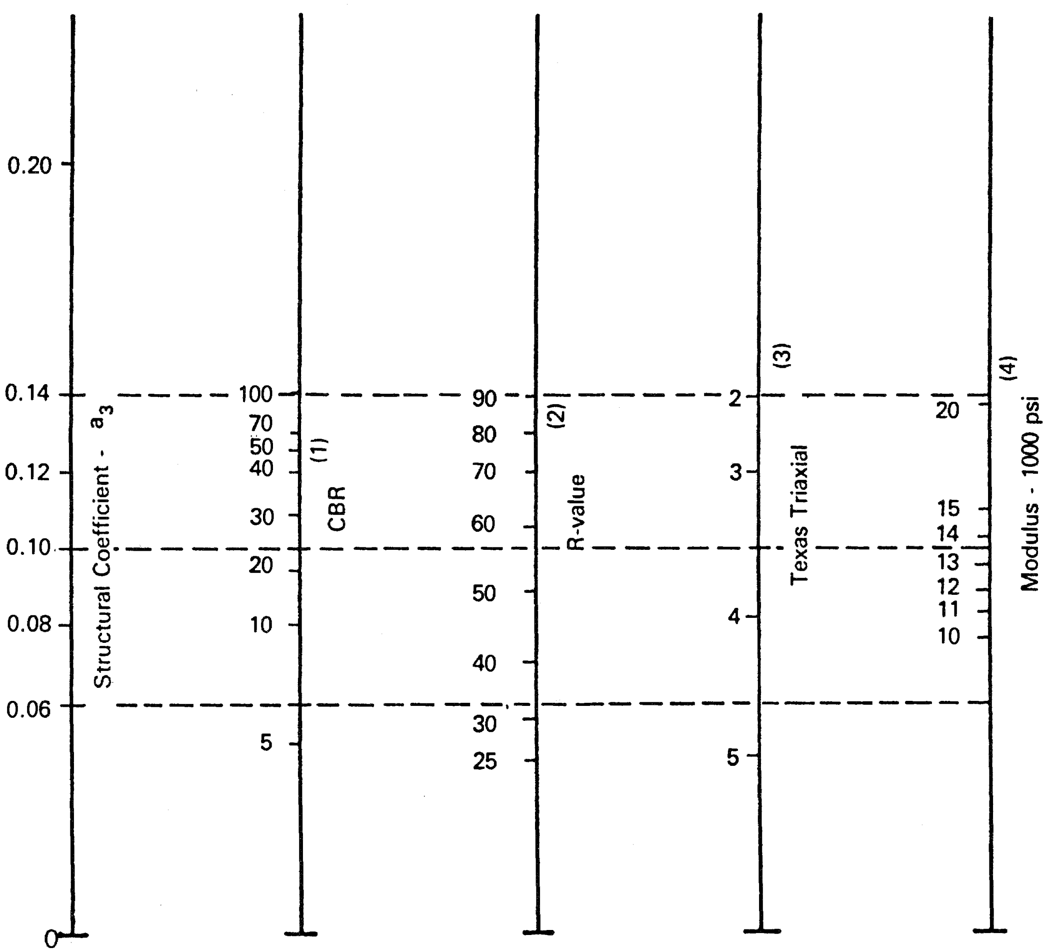


Figure 2-3.20. Suggested AASHO layer coefficient nomographs. (a) Variation in a_1 with surface course strength parameters; (b) variation in a_3 for granular subbase and subbase strength parameters; (c) variation in a_2 for bituminous-treated bases with base strength parameters; (d) variation in granular coefficients a_2 with base strength parameters; and (e) variation in a_2 for cement treated bases with base strength parameter

Use: Enter any of the appropriate laboratory determined parameter on the right hand scale and read off the layer coefficient on the left scale. Laboratory determined parameters are: marshall stability, moduli of resilience, R value, Texas triaxial value, or unconfined compressive strength.



(1) Scale derived from correlations from Illinois.

(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.

(3) Scale derived from correlations obtained from Texas.

(4) Scale derived on NCHRP project *(3)*.

Figure 2-3.21. Suggested AASHTO Layer Coefficient Nomographs for Subbase Material.

5.5.1 Special Considerations for Rigid Pavements

The structural layer coefficients are used directly in the structural number concept for flexible pavements. In a rigid pavement design, there is no structural number calculation using the layer coefficients and thicknesses. The roadbed soil and subbase are combined into an effective support for the portland cement concrete slab termed a composite modulus of subgrade reaction, k. The resilient modulus of the roadbed soil and the resilient modulus of the subbase are required. The roadbed soil's resilient modulus can be determined as described in Module 2-2. The resilient modulus of the subbase can be determined from the nomographs presented in Figure 2-3.20 and Figure 2-3.21 for a base material. The two modulus values can be converted into the composite modulus of subgrade reaction using Figure 2-3.22.

Because of the necessity of having resilient modulus for the granular subbase material, it is highly recommended that AASHTO T274 testing be implemented to develop accurate modulus values to go along with the material test values which may already be catalogued on the materials being used in the state.

It must be recognized that the subbase under a rigid pavement must contain different properties than a subbase or base material for a flexible pavement. The subbase material must resist erosion which leads to a loss of support. This loss of support is a critical element in the design procedure for rigid pavements. Subbase materials for rigid pavements must not contain fines which can be eroded. They should be free draining and/or stabilized to resist pumping and faulting. This requirement typically calls for the use of very different materials than have normally been used in the design of a flexible pavement, increasing the importance of having accurate characterization of the resilient modulus of the material.

6.0 EXAMPLE PROBLEMS

6.1 Determine the load applications to failure in a PCC pavement if the modulus of rupture is 750 psi, and the tensile stress in the slab under load is 540 psi using the PCA curve.

Answer: 100,000 loadings

6.2 Calculate the tensile strength for a AC-20 asphalt concrete mixture if the load in the test is 2200 pounds. The sample has a diameter of 4 inches, and a thickness of 2.5 inches.

Answer: 140 psi.

6.3 Determine the allowable number of loadings using the AASHTO/ARE fatigue equation for asphalt concrete if the radial strain at the bottom of the asphalt concrete is 0.0001 in/in.

Answer: 4.25×10^6 loadings

6.4 If a pavement is to carry 10 million loadings, what must the maximum vertical strain on top of the subgrade be?

Answer: 0.000293 in/in

Example:

D_{SB} = 6 inches

E_{SB} = 20,000 psi

M_R = 7,000 psi

Solution: k_∞ = 400 pci

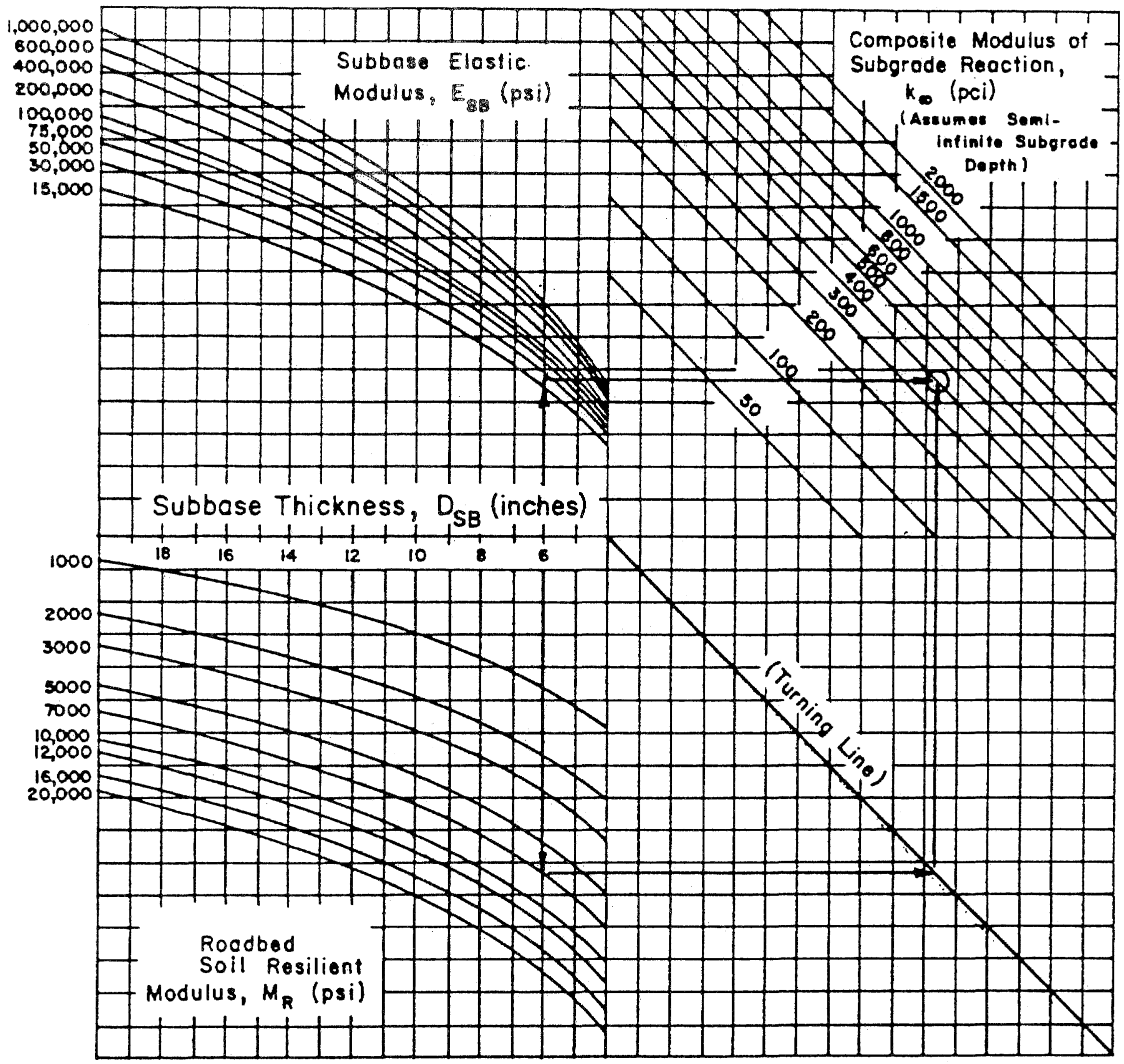


Figure 2-3.22. Chart for estimating composite modulus of subgrade reaction, k_∞, assuming a semi-infinite subgrade depth. (For practical purposes, a semi-infinite depth is considered to be greater than 10 feet below the surface of the subgrade.)

6.5 What modulus values would you use for asphalt concrete if you had the following structural layer coefficients?

a. 0.44
b. 0.22
c. 0.46

Answer:
a. 450,000 psi
b. 125,000 psi
c. 500,000 psi

6.6 What Marshall Stability would you expect with a structural layer coefficient of 0.4?

Answer: 1700

6.7 What layer coefficient and resilient modulus would you expect to have for a cement-treated base with a seven day compressive strength of 400 psi.

Answer: coefficient = .15
modulus = 600,000 psi

6.8 For a subbase modulus of 10,000 psi, what layer coefficient would you expect to have.

Answer: 0.08

7.0 SUMMARY

This module has presented a summary of the properties of pavement components that have an impact on pavement design and performance, including the selection of materials for construction use, design input requirements for each material, physical and engineering properties, and various response parameters.

In concrete pavements, for design purposes, the allowable stress is calculated using the modulus of rupture, which is taken as the extreme fiber stress under breaking load. The rigidity of the pavement slab and its ability to distribute loads is represented by the concrete modulus of elasticity, Ec. Concrete pavement performance is directly influenced by this property which is a function of the compressive strength and varies with mixture variables, time and temperature.

Pavement distress is affected by many factors, such as loads and stress, environmental conditions, material properties, construction and maintenance methods. Fatigue cracking is one of the two major traffic-associated failure distress modes for asphaltic pavements. Various test methods for fatigue were described. In the review of pavement distress manifestations, it was noted that distresses are related to governing parameters in a very complex way and that the mechanisms are not well understood. Predictive distress models have been developed, however, for both rigid and flexible pavements.

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MODULE 2-4

DRAINAGE DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module presents the principles of drainage and the influence of moisture on material performance. The interaction of moisture with materials is presented with sensitivity analyses to show the impact of this interaction on pavement performance and thickness design. An evaluation is presented which allows the engineer to determine whether drainage will improve the performance of a pavement or not. Design procedures are presented and the requirements for constructing an effective drainage system are discussed. The principles and reasons for incorporating subdrainage into the AASHTO Design Guide are presented.

Upon completion of this module the participants will be able to accomplish the following:

1. List the sources of moisture in a pavement and identify the influence of moisture on different materials by describing distresses which could result.

2. Describe different drainage systems and the sources of moisture they are designed to remove.

3. Identify material requirements needed to ensure adequate performance in the drainage installation.

4 List the steps required in designing a subdrainage system and be able to use the subdrainage design manual to design the system.

5. List the steps in the procedure to calculate drainability of the pavement system and select m for flexible pavements, or C_d and loss of support factors for rigid pavements.

2.0 PAVEMENT DRAINAGE CONCEPTS

2.1 Introduction

Water can produce detrimental effects on a highway in several different ways. Most failures caused by ground water and seepage (1) can be classified into two categories:

1. Those which take place when soil particles migrate to an escape exit, causing piping or erosional failures.

2. Those which are caused by uncontrolled seepage patterns and lead to saturation, internal flooding, excessive uplift, or excessive seepage forces.

Failures caused by surface infiltration generally result from continual exposure to moisture, and can be placed in two categories:

1. Softening of foundation layers as they become saturated and remain saturated for prolonged periods of time.

2. Degradation of material quality from the interaction of an increased moisture content with the environment, stripping and D-Cracking.

A given pavement can be stable at a given moisture content but become unstable if the soils become saturated. High water pressures can develop in saturated soils when subjected to dynamic loadings. Subsurface water can freeze, expand and exert forces of considerable magnitude on a given pavement. Water in motion can carry soil particles, causing any number of different problems, from clogging drains to eroding embankments. These circumstances must be recognized and accounted for in the design of a pavement.

In the design of a highway, a major objective should be to keep the base, subbase, subgrade, and/or other specific paving materials from becoming saturated or even exposed to constant high moisture levels which may be below saturation. There are three approaches which should be considered for controlling or eliminating the problems caused by moisture:

1. Seal the pavement properly and do not allow the water to enter the pavement layers.

   a. Use proper sealing materials and techniques for concrete slab joints and seal cracks in flexible pavements.
   b Design utilizing impervious membranes.
   c. Use impervious wearing surfaces, bases, subbases, and impervious shoulders.
   d. Install interceptor drains to prevent moisture from entering a pavement section.

2. Use materials that are moisture insensitive and will not contribute to moisture-related distress.

   a. Use stabilized materials for granular layers (lime, cement, bituminous).
   b. Select granular materials with low fines, and low plasticity which resist the effects of moisture better than dense-graded materials.

3. Provide adequate drainage to effectively remove any moisture that may enter the pavement from the materials before damage can be initiated.

   a. Design a drainage system which permanently lowers the water table under a given pavement or adequately removes any infiltration which is seen to enter the pavement system.
   b. Use pervious bases and subbases designed not only as structural components but also as drainage layers. Water which enters the pavement from above will drain in the horizontal direction from beneath the highway rather than continuing downward into the subgrade.
   c. Add drainage blankets beneath embankment sections.

It should be noted that the above solutions require adequate surface water drainage facilities. To accomplish the second concept above, a detailed understanding of material behavior as related to moisture must be developed. There are a number of new materials which have reduced susceptibility to moisture damage, which will be discussed in the following sections. However, when working with local materials within a given economic condition it is not always possible to justify the use of special materials. Additionally, good maintenance practices to maintain an impermeable surface are not always obtained. Thus, it is always recommended that adequate drainage be provided if it can be demonstrated that drainage is required to maintain a high level of performance in the pavement.

2.2 Moisture-Induced Pavement Distress

Surface infiltration, high groundwater, capillary rise, and excess seepage water are primary causes of pavement distress. Moisture-related flexible pavement failures are characterized by excessive deflection, cracking, reduced load-bearing capacity, raveling, and disintegration. A simplified description of moisture related distress in flexible pavements are given in Figure 2-4.1 (14). Subgrade instability, pumping and the subsequent loss of support, as well as deterioration of concrete due to the "D" cracking phenomenon are common indicators of moisture-induced damage in rigid pavements. Distresses are presented in Figure 2-4.2 for rigid pavements (14).

The influence of moisture on the load-carrying capacity of subgrades has long been recognized. The classical pavement design methods are based on a saturated subgrade strength and the resulting loss of support due to the excessive moisture. Pulsating pore pressures developed in a subgrade as a result of moving loads significantly influence the subgrade's load-carrying capacity. According to Cedergren (1), it is possible that a saturated subgrade might become supersaturated where the water holds the soil particles apart, resulting in a complete loss of soil strength.

Studies of moisture-induced damage in flexible pavements (4) confirm that the strength and moduli of asphaltic concrete mixtures are adversely affected by the presence of moisture. Pavement structural evaluations conducted at the University of Illinois (5) using a circular test tract have similarly confirmed that wheel loads on flooded sections are many times more damaging than those on a dry pavement.

Free water at the subgrade-pavement interface has been similarly identified as a significant parameter contributing to pavement deterioration. Cedergren indicated that the moving pressure waves created by dynamic loads develop large hydrostatic pressures resulting in the movement of soil particles at the pavement interface (1,2). Studies in Georgia suggest that soil and subbase particles are indeed displaced at the pavement interface and near joints under the effect of moving loads and that cavities and void spaces are developed (3). Such a condition is visually observed in pumping of subbase fines in a rigid pavement, an action that leads to loss of support of the pavement slab, thereby allowing cracking and failure of the pavement structure.

TYPE	DISTRESS MANIFESTATION	MOISTURE PROBLEM	CLIMATIC PROBLEM	MATERIAL PROBLEM	LOAD ASSOCIATED	STRUCTURAL DEFECT BEGINS IN		
						ASPHALT	BASE	SUBGRADE
SURFACE DEFECT	ABRASION	NO	NO	AGGREGATE	NO	YES	NO	NO
	BLEEDING	NO	ACCENTUATED BY HIGH TEMP.	BITUMEN	NO	YES	NO	NO
	RAVELLING	NO	NO	AGGREGATE	SLIGHTLY	YES	NO	NO
	WEATHERING	NO	HUMIDITY AND LIGHT-DRIED BITUMEN	BITUMEN	NO	YES	NO	NO
SURFACE DEFORMATION	BUMP OR DISTORTION	EXCESS MOISTURE	FROST HEAVE	STRENGTH-MOISTURE	YES	NO	YES	YES
	CORRUGATION OR RIPPLING	SLIGHT	CLIMATIC & SUCTION RELATIONS	UNSTABLE MIX	YES	YES	YES	YES
	SHOVING	NO		UNSTABLE MIX LOSS OF BOND	YES	YES	NO	NO
	RUTTING	EXCESS IN GRANULAR LAYERS	SUCTION & MATERIAL	COMPACTION PROPERTIES	YES	YES	YES	YES
	WAVES	EXCESS	SUCTION & MATERIALS	EXP. CLAY FROST. SUSC.	NO	NOT INITIALLY	NO	YES
	DEPRESSION	EXCESS	SUCTION & MATERIALS	SETTLEMENT, FILL MATERIAL	YES	NO	NO	YES
	POTHOLES	EXCESS	FROST HEAVE	STRENGTH-MOISTURE	YES	NO	YES	YES
CRACKING	LONGITUDINAL	YES	SPRING-THAW STRENGTH LOSS		YES	FAULTY CONSTRUCTION	YES	YES
	ALLIGATOR	YES DRAINAGE		POSSIBLE MIX PROBLEMS	YES	YES MIX	YES	YES
	TRANSVERSE	YES	LOW-TEMP, F-T CYCLES	THERMAL PROPERTIES	NO	YES,TEMP SUSCEPTIBLE	YES	YES
	SHRINKAGE	YES	SUCTION, MOISTURE LOSS	MOISTURE SENSITIVE	NO	YES, HARDENING	YES	YES
	SLIPPAGE	YES	NO	LOSS OF BOND	YES	YES-BOND	NO	NO

Figure 2-4.1. Moisture Related Distresses in Flexible Pavements.

TYPE	DISTRESS MANIFESTATION	MOISTURE PROBLEM	CLIMATIC PROBLEM	MATERIAL PROBLEM	LOAD ASSOCIATED	STRUCTURAL DEFECT BEGINS IN		
						SURFACE	BASE	SUBGRADE
SURFACE DEFECTS	SPALLING	POSSIBLE	NO		NO	YES	NO	NO
	SCALING	YES	F-T CYCLING	CHEMICAL INFLUENCE	NO	YES - FINISHING	NO	NO
	D-CRACKING	YES	F-T CYCLING	AGGREGATE	NO	YES	NO	NO
	CRAZING	NO	NO	RICH MORTAR	NO	YES - WEAK SURFACE	NO	NO
SURFACE DEFORMATION	BLOW-UP	NO	TEMPERATURE	THERMAL PROPERTIES	NO	YES	NO	NO
	PUMPING	YES	MOISTURE	FINES IN BASE MOISTURE SENSITIVE	YES	NO	YES	YES
	FAULTING	YES	MOISTURE-SUCTION	SETTLEMENT DEFORMATION	YES	NO	YES	YES
	CURLING	POSSIBLE	MOISTURE AND TEMP.		NO	YES	NO	NO
CRACKING	CORNER	YES	YES	FOLLOWS PUMPING	YES	NO	YES	YES
	DIAGONAL TRANSVERSE LONGITUDINAL	YES	POSSIBLE	CRACKING FOLLOWS MOISTURE BUILDUP	YES	NO	YES	YES
	PUNCH OUT	YES	YES	DEFORMATION FOLLOWING CRACKING	YES	NO	YES	YES
	JOINT	PRODUCES DAMAGE LATER	POSSIBLE	PROPER FILLER AND CLEAN JOINTS	NO	JOINT	NO	NO

Figure 2-4.2. Moisture Related Distresses in Rigid Pavements.

2.3 Source of Moisture in Pavements

Moisture in the subgrade and pavement structure can come from many sources. The water may seep upward from a high groundwater table, or it may flow laterally from the pavement edges and shoulder ditches, as shown in Figure 2-4.3. Capillary effect and moisture-vapor movement are also responsible for water accumulating beneath a pavement structure. Moisture-vapor movement is associated with fluctuating temperature and other climatic conditions. The water in a pavement can also result from infiltration through the surface. Joints, cracks, shoulder edges, and various defects in the surface represent easy access paths for water.

Many highway engineers believe that groundwater and high water tables are the primary causes of moisture-induced damage. This is evidenced in the fact that most highway departments have a practice of installing underdrain and drainage facilities primarily to remove the groundwater and lower the water table (6, 7). Despite this common belief, it can be shown that surface water is a major contributor to moisture accumulation in the subgrade. The effect of the infiltration of surface water has been directly related to the amount of precipitation and the pavement condition (7). The amount of surface infiltration depends on the permeability characteristics of pavement surface. Table 2-4.1 displays typical ranges of permeability of various old and new flexible pavements. As shown in this table, the permeability of flexible pavements decreases with pavement life as traffic seals the porous surface.

According to Cedergren's data (6), the permeability of rigid pavements, taking into account joints etc., can be assumed as 0.20 inch/hour. The permeability of flexible pavements, on the other hand, might be selected at an approximate value of 0.50 inch/hour. The permeability and porosity of each pavement layer, the base and subbase, similarly influence the outflow and storage characteristics of the total pavement structures.

3.0 BASIC PRINCIPLES OF SUBSURFACE DRAINAGE DESIGN

The analysis and design of highway subsurface drainage systems involve the consideration of subsurface water from a wide variety of sources. It is convenient to consider these sources of drainable subsurface water in two broad categories:

1. Groundwater, which is defined as the water existing in the zone of saturation at the water table.

2. Infiltration, which is defined as surface water that gets into the pavement structural section by seeping down through joints or cracks in the pavement surface, through voids in the pavement itself, or from ditches along the side of the road.

Although free water from melting ice lenses commonly exists above the water table, it is generally considered as groundwater. The water that feeds the growth of ice lenses originates at the base of the capillary fringe (i.e., at the water table); no frost action could take place without water from this source.

Table 2-4.1. Permeabilities of Old and New Asphalt Concrete Pavements (10).

SOURCE OF DATA	PERMEABILITY, K, FT/DAY
NEW PAVEMENT:	
AIR PERMEABILITY OF US 101, BY KARI-SANTUCCI	150
US 101, LEFT WHEEL PATH	46
US 101, BETWEEN LEFT AND RIGHT WHEEL PATH	90
CALIFORNIA DIVISION OF HIGHWAY SPEC.	40
CEDERGREN	
OLD PAVEMENTS:	
OLD PAVEMENTS, SOUTH AFRICA, CRACKED SURFACES	2.0
OLD PAVEMENT, BELGIUM	7.0
OLD PAVEMENT, UNIVERSITY OF CONNECTICUT, TRAFFIC LANE	4.4
OLD PAVEMENT, UNIVERSITY OF CONNECTICUT, SHOULDER	7.0

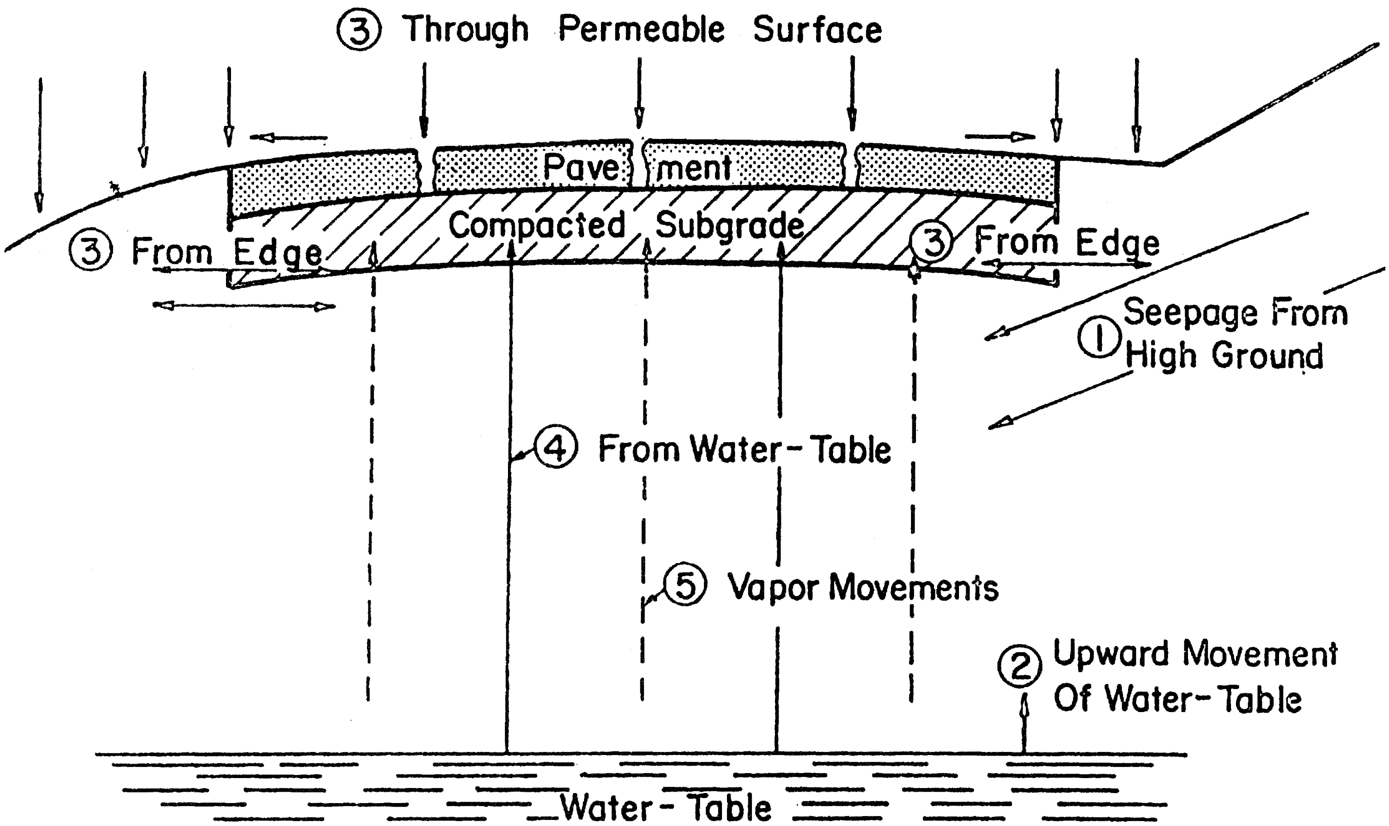


Figure 2-4.3. Sources of Moisture in Pavement Systems.

The infiltration of water into the pavement structural section would appear to be a simple phenomenon. However, the interaction between the type and frequency of openings permitting infiltration, the rate of water supply, and the permeability and ambient moisture conditions of the underlying materials is very complex. The interaction of moisture with different materials has a complex influence on design which produces another complicating factor if drainage analysis in the design process is to control distress. Thus, the estimation of the amount of infiltration that must be handled by subsurface drainage requires careful consideration.

Important considerations in the design of subsurface drainage include:

1. Seepage - the movement, or flow, of water through a permeable porous medium.

2. Porosity - the ratio of the volume of the pore spaces to the total volume of the material. The extent to which porous media will permit fluid flow is governed by the permeability of the material.

3. Permeability - the ease with which water passes through a media, is dependent upon the size, shape, and extent to which the pore spaces are interconnected (8, 9).

The coefficient of permeability varies over a very wide range, depending on the nature of the porous media through which flow is taking place. In natural deposits, and even in some compacted soils, it may be much greater in one direction than in another (8, 15, 16, 17, 18). This phenomenon should be considered, whenever possible in arriving at practical solutions in highway subdrainage problems.

Movement of groundwater in the vicinity of a highway may be the result of natural phenomena and hydraulic gradients that are the direct outgrowth of the controlling topographic, hydrologic and geological features in the area of the pavement. More often than not, however, the highway construction causes some kind of disruption to the natural pattern of moisture flow. For example, a highway cut may intersect the existing water table, or a fill may serve to dam the natural flow of groundwater. The installation of subsurface drainage to control this groundwater results in a further alteration of the flow pattern. The final configuration of the flow is dependent upon both the initial groundwater flow conditions and the characteristics of the subsurface drainage system that is installed.

The movement of infiltration within the pavement structural section is governed by the permeability of the materials used in the pavement system, the longitudinal grade of the roadway and the pavement cross (transverse) slope. The general patterns of surface and subsurface flow associated with infiltration are shown for a portland cement concrete pavement in Figure 2-4.4. Although the joint and crack patterns (points of inflow) are different for a bituminous concrete pavement, the geometry and subsurface flow are essentially the same as shown in Figure 2-4.4.

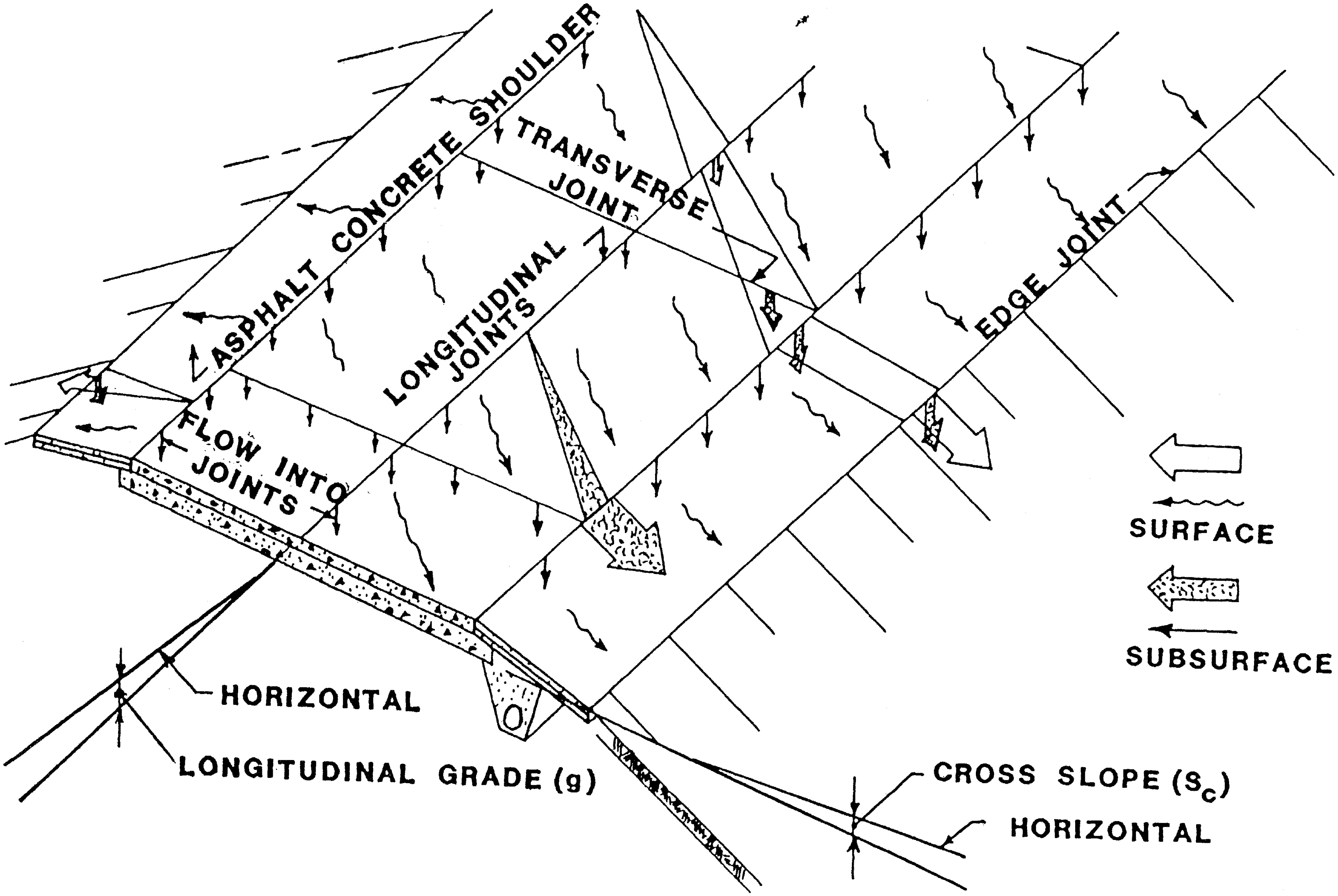


Figure 2-4.4. Paths of Flow of Surface and Subsurface Water in PCC Pavement.

3.1 Drainage Requirements

A rational design strategy that satisfies the requirements of a long-lasting subsurface drainage system should incorporate the following design criteria:

1. The pavement system including its shoulders and adjacent areas should be designed and maintained as impervious as possible to minimize the infiltration of surface, capillary and groundwater into critical areas.

2. To minimize moisture-induced damage the drainage facility should be designed with a water-removing capability such that infiltrating water can be removed in a very short period of time.

3. The drainage system designed must be a structural member of the pavement structure. It must not decrease the performance of the pavement or require exceptional measures to compensate for material problems.

The time considered adequate for a drainage system to remove water from a pavement system depends upon the allowable severity of moisture-induced damage and prevailing climatic conditions. These two factors represent the intrinsic and extrinsic areas of study in drainage of a pavement. The amount of water to be removed is the extrinsic factor, and the material properties which impede or assist this removal of water are the intrinsic factors. In areas with an expected freeze effect, the flooded pavements should be drained within a half-hour to one hour period to minimize the long term effect of moisture presence in the pavement system. As a comparison, a typical pavement structure without any effective drainage system generally require as long as 20 to 50 hours to drain.

For a pavement to satisfy its structural requirements, the drainage system must be designed as an integral part of the pavement structure. This requires that the structural properties of the materials used in the drainage layer be carefully determined before they are used. Often the materials best suited for drainage require special construction practices or material handling precautions. The presence of a drainage layer should not adversely affect the structural performance of the roadway, and should actually improve its performance by decreasing the time the pavement will be exposed to moisture.

The procedures for the determination of the total amount of water which must be removed from a pavement section will be discussed in detail in a later section of this module where specific design criteria will be developed. Once the total quantity of moisture required to be handled by the drainage system has been determined, the material properties must be used to size the drainage layers to ensure they are capable of handling the water. The material properties which are altered by the presence of moisture in a pavement system, and the magnitude of this alteration, represent an area where a great deal of uncertainty exists in current design philosophy. The effect of water on design can be quantified only when the amount of water entering the pavement can be quantified.

3.2 Sources of Water Inflow

3.2.1 Groundwater

Groundwater may come from gravity drainage (q) or from artesian flow (q_a). These flow quantities can be computed by means of electric analogs, hydraulic models, numerical methods, or by graphical flow nets. The use of flow nets illustrated in Figure 2-4.5 allows total seepage quantities to be estimated from a series of equations. The use of flow nets is detailed in the FHWA Highway Subdrainage Design Manual (20) which should be required reading for any design problem.

3.2.2 Melt Water from Ice Lenses

In areas subject to deep frost penetration, the potential for frost heave is great, and when certain soils are present, the amount of frost heave can be significant. Frost heave is the result of water being drawn up from the water table, through the soil to the freezing front at the depth of frost penetration. When a winter is cold for extended periods of time, and the frost line is stationary, the amount of moisture drawn up to the frost line can be significant. As the water freezes, the pavement surface heaves. When the water thaws during the spring there will be an excess of water which was not there previously, and which should be drained away as quickly as possible. The quantity of water that accumulates in the form of ice in a pavement subgrade as a result of frost is a function of the subgrade soil types, availability of groundwater and severity and duration of the freezing temperature.

In the Highway Drainage Manual (20), it is shown that the amount of water from a melting ice lens, q_m can be determined from Figure 2-4.6 and a value of the heave rate or frost susceptibility classification shown in Figure 2-4.7. The value σp in Figure 2-4.6 is the subgrade stress (pcf).

3.2.3 Vertical Outflow

Water that enters a pavement system will seep out of the pavement layers through the underlying soil strata. The rapidity with which this seepage occurs is a direct function of the permeability and moisture characteristics of the subgrade soil. The flow (q_v) can be easily computed through the use of an equation or estimated from a chart such as that shown in Figure 2-4.8.

While the average hydraulic gradient can be expected to decrease considerably from its initial value of unity, it can sometimes remain at a relatively high value for some time following the initiation of flow. It should be recognized that the effect of infiltration, other than that introduced through the pavement, has been ignored. In reality, rainfall of long duration, which could be expected to produce infiltration through the pavement for prolonged period of time, would also produce downward percolation through the surrounding soil. This percolation would raise the water table reducing the outflow from the pavement section. Consequently, it is recommended that caution be exercised in applying the above method to estimate vertical outflow toward an underlying horizontal water table. If it can be demonstrated with reliability that the water table will remain well below the level of the pavement even during prolonged wet weather, then

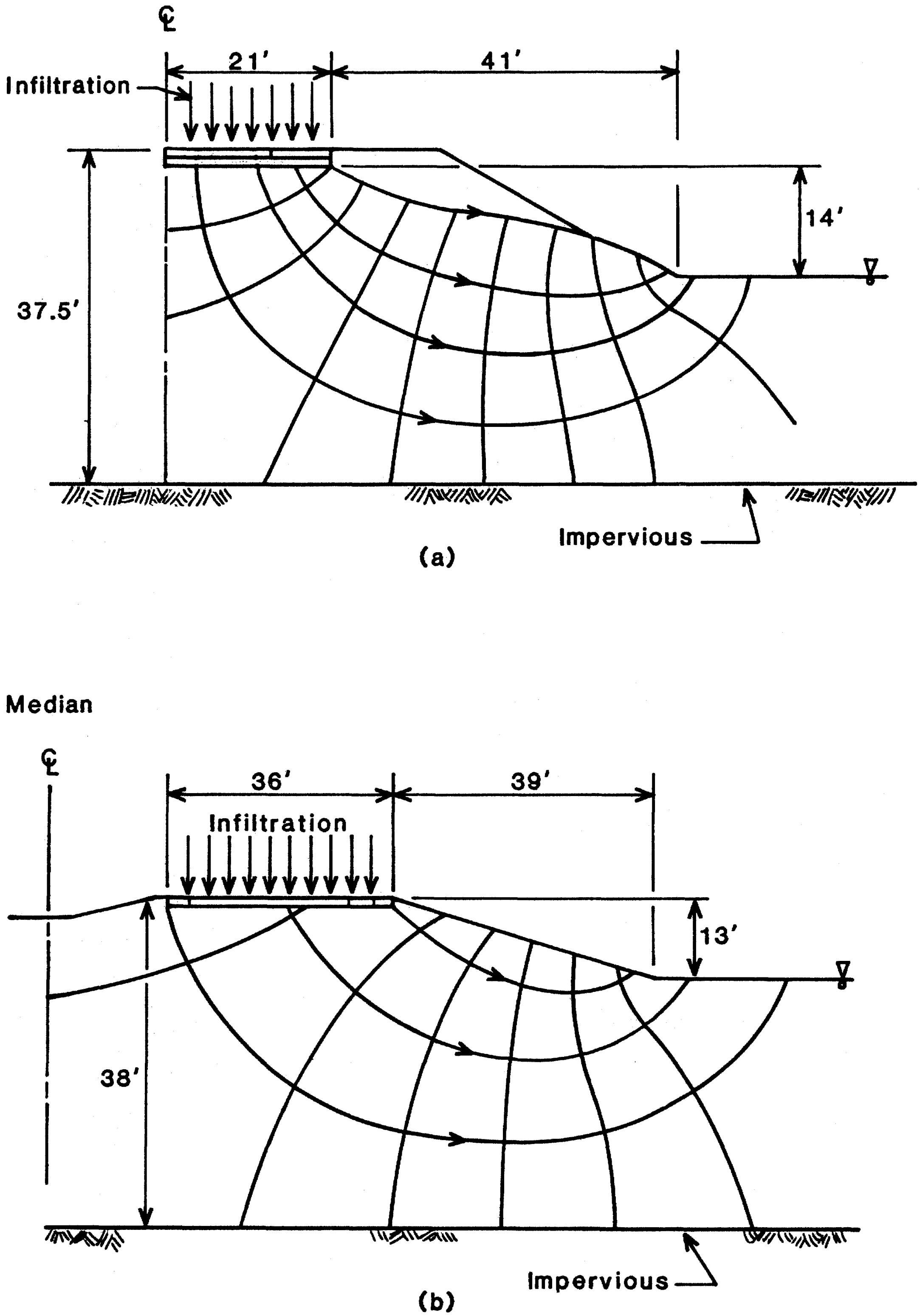


Figure 2-4.5. Flow Nets for Calculating Vertical Flow.

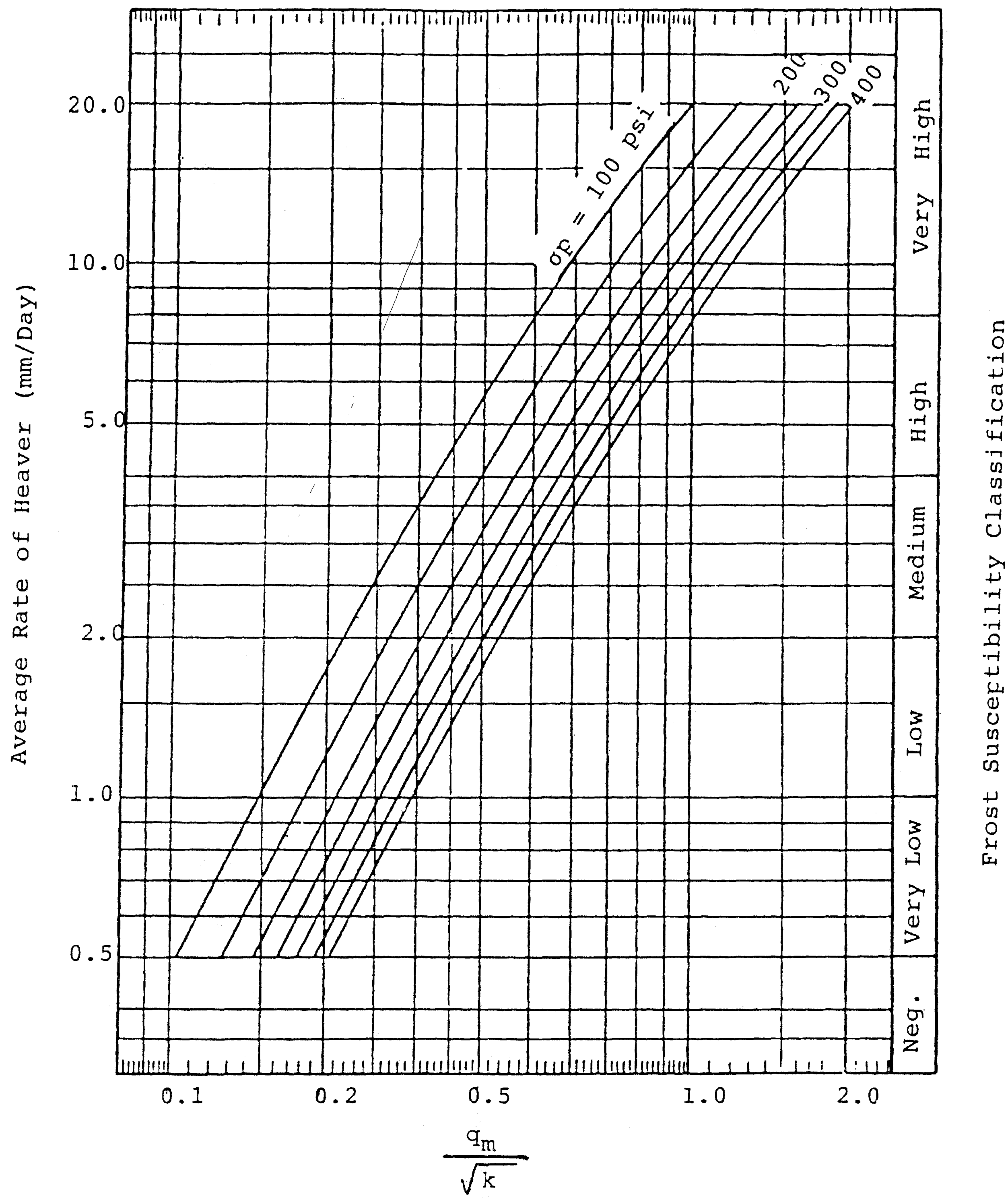


Figure 2-4.6. Chart for Estimating Design Inflow Rate of Melt Water From Ice Lenses.

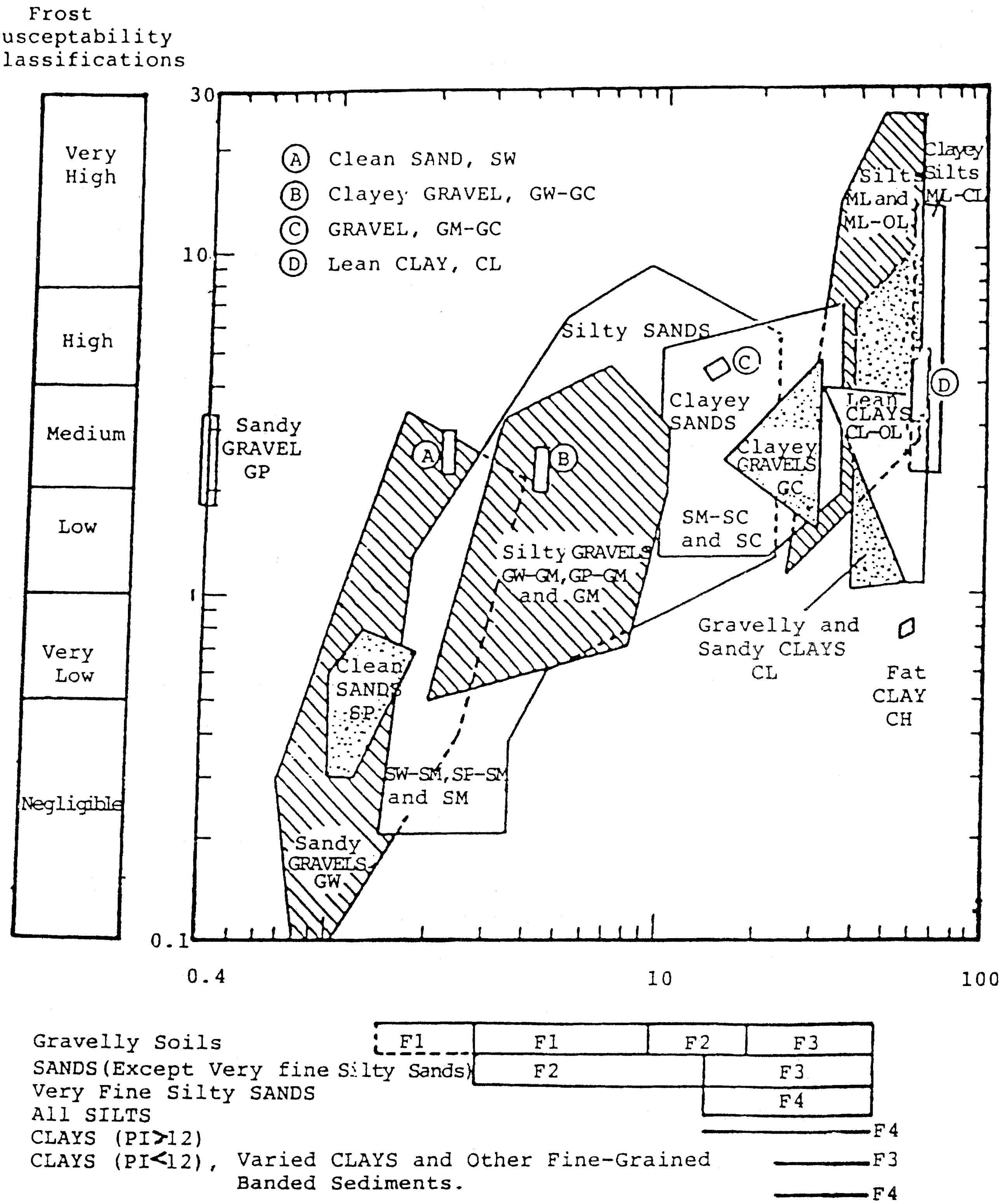


Figure 2-4.7. Summary of Results of Laboratory Freezing Tests Performed by Corps of Engineers.

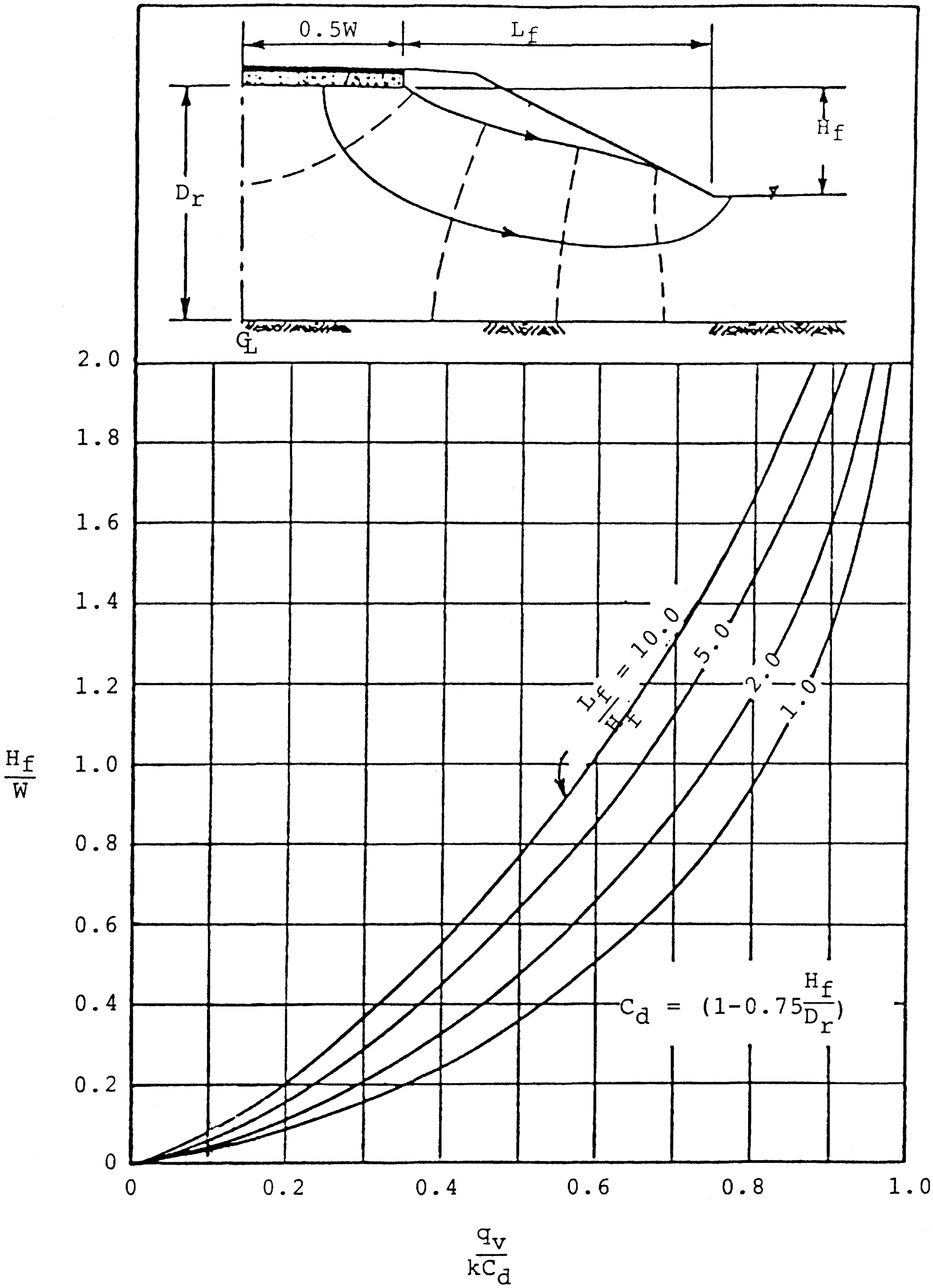


Figure 2-4.8. Chart for Estimating Vertical Outflow.

vertical outflow of this type should be considered in design. However, in the absence of such data, it is advisable for design purposes to consider vertical outflow toward a horizontal water table to be negligible.

3.2.4 Net Pavement Inflow

Consideration of all the possible sources of water allows for the determination of the net inflow (q_n). Combining various inflows and outflows gives the following set of relationships:

$$q_n = q_i + q_g + q_a + q_m + q_v$$

The Highway Subdrainage Manual (10) recommends that infiltration flow (q_i) always be included in computation of net inflow.

3.3 Types and Uses of Highway Subdrainage

Systems of highway subsurface drainage can be classified in a variety of ways according to:

1. The source of the subsurface water they are designed to control.
2. The function they perform.
3. Their location and geometry.

It is important that these classifications be put in perspective and that the associated terminology be understood.

A groundwater control system refers to subsurface drainage specifically designed to remove and/or control the flow of groundwater. Similarly, an infiltration control system is designed to remove water that seeps into the pavement structural section. Often the subdrainage may be required to control water from both sources. The physical features of the two systems may be very much alike although the desired result is very different.

A subsurface drainage system may perform one or more of the following functions:

1. Interception or cutoff of the seepage above an impervious boundary.
2. Draw-down or lowering of the water table.
3. Collection of the flow from other drainage systems.

Although a subdrainage system may be designed to serve one particular function, it will commonly be expected to serve more than one function. For example, an interceptor drain not only cuts off the flow from higher ground, but it draws down the water table so that it does not break out through a cut slope, for example.

The most common way of identifying subdrainage systems is in terms of their location and geometry. Familiar classifications of this type include:

1. Longitudinal drains.

2. Transverse and horizontal drains

3. Drainage blanket.

4 Well systems.

It should be noted that these types of subdrainage may be designed to control both groundwater and infiltration and/or to perform any of the functions outlined above.

3.3.1 Longitudinal Drains

As the name implies, a longitudinal drain is located parallel to the roadway centerline both in horizontal and vertical alignment. It may involve a trench of specified depth, a collector pipe and a protective filter of some kind, as shown in Figure 2-4.9. It may be less elaborate, as shown in Figure 2-4.10. The degree of sophistication employed in the design of longitudinal drains depends upon the source of the water that is to be drained and the manner in which the drain is expected to function.

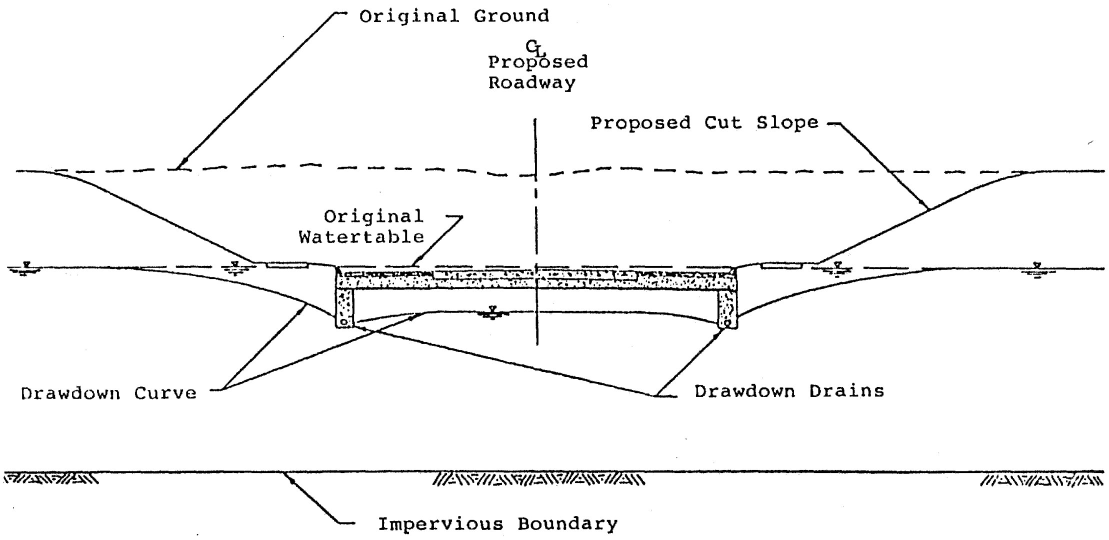
Sometimes, systems of longitudinal drains of different types can be employed effectively. An example of such an application is presented in Figure 2-4.11, which shows a multiple drain installation in a superelevated section of an expressway cut in a wet hillside. In order to intercept the flow and draw down the water table below the left cut slope, it was necessary to use two lines of relatively deep longitudinal drains. As shown in Figure 2-4.11, the collector drain (beneath the left shoulder) serves to drain any water that may get into the base or subbase of the left lanes as a result of infiltration or frost action. A similar function is performed by the shallow collector drain along the left edge of the right lanes.

The combination of groundwater conditions and highway cross-sections shown in Figure 2-4.9 and Figure 2-4.10 were such that the groundwater could be intercepted and/or drawn down well below the pavement sections with no more than two lines of longitudinal underdrains. However, this is not always possible, particularly when the water table is very high and the roadway section is very wide, as shown in Figure 2-4.12. In this case, the flow of groundwater might have saturated the subgrade and the pavement structural section over at least a part of its width if the third longitudinal drain had not been installed beneath the median. Even more complicated roadway geometries are possible, and more elaborate subdrainage configurations may be required for modern highways, particularly in the vicinity of interchanges.

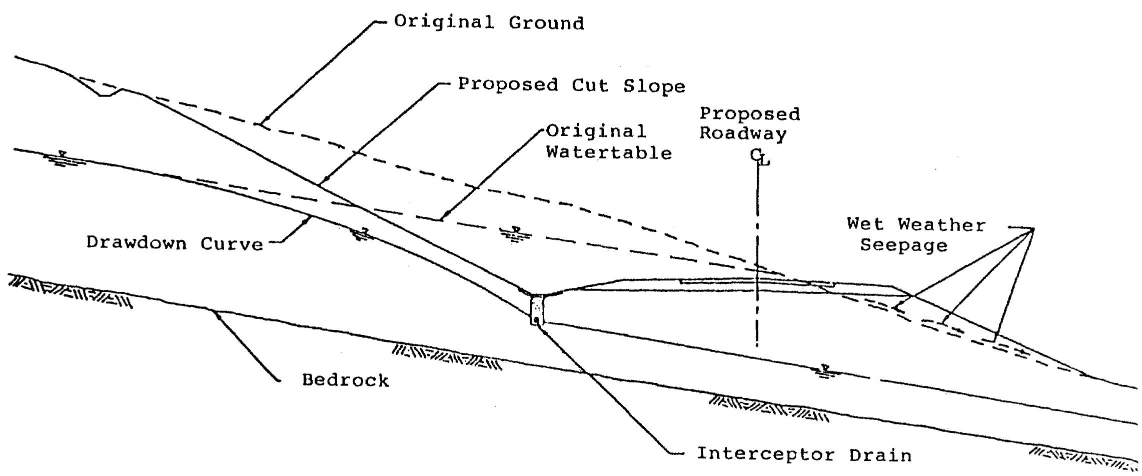
3.3.2 Transverse and Horizontal Drains

Subsurface drains that run laterally beneath the roadway are classified as transverse drains. These are commonly located at right angles to the roadway centerline although in some instances they may be skewed in the so-called "herringbone" pattern.

Transverse drains have been used at pavement joints to drain infiltration and groundwater in bases and subbases. This is particularly desirable where the relationship between the transverse and longitudinal



Synnetrical Longitudinal Drains Used to
Lower the Water Table



Longitudinal Interceptor Drain Used to Cut Off
Seepage and Lower the Groundwater Table

Figure 2-4.9. Longitudinal Drains.

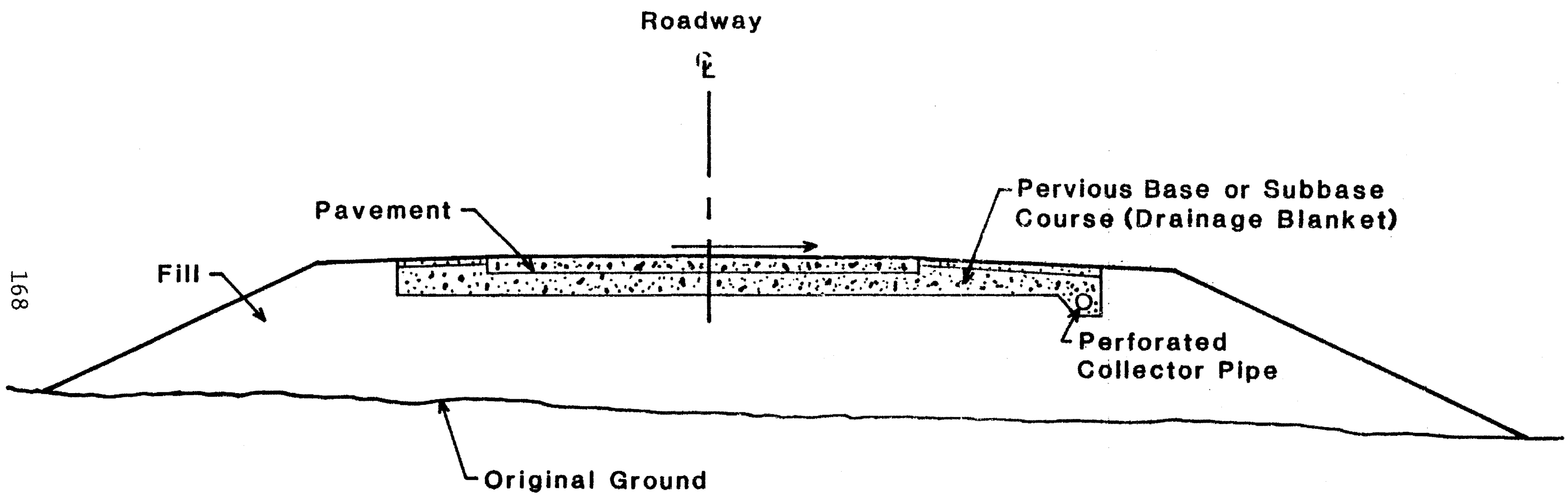


Figure 2-4.10. Drainage Blanket.

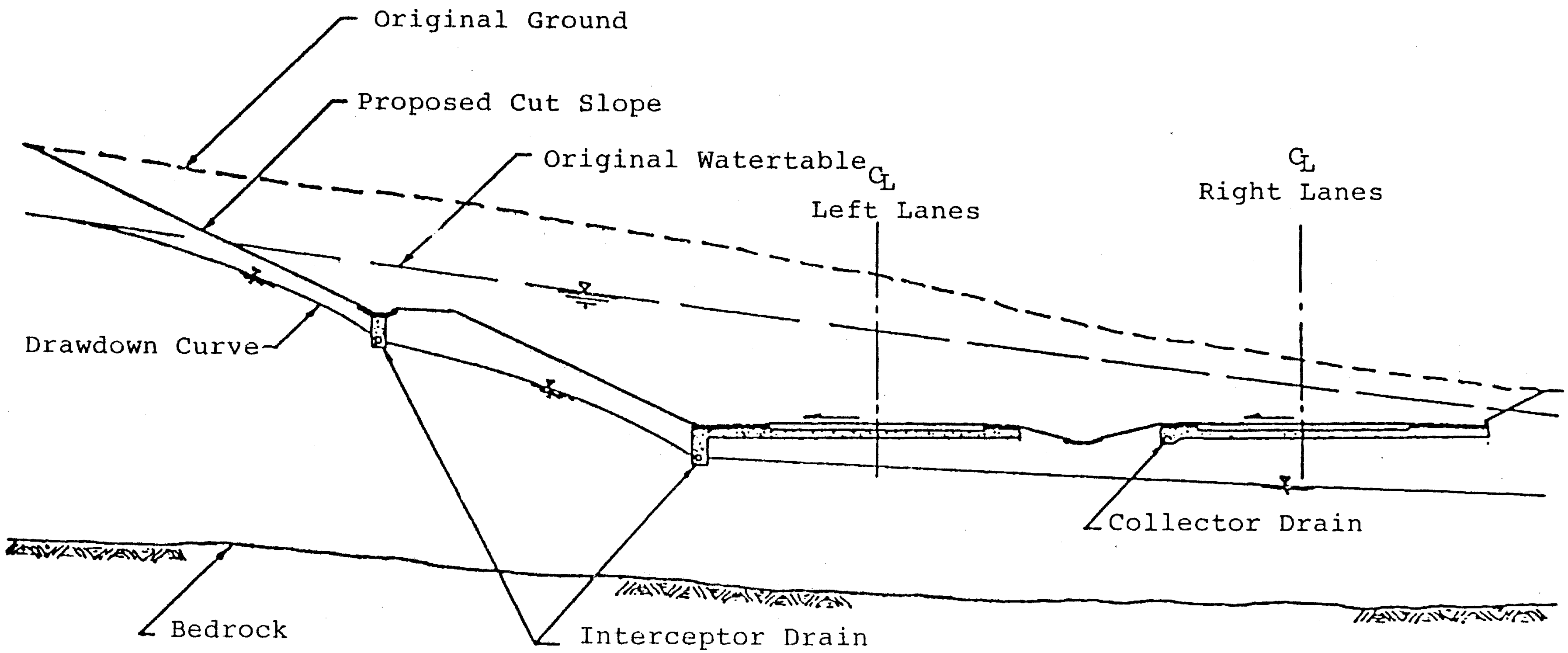


Figure 2-4.11. Multiple, Multipurpose, Longitudinal Drain Installation.

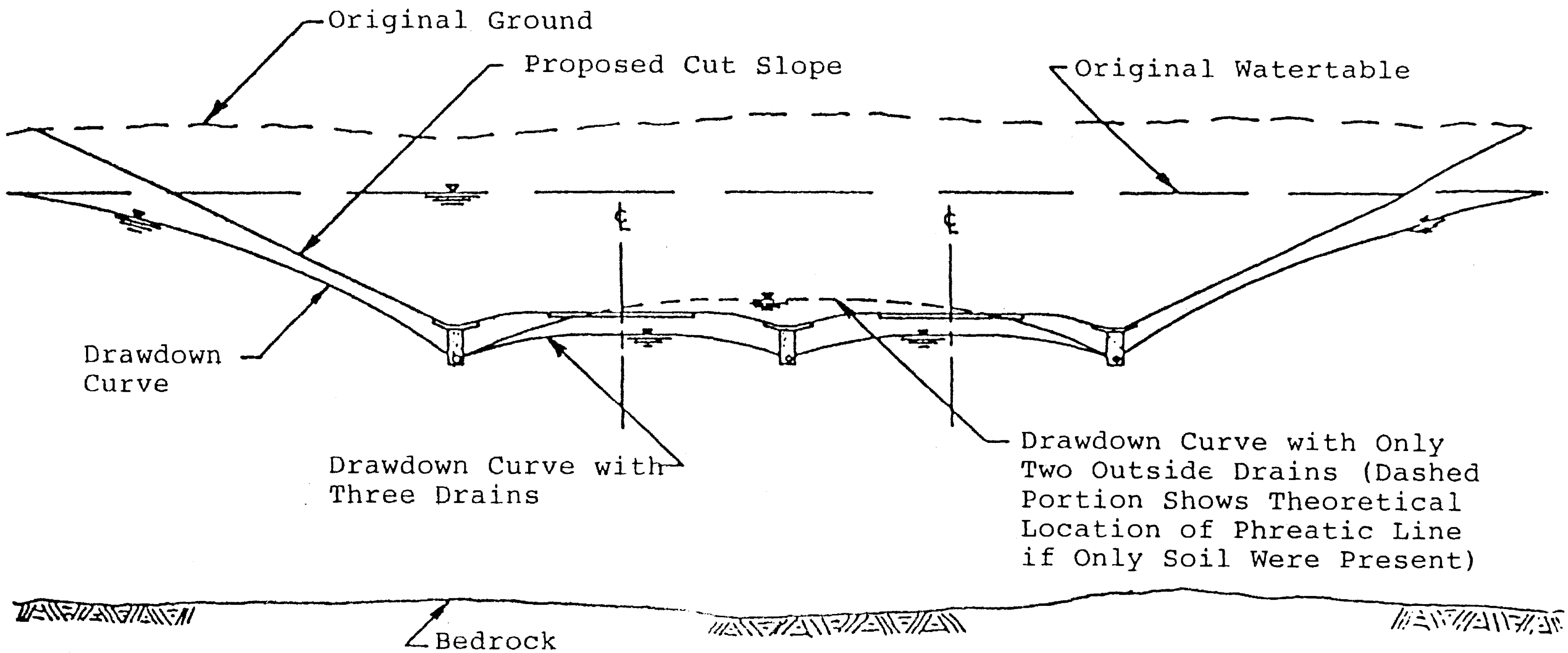


Figure 1-4.12. Multiple Longitudinal Drawdown Drain Installation.

grades is such that flow tends to take place more in the longitudinal direction than in the transverse direction. An example of this type of installation is shown in Figure 2-4.13. In this illustration, the transverse drains have been used in conjunction with a horizontal drainage blanket and longitudinal collector drain system. This can provide a very effective means for rapid removal of water from the pavement section.

Transverse drains may involve a trench, collector pipe and protective filter, as shown in Figure 2-4.13, or they can consist of simple "french drains" (i.e., shallow trenches filled with open graded aggregate), although this is not generally recommended. As with longitudinal drains, the degree of sophistication employed depends on the source and amount of subsurface water and the function of the drain.

When the general direction of the groundwater flow tends to be parallel to the roadway (this occurs commonly when the roadway is cut more or less perpendicular to the existing contours), transverse drains can be more effective than longitudinal drains in intercepting and/or drawing down the water table. This application is illustrated in Figure 2-4.14.

Some caution should be exercised in the use of transverse drains in areas of seasonal frost, since there has been some experience with pavements undergoing a general frost heaving except where transverse drains were installed, thus leading to poor riding quality during winter months.

Horizontal drains consist of nearly horizontal pipes drilled into cut slopes or sidehill fills to tap springs and relieve porewater pressures. In ordinary installations, the ends of the perforated small diameter drain pipes are simply left projecting from the slope and the flow is picked up in drainage ditches. However, in more elaborate installations, drainage galleries or tunnels may be required to carry large flows, and some type of pipe collector system may be used to dispose of the water outside of the roadway limits. An example of a drainage installation of this type, used in connection with a landslide stabilization project is shown in Figure 2-4.15.

3.3.4 Drainage Blankets

The term drainage blanket is applied to a very permeable layer whose width and length (in the direction of flow) is large relative to its thickness. Properly designed blankets can be used for effective control of both groundwater and infiltration, depending on the existing conditions.

The horizontal drainage blanket can be used beneath, or as an integral part of, the pavement structure to remove infiltration or to remove groundwater from both gravity and artesian sources. Although relatively pervious granular materials are often utilized for base and subbase courses, these layers will not function as drainage blankets unless they are specifically designed and constructed to do so. This requires an adequate thickness of material with a very high coefficient of permeability, a positive outlet for the water collected, and, in most instances, the use of one or more protective filter layers.

Two types of horizontal drainage blanket systems are shown in Figure 2-4.16. Here, a horizontal blanket drain is used in connection with shallow longitudinal collector drains to control both infiltration and the flow of

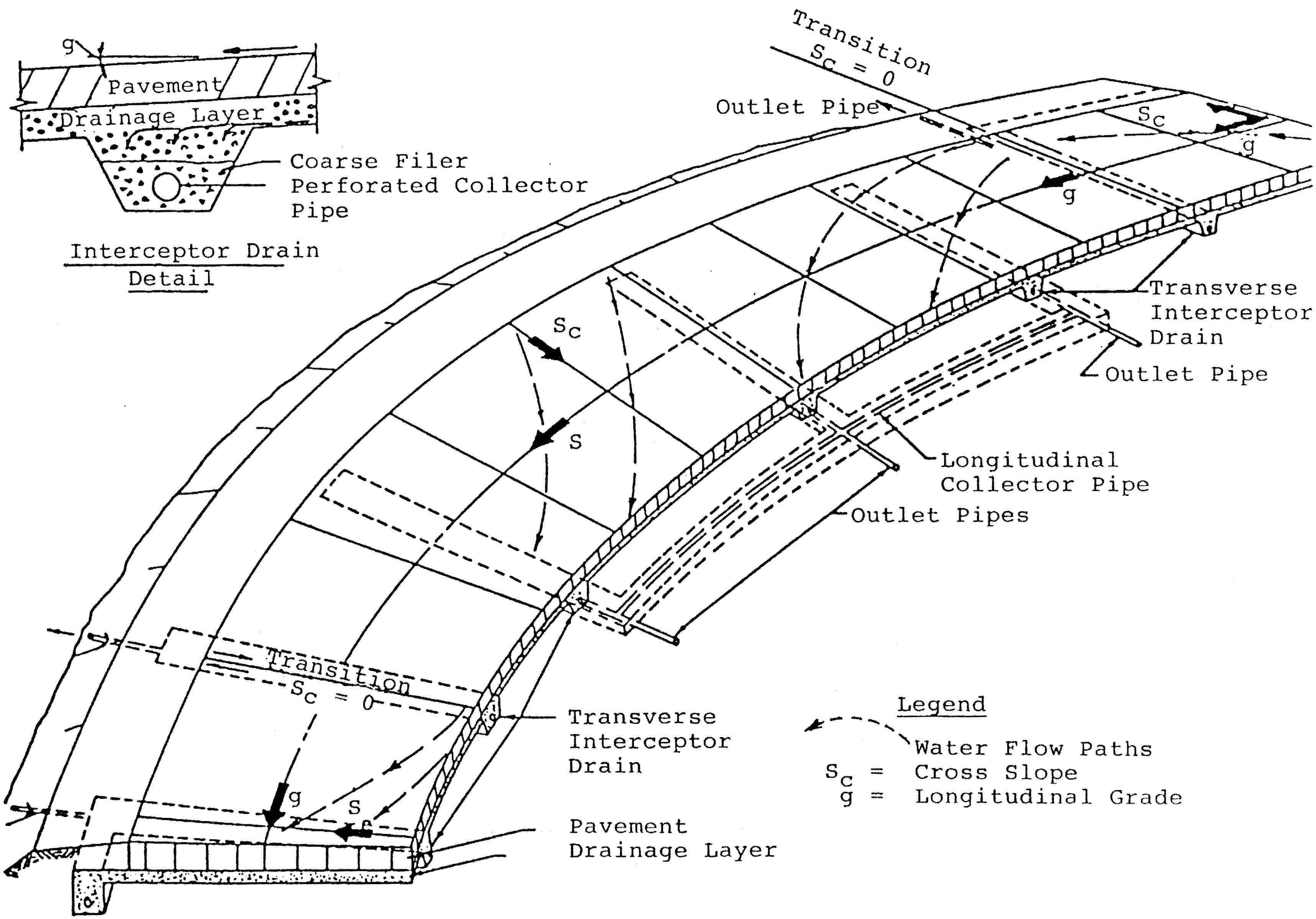


Figure 2-4.13. Transverse Drains on Superelevated Curve.

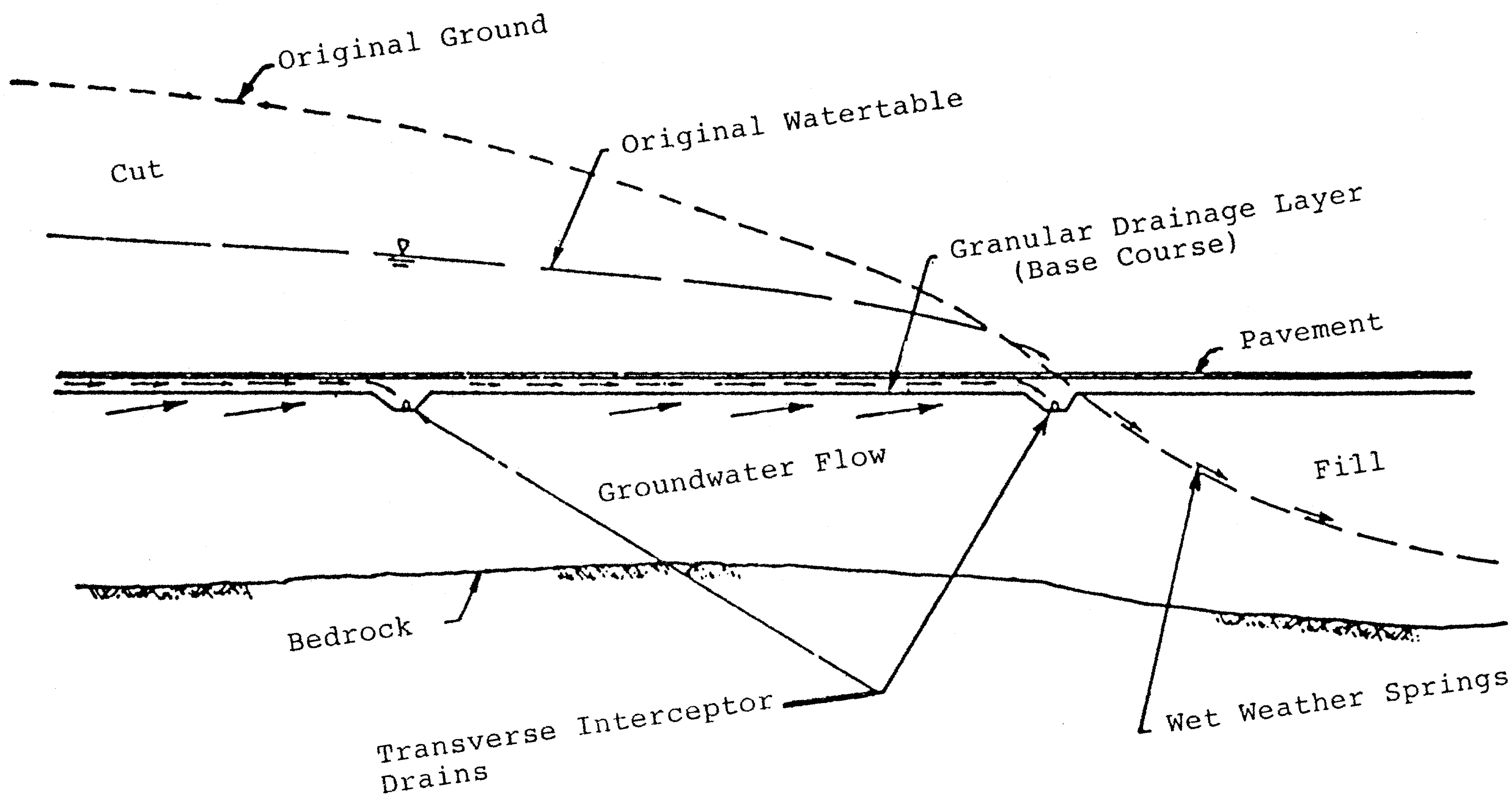
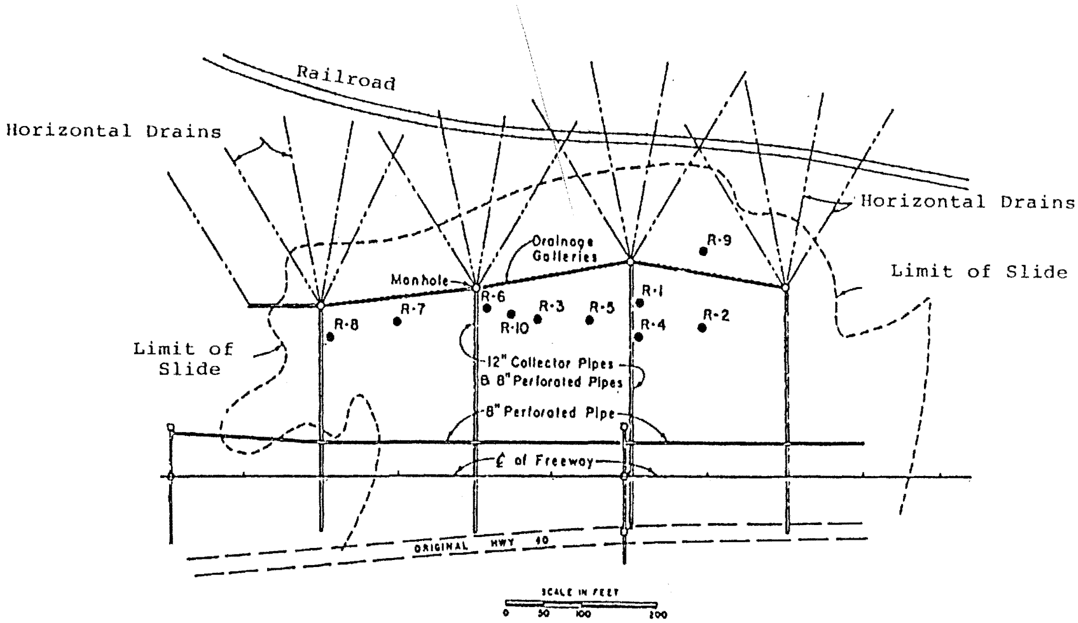
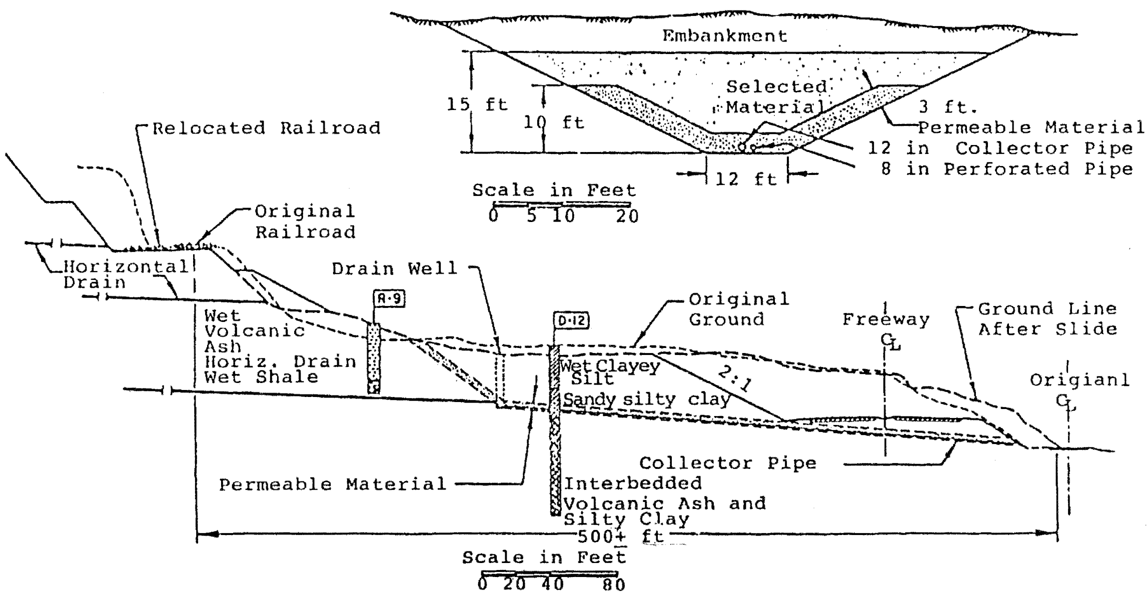


Figure 2-4.14. Transverse Interceptor Drain Installation in Roadway Cut with Alignment Perpendicular to Existing Contours.

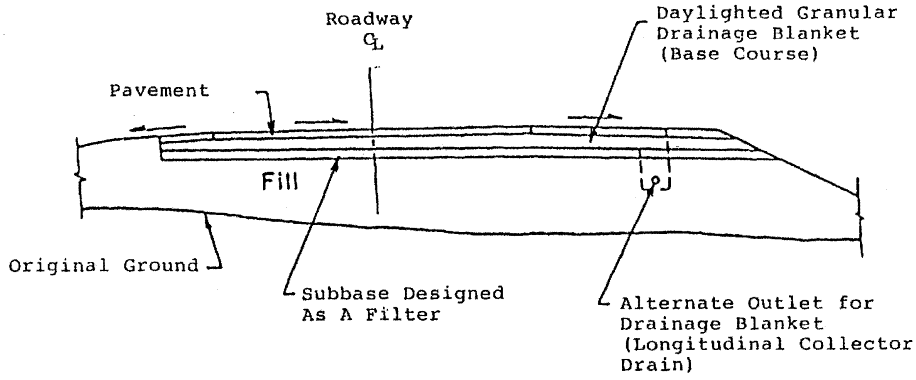


Plan Showing Drainage Details and Boring Locations at Towle Slide (20)

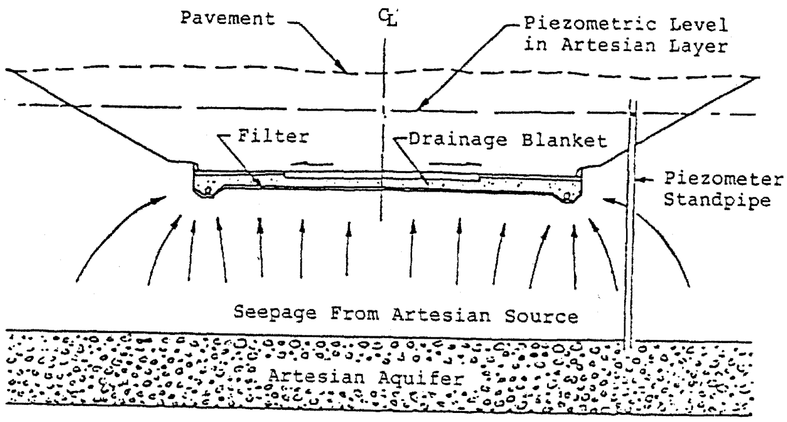


Profile and Typical Section of Drainage Trench at Towle Slide (20)

Figure 2-4.15. Horizontal Drains.



(b)



(a)

Figure 2-4.16. Applications of Drainage Blankets.

groundwater from the artesian source. Note that a protective filter layer has been used to prevent the subgrade soil from being washed into and, thus clogging the drainage layer. In Figure 2-4.16-b, a horizontal blanket drain is used to remove water that has seeped into the pavement by infiltration alone. In this case, the outlet has been provided by "daylighting" the drainage blanket. This type of outlet typically becomes clogged and ceases to function effectively. A more positive means of outletting the drainage blanket would have been to use the longitudinal drain shown dashed in Figure 2-4.16-b.

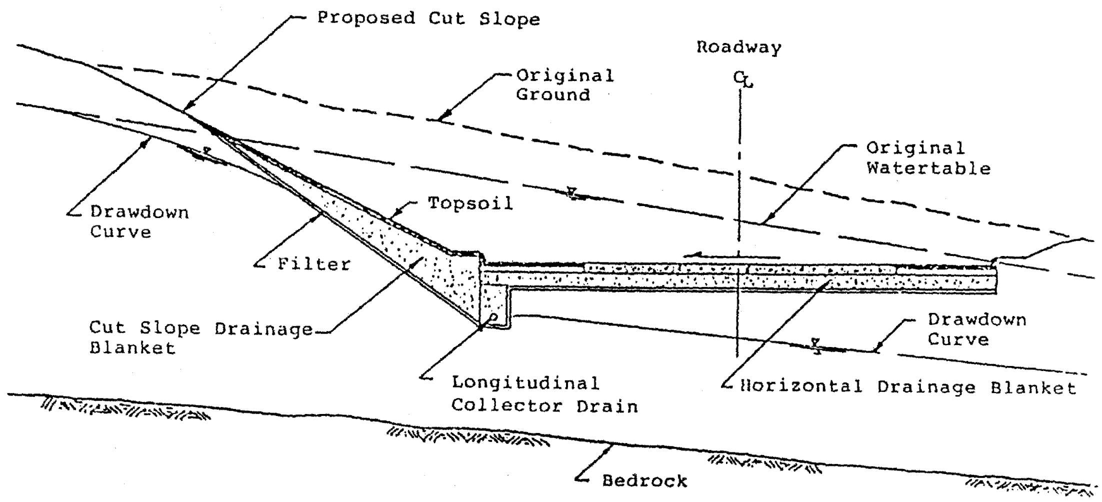
Additionally, the subbase has been designed as a filter in this instance to prevent intrusion of the subgrade soil into the base course under the action of traffic. When the longitudinal grade is large enough to control the direction of flow, transverse drains may be required to outlet the drainage blanket as shown in Figure 2-4.13. Drainage blankets can be used effectively to control the flow of groundwater from cut slopes and beneath sidehill fills.

Examples of these uses are illustrated in Figure 2-4.17. As shown in Figure 2-4.17-a, the drainage blanket used in connection with longitudinal drain can help to improve the surface stability (relieve sloughing) of cut slopes by preventing the development of a surface of seepage and by its buttress action. The blanket drain shown in Figure 2-4.17-b prevents the trapping of wet weather flow beneath the fill and minimizes the buildup of high porewater pressures that can lead to slope instability.

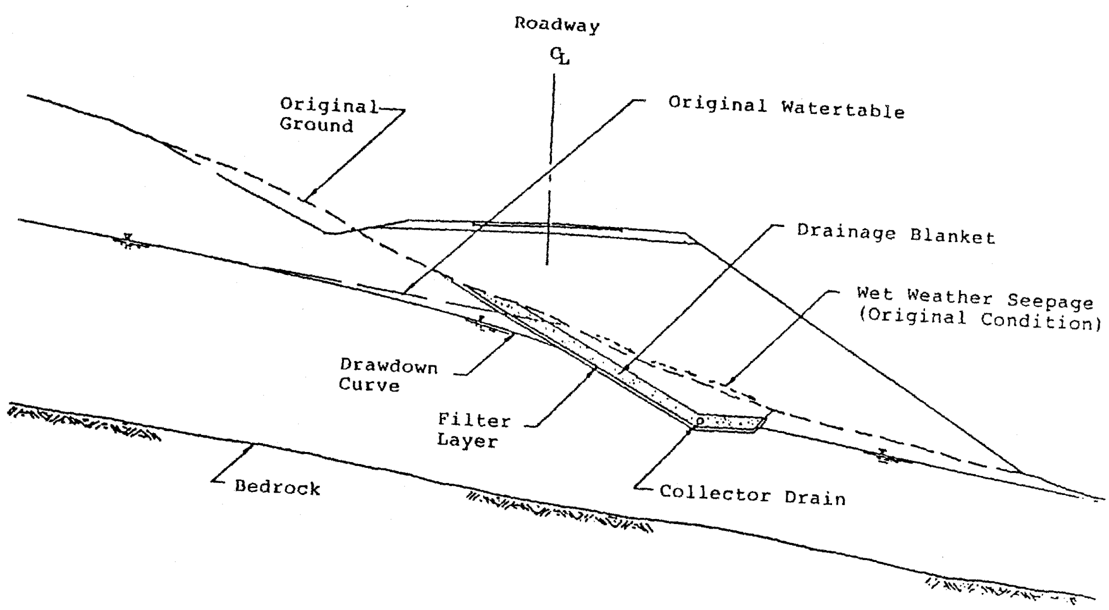
3.3.4. Well Systems

Systems of vertical wells can be used to control the flow of groundwater and relieve porewater pressures in potentially troublesome highway slopes. In this application, they may be pumped for temporary lowering of the water table during construction or simply left to overflow for the relief of artesian pressures. More often, however, they are provided with some sort of collection system so that they are freely drained at their bottoms. This may be accomplished by the use of tunnels, drilled-in pipe outlets or horizontal drains. Typical well drainage systems that were used to help in the stabilization of wet slopes were shown in Figure 2-4.15.

Sand filled vertical wells (sand drains) can be used to promote accelerated drainage of soft and compressible foundation materials which are undergoing consolidation (the squeezing out of water) as a result of the application of a surface loading such as that produced by a highway embankment. An installation of this type is illustrated schematically in Figure 2-4.18. The design and construction of sand drains for foundation stabilization is a rather specialized undertaking requiring detailed consideration and understanding of the three-dimensional consolidation process. When used, this form of drainage is very expensive, and other materials may be installed such as geotextile wick drains for water removal for a more cost-effective drainage installation. This aspect of highway subdrainage is considered to be outside the scope of this course and will not be given further consideration.



Drainage Blanket (Wedge) on cut Slope
Drained by Longitudinal Collector Drain



Drainage Blanket Beneath Sidehill Fill
Outletted by Collector Drain

Figure 2-4.17. Drainage Blankets.

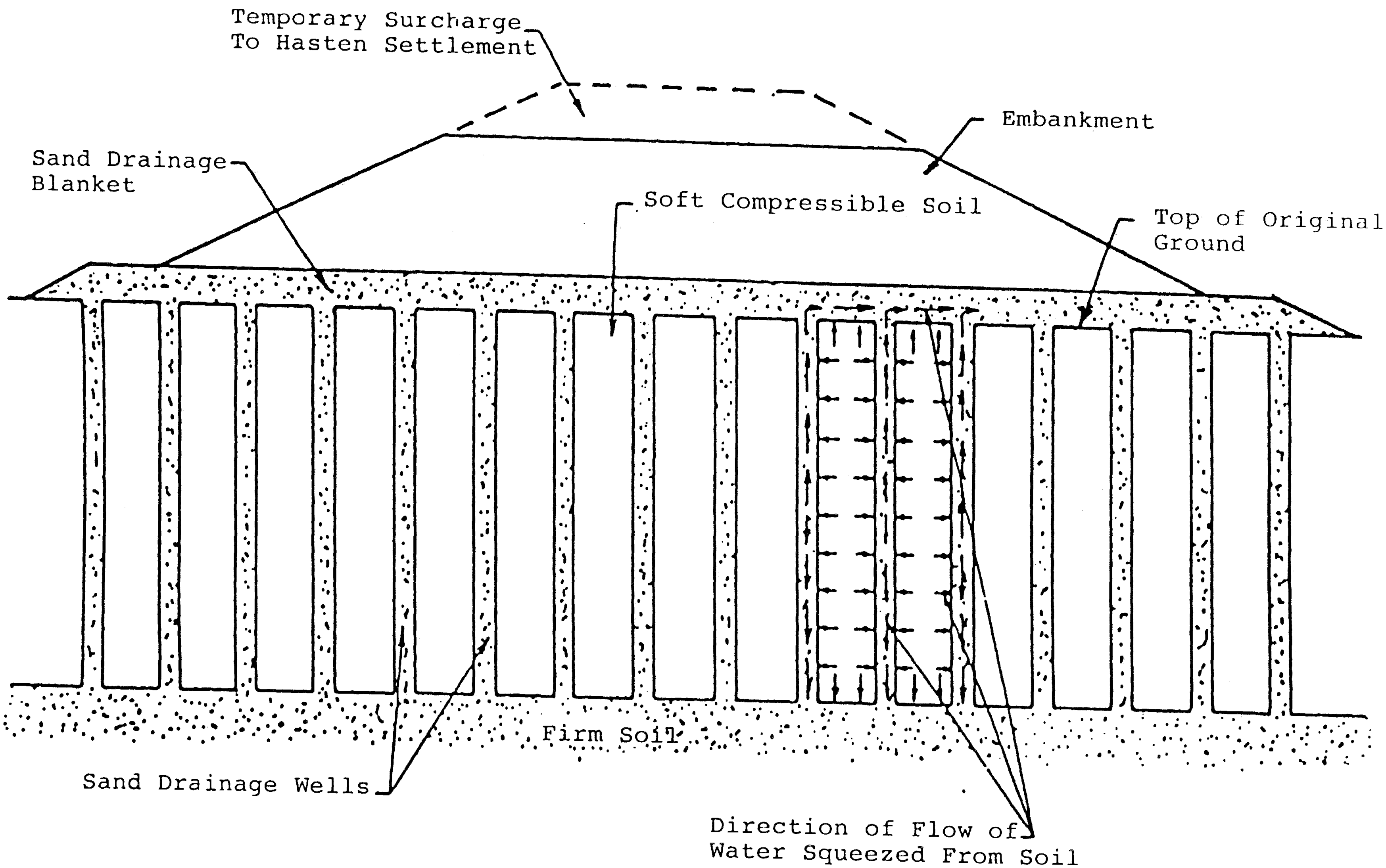


Figure 2-4.18. Typical Sand Drainage Well Installation.

3.3.5 Miscellaneous Drainage

Frequently, during the course of highway construction and maintenance operations, local seepage conditions are encountered which require subsurface drainage to remove the excess moisture or relieve porewater pressures. These conditions may require small drainage blankets with pipe outlets, longitudinal or transverse drains or some combination of these drainage systems. Although subdrainage of this type is highly individualized, its importance should not be minimized and its design should be approached with the same care as the design of more elaborate subdrainage systems.

4.0 FHWA HIGHWAY SUBDRAINAGE DESIGN MANUAL

4.1 Content

The Highway Subdrainage Design Manual was prepared to present a comprehensive approach to evaluating moisture in a pavement and designing drainage to remove this moisture in pavements. The manual contains the following chapters:

1. General Considerations.
2. Data Required for Analysis and Design.
3. Pavement Drainage.
4. Control of Groundwater.
5. Construction and Maintenance

The content of this manual provides nomographs, charts, flow nets, and other relationships to determine the inflow of water into the pavement system, lay out the drainage system to effectively remove the water which must be removed, and size the drainage system to ensure it removes all the water entering the pavement system. A detailed example problem is presented in the example problem section of this module to illustrate a complete drainage design.

5.0 DRAINAGE MATERIALS

When the drainage system has been analyzed and sized to handle the appropriate amount of water that will infiltrate the pavement, the materials must be selected carefully to ensure that the properties in the layers promote drainage and do not interfere with the flow of the water. The necessary components of the drainage system must function in unity for the system to be effective. These components include the portion which intercepts the water, the component which collects the water to a central point, and the component which removes the water from the pavement system so that it cannot do any damage.

5.1 Drainage Pipe

Presently, several different types of drainage pipe of various lengths and diameters are being used in pavement subsurface drainage. Some of these are as follows:

1. Clay tile.
2. Concrete tile and pipe.
3. Vitrified clay pipe.
4. Perforated plastic bituminous fiber pipe.
5. Perforated corrugated-metal pipe.
6. Corrugated plastic tubing.

The clay and concrete tile can be obtained in 1 to 3 ft. (0.3 to 0.9 m) lengths. Metal and fiber pipes are usually manufactured in lengths of 8 ft (2.4 m) or longer. The thick-walled, semi-rigid plastic tubing may be obtained in about 20 ft. (6 m) lengths. The corrugated plastic tubing is manufactured in rolls about 200 to 300 ft. (61-91 m) long. For subsurface drainage, the pipe diameter generally ranges between 4 in. and 6 in. (10 and 15 cm). However, the California Department of Transportation has used slotted plastic pipe with an inside diameter of 2 inches, and has recently gone to 3 inches.

Most of the newer drainage pipes are flexible conduits rather than rigid conduits such as clay, concrete, or metal conduit. The flexible plastic drains can fail as a result of excessive deflection if inadequately installed. For this reason, the load-deflection characteristics are important considerations when this material is being used in subsurface drainage design. Impact resistance is also important from the standpoint of damage to the pipe while it is being placed.

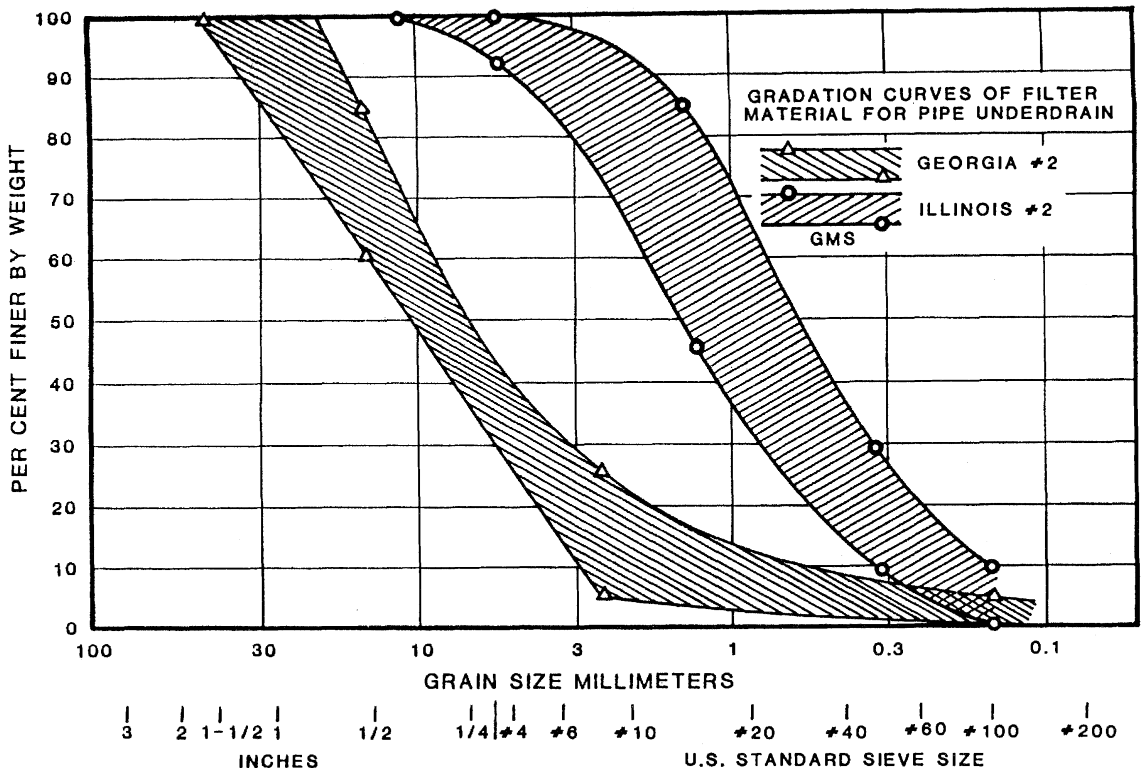
5.2 Drainage Filter or Envelope Materials

5.2.1 Envelope Material

When considering open-graded transverse drains, longitudinal drains, drainage blankets, and drainage wells, it is necessary to evaluate the filter or envelope material. The primary functions of the envelope material around subsurface drains are as follows:

1. To prevent the movement into the drains of soil particles which might settle and clog the drain.
2. To provide material in the immediate vicinity of the drain openings which is more permeable than the surrounding soil.
3. To provide a suitable bedding for the drain.
4. To stabilize the soil in which the drain is being laid.

Until recently, the most commonly used envelope materials were naturally graded coarse sands and gravels. There is a considerable range of gradations used for drainage envelopes. Figure 2-4.19 shows a comparison of the range that can be found between two different state transportation departments. The general procedure for designing the drainage envelope for a given soil is to make a mechanical analysis of both the soil and the proposed



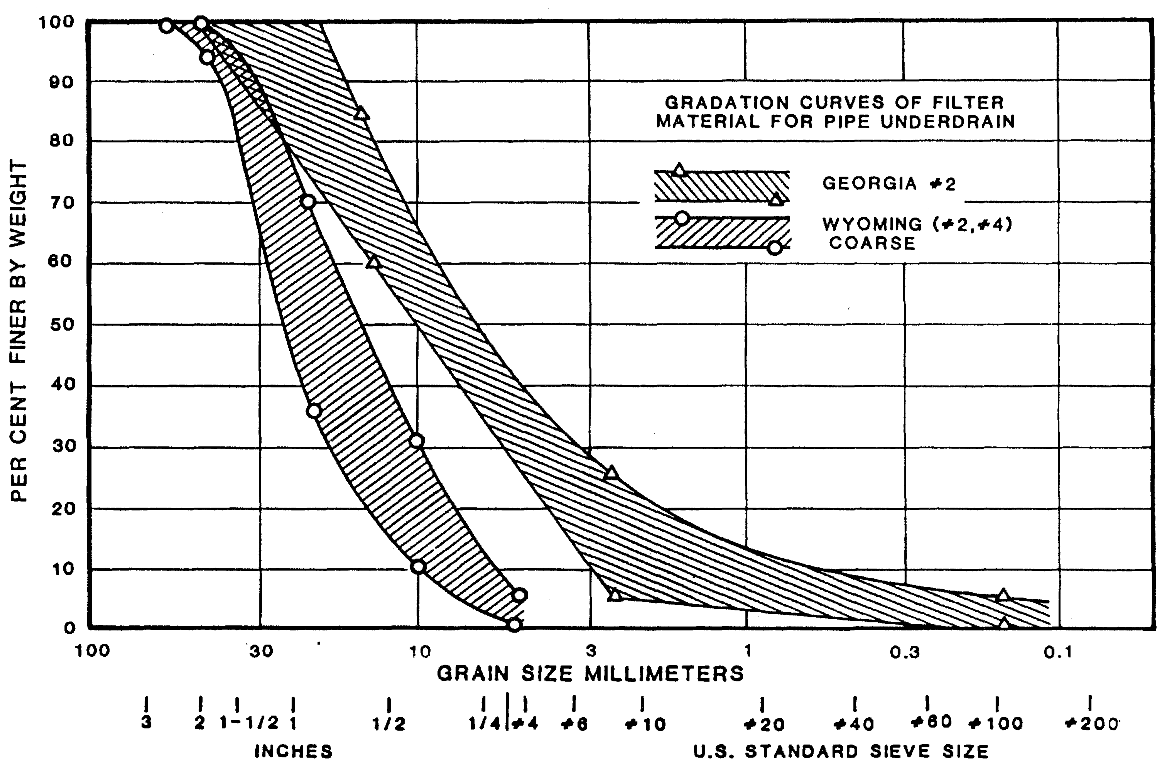


Figure 2-4.19. Gradations of Filter Materials.

envelope material, compare the two particle size distribution curves, and use Terzaghi's (6) gradation matching criteria to determine whether the envelope material is satisfactory.

The piping criteria are written as:

D_{15}(drain)/D_{85}(filter) ≤ 5

D_{50}(filter)/D_{50}(subgrade) ≤ 25

D_{15} (subgrade) $\leq d_{15}$(filter)$\leq D_{85}$(subgrade

These requirements result in a filter that is sandwiched between the subgrade soil, and the drain material. The function of the filter material is to protect the drain from clogging with fines moving out of the subgrade soil.

To prevent the entrance of filter material into slots and holes of the drainage pipes, the following requirements have been recommended:

For pipes with slots:

D_{85}(filter)/(Slot Width) > 1.2

For pipes with circular holes:

D_{85}(filter)/(hole diameter) > 1.0

According to Cedergan's report, the Bureau of Reclamation requires that:

D_{85}(filter)/(Max opening of pipe) > 2

It should be noted that one of the important considerations in filter performance is the local condition. As an example, the aggregate segregation and construction variables during material placement could significantly affect the performance of the drainage system.

5.2.2 Filter Fabric

A recent innovation that has been widely used in highway subdrainage is the use of filter, or geotextile fabric. These fabrics are either woven or non-woven mats constructed of polypropolene or nylon fibers. The fabrics take the place of the graded filter material. They serve the very same function as the filter material. As such, fabrics have the same considerations in matching the fabric to the subgrade soil. Fabrics have an "Equivalent Opening Size" (EOS) which is much the same as a particle size. The EOS must be matched to the subgrade soil to ensure that the fabric will prevent piping of the soil fines out of the subgrade (12). ASCE and ASTM are in the process of publishing guidelines for selection of EOS for fabrics.

The elimination of the granular filter material reduces the cost of the drainage installation, offsetting the increased cost of the fabric. Fabrics have been used to allow innovative drainage installations. Pipe can be covered directly with the fabric and installed in a trench. Vertical fin drains have been designed using fabric wrapped around a supporting plastic core. Their selection is not a matter of chance, and should be selected and analyzed very carefully.

6.0 SUBSURFACE DRAINAGE EVALUATION TO IMPROVE INITIAL DESIGNS

As a part of a design process the drainage existing in functioning pavements should be carefully evaluated to determine if it is performing up to expectations. If the drainage is to be counted on for increased support during the life of a pavement, the drainage must be functioning. This evaluation must consist of two parts, a field evaluation of the existing drainage characteristics of the pavement, and a laboratory evaluation of the materials in the pavement. The relative field performance can be used to assist the engineer in selecting drainage designs which will function best.

6.1 Visual Survey

The visual survey must include a determination of the moisture related distress on the pavement. This will show the engineer what assumptions in the design are not being used in the field. The functioning of any existing drainage must be established. This includes an evaluation of any subdrainage which may be functioning poorly, and an examination of the drainage ditches along the roadway which serve to remove water from the pavement system proper. The integrity of the surface must be evaluated. Cracks and joints that are not adequately sealed will increase the need for appropriate drainage considerations in new design which may require good sealing.

6.2 Material Survey

Subsurface drainage should be designed and constructed with long term performance and maintenance in mind. Procedures for cleaning collector pipes and maintaining outlets are necessary. Drainage systems require periodic inspections to check performance. Outflow measurements when first constructed and at later periodic intervals will indicate whether the drain is functioning properly.

Climate factors, such as rainfall precipitation, frost depth and temperature are among the most important parameters affecting pavement performance. The influence of precipitation on pavement performance is reflected by the changes in weakening of the pavement support condition and moisture-related damage in various pavement component layers.

Surface water infiltration, high ground water, and capillary rise in the pavement structure contribute significantly to the pavement distresses. The damaging effects of adverse drainage on pavement performance have been documented, and show that if a pavement system is expected to perform well over its expected design life, an adequate drainage system should be designed and installed.

The infiltration of excess water into a concrete pavement system can result in several distresses which would significantly reduce the life of the pavement. The fact that moisture problems may appear in any layer emphasizes the necessity of having a logical procedure for determining where the problem is most likely to be originating from so that it may be addressed in new designs. The amount of moisture in a pavement and the impact of that moisture on performance are primarily due to climatic factors. A large number of climatic variables have been studied and catalogued for nearly every region in the United States. FHWA studies have provided guidelines to identify climatic regions for the United States. These regions provide areas of similar expected pavement performance based on moisture availability in the roadbed soil and the influence of temperature. There are nine distinct moisture zones as shown in Figure 2-4.20, which have been divided into six for the AASHTO pavement design Guide.

The different zones are based on a yearly average values, and some discrepancies may exist in a local area. These should be checked by performing the calculations for any particular locality.

It has been recommended that in regions where relatively high annual rainfalls exist or where significant groundwater exists, consideration should be given to providing subsurface drainage systems. In a study by Cedergren et al. (1) on subsurface drainage, it was recommended that a subsurface drainage system is required if:

1. The average annual precipitation is more than 10 inches, (254 mm).

2. The pavement is expected to be subjected to more than 250, 18 Kip (80 kN) equivalent axle loads per day during the design life of the pavement.

6.3 Drainability of Base Course

Relative times for a base course to drain water is a direct indication of the ability of the base to resist the detrimental effects of moisture on pavement performance. The procedure to perform this calculation is presented,and the DRAINIT spreadsheet program is used to perform the calculations (15).

The form presented in Figure 2-4.21 must be filled in to calculate the drainability of the granular layers. First, the pavement cross sectional properties must be recorded. These include the following, which should be recorded in the appropriate place on Figure 2-4.21:

1. Longitudinal Slope, gl, ft/ft.

2. Transverse Slope, gt, ft/ft.

3. Thickness of Drainage Layer, H, ft.

4. Width of Drainage Layer, D. ft.

Sections having different cross-section properties must be analyzed separately. The terminology used to differentiate each pavement section should be recorded in the appropriate block. Three calculations must be performed as indicated in Figure 2-4.21 for the cross-sectional properties.

1. Le = effective length of drainage = D $(ge/gt)^2 + 1$

2. ge = effective slope of drainage path = $gl^2 + gt^2$

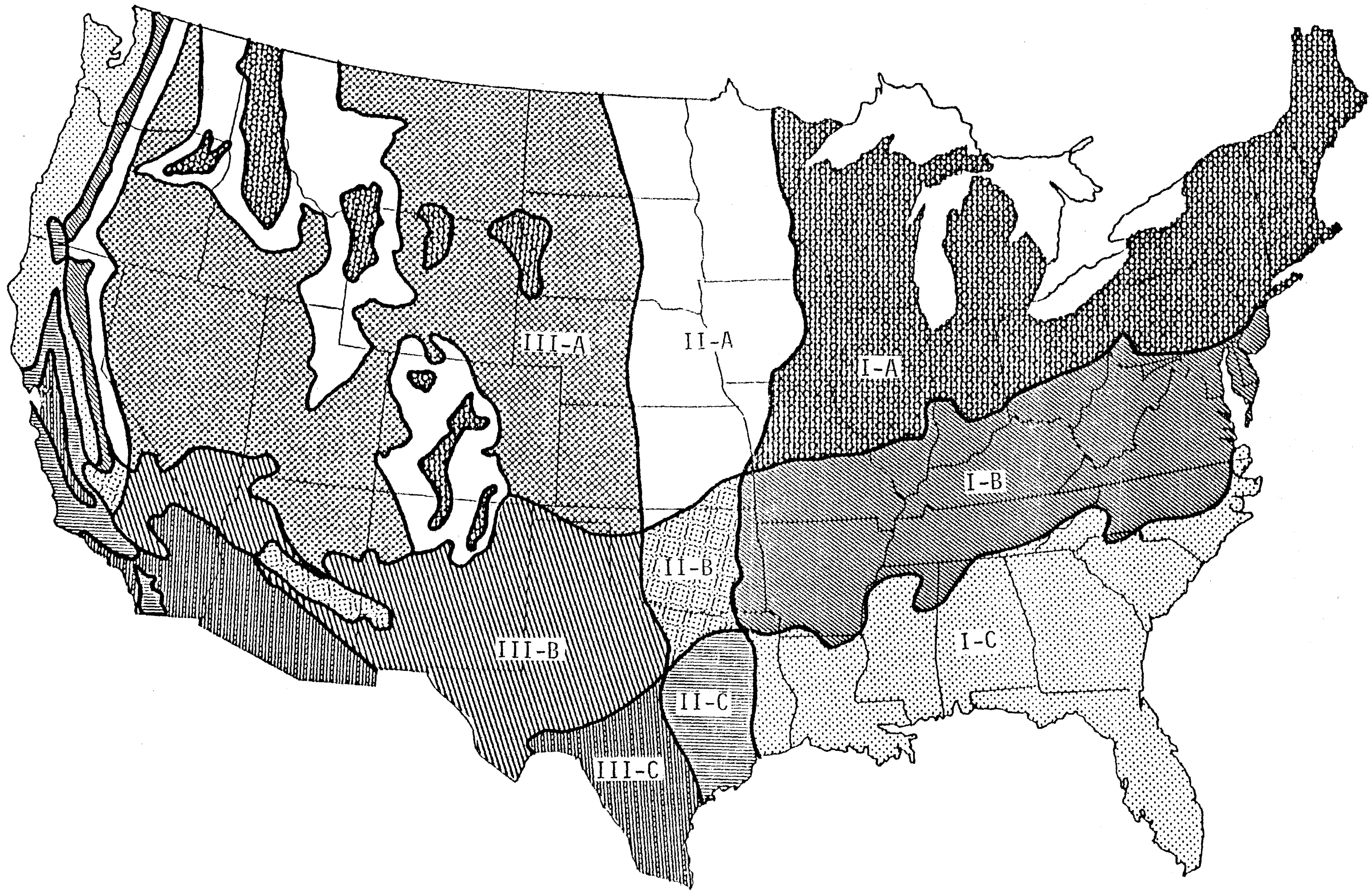


Figure 2-4.20. Environmental Zones.

```
****************************************************************************
DRAINAGE TIME ANALYSIS OF GRANULAR MATERIALS = AASHTO CRITERIA =           *
                                                                           *
         TITLE:Example Run                                                 *
---------------------------------------------------------------------------*
                  PAVEMENT SECTION PROPERTIES                              *
                           << INDICATES INPUT                              *
PERCENT FINES:           6 <<                                              *
TYPE OF FINES:           5 INERT=3  SILT=4   CLAY=5   <<                   *
D 10 EFF. SIZE:         .8 mm <<                                           *
MATERIAL TYPE:           2 GRAVEL=1, SAND=2. <<                            *
DENSITY                122 pcf <<                                          *
SPEC. GRAVITY        2.684 <<                                              *
PERMEABILITY:     26.21464 ft/day   Figure 2-4.23                          *
                                                                           *
---------------------------------------------------------------------------*
```

```
---------------------------------------------------------------------------*
                                                                           *
                  CROSS SECTION DATA                                       *
DRAINAGE WIDTH:         24 ft.   <<                                        *
BASE THICK:            .75 ft.   <<                                        *
Gt:                   .005 ft/ft   <<                                      *
Gl:                    .01 ft/ft   <<                                      *
Lr:               53.66563                                                 *
Sr:               .0111803                                                 *
SLOPE FACTOR:         1.25        X= 1.43894                               *
                                                                           *
---------------------------------------------------------------------------
```

```
---------------------------------------------------------------------------*
                                                                           *
                  MATERIAL CALCULATIONS                                    *
Ws:                  1.952 gm/cc      weight of solids                     *
Vs:               .7272727 cc         volume of solids                     *
Vv:               .2727273 cc         volume of voids in soil              *
EST. WATER LOSS         15            FIGURE 2-4.23                        *
SPECIFIC YIELD:.0409091               Estimated from Figure 2-4.22         *
SPECIFIC YIELD:.0573235               Figure 30, FHWA Manual.              *
                  ^Average is used in program                              *
---------------------------------------------------------------------------*
```

Figure 2-4.21. Input Screens for DRAINIT Spreadsheet to Calculate Drainage Times.

3. S = Slope Factor = H/(Le x ge)

The next material which must be examined is the roadbed soil. The gradation and plasticity characteristics must be known. These can be obtained from construction records, tests run on core samples, or county soil maps. Initial results can be developed from construction records, but final recommendations for rehabilitation or drainage work must be based on actual core data. New designs must be done with material specifications or actual data from similar materials used in the field. The information to be recorded includes:

1. Percent fines (- #200).

2. Types of fines

   a. Inert - Substantially below "A" line in Unified Classification system, PI below 1.
   b. Silty - Material plots near "A" line. PI above 1, but below "A" line.
   c. Clay - Material has high PI, it plots above the "A" line in the Unified System.

3. D_{10}, effective grain size with 10 percent of the material passing this size, mm.

4. Dry density, pcf and gm/cc.

5. Specific gravity of solids, G_s. This may be obtained from construction records and initial material tests, and will not vary from section to section.

These should be recorded in the appropriate blank in Figure 2-4.21.

The next section to be completed on Figure 2-4.21 involves calculation of drainability properties of the pavement section. This section performs some calculations as follows:

1. Assume Ws = 1.0.

2. Calculate Vs = Ws/Gs.

3. Calculate Vv = 1 - Vs = $N_e$max (= B). Nemax is the volume of water that completely fills the voids in the material.

4. From Figure 2-4.22 select the estimated water loss, C. Consult plasticity and grain size data for the material.

5 Calculate the specific yield, Ne = ($N_e$max) x C/100 = (B x C/100).

6. Calculate X, X = (Ne x Le)/(H x k) k, the permeability may be estimated from Figure 2-4.23, the spreadsheet automatically calculates permeability from material properties supplied.

These data are used to calculate drainage times and saturation levels. The spreadsheet automatically performs these calculations as shown in Figure 2-4.24. The procedure is as follows:

AMOUNT OF FINES	2.5% FINES			5% FINES			10% FINES		
TYPE OF FINES	INERT FILLER	SILT	CLAY	INERT FILLER	SILT	CLAY	INERT FILLER	SILT	CLAY
GRAVEL	70	60	40	60	40	20	40	30	10
SAND	57	50	35	50	35	15	25	18	8

*Gravel, 0% fines, 75% greater than #4: 80% water loss

*Sand, 0% fines, well graded: 65% water loss

* Gap graded material will follow the predominant size

Figure 2-4.22. Estimated Values of Water Loss for Calculating Specific Yield, C

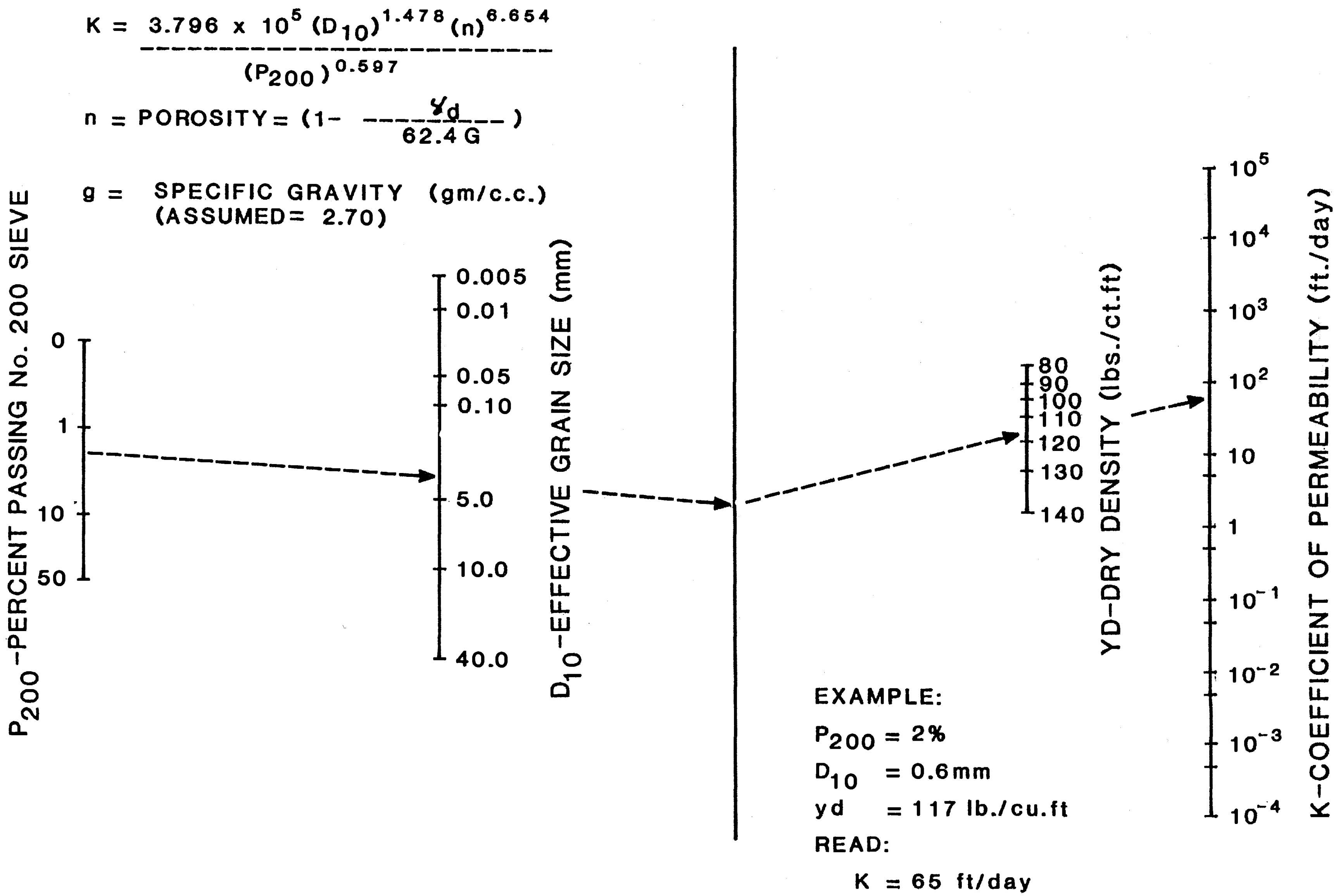


Figure 2-4.23. Chart to Estimate Permeability.

DRAINAGE TIME CALCULATION

U PERCENT	T FACTOR	(T)*24 HOURS	Ne*U	Vv-Ne*U	SATURATION %
0		0			100
.1	.0180937	.6248596	.0049116	.2678156	98.19907
.2	.0662004	2.286203	.0098233	.262904	96.39814
.3	.1374342	4.746232	.0147349	.2579924	94.59721
.4	.2270371	7.840629	.0196465	.2530808	92.79628
.5	.3315833	11.45109	.0245581	.2481691	90.99535
.6	.463284	15.99931	.0294698	.2432575	89.19442
.7	.6559723	22.65372	.0343814	.2383459	87.39349
.8	.9701154	33.50252	.039293	.2334343	85.59256
.9	1.608647	55.55394	.0442046	.2285226	83.79163

Figure 2-4.24. Calculation Screen for DRAINIT Spreadsheet to Calculate Drainage Times.

1. From Figure 2-4.25 select a time factor T for every value of U. The slope factor, S, previously calculated is used to select the proper curve.

2. Calculate the drainage time, in hours, for Column 3. (Column 2) x X x 24 = hours.

3. Specific yield, Ne, time U gives the amount of water drained during this time. Record this in Column 4.

4. Subtract Column 4 from $N_e$max (labeled B). This is the amount of water remaining in the sample and goes in Column 5.

5. Column 5 divided by Nemax (labeled B) time 100 gives the saturation level of the sample and is recorded in Column 6.

The values of t in hours and the percent saturation should be plotted on Figure 2-4.26 to determine the suitability of the granular layer for drainage purposes. This classification will be either acceptable (a), marginal (m), or unacceptable (u). These times can be altered to match the AASHTO design requirements for a specific level of saturation.

If very different materials are being used, each section with a different granular material should be evaluated separately. Each section will receive a separate rating for granular drainability. Areas which receive similar ratings may be combined. The areas of granular drainability should be noted on a strip map of the project to show their locations.

6.4 Drainability of Subgrade

The first step in evaluating the subgrade for potential contribution to moisture damage is to determine the type and distribution of subgrade materials present under the project. The first choice to obtain this information is the USDA County Soils Map discussed in Module 2-1, which will provide a very detailed picture of the soils present. A second choice would be to use soil test results taken from construction records which were used to delineate soil types for the original design.

When the county soil maps are available, subgrade boundaries and types can be marked directly on a strip map of the project. The drainage class of each subgrade type can be noted from the soils map information and the Natural Drainage Index value selected from Figure 2-4.27. When using only soil classification data, the approximate relationships in Figure 2-4.28 can be used to determine the Natural Drainage Index. The problem of extensive reworking of soils during grading, for example, will not produce a change in the NDI which will develop over several years once the pavement is completed. When a pavement is being investigated for rehabilitation, the intermixing will have been negated and the altered soils will have assumed the properties of the undisturbed underlying soil. Thus, the soil maps will very likely still accurately reflect the soil under the pavement. Extensive cuts or fills, greater than 4-6 feet (1.2-1.8 m) may take much longer to approach the condition of the original soil. For these localized areas, they should be examined individually and assigned an average value indicating whether the cut or fill improved the material present under the roadway and improved the position relative to the water table.

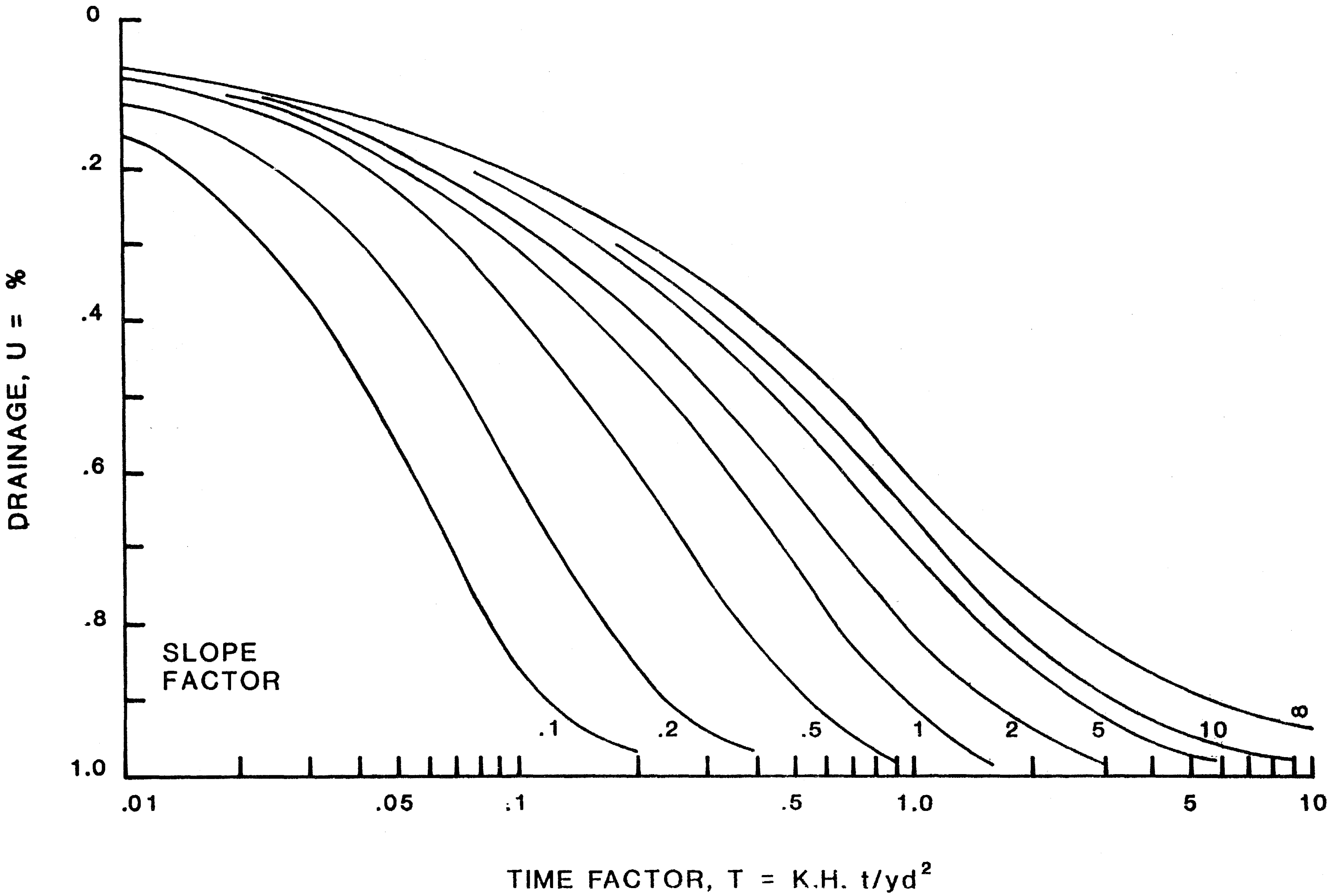


Figure 2-4.25. Chart for Determining Drainage Percentage.

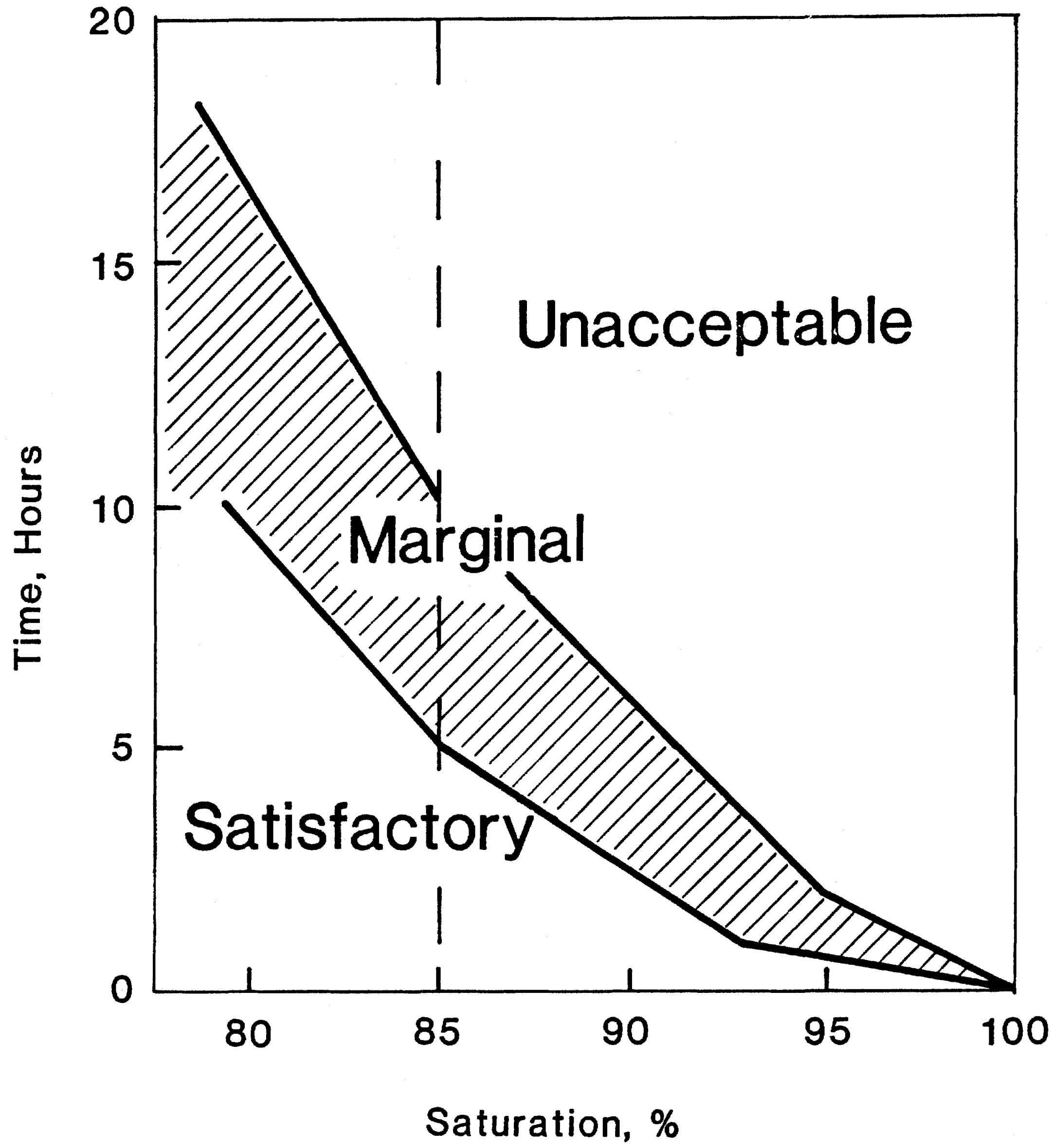


Figure 2-4.26. Graph Illustrating Three Levels of Drainage Quality, Calculated from DRAINIT Program.

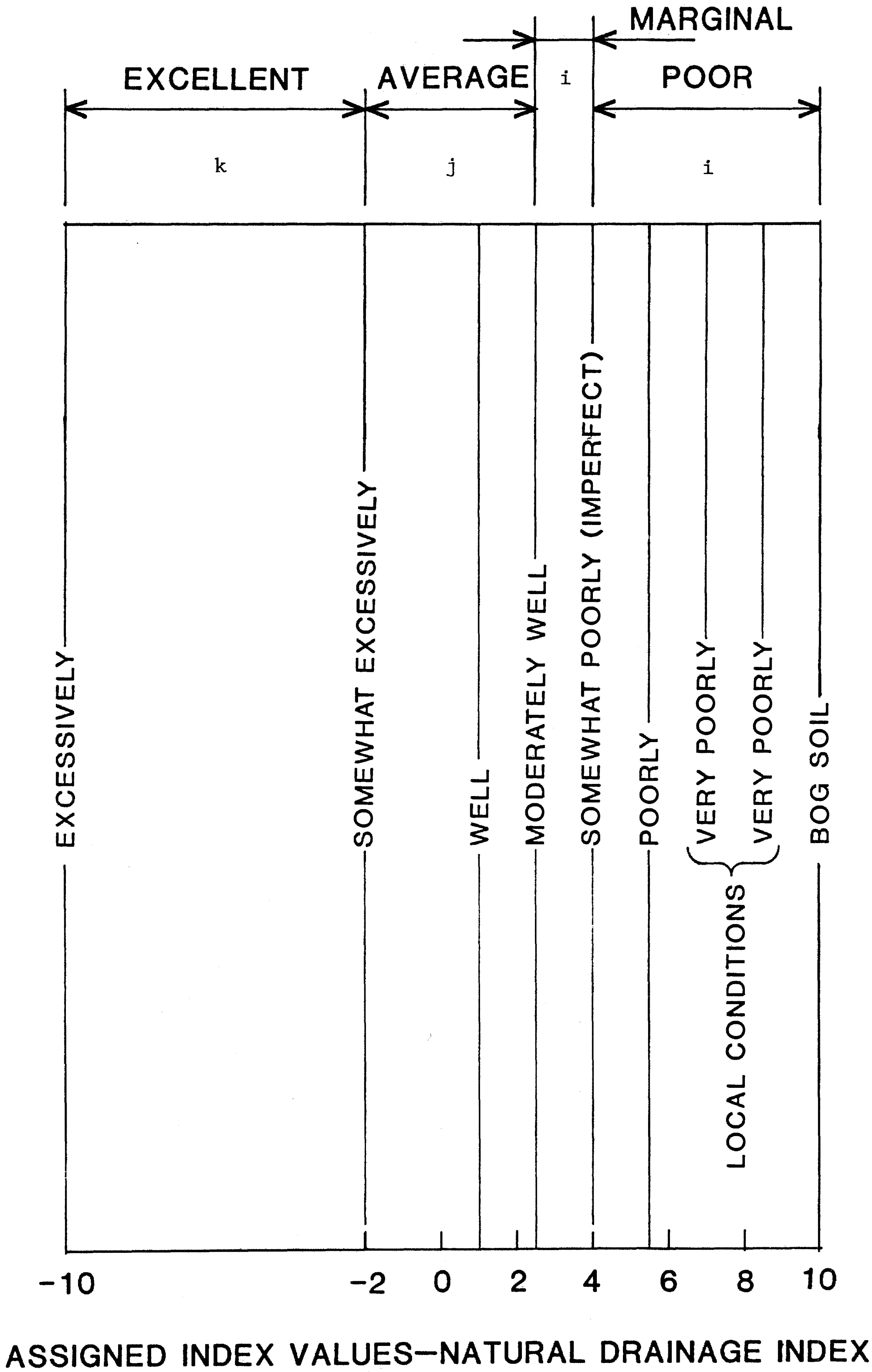


Figure 2-4.27. Chart for Assigning Natural Drainage Index Value From Soil Map Data.

AASHTO Class. \ Position in Topography.	Top of Hills	Sides of Hills	Depressions
A-1 A-3	k	k	k
A-2-4 A-2-5	k	k	
A-2-6 A-2-7	k	k	j
A-4	k	j	j
A-5	j	j	i
A-6	j	i	i
A-7-5 A-7-6	i	i	i

A group index above 20 will alter the NDI rating, k → j, j → i.

A group index below 5 will alter the NDI rating, i → j, j → k.

Figure 2-4.28. Approximate Relationships for Obtaining the Natural Drainage Index from Soil Classification Data.

These two parameters to describe the granular material and the subgrade provide information which can describe the potential for the pavement to perform well without drainage or require drainage for good performance.

7.0 INFLUENCE OF DRAINAGE ON PAVEMENT DESIGN

Properly designed and installed drainage should remove water from the pavement materials. This water removal increases the support capacity of the roadbed materials which prolongs the life of the pavement. The relationship between resilient modulus and saturation has been shown previously in the Modules 2-1 and 2-2. If a pavement can be drained adequately, the thickness selection should reflect a decreased requirement with a more reliable design to resist moisture.

The AASHTO Design Guide recommends the following drainage times be considered for pavement design:

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	(Water will not drain)

The calculation time procedure demonstrated in the previous section can be used to generate relative comparisons of drainage potential. Selection of a specific level of saturation to be achieved in drainage (85 percent recommended) will alter the actual magnitude of the time increment used to select the specific quality of drainage. Table 2-4.2 presents the recommended values, m, to use to adjust structural layer coefficients in the flexible design procedure. Table 2-4.3 shows the recommended drainage parameters, C_d to be used in rigid pavement design. These tables require a quality of drainage to be selected based on the ability of the base to drain freely, and the level of moisture to which the pavement will be exposed.

The calculation schemes presented in this section can be grouped to provide an approximate indication of the quality of drainage as shown in Figure 2-4.29. This chart provides a means of using material properties and soil information to arrive at a rational indicator of drainage quality to be used in pavement design. One further consideration not directly shown here is the amount of water available in the pavement system. The greater period during the year in which the pavement is exposed to saturation levels of water, the greater the deterioration in performance. Table 2-4.4 contains recommendations as to the period of the year in which the pavement structure will have water present. These can be used with the calculations previously shown to select the drainage coefficients shown previously which are used in the pavement design process to modify material properties. The smaller the drainage coefficient, the worse the drainage in the pavement, and the greater the need for some means to remove water in a positive manner.

This material evaluation to establish the drainage factor is important in establishing the long term performance of the pavement. A thorough understanding of the soil-moisture interaction is required to select the appropriate factor. The selection of increased thickness must not be used in place of selecting the appropriate drainage factor. Local pavements can be

Table 2-4.2. Recommended m_i Values For Modifying Structural Layer Coefficients of Untreated Base and Subbase Materials in Flexible Pavements.

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

Table 2-4.3. Recommended C_d Values for Modifying Structural Layer Coefficients of Untreated Base and Sub-base Materials in Rigid Pavements.

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.25 - 1.20	1.20 - 1.15	1.15 - 1.10	1.10
Good	1.20 - 1.15	1.15 - 1.10	1.10 - 1.00	1.00
Fair	1.15 - 1.10	1.10 - 1.00	1.00 - 0.90	0.90
Poor	1.10 - 1.00	1.00 - 0.90	0.90 - 0.80	0.80
Very Poor	1.00 - 0.90	0.90 - 0.80	0.80 - 0.70	0.70

Table 2-4.4. Suggested Seasons Length (Months) For The Six U. S. Climatic Regions.

U.S. Climatic Region	Season (Roadbed Soil Moisture Condition)			
	Winter (Roadbed Frozen)	Spring-Thaw (Roadbed Saturated)	Spring/Fall (Roadbed Wet)	Summer (Roadbed Dry)
I	0.0*	0.0	7.5	4.5
II	1.0	0.5	7.0	3.5
III	2.5	1.5	4.0	4.0
IV	0.0	0.0	4.0	8.0
V	1.0	0.5	3.0	7.5
VI	3.0	1.5	3.0	4.5

*Number of months for the season.

		Base Drainabilities		
		A Acceptable	M Marginal	U Unacceptable
Subgrade Drainability	G Good k	EXC	G	F to P
	F Fair j	G	F	P to VP
	P Poor i	F to P	P to VP	VP

Quality of Drainage Criteria

EXC - Excellent
G - Good
F - Fair
P - Poor
VP - Very Poor

Figure 2-4.29. Chart for Estimating Quality of Drainage From Base and Subgrade Drainability Calculations.

evaluated to assist the design engineer in establishing the parameters of drainage as they exist on his pavements by evaluating existing pavements.

8.0 EXAMPLE PROBLEM

The pavement is located in Central Illinois. It is to be a four lane divided Interstate pavement. The pavement type will be plain jointed with 15 foot joint spacing. The subgrade soil is predominantly A-6 (19). The topography is level with a water table depth of 4-6 feet which seasonally can be as high as the pavement surface for a structure at grade. The cross section of this pavement is shown in Figure 2-4.30.

The granular base proposed for this pavement will have the following properties:

	Crushed Stone Base	Gravel Base
Top Size	1.5 inches	1.5 inches
-#200 material	11 percent	9 percent
Plasticity Index	NP	3.5
Compacted Density	139 pcf	140 pcf
Gradation		
Sieve Size	Percent Passing	
1.5	100	100
1	90	98
3/4	80	-
1/2	68	74
#4	50	49
#40	21	23
#200	11	9

The subgrade is an A-6 material with a group index of 19. The topography is flat with no relief.

8.1 Drainage Coefficients

1. Calculate the time for drainage for this base material.
2. Estimate the amount of time this base material will be saturated.
3. Determine the drainage coefficient for either a flexible or rigid pavement to be constructed here.

8.2 Drainage System

Design a drainage layer to function as a base for this pavement, and the associated longitudinal drain for removal of the water collected.

1. Determine Net Water Inflow

$q_i = 0.71$ cfd/ft^2
q_v = negligible
$q_m = 2$ mm/day

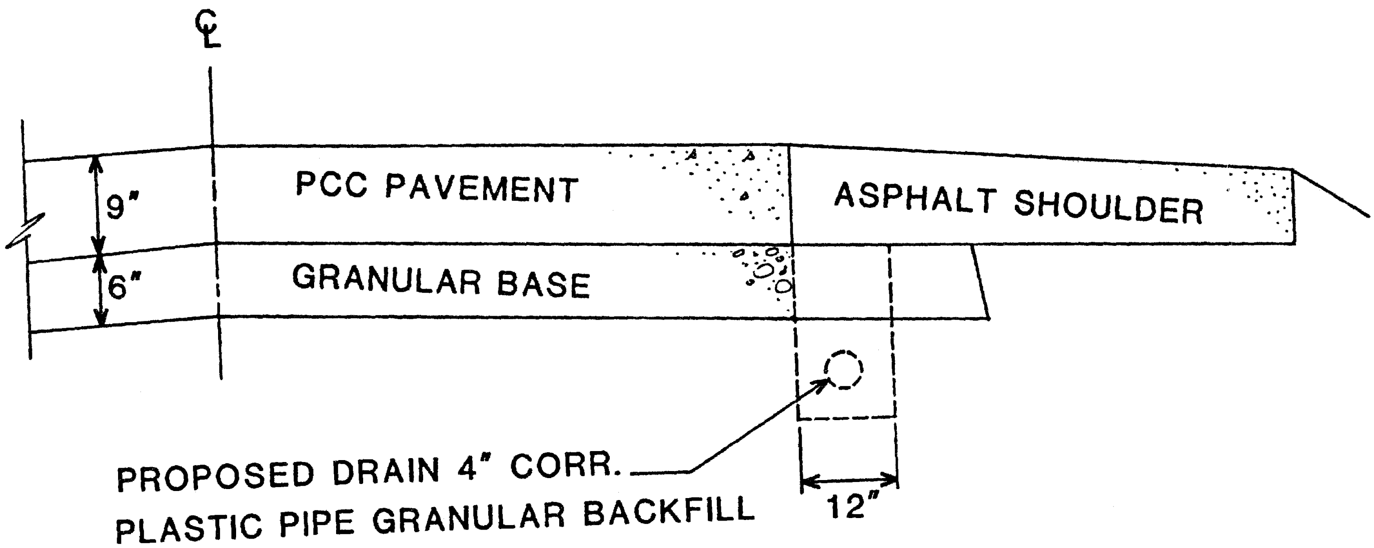


FIGURE G17. CROSS SECTION FOR DESIGN EXAMPLE

Figure 2-4.30. Cross Section For Design Example.

2. Check thickness and permeability of granular layer

3. Check drainage trench width

4. Check Filter Design

5. Determine Diameter of Pipe Collector System

9.0 SUMMARY

Various subdrainage systems were identified and the material and drainage requirements were discussed. The necessary steps for designing subsurface drainage were defined and an example design was presented using the FHWA Highway Subdrainage Design Manual. A subsurface drainage evaluation system (MAD System) has been explained and discussed.

Water may permeate the sides of a pavement structure, particularly where coarse-grained layers are present or where surface drainage facilities within the vicinity are inadequate. The water table may rise in the winter and spring seasons producing seasonal periods of low support in the roadbed soils and low resilient modulus in the paving materials which translates to lowered structural layer coefficients.

Surface water will enter joints and cracks in the pavement, penetrate at the edges of the surfacing, or percolate through the surfacing and shoulders. This form of water entry can be alleviated by adequate sealing and maintenance. Water may move vertically in capillaries or interconnected water films. The possibility of this is greatest in silty soils and produces excessive frost heave problems. Water may move in vapor form, depending upon adequate temperature gradients and air void space, this movement typically involves small total amounts of moisture.

Materials can and should be evaluated to establish their drainability capabilities. A procedure has been presented which allows these calculations to be completed to provide input into the AASHTO Design selection procedure for drainage adjustments.

10.0 REFERENCES

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MODULE 2-5

MEASURES OF PAVEMENT PERFORMANCE

1.0 INSTRUCTIONAL OBJECTIVES

This module discusses pavement performance, the types of pavement performance and the significant indicators by which pavement performance is measured. The four major performance indicators, distress, serviceability, structural capacity, and surface friction, are briefly discussed.

Upon completion of this module the participants will be able to accomplish the following:

1. Describe the differences between functional and structural pavement performance.

2. List and describe the pavement performance characteristics of the four major performance indicators.

3. Discuss the relative magnitudes of the characteristics of the four performance indicators as they relate to pavement performance.

4. Relate distresses to pavement failure in the different pavement types.

2.0 DEFINITION OF PAVEMENT PERFORMANCE

The measurable adequacy of a pavement's structural and functional service over a specified design period is termed its "performance." The public assesses pavement performance in subjective ways. As users, they are concerned with ride quality, safety, appearance, and convenience. As taxpayers, they expect pavements to last long enough to justify the cost of their construction.

A pavement provides functional service by giving users a safe and comfortable ride for a specified range of speed. Functional service is comprised of several factors, including:

1. Acceptable ride quality.
2. Adequate surface friction for safety.
3. Appropriate geometry for safety.
4. Appearance of geometric adequacy.
5. Appearance of condition.

(Note: Geometric safety and appearance, while important to user satisfaction, are beyond the scope of this manual. For more information on these topics, the reader is referred to A Policy on Geometric Design of Highways and Streets by the American Association of State Highway and Transportation Officials.)

A pavement provides structural service by supporting traffic loadings and withstanding environmental influences. The types and thicknesses of materials used to construct the pavement layers dictate how the pavement will perform structurally.

Structural and functional adequacy are closely related, but are not entirely interdependent. Structural deterioration of a pavement is manifested to some extent in diminished functional adequacy, in the forms of increased roughness, noise, and even hazard to vehicles and their occupants. However, some types of structural deterioration can occur and progress to fairly advanced stages without being noticeable to users. It is also possible for a pavement's functional adequacy to decrease without any significant change in structural adequacy (e.g., loss of skid resistance).

3.0 PERFORMANCE INDICATORS

User assessments of pavement performance are, by their subjective nature, difficult to utilize directly in pavement design. There are, however, characteristics of pavements which (1) can be measured quantitatively and (2) can be correlated to the users' subjective assessments of performance. These characteristics are called "performance indicators." The four major performance indicators are:

1. Visible distress.
2. Structural adequacy.
3. Surface friction.
4. Roughness/serviceability.

How these indicators are related to performance and how they can be measured are described in the remainder of this module.

3.1 Visible Distress

Distress occurs in pavements as a result of complex interactions of design, construction, materials, traffic, environment, and maintenance procedures. Visible distress should be quantified with respect to the following three parameters:

1. Type.
2. Severity.
3. Quantity.

The most significant distress types which occur in asphalt and concrete pavements are described in this section. Severity levels have been defined for each of the distress types (1). Although the thresholds between distress levels are somewhat arbitrary, in general, they reflect the relative severity of the distress state. Low-severity distress is evidence that deterioration mechanisms are occurring, but the distress is not serious enough to significantly affect ride quality or warrant immediate repair. At the other extreme, high-severity distress is evidence of substantial deterioration that is likely to contribute to poor ride quality, and if it poses a safety hazard, warrants immediate repair.

Distress quantities are measured in one of several ways, such as:

1. Average magnitude in inches or millimeters over the entire project (e.g., for faulting and rut depth).
2. Total linear quantity (e.g., for transverse cracking).
3. Total area quantity (e.g., for block cracking).
4. Percent of pavement area affected (e.g. for map cracking).
5. Number of occurrences (e.g., for settlements and heaves).

Distress types, severities, and quantities are determined during a distress survey of the pavement. The information is manually observed and either recorded on paper or entered into portable computers. Photographs can also be useful in noting distress locations. Work is underway on high-speed survey equipment which eliminates the need for manual interpretation (4). See Reference 1 for more information on distress types, quantities, and measurements.

3.1.1 Distresses in Concrete Pavements

This section describes the appearance and probable causes of distresses which occur to jointed plain, jointed reinforced, and continuously reinforced concrete pavements, and concrete overlays.

Blowup

Blowups occur in concrete pavement joints or cracks when high temperature, infiltration of incompressives into the joints or cracks or presence of reactive aggregate expands the concrete which produces excessive compressive stress in the slab. The compressive stress built up in the pavement is relieved by shattering or by buckling upward at the joint or crack.

The accumulated infiltration of incompressibles into joints and cracks over a period of years, high temperatures, joint spacing, and the presence of "D" cracking or reactive aggregates are major factors in blowups. Blowups seldom occur in pavements with joint spacings less than 20 feet.

Most blowups occur during the spring or early summer. Blowups usually occur in the late afternoon when the temperature peaks. Blowups have been known to occur in CRC pavements at transverse cracks where the steel has ruptured, permitting large crack openings and infiltration of incompressibles.

Corner Break

A corner break is a crack that intersects a transverse joint and the pavement edge in a jointed concrete pavement at a distance less than 6 feet on each side from the corner of the slab. A corner break extends vertically through the entire slab thickness. It should not be confused with a corner spall, which is a crack running at an angle through the depth of the slab, and which is typically within 1 foot of the slab corner.

Heavy repeated loads, loss of slab support, poor load transfer across the joint, and thermal curling and moisture warping stresses all contribute to corner breaks.

Durability "D" Cracking

Durability ("D") cracking is a series of closely spaced crescent-shaped cracks that appear at a concrete slab pavement surface adjacent and roughly parallel to transverse and longitudinal joints and the free edge of the pavement.

"D" cracking is caused by freezing and thawing of saturated aggregates in the concrete. Many varieties of chert and limestone found in the Midwest are susceptible to this type of distress. Typically, "D" cracking is more severe at the bottom of the pavement than at the top. It first becomes evident on the surface at transverse joints and cracks. It then appears along longitudinal and shoulder joints and progresses toward the center of the slab.

Faulting

Faulting is a difference in elevation of two adjacent slabs at a joint or crack in a jointed concrete pavement. Faulting is caused by build up of loose materials under the approach slab and depression of the heave slab. The build-up of eroded or infiltrated materials is caused by pumping of water under the slab as heavy wheel loads pass over the joint or crack. Important factors in the development of faulting is the lack of good load transfer across the transverse joints and cracks and the pressure of an erodable subbase material.

Joint Seal Damage

Joint seal damage exists when incompressibles and/or water can infiltrate into the joints. Sealant failures may be due to poor durability, inappropriate reservoir shape, or sealant properties. Many sealant failures can be attributed to the fact that the depth-to-width ratio (shape factor) of the sealant reservoir is not appropriate for the sealant type and the magnitude of joint movements. Common types of joint sealant failures are:

1. Extension of the sealant from the joint.

2. Weed growth.

3. Hardening of the sealant (oxidation).

4. Loss of bond between the sealant and the joint reservoir sides (adhesive failure).

5. Absence of sealant.

6. Splitting of the sealant (cohesive failure).

Longitudinal Cracking

Longitudinal cracks run generally parallel to the centerline of the pavement. Improper construction of longitudinal joints, warping or curling of the concrete slab, and foundation movement due to swelling soils or frost heave are the major causes of longitudinal cracks.

Pumping

Pumping is caused by vertical movement of the slab at joints and cracks under wheel loads, which results in ejection of loose materials and water from under the pavement through the cracks and joints. Pumping becomes serious when the volume of displaced materials is such that large areas under slab corners are left unsupported. This results in increased stresses under loads, increased deflections, and eventually slab cracking. Pumping can also cause loose particles to collect between the joint faces and restrict slab expansion, which can lead to blowups.

Punchout

Punchouts are the major structural distress in CRC pavements. A punchout occurs when a section of concrete slab between two closely spaced transverse shrinkage crack breaks and is depressed into the subbase under repeated loads. This usually occurs at the outside edge of the truck lane. Evidence of pumping (fines on the pavement or shoulder surface) is often found near punchouts.

Reactive Aggregate Distress

Reactive aggregates contain silicates or carbonates which react with alkalies in Portland cement in the presence of moisture and cause expansion of the concrete. This expansion can be sufficient to cause build-up of compressive stress in the slab, resulting in fine, closely spaced longitudinal or map cracks and eventually severe spalling. Reactive aggregate distress may affect the entire slab area, but the resulting spalling usually begins at joints and cracks and progresses to larger portions of the slab area.

Scaling and Map Cracking

Map cracking, or crazing, is a network of shallow, fine, hairline cracks extending only into the upper surface of the slab. Scaling is the disintegration and loss of material from the concrete surface. Map cracking is caused by over-finishing, and may progress to scaling. Scaling is also caused by reinforcing steel being too close to the slab surface.

Spalling at Joints and Cracks

Spalling is cracking, breaking or chipping of the pavement at a joint or crack. Spalling differs from cracking in that the spall results in a crack running diagonally from the surface of the pavement to the face of the joint or crack.

Spalling is caused by infiltration of incompressibles into the joints or cracks, joint lock-up, misaligned or corroded dowel bars, poorly designed joint forming inserts, or "D" cracking incompressibles.

Transverse and (Diagonal) Cracking

Transverse cracks are cracks through the pavement slab that run generally perpendicular to the centerline across the slab. Repeated traffic loadings, curling and/or warping and drying shrinkage are the major causes of transverse cracking.

In contrast to plain and jointed reinforced concrete pavements, transverse cracks are an expected phenomenon in CRC pavements and are generally not considered a distress. The shrinkage of the concrete produces tensile stress in the concrete, which is opposed by subbase friction and steel reinforcement. The reinforcing steel holds the cracks tight and insures load transfer through aggregate interlock. The transverse cracks become a distress when the reinforcing steel ruptures, the cracks open, and their widening leads to the intrusion of water, incompressibles and deicing chemicals into the cracks and loss of aggregate interlock.

3.1.2 Distresses in Bituminous Pavements

This section describes the appearance and probable causes or distresses which occur in asphalt-surface pavements and asphalt overlays.

Alligator Cracking

Alligator or fatigue cracking is a series of interconnecting cracks caused by fatigue failure of the asphalt concrete surface (or stabilized base) under repeated traffic loading. The cracking generally initiates at the bottom of the asphalt surface (or stabilized base) where tensile stress and strain is highest under a wheel load. The cracks propagate to the surface initially as one or more longitudinal parallel cracks. After repeated traffic loading the cracks connect, forming many-sided, sharp-angled pieces that develop a pattern resembling chicken wire or the skin of an alligator. The pieces are usually less than 1 foot on the longest side. Alligator cracking occurs only in areas that are subjected to repeated traffic loadings. Therefore, it would not occur over an entire area unless the entire area was subjected to traffic loading. Alligator cracking does not occur in asphalt overlays of concrete pavements, unless the slabs have disintegrated. Alligator cracking is considered a major structural distress.

Bleeding

Bleeding is a film of bituminous material on the pavement surface which creates a shiny, glass-like, reflecting surface that usually becomes quite sticky. Bleeding is caused by excessive amounts of asphalt cement in the mix and/or low air void contents. It occurs when asphalt fills the voids of the mix during hot weather and then expands out onto the surface of the pavement. Since the bleeding process is not reversible during cold weather, asphalt will accumulate on the surface.

Longitudinal Cracking

Longitudinal cracking other than alligator cracking is parallel to the pavement's centerline or laydown direction within the lane width. It may be caused by:

1. A poorly constructed paving lane joint.

2. Shrinkage of the AC surface due to low temperatures or hardening of the asphalt.

3. A reflective crack caused by cracks beneath the surface course, including cracks in PCC slabs (but not at lane edge longitudinal joints).

Raveling and Weathering

Raveling and weathering are the wearing away of the pavement surface caused by the dislodging of aggregate particles (raveling) and loss of asphalt binder (weathering). They generally indicate that the asphalt binder has hardened significantly.

Rutting

A rut is a longitudinal surface depression in the wheel path. Pavement uplift may occur along the sides of the rut. In many instances ruts are noticeable only after a rainfall, when the wheel paths are filled with water. Rutting usually stems from permanent deformation in any or all of the pavement layers or subgrade, usually caused by consolidation or lateral movement of the materials due to traffic loads. Rutting may be caused by plastic movement in the mix in hot weather, inadequate compaction during construction, or abrasion by studded tires. Significant rutting can lead to major structural failure of the pavement and hydroplaning potential.

Transverse Cracking

Transverse (non-reflective) cracks extend across the pavement centerline or direction of laydown and are caused by shrinkage of the AC surface due to low temperatures or hardening of the asphalt. These types of cracks are not usually load associated, although loads can cause them to develop.

Reflection Cracking

This distress occurs on pavements having an asphalt concrete (AC) surface over a jointed Portland cement concrete (PCC) slab. The cracks occur in the AC over cracks or joints in the underlying PCC slab. Reflection cracking is caused mainly by movement of the PCC slab beneath the AC surface because of thermal and moisture changes; it is generally not load associated. However, traffic loading may cause a breakdown of the AC near the initial crack, resulting in spalling. A knowledge of slab dimensions beneath the AC surface will help to identify these cracks.

3.2 Structural Adequacy

Structural design of a pavement begins with a forecast of the types and volumes of vehicle traffic expected to use the pavement over a specified future period of time. Pavement layer materials are then selected, and thicknesses of these layers are determined which will provide a structure capable of supporting the forecasted traffic over the time period without failing.

In Blocks 3 and 4, the various procedures available for designing flexible and rigid pavements are described in detail. Any procedure used has some criterion associated with it for judging whether or not a pavement is structurally adequate. This criterion is generally some measure of pavement condition which has been arbitrarily defined to represent "failure." The failure criterion may be distress related, such as:

1. 50 percent of slabs cracked in a jointed concrete pavement.

2. 10 percent of the pavement area alligator-cracked in an asphalt concrete pavement.

3. Average rutting of 0.5 inch on an asphalt concrete pavement.

Structural adequacy can also be assessed by nondestructive testing, in which electronic sensors measure the deflection of the pavement under a load of known magnitude. A magnitude of deflection which corresponds to poor structural capacity can be used as a failure criterion for either concrete or asphalt pavements.

3.3 Surface Friction

The term "surface friction" refers to the characteristic of pavement surfaces that inhibits skidding of tires. Three factors influence a pavement's surface friction: microtexture, macrotexture, and transverse slope. Microtexture refers to the "roughness" of the coarse aggregate particle surfaces and of the binder (either asphalt or cement paste) including the fine aggregate particles. Microtexture contributes to friction by adhesion with vehicle tires. Macrotexture refers to the overall texture of the pavement, which is controlled by the surface finishing technique in concrete pavements and by the aggregate size and gradation in asphalt pavements. Transverse slope contributes to surface friction by removing water from the pavement surface. A slope of at least 1 percent is necessary for adequate surface drainage.

Inadequate surface friction of a newly constructed pavement may be attributed to:

1. Poor construction techniques.

2. Poor materials selection or mix design.

3. Poor transverse slope.

Construction is typically controlled by field inspection and thus is beyond the control of the pavement design engineer. The design engineer does have control of materials selection, mix design, and transverse slope requirements.

Several methods exist for measuring surface friction. In general, the methods involve dragging a test tire over the wet pavement surface, and measuring the resistance between the pavement surface and the tire. See References 3 and 4 for further information on friction measuring equipment and procedures.

3.4 Roughness/Serviceability

Users assess the condition of a pavement largely in terms of ride quality. Serviceability is thus defined as the ability of a pavement to provide a safe and comfortable ride to users. Serviceability is quantified in terms of the Present Serviceability Rating (PSR). A group of individuals ride the pavement and rate it on a scale from 0 to five, where:

0-1 = Very poor
1-2 = Poor
2-3 = Fair
3-4 = Good
4-5 = Very good

A PSR of five indicates a perfect pavement, whereas a rating of zero represents an impassable pavement.

At the AASHO Road Test, PSRs were correlated with measurements of roughness and distress (patching and cracking). The regression analysis done with the Road Test data resulted in equations for a Present Serviceability Index (PSI), which predicts serviceability from roughness and distress.

The general form of the PSI regression equation is:

$$PSI = A_o + A_1 R + A_2 F_1 + A_3 F_2$$

where:

PSI = Pavement Serviceability Index
A_o, A_1, A_2, A_3 are constants
R = Measure of roughness
F_1, F_2 = Physical measurement of cracks and patches

Thus the PSI is a computed number obtained from the regression equation containing roughness and distress terms which correlates well with the subjective rating (PS) of a panel of users. The original PSI equations developed from the AASTHO Road Test are given as follows:

<u>Flexible Pavement</u>

$$PSI = 5.03 - 1.91 \log (1 + SV) - 1.38 (RD)^2 - 0.01 (C+P)^{0.5}$$

where:

SV = slope variance over the section from CHLOE profilometer, X 10 $(in/ft)^2$
RD = mean rut depth, inches
C = Class 2 + Class 3 alligator cracking, $ft^2/1000\ ft^2$
P = Patching, $ft^2/1000\ ft^2$

The slope variance, which represents roughness, is a statistical measure of the profile of the pavement measured in both wheel paths, as given by:

$$SV = [\ (\Sigma y^2 - (1/n)(\ \Sigma y)^2)]/(n-1)$$

where:

$y =$ the difference in elevation between two points one foot apart, in/ft

$n =$ number of measurements

Rigid Pavement

$$PSI = 5.41 - 1.78 \log (1 + SV) - .09(C + P)^{0.5}$$

where:

C = Class 2 and sealed cracks, $ft^2/1000\ ft^2$
P = Patching, $ft^2/1000\ ft^2$

Many agencies have converted the slope variance term to a roughness index as measured with any of several types of roughness measuring equipment. More recently, non-contact roughness measuring equipment has been introduced for rapid data collection procedures (4).

In both of these equations, measured roughness (indicated by the slope variance) dominates the computed value of serviceability. The practical implication of this is that roughness most significantly affects the users' assessment of ride quality.

Even though the regression equations contain distress terms, the distress does not contribute much to the accuracy of the equations because the roughness term dominates. Many agencies have simply correlated the subjective panel rating directly with roughness measurements. The correlation of the rating panel of highway users with roughness can provide an entirely adequate equation by which to compute the PSI. See References 3 and 4 for more information on roughness measuring equipment and procedures. Development of the serviceability concept is explained in detail in Reference 2.

4.0 SUMMARY

Pavement performance is the measurable adequacy of a pavement's functional and structural service over a specified design period. Pavement performance is a very real concern to the public, as roadway users and as taxpayers.

Users assess pavement performance in subjective and imprecise ways. In order to achieve good performance through design, the engineer must understand how quantitative measures are called "performance indicators." The four major performance indicators are visible distress, structural adequacy, surface friction, and roughness/serviceability.

5.0 REFERENCES

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MODULE 2-6

VEHICLE AND TRAFFIC CONSIDERATIONS

1.0 INSTRUCTIONAL OBJECTIVES

This module presents traffic considerations which must be included to design a pavement structure. The important conversion of mixed traffic to equivalent single axle loads is presented. The concept of a load equivalency factor and a damage factor and the role in calculating equivalent axle loads are presented. The interaction between tire pressure, axle type, axle load, tire type, and vehicle configuration as they affect pavement performance are discussed and the influence of these parameters on pavement design are presented.

Upon completion of this module the participant will be able to accomplish the following:

1. Use traffic vehicle classification counts and W-4 tables to calculate the 18 kip equivalent single axle loads (ESAL) for a pavement design project.

2. Describe differences in damage factors and explain where differences arise and how the differences can impact a pavement design procedure.

3. Describe the difference between projections of traffic based on total traffic, trucks, vehicle classification and the impact on ESAL predictions by using different counts.

4. Discuss the impact of tire pressures, tire types and vehicle configuration on the calculation of damage factors and the 18 Kip ESAL.

5. Discuss the need for adequate traffic sampling plans, and conversion of data to project specific data, and how weigh in motion (WIM) information can improve the evaluation process.

6. Determine truck lane distribution factors for different highway conditions and describe the importance of this factor.

2.0 INTRODUCTION

One of the most important input parameters for any pavement thickness design method is traffic. The traffic on any pavement is a combination of many different types of vehicles having different gross weights and distributions of axle weights. This is unlike the AASHO Road Test which used uniform load conditions on each loop and compared the rate of deterioration of each pavement.

For many design procedures the loads applied to the pavement structure by each car and truck during a period must be converted into a number of equivalent 18 kip single axle loads (ESAL) which is used as the standard by nearly all highway agencies. The process of collecting mixed traffic data and converting it into equivalent 18-kip ESALs is complex. It is also important to realize that axle type and weight are far more critical for pavement performance than vehicle gross weight. Two different trucks could have the same gross weight but cause greatly different amounts of damage to a pavement, depending upon their axle configuration.

To accomplish the above, the pavement engineer must understand the process of gathering, projecting, and converting traffic data into equivalent 18 kip single axle loads using the damage factor concept. Each vehicle loading causes the pavement layers to deflect under the applied load. The method used to convert these mixed traffic loadings to an equivalent number of standard load applications is often done through theoretical comparisons of the damage done by a load to the damage done by the standard load.

3.0 TRAFFIC ANALYSIS

A complete analysis of the traffic to develop adequate ESALs for design must include an analysis of the traffic volume and the weight distribution among the specific vehicles using the highway.

3.1 Traffic Volume

The minimum traffic data collected for a pavement design is the average daily traffic (ADT), and the average daily truck traffic (ADTT). This can be obtained from actual traffic counts on the roadway being designed, if the existing pavement structure is to be reconstructed. Counts on nearby highways, although less accurate, can be used for new pavements. Traffic volume maps which show the traffic counts over time on various roadways within a given area can also be used although they are far less accurate than an actual count. The designer must remember to adjust the numbers collected for the changes in traffic which occur on a daily (weekday versus weekend) basis and on a seasonal (summer versus winter) basis. Figure 2-6.1 illustrates this information plotted over time.

As will be discussed later in this module, the AADT is not the only factor which must be used in the calculation of the ESALs expected to use the pavement. Additional data includes the rate of traffic growth, directional distribution, and lane distribution. Two common approaches to develop the average traffic over the design period are used. Simple rate formulas, and more conveniently, tables and graphs, are available to project the initial ADT to the ADT at the end of the design period. An average value of ADT initial and ADT final will then be calculated and called the design ADT. Alternatively, simple growth factor formulas can be used to calculate a traffic projection factor that when multiplied by the initial ADT will give the design ADT.

3.2.1 Truck Volume

When the amount of traffic has been determined, two problems still exist before the traffic data can be used for pavement design. First, the percentage of trucks in the traffic stream must be determined. Second, and most importantly, the classifications of the trucks in the traffic stream must be determined. Different classes of trucks will carry different loadings, and they should not be combined in a gross manner. The use of percent trucks to calculate an ESAL value per truck must be done from precise determinations of the different classes of trucks in the traffic stream.

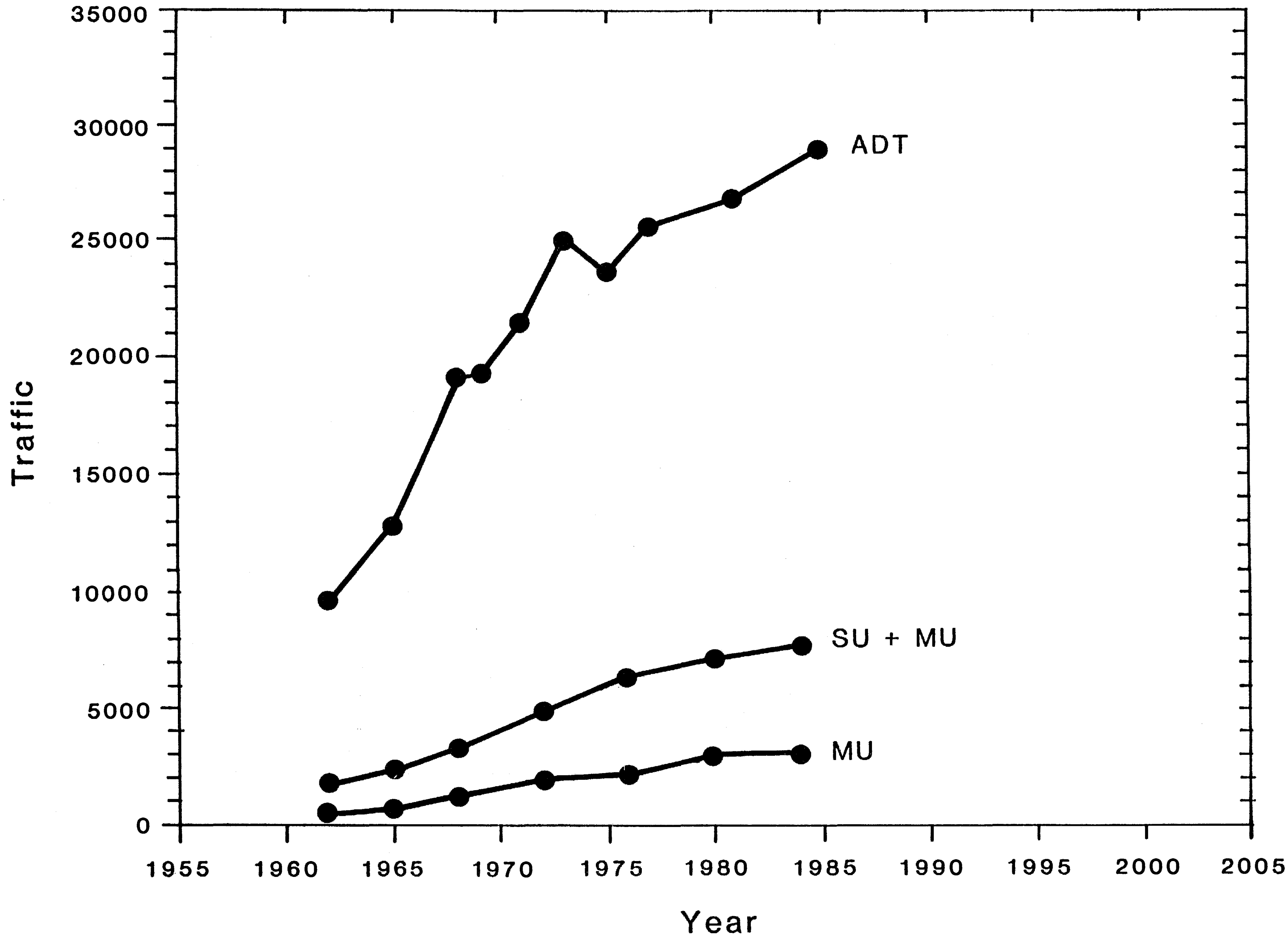

Figure 2-6.1. Growth in ADT Over Time.

This truck classification information must be determined accurately. Figure 2-6.2 shows a summary of truck distributions on various highway classes in the United States. The average percentage of commercial trucks (excluding four tired panels and pickups) is about 15 percent, but varies up to 50 percent. Tables such as these should not be used as an estimator as they are traditionally low because there have been large increases in truck traffic over the past twenty years.

The number and class of trucks which will travel over a new or reconstructed highway can best be determined from a vehicle count at that specific location. Traffic count data collected on other routes, or from the state's standard collection procedures must be thoroughly evaluated before deciding that a count at the project is not needed. Although this general information is very useful, the project specific data can be much more accurate if performed properly. However, if the project specific survey is not performed properly, the results can have errors of 50 percent or higher (1).

3.2.2 Vehicle Classifications

There are thirteen classes of vehicles which should be used to develop the design ESALs. These classes are:

1. Motorcycles (optional).
2. Passenger Cars.
3. Other Two-Axle, Four-Tire Single Unit vehicles.
4. Buses.
5. Two-Axle, Six-Tire, Single Unit truck.
6. Three-Axle Single Unit Trucks.
7. Four or more Axle Single Unit Trucks.
8. Four or Less Axle Single Trailer Trucks.
9. Five-Axle Single Trailer Trucks.
10. Six or More Axle Single Trailer Trucks.
11. Five or Less Axle Multi-Trailer Trucks.
12. Six-Axle Multi-Trailer Trucks.
13. Seven or More Axle Multi-Trailer Trucks.

It is critical to record the actual numbers of each vehicle class for design purposes. As will be discussed later, vehicle class can have a significant impact on the calculation of 18-kip ESAL values for structural design of a new pavement.

3.3 Truck Weights

Just as the distribution of truck types is important, the weights of the trucks in each classification is equally important. As the shipping industry continues to undergo changes with deregulation and changing population centers, the distribution of classes of trucks, and their cargoes will continue to change. If these changes are not accurately recorded, predictions for pavement design cannot be done accurately. The two factors which must be considered together for pavement design are the gross weight of the truck, and the axle distribution of this weight.

		Traffic			Maximum axle loads, kips	
			ADTT**			
Axle-load category	Description	ADT	%	Per day	Single axles	Tandem axles
1	Residential streets Rural and secondary roads (low to medium*)	200-800	1-3	up to 25	22	36
2	Collector streets Rural and secondary roads (high*) Arterial streets and primary roads (low*)	700-5000	5-18	40-1000	26	44
3	Arterial streets and primary roads (medium*)	3000-12,000 2 lane	8-30	500-5000+	30	52
	Expressways and urban and rural Interstate (low to medium*)	3000-50,000+ 4 lane or more				
4	Arterial streets, primary roads, expressways (high*)	3000-20,000 2 lane	8-30	1500-8000+	34	60
	Urban and rural Interstate (medium to high*)	3000-150,000+ 4 lane or more				

*The descriptors high, medium, or low refer to the relative weights of axle loads for the type of street or road; that is, "low" for a rural Interstate would represent heavier loads than "low" for a secondary road.
*Trucks — two-axle, four-tire trucks excluded.

Figure 2-6.2. Percentages of Four-Tire Single Units and Trucks on Various Highway Systems. (10)

3.3.1 Truck Weigh Stations

Knowledge of truck weights and axle load distribution has largely been determined using truck weigh stations. Several deficiencies exist with this procedure, however. First, the number of stations in any given state is limited. According to a recent TRB survey, the number of stations vary from a low of 5 in one state to a high of 64 in another state (8). Not all of these locations may be permanent sites, however, and the average number of sites per state (excluding the one state with 64 reported loadometer stations) was only 15 locations. Unless a loadometer station is located close to the area of the pavement being designed, it is questionable as to whether the load and distribution data can be applied accurately to the roadway under design.

Second, not many weigh stations operate continuously. Some are open only on weekdays or only during daylight hours. Numerous studies have demonstrated that the truck traffic and weight distribution varies significantly during the week, as well as during the day (5). Other weigh stations operate on a 24 hour basis but only for one or two days per week. The data thus obtained is biased and must be adjusted. Third, it is well known that overloaded trucks can bypass the loadometer stations when they are open. The data collected will represent a sample, and the collection procedures must insure that they are unbiased, or the sample will not represent the actual weight distribution.

3.3.2 Equipment

Again, it is vital to use actual data collected from the project in question. Actual data on the project will provide a more accurate design. This requires portable devices that can be taken to the project in question. This equipment can consist of portable static scales, or the newer weigh in motion scales (WIM). At present, the static scales have more accuracy than the WIM scales, but new advances are closing this gap. The WIM devices have advantages that can surpass any inaccuracy. The high speed WIM offers a high degree of flexibility in data collection and reporting with the use of high speed digital processors. There is little or no disruption to the traffic stream. They provide for some measure of concealment which can enhance the data credibility and more than offset any inaccuracy in the actual load measurement because vehicles which might have deliberately avoided a standard scale, will be captured on the WIM. Further, they are highly practical in heavy volume areas because they do not interrupt the traffic stream, and are relatively easy to install. They can be installed in each lane of a multi-lane facility to provide an accurate distribution of the loadings and traffic in each lane which can be used in design.

3.4 Components of a Monitoring Program

To obtain accurate 18 kip ESAL data for a pavement design, a weighing program should provide the following (1):

1. Truck volumes by truck classification.

2. Volume growth rate for each truck type.

3. 18-kip ESAL factor, or truck factor, for each truck classification and the corresponding growth rate for this ESAL factor.

4. Lane distribution for the truck traffic, preferably by truck type.

5. Variations in the average weight of each truck classification by lane, reflecting the assumption that trucks traveling in the slow lanes are more heavily loaded than those in the fast lanes and thus have above-average truck factors.

6. The percentage of equivalent axle loads (ESALs) occurring during the spring freeze-thaw cycle months.

7. The percentage of truck traffic expected to experience creep speeds during the hot summer months.

The importance of collecting accurate data on the weight and volumes of each classification of truck using the pavement has been shown in a recent series of data collected by FHWA, and in individual states using Weigh in Motion equipment. While the growth rate for all vehicles may be only 3 - 5 percent, the growth rate for multi-axle trucks is some 3 - 5 times greater. This is shown in Table 2-6.1 (4). If this difference is not accounted for in the design it will produce a significant error in the total number of ESALs used to design the pavement.

3.5 Directional Distribution Estimates

3.5.1 Volume Estimates for Trucks

For pavement design purposes it is necessary to divide the total number of cars and trucks into two parts, one part for each direction of travel. In most cases this is done by simply dividing the ADT and ADTT volume in half. For some situations, however, more vehicles may be travelling in one direction than the other. Traffic count data should indicate any bias in directional travel.

3.5.2 Axle Weights for Trucks

More importantly, there is often a significant difference in the gross weight of the vehicles travelling in one particular direction. A large ocean port facility where heavily loaded trucks are carrying products to the port, but returning much more lightly loaded is one such example. This must be accounted for through the truck axle weight distribution and design truck factor to be discussed later.

3.6 Lane Distribution Estimates

A design lane must be selected to design the pavement. The design lane is the lane carrying the larger number of ESALs. For a two lane highway, assuming no directional differences, either lane can be used as the design lane and 50 percent of the total ADTT volume and ESAL numbers will be the same in each of the two lanes. On roadways having four or more lanes, the design lane is usually taken as the outside driving lane. Most of the truck traffic, and therefore most of the equivalent axle loads, will operate in

Table 2-6.1. Growth Rates For Different Classes of Trucks.

LOCATION	ANNUAL GROWTH RATES (PERCENT)			
	ALL VEHICLES	ALL TRUCKS	TRUCKS 5 AXLE OR GREATER	18 KIP EAL'S
MT - I-94, Wilbaux to ND	3.4	5.4	6.3	10.3
MT - I-90, Billings to Laurel	4.0	8.1	13.1	18.9
MT - I-90, Butte	2.6	4.2	9.9	N/A
MT - I-90, Superior West	3.9	9.5	10.4	10.4
WA - I-90, Cle Elum, WA	2.1	N/A	5.6	8.5
WA - I-5, Vancouver to Olympia, WA	3.6	N/A	10.1	13.2
OR - I-5, Ashland, OR	4.1	8.8	11.7	12.6
OR - I-84, Oregon-Idaho Border	4.4	8.0	10.4	11.1
AVERAGE	3.5	7.33	9.69	12.1

this lane. The actual distribution of loads, however, will vary with the particular roadway and the number of lanes in each direction, and should be determined on the project of interest. Table 2-6.2 provides approximate data on the percentage of truck traffic which might be travelling in the design lane (6). While these numbers should be used with great caution, the principle behind these percentages must be used to accurately determine the exact number of ESALs to be used in the thickness design. Further, differences between lanes can be used as support for designing different thicknesses for the different lanes if this is an available option.

4.0 CONVERSION OF MIXED TRAFFIC TO ESALs FOR PAVEMENT DESIGN

The principle of design is that different wheel loads on a pavement structure produce different stresses and strains in the pavement layers. Further, different layer thickness or materials will also produce different stresses. It is the stresses or strains which produce deterioration in the pavement. Because different loadings produce different stresses they also produce different rates of deterioration in the pavement. Because of these differences it is normally necessary to reduce the mixture of traffic to a common value for design. This common base is the 18-kip ESAL, as discussed previously, and the conversion is done with the load equivalency factors.

4.1 Equivalency Factors

The AASHO Road Test forms the experimental model for converting mixed traffic loads to a common input parameter. At the Road Test, similar pavement designs were loaded with different axle types and loadings so that the direct effect of each axle type and load on the loss of present serviceability could be determined independently over a range of pavement designs.

The traffic equivalency factor as developed from the Road Test is a numerical factor that expresses the relationship of a given axle load to a standard axle load for the pavement to reach a given present serviceability index. The relationship is as follows:

$$\text{LEF} = \frac{\text{Number of 18-kip single axle load applications to cause a given loss of serviceability}}{\text{Number of X-kip single (or tandem) axle load applications to cause the same loss of serviceability}}$$

For example, consider two identical pavement structures that carried the following loads until the serviceability dropped from 4.2 to 2.5:

1. 100,000 load applications of an 18-kip single axle

2. 14,347 load applications of a 30-kip single axle

The load equivalency factor would be 100,000/14,347 for an LEF of 6.97 for the 30-kip single axle. this means that 14,347 passes of the 30-kip single axle produces as much damage as 100,000 applications of the 18-kip single axle.

Table 2-6.2. Truck Distribution for Multiple Lane Highways (129 Counts in 6 States, 1982-83) (6).

One-Way ADT	2 Lanes (One-Direction)		3+ Lanes (One-Direction)		
	Inner	Outer	Inner*	Center	Outer
2,000	6**	94	6	12	82
4,000	12	88	6	18	76
6,000	15	85	7	21	72
8,000	18	82	7	23	70
10,000	19	81	7	25	68
15,000	23	77	7	28	65
20,000	25	75	7	30	63
25,000	27	73	7	32	61
30,000	28	72	8	33	59
35,000	30	70	8	34	58
40,000	31	69	8	35	57
50,000	33	67	8	37	55
60,000	34	66	8	39	53
70,000	--	--	8	40	52
80,000	--	--	8	41	51
100,000	--	--	9	42	49

* Combined inner one or more lanes.

** Percent of all trucks in one direction (note that the proportion of trucks in one direction sums to 100 percent).

Because each pavement structure responds differently to any one axle load, each pavement type or structure will have different load equivalency factors. If the point of failure is changed, the relationship between number of loadings to reach the level of failure will also change. This is why there are load equivalency factors for both flexible and rigid pavements, and why they change for different structural levels of the pavement (SN) and why they change for different failure levels. Examples of the AASHTO load equivalency factors for flexible and rigid pavements are shown in Figure 2-6.3 and Figure 2-6.4 respectively for a terminal serviceability (P_t) of 2.5.

4.2 Calculations of ESAL Applications

The traffic stream must be defined and broken into units corresponding to data available in the load equivalency tables in order to convert the mixed traffic stream into an equivalent number of 18-kip single axle load applications. For example:

Axle Type	Weight (Pounds)	Number of Axles	Equivalency Factor	Number of 18-kip ESALs
Single	18,000	100	1.00	100
Single	22,000	100	2.18	218
Tandem	18,000	1,000	0.08	80
Tandem	48,000	10	4.17	42
Totals		1,210		440

Thus, a total of 440 18-kip ESALs would pass over this pavement in the time period covered by the measurement of the traffic stream. It is important to have an accurate count of axles per truck and the weight per axle on the truck. With this information, the truck factor can be calculated for use in this conversion of mixed traffic.

It is necessary to compute the average 18-kip equivalent single axle load per truck (the truck factor). It is recommended that a truck factor be computed for each general truck classification (e.g., six or more axle single-trailer trucks, five or less axle multi-trailer trucks, six-axle multi-trialer trucks, etc.). Some design procedures use an average truck factor for all trucks in the traffic stream. This is an approximate method and not recommended for design, however. Engineers should obtain loadometer date (e.g., specific weigh station W-4 tables, weigh-in-motion scales) for the specific highway under consideration and not use these averages if at all possible.

Due to many economic, legal, political and other factors, there have been and are currently underway, changes in the axle configuration for trucks. It also appears that a larger number of trucks are running with more cargo. This has produced a dramatic shift in recent years in the mean truck factor (ESAL per truck), and the use of today's mean truck factor will result in grossly underestimating the total 18-kip ESAL over the next ten or twenty years.

The observation of historical increases in the truck factor is very important to understanding the amount of change that has taken place and that

Figure 2-6.3. AASHTO Axle Load Equivalency Factors, Flexible Pavement. (2).

Table D.4. Axle load equivalency factors for flexible pavements, single axles and p_t 2.5.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0004	.0004	.0003	.0002	.0002	.0002
4	.003	.004	.004	.003	.002	.002
6	.011	.017	.017	.013	.010	.009
8	.032	.047	.051	.041	.034	.031
10	.078	.102	.118	.102	.088	.080
12	.168	.198	.229	.213	.189	.176
14	.328	.358	.399	.388	.360	.342
16	.591	.613	.646	.645	.623	.606
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.57	1.49	1.47	1.51	1.55
22	2.48	2.38	2.17	2.09	2.18	2.30
24	3.69	3.49	3.09	2.89	3.03	3.27
26	5.33	4.99	4.31	3.91	4.09	4.48
28	7.49	6.98	5.90	5.21	5.39	5.98
30	10.3	9.5	7.9	6.8	7.0	7.8
32	13.9	12.8	10.5	8.8	8.9	10.0
34	18.4	16.9	13.7	11.3	11.2	12.5
36	24.0	22.0	17.7	14.4	13.9	15.5
38	30.9	28.3	22.6	18.1	17.2	19.0
40	39.3	35.9	28.5	22.5	21.1	23.0
42	49.3	45.0	35.6	27.8	25.6	27.7
44	61.3	55.9	44.0	34.0	31.0	33.1
46	75.5	68.8	54.0	41.4	37.2	39.3
48	92.2	83.9	65.7	50.1	44.5	46.5
50	112.	102.	79.	60.	53.	55.

Table D.5. Axle load equivalency factors for flexible pavements, tandem axles and p_t of 2.5.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0001	.0001	.0001	.0000	.0000	.0000
4	.0005	.0005	.0004	.0003	.0003	.0002
6	.002	.002	.002	.001	.001	.001
8	.004	.006	.005	.004	.003	.003
10	.008	.013	.011	.009	.007	.006
12	.015	.024	.023	.018	.014	.013
14	.026	.041	.042	.033	.027	.024
16	.044	.065	.070	.057	.047	.043
18	.070	.097	.109	.092	.077	.070
20	.107	.141	.162	.141	.121	.110
22	.160	.198	.229	.207	.180	.166
24	.231	.273	.315	.292	.260	.242
26	.327	.370	.420	.401	.364	.342
28	.451	.493	.548	.534	.495	.470
30	.611	.648	.703	.695	.658	.633
32	.813	.843	.889	.887	.857	.834
34	1.06	1.08	1.11	1.11	1.09	1.08
36	1.38	1.38	1.38	1.38	1.38	1.38
38	1.75	1.73	1.69	1.68	1.70	1.73
40	2.21	2.16	2.06	2.03	2.08	2.14
42	2.76	2.67	2.49	2.43	2.51	2.61
44	3.41	3.27	2.99	2.88	3.00	3.16
46	4.18	3.98	3.58	3.40	3.55	3.79
48	5.08	4.80	4.25	3.98	4.17	4.49
50	6.12	5.76	5.03	4.64	4.86	5.28
52	7.33	6.87	5.93	5.38	5.63	6.17
54	8.72	8.14	6.95	6.22	6.47	7.15
56	10.3	9.6	8.1	7.2	7.4	8.2
58	12.1	11.3	9.4	8.2	8.4	9.4
60	14.2	13.1	10.9	9.4	9.6	10.7
62	16.5	15.3	12.6	10.7	10.8	12.1
64	19.1	17.6	14.5	12.2	12.2	13.7
66	22.1	20.3	16.6	13.8	13.7	15.4
68	25.3	23.3	18.9	15.6	15.4	17.2
70	29.0	26.6	21.5	17.6	17.2	19.2
72	33.0	30.3	24.4	19.8	19.2	21.3
74	37.5	34.4	27.6	22.2	21.3	23.6
76	42.5	38.9	31.1	24.8	23.7	26.1
78	48.0	43.9	35.0	27.8	26.2	28.8
80	54.0	49.4	39.2	30.9	29.0	31.7
82	60.6	55.4	43.9	34.4	32.0	34.8
84	67.8	61.9	49.0	38.2	35.3	38.1
86	75.7	69.1	54.5	42.3	38.8	41.7
88	84.3	76.9	60.6	46.8	42.6	45.6
90	93.7	85.4	67.1	51.7	46.8	49.7

Table D.6. Axle load equivalency factors for flexible pavements, triple axles and p_t of 2.5.

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0000	.0000	.0000	.0000	.0000	.0000
4	.0002	.0002	.0002	.0001	.0001	.0001
6	.0006	.0007	.0005	.0004	.0003	.0003
8	.001	.002	.001	.001	.001	.001
10	.003	.004	.003	.002	.002	.002
12	.005	.007	.006	.004	.003	.003
14	.008	.012	.010	.008	.006	.006
16	.012	.019	.018	.013	.011	.010
18	.018	.029	.028	.021	.017	.016
20	.027	.042	.042	.032	.027	.024
22	.038	.058	.060	.048	.040	.036
24	.053	.078	.084	.068	.057	.051
26	.072	.103	.114	.095	.080	.072
28	.098	.133	.151	.128	.109	.099
30	.129	.169	.195	.170	.145	.133
32	.169	.213	.247	.220	.191	.175
34	.219	.266	.308	.281	.246	.228
36	.279	.329	.379	.352	.313	.292
38	.352	.403	.461	.436	.393	.368
40	.439	.491	.554	.533	.487	.459
42	.543	.594	.661	.644	.597	.567
44	.666	.714	.781	.769	.723	.692
46	.811	.854	.918	.911	.868	.838
48	.979	1.015	1.072	1.069	1.033	1.005
50	1.17	1.20	1.24	1.25	1.22	1.20
52	1.40	1.41	1.44	1.44	1.43	1.41
54	1.66	1.66	1.66	1.66	1.66	1.66
56	1.95	1.93	1.90	1.90	1.91	1.93
58	2.29	2.25	2.17	2.16	2.20	2.24
60	2.67	2.60	2.48	2.44	2.51	2.58
62	3.09	3.00	2.82	2.76	2.85	2.95
64	3.57	3.44	3.19	3.10	3.22	3.36
66	4.11	3.94	3.61	3.47	3.62	3.81
68	4.71	4.49	4.06	3.88	4.05	4.30
70	5.38	5.11	4.57	4.32	4.52	4.84
72	6.12	5.79	5.13	4.80	5.03	5.41
74	6.93	6.54	5.74	5.32	5.57	6.04
76	7.84	7.37	6.41	5.88	6.15	6.71
78	8.83	8.28	7.14	6.49	6.78	7.43
80	9.92	9.28	7.95	7.15	7.45	8.21
82	11.1	10.4	8.8	7.9	8.2	9.0
84	12.4	11.6	9.8	8.6	8.9	9.9
86	13.8	12.9	10.8	9.5	9.8	10.9
88	15.4	14.3	11.9	10.4	10.6	11.9
90	17.1	15.8	13.2	11.3	11.6	12.9

Figure 2-6.4. AASHTO Axle Load Equivalency Factors, Rigid Pavement. (2).

Table D.13. Axle load equivalency factors for rigid pavements, single axles and p_t of 2.5.

Axle Load (kips)	Slab Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002	.0002
4	.003	.002	.002	.002	.002	.002	.002	.002	.002
6	.012	.011	.010	.010	.010	.010	.010	.010	.010
8	.039	.035	.033	.032	.032	.032	.032	.032	.032
10	.097	.089	.084	.082	.081	.080	.080	.080	.080
12	.203	.189	.181	.176	.175	.174	.174	.173	.173
14	.376	.360	.347	.341	.338	.337	.336	.336	.336
16	.634	.623	.610	.604	.601	.599	.599	.599	.598
18	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	1.51	1.52	1.55	1.57	1.58	1.58	1.59	1.59	1.59
22	2.21	2.20	2.28	2.34	2.38	2.40	2.41	2.41	2.41
24	3.16	3.10	3.22	3.36	3.45	3.50	3.53	3.54	3.55
26	4.41	4.25	4.42	4.67	4.85	4.95	5.01	5.04	5.05
28	6.05	5.75	5.92	6.29	6.51	6.81	6.92	6.98	7.01
30	8.16	7.57	7.79	8.29	8.79	9.14	9.35	9.45	9.52
32	10.8	10.1	10.1	10.7	11.4	12.0	12.3	12.6	12.7
34	14.1	13.0	12.9	13.6	14.5	15.4	16.0	16.4	16.5
36	18.2	16.7	16.4	17.1	18.3	19.5	20.4	21.0	21.3
38	23.1	21.1	20.6	21.3	22.7	24.3	25.6	26.4	27.0
40	29.1	26.5	25.7	26.3	27.9	29.9	31.6	32.9	33.7
42	36.2	32.9	31.7	32.2	34.0	36.3	38.7	40.4	41.5
44	44.6	40.4	38.8	39.2	41.0	43.8	46.7	49.1	50.6
46	54.5	49.3	47.1	47.3	49.2	52.3	55.9	59.0	61.4
48	66.1	59.7	56.9	56.8	58.7	62.1	66.3	70.3	73.4
50	79.4	71.7	68.2	67.8	69.6	73.3	78.1	83.0	87.1

Table D.14. Axle load equivalency factors for rigid pavements, tandem axles and p_t of 2.5.

Axle Load (kips)	Slab Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0006	.0006	.0005	.0005	.0005	.0005	.0005	.0005	.0005
6	.002	.002	.002	.002	.002	.002	.002	.002	.002
8	.007	.006	.006	.005	.005	.005	.005	.005	.005
10	.015	.014	.013	.013	.012	.012	.012	.012	.012
12	.031	.028	.026	.026	.025	.025	.025	.025	.025
14	.057	.052	.049	.048	.047	.047	.047	.047	.047
16	.097	.089	.084	.082	.081	.081	.080	.080	.080
18	.155	.143	.136	.133	.132	.131	.131	.131	.131
20	.234	.220	.211	.206	.204	.203	.203	.203	.203
22	.340	.325	.313	.308	.305	.304	.303	.303	.303
24	.475	.462	.450	.444	.441	.440	.439	.439	.439
26	.644	.637	.627	.622	.620	.619	.618	.618	.618
28	.855	.854	.852	.850	.850	.850	.849	.849	.849
30	1.11	1.12	1.13	1.14	1.14	1.14	1.14	1.14	1.14
32	1.43	1.44	1.47	1.49	1.50	1.51	1.51	1.51	1.51
34	1.82	1.82	1.87	1.92	1.95	1.96	1.97	1.97	1.97
36	2.29	2.27	2.35	2.43	2.48	2.51	2.52	2.52	2.53
38	2.85	2.80	2.91	3.03	3.12	3.16	3.18	3.20	3.20
40	3.52	3.42	3.55	3.74	3.87	3.94	3.98	4.00	4.01
42	4.32	4.16	4.30	4.55	4.74	4.86	4.91	4.95	4.96
44	5.26	5.01	5.16	5.48	5.75	5.92	6.01	6.06	6.09
46	6.36	6.01	6.14	6.53	6.90	7.14	7.28	7.36	7.40
48	7.64	7.16	7.27	7.73	8.21	8.55	8.75	8.86	8.92
50	9.11	8.50	8.55	9.07	9.68	10.14	10.42	10.58	10.66
52	10.8	10.0	10.0	10.6	11.3	11.9	12.3	12.5	12.7
54	12.8	11.8	11.7	12.3	13.2	13.9	14.5	14.8	14.9
56	15.0	13.8	13.6	14.2	15.2	16.2	16.8	17.3	17.5
58	17.5	16.0	15.7	16.3	17.5	18.6	19.5	20.1	20.4
60	20.3	18.5	18.1	18.7	20.0	21.4	22.5	23.2	23.6
62	23.5	21.4	20.8	21.4	22.8	24.4	25.7	26.7	27.3
64	27.0	24.6	23.8	24.4	25.8	27.7	29.3	30.5	31.3
66	31.0	28.1	27.1	27.6	29.2	31.3	33.2	34.7	35.7
68	35.4	32.1	30.9	31.3	32.9	35.2	37.5	39.3	40.5
70	40.3	36.5	35.0	35.3	37.0	39.5	42.1	44.3	45.9
72	45.7	41.4	39.6	39.8	41.5	44.2	47.2	49.8	51.7
74	51.7	46.7	44.6	44.7	46.4	49.3	52.7	55.7	58.0
76	58.3	52.6	50.2	50.1	51.8	54.9	58.6	62.1	64.8
78	65.5	59.1	56.3	56.1	57.7	60.9	65.0	69.0	72.3
80	73.4	66.2	62.9	62.5	64.2	67.5	71.9	76.4	80.2
82	82.0	73.9	70.2	69.6	71.2	74.7	79.4	84.4	88.8
84	91.4	82.4	78.1	77.3	78.9	82.4	87.4	93.0	98.1
86	102.	92.	87.	86.	87.	91.	96.	102.	108.
88	113.	102.	96.	95.	96.	100.	105.	112.	119.
90	125.	112.	106.	105.	106.	110.	115.	123.	130.

Table D.15. Axle load equivalency factors for rigid pavements, triple axles and p_t of 2.5.

Axle Load (kips)	Slab Thickness, D (inches)								
	6	7	8	9	10	11	12	13	14
2	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001	.0001
4	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003	.0003
6	.001	.001	.001	.001	.001	.001	.001	.001	.001
8	.003	.002	.002	.002	.002	.002	.002	.002	.002
10	.006	.005	.005	.005	.005	.005	.005	.005	.005
12	.011	.010	.010	.009	.009	.009	.009	.009	.009
14	.020	.018	.017	.017	.016	.016	.016	.016	.016
16	.033	.030	.029	.028	.027	.027	.027	.027	.027
18	.053	.048	.045	.044	.044	.043	.043	.043	.043
20	.080	.073	.069	.067	.066	.066	.066	.066	.066
22	.116	.107	.101	.099	.098	.097	.097	.097	.097
24	.163	.151	.144	.141	.139	.139	.138	.138	.138
26	.222	.209	.200	.195	.194	.193	.192	.192	.192
28	.295	.281	.271	.265	.263	.262	.262	.262	.262
30	.384	.371	.359	.354	.351	.350	.349	.349	.349
32	.490	.480	.468	.463	.460	.459	.458	.458	.458
34	.616	.609	.601	.596	.594	.593	.592	.592	.592
36	.765	.762	.759	.757	.756	.755	.755	.755	.755
38	.939	.941	.945	.948	.950	.951	.951	.951	.951
40	1.14	1.15	1.16	1.17	1.18	1.18	1.18	1.18	1.18
42	1.38	1.38	1.41	1.44	1.45	1.46	1.46	1.46	1.46
44	1.65	1.65	1.70	1.74	1.77	1.78	1.78	1.78	1.79
46	1.97	1.96	2.03	2.09	2.13	2.15	2.16	2.16	2.16
48	2.34	2.31	2.40	2.49	2.56	2.58	2.59	2.60	2.60
50	2.76	2.71	2.81	2.94	3.02	3.07	3.09	3.10	3.11
52	3.24	3.15	3.27	3.44	3.56	3.62	3.66	3.68	3.68
54	3.79	3.66	3.79	4.00	4.16	4.26	4.30	4.33	4.34
56	4.41	4.23	4.37	4.63	4.84	4.97	5.03	5.07	5.09
58	5.12	4.87	5.00	5.32	5.59	5.76	5.85	5.90	5.93
60	5.91	5.59	5.71	6.08	6.42	6.64	6.77	6.84	6.87
62	6.80	6.39	6.50	6.91	7.33	7.62	7.79	7.88	7.93
64	7.79	7.29	7.37	7.82	8.33	8.70	8.92	9.04	9.11
66	8.90	8.29	8.33	8.83	9.42	9.88	10.17	10.33	10.42
68	10.1	9.4	9.4	9.9	10.6	11.2	11.5	11.7	11.8
70	11.5	10.6	10.6	11.1	11.9	12.6	13.0	13.3	13.5
72	13.0	12.0	11.8	12.4	13.3	14.1	14.7	15.0	15.2
74	14.6	13.5	13.2	13.8	14.8	15.8	16.5	16.9	17.1
76	16.5	15.1	14.8	15.4	16.5	17.6	18.4	18.9	19.2
78	18.5	16.9	16.5	17.1	18.2	19.5	20.5	21.1	21.5
80	20.6	18.8	18.3	18.9	20.2	21.6	22.7	23.5	24.0
82	23.0	21.0	20.3	20.9	22.2	23.8	25.2	26.1	26.7
84	25.6	23.3	22.5	23.1	24.5	26.2	27.8	28.9	29.6
86	28.4	25.8	24.9	25.4	26.9	28.8	30.5	31.8	32.8
88	31.5	28.6	27.5	27.9	29.4	31.5	33.5	35.1	36.1
90	34.8	31.5	30.3	30.7	32.2	34.4	36.7	38.5	39.8

will affect the future. The following data illustrate the change in average U. S. Interstate rural highway flexible pavement ESAL per truck from 1971 to 1982:

Year	ESAL/Truck
1971	0.595
1975	0.691
1979	0.766
1982	0.929

An example of the change in truck factor for one state on Interstate highways is shown in Figure 2-6.5, in the W-4 data curve. However, even reliance on historical data may be erroneous if truck categories are grouped in large classifications as shown here, rather than separated by weight classifications. More accurate breakdown by truck type and weight classification is required for accurate predictions of the truck factor. Figure 2-6.6 also shows that the mean truck factor (for all trucks in this case) may be considerably different when the standard W-4 tables are used than if weigh-in-motion scales are used. The WIM mean truck factor is a full twenty percent greater than the W-4 value.

4.3 Calculation Procedure for ESAL Applications

To calculate ESAL applications, first convert the traffic data into a truck load factor. This is done with the W-4 data discussed previously. The truck factor must not be calculated as a general ESAL factor for all trucks in the traffic stream. The truck factor must be calculated for each class of truck, as described previously. Figure 2-6.7 contains data from the weighing of 5-axle, tractor semi-trailer trucks at a specific weigh station (2). The traffic equivalency factors were obtained from Figure 2-6.3 and Figure 2-6.4. The number of axles recorded represents the grouping or distribution of weights within the axle load intervals indicated. The ESAL's by axle load interval are summed to produce the total ESAL's for 165 trucks of the type which were weighed. Similar calculations must be made for each class of truck in the traffic stream at this weigh station.

The truck load factors for each classification are then used in Figure 2-6.8 to calculate the total 18-kip ESAL applications for the pavement over its analysis period. The following components must be completed:

1. Column A is the daily volume count of each vehicle type taken from data collected at classification count stations representative of the design location, for the base year.

2. Column B contains the growth factor assigned to each class of vehicle as taken from Figure 2-6.9. This accounts for the stated fact that not all vehicles are increasing at the same rate.

3. Column C is the product of Column A times Column B multiplied by 365 days to produce the accumulated applications of specific vehicle types during the analysis period.

4. Column D is the individual ESAL factor for each truck type (truck load factor calculated as in Figure 2-6.7).

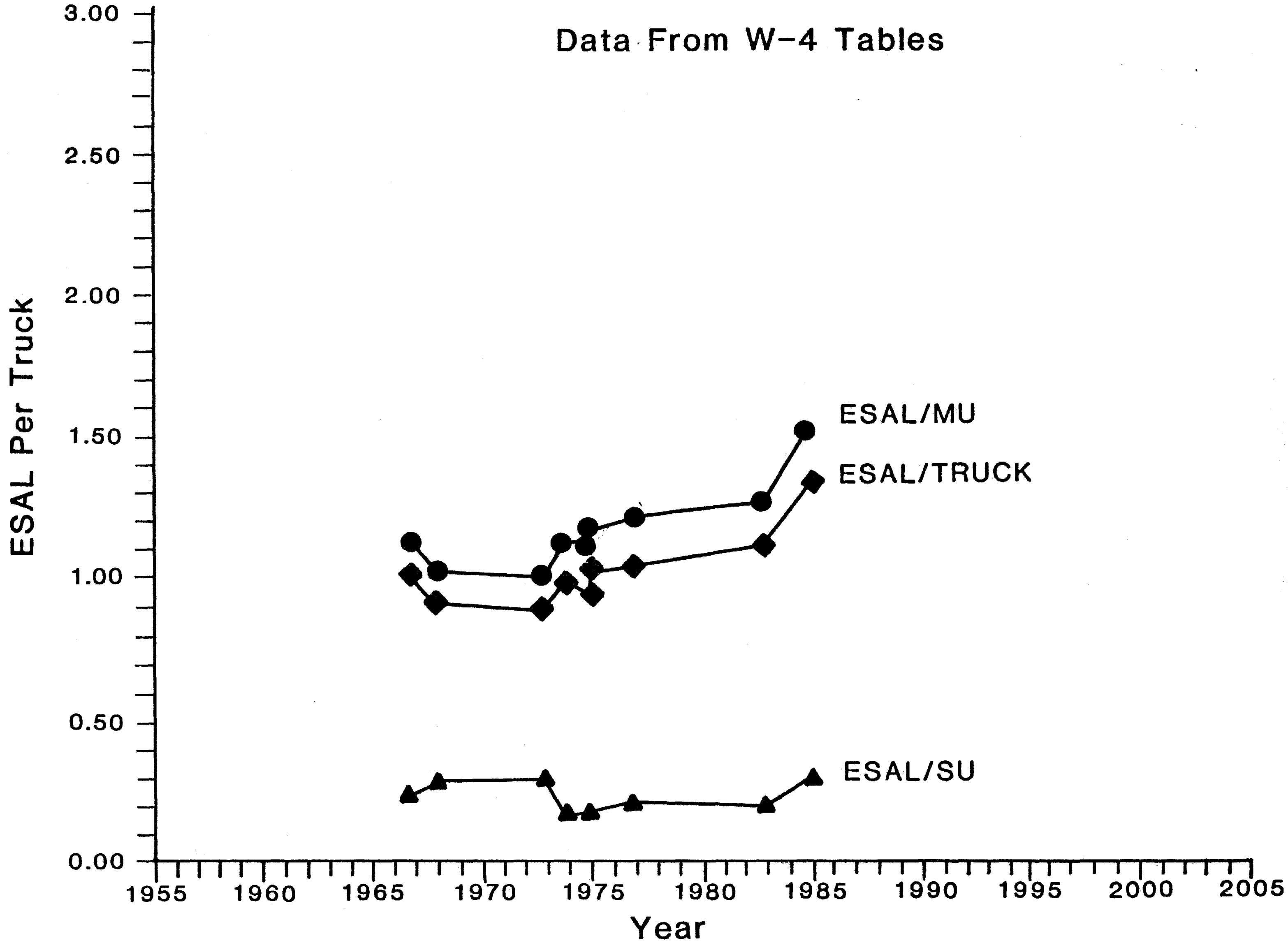

Figure 2-6.5. Change in Truck Factor for Different Classes of Vehicle.

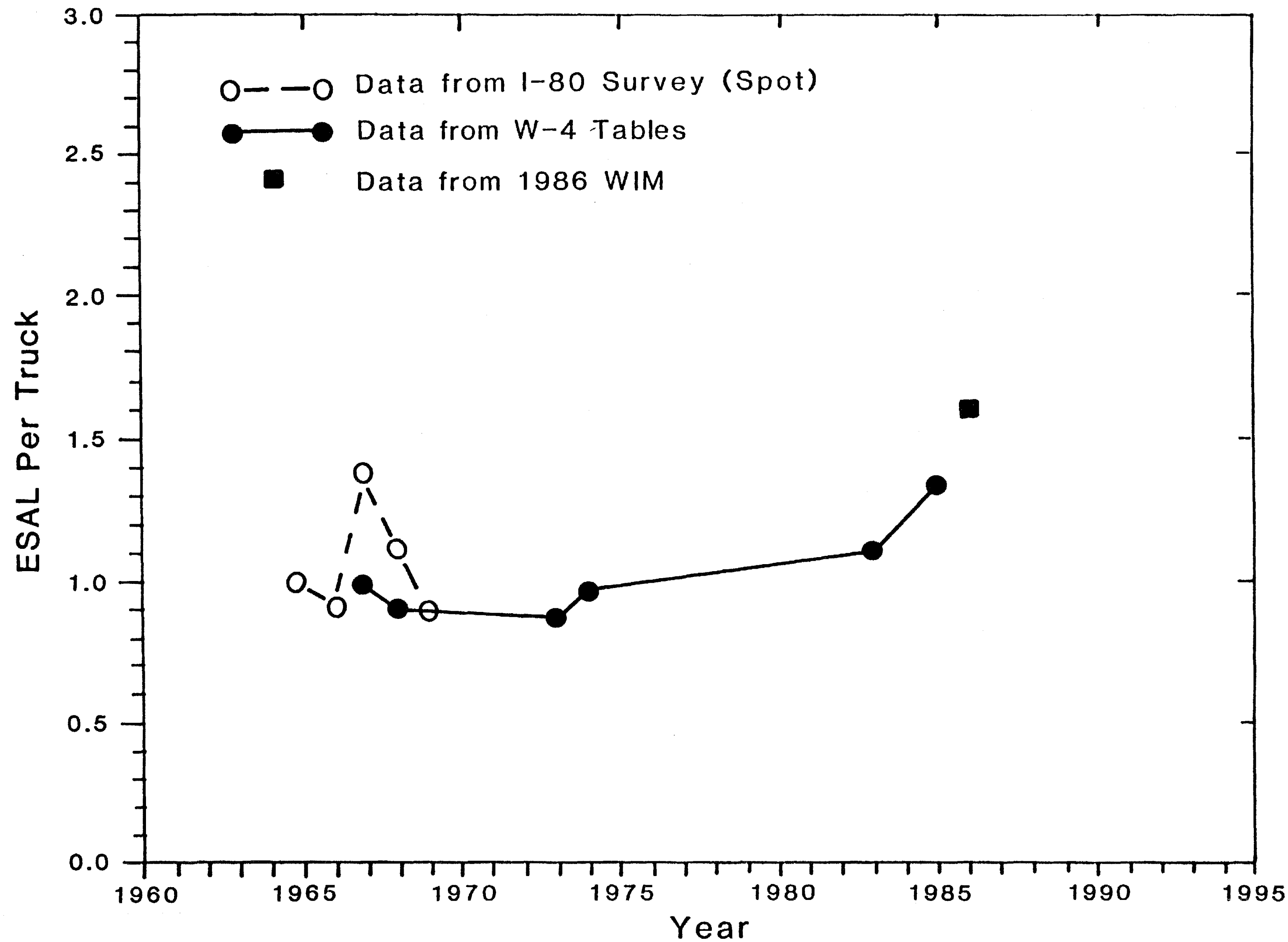

Figure 2-6.6. Truck Factor for all Trucks With Weight in Motion (WIM) Calculated Value Indicated.

Axle Load	Traffic Equivalency Factor		Number of Axles		A18 Kip EAL's
Single Axles	P = 2.5, SN = 5				
Under 3,000	0.0002	X	0	=	0.000
3,000 - 6,999	0.0050	X	1	=	0.005
7,000 - 7,999	0.0320	X	6	=	0.192
8,000 - 11,999	0.0870	X	144	=	12.528
12,000 - 15,999	0.3600	X	16	=	5.760
26,000 - 29,999	5.3890	X	1	=	5.3890
Tandem Axle Groups					
Under 6,000	0.0100	X	0	=	0.000
6,000 - 11,993	0.0100	X	14	=	0.140
12,000 - 17,999	0.0440	X	21	=	0.924
18,000 - 23,999	0.1480	X	44	=	6.512
24,000 - 29,999	0.4260	X	42	=	17.892
30,000 - 32,000	0.7530	X	44	=	33.132
32,001 - 32,500	0.8850	X	21	=	18.585
32,501 - 33,999	1.0020	X	101	=	101.202
34,000 - 35,999	1.2300	X	43	=	52.890
18 Kip EAL's for all trucks wieghed				=	255.151

$$\text{Truck Load Factor} = \frac{\text{18 Kip EAL's for all trucks weighed}}{\text{Number of trucks weighed 1654}} = \frac{255.151}{165} = 1.5464$$

Figure 2-6.7. Computation of the Truck Load Factor for 5-Axle or Greater Trucks on Flexible Pavement (2).

Figure 2-6.8. Example Table for Calculating Design ESAL by Vehicle Class (2).

Analysis Period = ____________ Years

Location ____________________ Assumed SN or D = ________

Vehicle Types	Current Traffic (A)	Growth Factors (B)	Design Traffic (C)	E.S.A.L. Factor (D)	Design E.S.A.L. (E)
Passenger Cars Buses					
Panel and Pickup Trucks Other 2-Axle/4-Tire Trucks 2-Axle/6-Tire Trucks 3 or More Axle Trucks All Single Unit Trucks					
3 Axle Tractor Semi-Trailers 4 Axle Tractor Semi-Trailers 5+ Axle Tractor Semi-Trailers All Tractor Semi-Trailers					
5 Axle Double Trailers 6+ Axle Double Trailers All Double Trailer Combos.					
3 Axle Truck-Trailers 4 Axle Truck-Trailers 5+ Axle Truck-Trailers All Truck-Trailer Combos.					
All Vehicles				Design E.S.A.L.	

Figure 2-6.9. Growth Factors for Traffic Estimates (2).

Analysis Period Years (n)	Annual Growth Rate, Percent (g)							
	No Growth	2	4	5	6	7	8	10
1	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
2	2.0	2.02	2.04	2.05	2.06	2.07	2.08	2.10
3	3.0	3.06	3.12	3.15	3.18	3.21	3.25	3.31
4	4.0	4.12	4.25	4.31	4.37	4.44	4.51	4.64
5	5.0	5.20	5.42	5.53	5.64	5.75	5.87	6.11
6	6.0	6.31	6.63	6.80	6.98	7.15	7.34	7.72
7	7.0	7.43	7.90	8.14	8.39	8.65	8.92	9.49
8	8.0	8.58	9.21	9.55	9.90	10.26	10.64	11.44
9	9.0	9.75	10.58	11.03	11.49	11.98	12.49	13.58
10	10.0	10.95	12.01	12.58	13.18	13.82	14.49	15.94
11	11.0	12.17	13.49	14.21	14.97	15.78	16.65	18.53
12	12.0	13.41	15.03	15.92	16.87	17.89	18.98	21.38
13	13.0	14.68	16.63	17.71	18.88	20.14	21.50	24.52
14	14.0	15.97	18.29	19.16	21.01	22.55	24.21	27.97
15	15.0	17.29	20.02	21.58	23.28	25.13	27.15	31.77
16	16.0	18.64	21.82	23.66	25.67	27.89	30.32	35.95
17	17.0	20.01	23.70	25.84	28.21	30.84	33.75	40.55
18	18.0	21.41	25.65	28.13	30.91	34.00	37.45	45.60
19	19.0	22.84	27.67	30.54	33.76	37.38	41.45	51.16
20	20.0	24.30	29.78	33.06	36.79	41.00	45.76	57.28
25	25.0	32.03	41.65	47.73	54.86	63.25	73.11	98.35
30	30.0	40.57	56.08	66.44	79.06	94.46	113.28	164.49
35	35.0	49.99	73.65	90.32	111.43	138.24	172.32	271.02

*Factor = $\frac{(1+g)^n - 1}{g}$, where $g = \frac{\text{rate}}{100}$ and is not zero. If annual growth rate is zero, the growth factor is equal to the analysis period.

Note: The above growth factors multiplied by the first year traffic estimate will give the total volume of traffic expected during the analysis period.

5. Column E is the product of Column D times Column C. The vertical summation of Column E is the design 18-kip ESAL applications to be used in the pavement structural design process.

The number developed from this table must be corrected for the lane distribution factor developed from project specific studies so that the pavement can be designed for the actual traffic which will use the pavement in any of the lanes.

The calculations discussed here can be performed using a microcomputer spreadsheet program called "ESALCALC". This is a Lotus 123 compatible spreadsheet that performs exactly the same calculations contained in Figure 2-6.7. The input screen for ESALCALC is shown in Figure 2-6.10. This spreadsheet is user friendly and allows data to be used in several formats to fit the form of data available to the engineer. This program will be demonstrated on the example problem at the end of this module and will be used to generate design data for the workshop problems.

5.0 FACTORS AFFECTING ESALs

As previously mentioned there are a number of variables that can alter the calculation of the load equivalency factor and the truck factor. With current changes in these variables occurring more rapidly than at any time in the past an understanding of where variability can come from is required.

5.1 Pavement Selection Criteria

5.1.1 Effect of Terminal Serviceability Value (P_t)

The selection of the terminal serviceability for the pavement has a significant impact on the equivalency factors used to calculate ESAL from the traffic stream. This results from the non-linear relationship between loadings and decrease in PSR per loading. Additionally, distress produces a decrease in serviceability which is related to the number of loadings in a logarithmic fashion. The selection of the P_t value to be used in designing the pavement must be done carefully to ensure that the appropriate number of ESALs are calculated.

5.1.2 Effect of Pavement Type

The equivalency tables clearly indicate a difference in equivalency factors for rigid and flexible pavements. Because these pavements respond differently to similar loads, the deterioration of each pavement type will be different. For single axle loads less than 18 kips the equivalency factor is less for a rigid pavement than for a flexible pavement. For heavier loads, the equivalency factor for rigid pavements is greater. This difference, developed from the AASHO Road Test, indicates the different load carrying ability of a flexible vs. a rigid pavement, and the difference in which the stresses produced by a wheel load deteriorate each pavement. It is critical to use the appropriate equivalency factor for the type of pavement being designed.

HIGHWAY PAVEMENT DESIGN ESAL's NCHRP 255 PAGE 152 11/6/85 MOD 4/16/86
Written by JHK & Assoc (K. Hooper) & COMSIS Corporation (M. Roskin)
DESIGN PERIOD (1-40yrs)= 20 LANES = 4
DIRECTIONAL DISTRIB. (0.50 TO 1.00) = .5 ** INPUTS **

VEHICLE TYPES	BY%	BY ADT	FY GROWTH	FY%	FY ADT	ESAL FACTOR
PASSENGER CARS	50.0			63.0	14,805	.0008
BUSES	0.0				0	
PANEL/PICKUP TRUCKS	0.0				0	
OTHR 2-AXLE/4 TIRE TRKS	12.0			15.0	3,525	.02
2-AXLE/6-TIRE TRUCKS				11.0	2,585	.15
3 OR MORE AXLE TRUCKS					0	
3 AXLE TRACTOR SEMI					0	
4 AXLE TRACTOR SEMI					0	
5+ AXLE TRACTOR SEMI	6.0			11.0	2,585	1.25
5 AXLE DBL TRAILERS					0	
6+ AXLE DBL TRAILERS					0	
3 AXLE TRUCK-TLRS					0	
4 AXLE TRUCK-TLRS					0	
5+ AXLE TRUCK-TLRS					0	
** ALL VEHICLES **	68.0	10,500	- -	100.0	23,500	- -

Figure 2-6.10. Input Screen for ESALCALC Program to Calculate ESAL Factors for the Traffic Stream (11).

5.1.3 Effect of Relative Pavement Strength

The equivalency tables for the AASHTO Design Guide clearly show an influence of the pavement strength on the equivalency factor. The response to a load, a deflection of stress, is nonlinear, and the more non-linear material used in a pavement, the more variation in the response to the load. It is this response that produces distress which decreases the serviceability of the pavement. The actual difference in equivalency factors for different pavements is not great. It is not required that the design process become an iteration process where the final SN of the pavement match the assumed SN used to calculate the ESALs from the traffic stream. It is important to note that different pavement structures will respond differently to the same load. It is this response that produces distress which reduces the serviceability of the pavement. Different responses produce different rates of deterioration.

5.2 TRAFFIC VARIABLES

5.2.1 Effect of Mixed Traffic

The traffic stream is composed of various vehicle types and weights. Each pass of any vehicle causes the pavement to deflect, thereby inducing damaging effects. The damaging effects done to the pavement structure by automobiles and light trucks is so small that is is often ignored. The equivalency factor for an automobile indicates the error induced by this assumption will be very small. Conversely, the exclusion of even a small number of heavily loaded trucks from the traffic stream being analyzed can produce a significant error in the number of ESALs calculated for design purposes. Because of this sensitivity to heavily loaded trucks, the accuracy of the W-4 tables, and their representation of the traffic stream to date is critical to an accurate pavement design into the future.

For reasons previously discussed it is not sufficient to characterize the traffic stream with general categories of truck types. Detailed descriptions and data recording are necessary to accurately reflect the composition of the traffic stream to develop accurate ESAL values. Additionally, accurate weights on all truck types in the traffic stream must be collected along with the accurate distribution of truck types. Because slight load differences can have a pronounced effect on the ESAL calculation, the accuracy of the loadings and axle weight distribution is essential.

Weigh in Motion (WIM) devices have shown a significant difference in the data collected from Loadometer stations as reported on the W-4 tables. It is common for trucks to avoid an active weigh station for one reason or another which can bias the resulting ESAL values calculated for the pavement. Because the recorded data are being used to predict the traffic in the future it is important to have the most accurate data available. The weigh in motion equipment provides the most accurate determination of the complete traffic stream as it can be placed anywhere on the system and left for a specific period to record the total traffic stream, and all components of truck types and axle loads.

The significance of the WIM equipment's improvement in the data collection is illustrated in Figure 2-6.1 and Figure 2-6.6. In Figure 2-6.1 the ADT collected from W-4 tables for a rural interstate pavement in Central

Illinois is shown. The traffic stream has been broken down to multiple Units (MU) and Single Units (SU). When this information is reduced to ESAL per unit type, the individual curves are produced. Figure 2-6.6 is useful in predicting the growth in ESAL. The data through 1985 indicate a growth of approximately 4 percent per year in the ESAL per each truck. This growth factor is not uncommon, and indicates the increasing weight being carried by the trucking industry as trucks are being utilized to their capacity, and even beyond.

The interesting factor in Figure 2-6.6 is the inclusion of the 1986 WIM data. The WIM data clearly shows that there is a 20 percent difference between the on site survey and the WIM collected data. Because the WIM is a more complete collection device, and it is unannounced, it should include a more representative cross section of the traffic on the pavement. The data here represent two weeks of data collected at two different times, but the difference is significant. This 20 percent difference has been reported from other states with WIM equipment. This difference can be attributed to heavier trucks avoiding the permanent weigh stations as well as the increased accuracy provided by the WIM equipment for sampling and recording the entire traffic stream. There are, however, vehicle operating characteristics which alter the ESAL per truck figure. If this difference actually exists on the pavement, a prediction of ESAL 20 years into the future would be seriously in error if the starting point were off by 20 percent. This could result in an under-prediction of ESAL by many million, and result in premature failure of the pavement.

5.2.2 Gross Weight and Percent of Trucks

As goods movement shifted to the trucking industry, the number of trucks has increased in a disproportionate amount. Additionally, larger numbers of trucks are running with more cargo. These two facts have produced a dramatic shift in recent years in the ESAL per truck curves for many agencies, and reliance on historical data may be erroneous if truck categories are not separated by distinct weight classifications, but rather are grouped in rather large classifications. The detailed breakdown by truck type and weight classification is required for accurate future predictions of ESAL.

5.2.3 Axle Configurations

As was shown in the structural evaluation section, and in the ESAL section of this module, 36,000 pounds on a tandem axle is not the same as 18,000 on two single axles, even though the total load per wheel is the same. As gross loads continue to increase, different axle configurations are being used to maintain the per axle load in the same range as before. This practice does not guarantee a similar rate of deterioration in the pavement, however, as the number of axles is increased. The stress under the axle combination does not significantly decrease. This comparison requires detailed computer analyses to show the changes in stresses and strains produced by different tire and axle configurations, and cannot be directly extracted form the AASHTO equivalency tables. The appearance of the tridem axle is one indication of newer axle configurations included in AASHTO.

5.2.4 Tire Pressures

There has been an increased incidence of high tire pressures being reported on interstate pavements. Traditional computer analyses have assumed 70 to 80 psi values for a pavement analysis. Measurements from interstate highways in Illinois and Arizona have shown average tire pressures in the range of 90 psi with extreme values ranging to 130 psi (7). This increased tire pressure produces high stress levels in the surface course, and leads to rapid failure in asphalt concrete pavements in particular. These higher stresses produce different failure modes from the more commonplace failure modes existing when the AASHTO equivalency values were prepared, and require further research to establish equivalency values for higher tire pressures. In the traditional sense of load equivalency factors, it is not likely that a difference will become apparent from analytical studies of the entire pavement structure as the damage from higher tire pressures is limited to the surface layers.

6.0 OTHER PROCEDURES

There are other design procedures which have slightly different methods to determine the number of 18-kip ESAL applications for thickness design.

6.1 Asphalt Institute (9)

This procedure determines the equivalent 18 kip ESAL using Truck Factor constants for general truck types, and is therefore an approximate method of obtaining 18-kip ESALs. The difference between ESAL and Truck Factor is that ESAL represents the damage contributed by one passage of an axle, whereas TF is the number of equivalent single axle loads contributed by one passage of that vehicle.

The procedures for calculating the Truck Factor are as shown in Figure 2-6.7, in which the total ESAL is determined per 1000 vehicles using the AASHTO load equivalency factors shown previously in Figure 2-6.3 and Figure 2-6.4. The Truck Factor is given by:

$$TF = (\text{Number of axles x ESAL})/(\text{Number of Vehicles})$$

The distribution of Truck Factors for different classes of highways is presented in Table 2-6.5. It should be noted that TF values larger than 1.0 and as high as 5 have been reported for entrance roads to heavy commercial operations. The Asphalt Institute's method of calculation of Design ESAL is:

1. Determine the average number of each type of vehicle expected on the design lane during the first year of traffic.

2. Determine from axle-weight data, or select from Table 2-6.5, a Truck Factor for each vehicle type found in Step 1.

3. Select, from Figure 2-6.9, a single Growth Factor for all vehicles or separate Factors for each vehicle type, as appropriate.

4. Multiply the number of vehicles of each type times the Truck Factor and the Growth Factor (or factors) determined in Steps 2 and 3. Sum the values determined to obtain Design ESAL

Table 2-6.5. Distribution of Truck Factors (TF) For Different Classes of Highways and Vehicles-United States*

(After the Asphalt Institute, MS-17)

Vehicle Type	Truck Factors									
	Rural Systems						Urban Systems		All Systems	
	Interstate Rural		Other Rural		All Rural		All Urban			
	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range
Single-unit trucks										
2-axle, 4-tire	0.02	0.01-0.06	0.02	0.01-0.09	0.03***	0.02-0.08	0.03***	0.01-0.05	0.02	0.01-0.07
2-axle, 6-tire	0.19	0.13-0.30	0.21	0.14-0.34	0.20	0.14-0.31	0.26	0.18-0.42	0.21	0.15-0.32
3-axle or more	0.56	0.09-1.55	0.73	0.31-1.57	0.67	0.23-1.53	1.03	0.52-1.99	0.73	0.29-1.59
All single-units	0.07	0.02-0.16	0.07	0.02-0.17	0.07	0.03-0.16	0.09	0.04-0.21	0.07	0.02-0.17
Tractor semi-trailers										
3-axle	0.51	0.30-0.86	0.47	0.29-0.82	0.48	0.31-0.80	0.47	0.24-1.02	0.48	0.33-0.78
4-axle	0.62	0.40-1.07	0.83	0.44-1.55	0.70	0.37-1.34	0.89	0.60-1.64	0.73	0.43-1.32
5-axle or more**	0.94	0.67-1.15	0.98	0.58-1.70	0.95	0.58-1.64	1.02	0.69-1.69	0.95	0.63-1.53
All multiple units	0.93	0.67-1.38	0.97	0.67-1.50	0.94	0.66-1.43	1.00	0.72-1.58	0.95	0.71-1.39
All trucks	0.49	0.34-0.77	0.31	0.20-0.52	0.42	0.29-0.67	0.30	0.15-0.59	0.40	0.27-0.63

**Compiled from data supplied by the Highway Statistics Division, U.S. Federal Highway Administration.*

***Including full-trailer combinations in some states.*

****See Article 4.05 for values to be used when the number of heavy trucks is low.*

This procedure has a significant difference from the AASHTO procedure in that it allows for general truck factors to be used which may not relate to the actual loading conditions on the project. These values must be used very carefully as they will not be accurate. While they may be sufficient for long term planning, the final designs must be done from actual weight and classification data.

6.2 Portland Cement Association Method (10)

The PCA method does not use the load equivalency concept because PCA feels that the load equivalency is too dependent on pavement characteristics (slab thickness; concrete flexural strength, and subgrade modulus of reaction). Instead, the PCA method uses the results of the loadometer studies to determine the number of single and tandem axles of the various load groups, and determines the damage resulting from each load group and axle configuration separately.

When the precise data on axle-load distribution is not available, the axle load categories, percent trucks and maximum axle loads suggested in Figure 2-6.2 are recommended for use with the PCA design method.

The three main traffic estimates in the PCA method are design ADT, the average daily truck traffic (ADTT), and axle load per truck. The ADT is given by the projection method or by the capacity estimate as was illustrated before. The ADTT is counted only on trucks with 6 or more tires and is expressed as a percentage of the ADT or as an actual value.

For design purposes, the total number of trucks in the design period (T) is needed. This is obtained by multiplying design ADT by ADTT (percentage/100) times the number of days in the design period (365 x design period in years).

Data on the axle load distribution of the truck traffic is determined in one of three ways:

The difficulty with having all trucks, including the unwanted values for panels, pickups, and other four axle type vehicles, is overcome somewhat by adopting an adjusting factor.

The PCA method also uses a load safety factor (LSF) as follows:

1. For Interstate and other multilane projects where there will be uninterrupted traffic flow and high volumes of truck traffic, LSF = 1.2.

2. For highways and arterial streets where there will be moderate volumes of truck traffic, LSF = 1.1.

3. For roads, residential streets and other streets that will carry small volumes of truck traffic, LSF = 1.0

Exclusive of the load safety factors, a degree of conservatism is provided in the design procedure to compensate for such things as unpredicted truck overloads and normal construction variations in material properties and layer thicknesses. Above that basic level of conservatism (LSF = 1.0) the use of load safety factors of 1.1 or 1.2 is claimed to provide a greater allowance for the possibility of unpredicted heavy loads and volumes and a higher level of pavement serviceability appropriate for higher type pavement facilities.

Other features of this method are in the development of the analysis for Tridem axle (triple axle each spaced at 48 to 54 inches apart) where tables have been constructed for load equivalency based on fatigue and erosion criteria as will be shown later in Block 6.

7.0 SAMPLE PROBLEM

TRUCK FACTORS

Table 2-6.6 is the WIM data from an Interstate pavement in Central Illinois. It was collected during a one week period in August 1986. The calculation for ESAL per axle is shown in Table 2-6.7 and is as follows:

Single Axle:

$$(18.501/100)(48567/101349) = 0.0887$$

Tandem Axle:

$$(82.279/100)(52782/101349) = 0.429$$

Total: 0.518 ESAL per axle.

This value is for a concrete pavement with a slab thickness of nine inches.

If 2.75 axles are assumed per truck, the ESAL per truck becomes 1.424 ESAL per truck. This value represents only one season of the year, and must be averaged with other seasonal readings before calculating total ESAL on the pavement over a year from a truck count. This calculation should be further broken out for each truck classification before being used.

Examining <u>this data</u>, how many ESALs are applied to the pavement in the first year as shown by the data in Table 2-6.7?

Ans: Approximately 1,405,320 ESALs in the year.

If the total number of trucks is increasing by 4 percent per year, what will be the total number of ESALs over a 20 year design period?

Ans: Approximately 41,850,000 ESALs.

Further analysis shows that the 4 percent growth is composed of 3 percent increase in the single axle trucks, and 5 percent increase in the tandem axle trucks. Using these data, what is the design ESALs for the 20 year life?

Ans: Approximately 45,108,000 ESALs. This is a 7.75 percent difference. This difference becomes even greater when ESAL factors are calculated by axle weight grouping with percentage increases for each group, and not just by single vs. tandem groupings.

Table 2-6.6. WIM Data for August 1986.

DEPARTMENT OF TRANSPORTATION
CATEGORY 4-13, EXCLUDING OFFSCALE
TIME PERIOD 1986 AUG19 6_PM THRU AUG26 6_PM

	AXLE							
	SINGLE				TANDEM			
	HH				HH			
	6PM THRU 6AM		6AM THRU 6PM		6PM THRU 6AM		6AM THRU 6PM	
	N	PCTN	N	PCTN	N	PCTN	N	PCTN
WEIGHT								
UNDER 2000	89	0.42	191	0.70	.	.	3	0.01
2000 - 3999	654	3.05	1725	6.35	45	0.18	138	0.49
4000 - 5999	692	3.23	1394	5.13	107	0.43	269	0.96
6000 - 7999	1019	4.76	1596	5.88	232	0.94	505	1.79
8000 - 9999	8450	39.45	9601	35.37	695	2.82	1151	4.09
10000 - 11999	6292	29.38	7392	27.23	898	3.65	1405	4.99
12000 - 13999	1488	6.95	1855	6.83	1202	4.88	1551	5.51
14000 - 15999	1165	5.44	1326	4.88	1236	5.02	1556	5.52
16000 - 17999	876	4.09	1103	4.06	1410	5.73	1495	5.31
18000 - 19999	525	2.45	674	2.48	1540	6.26	1641	5.83
20000 - 21999	150	0.70	254	0.94	1484	6.03	1616	5.74
22000 - 23999	18	0.08	35	0.13	1435	5.83	1570	5.57
24000 - 25999	1	0.00	2	0.01	1512	6.14	1637	5.81
26000 - 27999	.	.	.	.	1981	8.05	2007	7.13
28000 - 29999	.	.	.	.	2718	11.04	2822	10.02
30000 - 31999	.	.	.	.	3150	12.79	3200	11.36
32000 - 33999	.	.	.	.	2533	10.29	2811	9.98
34000 - 35999	.	.	.	.	1717	6.97	1873	6.65
36000 - 37999	.	.	.	.	556	2.26	660	2.34
38000 - 39999	.	.	.	.	126	0.51	182	0.65
40000 - 41999	.	.	.	.	29	0.12	40	0.14
42000 - 43999	.	.	.	.	5	0.02	15	0.05
44000 - 45999	.	.	.	.	3	0.01	9	0.03
46000 - 47999	.	.	.	.	3	0.01	4	0.01
48000 - 49999	.	.	.	.	1	0.00	1	0.00
52000 - 53999	.	.	.	.	.	.	1	0.00
54000 - 55999	.	.	.	.	1	0.00	1	0.00
ALL	21419	100.00	27148	100.00	24619	100.00	28163	100.00

Table 2-6.7. Calculation of the ESAL per Axle.

b1986 Aug 19 thru Aug 26 (seven days)

Single Axles

Weight	6 PM thru 6 AM N	6 PM thru 6 AM Percent	6 AM thru 6 PM N	6 AM thru 6 PM Percent	Day and Night N	Day and Night Percent	Equivalency Factor	Percent x Equiv. Factor
under 2000	89	0.42	191	0.7	280	0.56	0.0002	0.000112
b2000 - 3999	654	3.05	1725	6.35	2379	4.7	0.00115	0.005405
b4000 - 5999	692	3.23	1394	5.13	2086	4.18	0.0061	0.025498
b6000 - 7999	1019	4.76	1596	5.88	2615	5.32	0.02125	0.11305
b8000 - 9999	8450	39.45	9601	35.37	18051	37.41	0.05705	2.1342405
b10000 - 11999	6292	29.38	7392	27.23	13684	28.305	0.12905	3.65276025
b12000 - 13999	1488	6.95	1855	6.83	3343	6.89	0.2585	1.781065
b14000 - 15999	1165	5.44	1326	4.88	2491	5.16	0.47215	2.436294
b16000 - 17999	876	4.09	1103	4.06	1979	4.075	0.80185	3.26753875
b18000 - 19999	525	2.45	674	2.48	1199	2.465	1.2833	3.1633345
b20000 - 21999	150	0.7	254	0.94	404	0.82	1.9541	1.602362
b22000 - 23999	18	0.08	35	0.13	53	0.105	2.85185	0.29944425
b24000 - 25999	1	0	2	0.01	3	0.005	4.01355	0.02006775
Total	21419	100	27148	99.99	48567	99.995		18.501172

Tandem Axles

Weight	6 PM thru 6 AM N	6 PM thru 6 AM Percent	6 AM thru 6 PM N	6 AM thru 6 PM Percent	Day and Night N	Day and Night Percent	Equivalency Factor	Percent x Equiv. Factor
under 2000	0	0	3	0.01	3	0.005	0.0001	0.0000005
b2000 - 3999	45	0.18	138	0.49	183	0.335	0.0003	0.0001005
b4000 - 5999	107	0.43	269	0.96	376	0.695	0.0012	0.000834
b6000 - 7999	232	0.94	505	1.79	737	1.365	0.00365	0.00498225
b8000 - 9999	695	2.82	1151	4.09	1846	3.455	0.009	0.031095
b10000 - 11999	898	3.65	1405	4.99	2303	4.32	0.0191	0.082512
b12000 - 13999	1202	4.88	1551	5.51	2753	5.195	0.03655	0.18987725
b14000 - 15999	1236	5.02	1556	5.52	2792	5.27	0.06465	0.3407055
b16000 - 17999	1410	5.73	1495	5.31	2905	5.52	0.1074	0.592848
b18000 - 19999	1540	6.26	1641	5.83	3181	6.045	0.1696	1.025232
b20000 - 21999	1484	6.03	1616	5.74	3100	5.885	0.25685	1.51156225
b22000 - 23999	1435	5.83	1570	5.57	3005	5.7	0.37555	2.140635
b24000 - 25999	1512	6.14	1637	5.81	3149	5.975	0.5328	3.18348
b26000 - 27999	1981	8.05	2007	7.13	3988	7.59	0.7362	5.587758
b28000 - 29999	2718	11.04	2822	10.02	5540	10.53	0.99385	10.4652405
b30000 - 31999	3150	12.79	3200	11.36	6350	12.075	1.3141	15.8677575
b32000 - 33999	2533	10.29	2811	9.98	5344	10.135	1.70525	17.28270875
b34000 - 35999	1717	6.97	1873	6.65	3590	6.81	2.1756	14.815836
b36000 - 37999	556	2.26	660	2.34	1216	2.3	2.7333	6.28659
b38000 - 39999	126	0.51	182	0.65	308	0.58	3.38645	1.964141
b40000 - 41999	29	0.12	40	0.14	69	0.13	4.14325	0.5386225
b42000 - 43999	5	0.02	15	0.05	20	0.035	5.0125	0.1754375
b44000 - 45999	3	0.01	9	0.03	12	0.02	6.0039	0.120078
b46000 - 47999	3	0.01	4	0.01	7	0.01	7.1285	0.071285
b48000 - 49999	1	0	1	0	2	0	8.3991	0
b50000 - 51999	0	0	0	0	0	0	9.8304	0
b52000 - 53999	0	0	1	0	1	0	11.43925	0
b54000 - 55999	1	0	1	0	2	0	13.2444	0
Total	24619	99.98	28163	99.98	52782	99.98		82.279319

ESALCALC

Given the vehicle classification counts shown in Figure 2-6.11, determine the total design ESAL using the ESALCALC program or by manually completing Figure 2-6.8.

The answer should be around 43.7 million ESALs, a total ESAL which must be adjusted for directional distribution, and for lane distribution before being used in a pavement thickness design procedure.

For comparison purposes, assume a 2 percent estimate for passenger cars and buses, and single unit trucks, a 4 percent growth in tractor semi-trailer and truck full trailer combinations, and a 5 percent trowth in double trailer combinations. This should calculate nearly 53.7 million ESALs, a 23 percent difference.

As a third example, what if the growth percentages are low by only two percentage units, and were actually 4, 6, and 7 percent respectively? Now the design ESAL is 66.4 million ESAL, a 50 percent difference with what is really a minor change in growth estimates.

NOTE: ESALCALC uses certain default lane distribution factors as, and lane numbers should be chosen to produce the desired percentage felt to fit your specific situation.

8.0 SUMMARY

This module has presented basic information on traffic loading evaluation which must be accomplished as part of the overall engineering evaluation of a pavement to determine the design requirements.

Load equivalency factors are defined in accordance with the AASHTO Design Guide. The use of historical traffic vehicle classification counts and axle load distribution data (W-4 Tables and WIM) to calculate the past and projected future cumulative 18-kip equivalent single axle loads (ESAL) for a pavement section was described. The need for accurate volume, classification and weight data specific to the project site were emphasized The great benefit of weigh-in-motion (WIM) information to the accuracy of the projection was emphasized

The past and projected future traffic loadings are two of the most important input parameters for designing a pavement. Results from the traffic evaluation provide information on estimation of past and current loadings, the structural adequacy of the existing pavement and overlay design requirements in the future for planned rehabilitation.

Analysis Period = 20 Years

Location Example 1

Assumed SN or D = 9"

Vehicle Types	Current Traffic (A)	Growth Factors (B)	Design Traffic (C)	E.S.A.L. Factor (D)	Design E.S.A.L. (E)
		2%			
Passenger Cars	5.925	24.30	52,551,787	.0008	
Buses	35	24.30	310,433	.6806	
Panel and Pickup Trucks	1,135	24.30	10,066,882	.0122	
Other 2-Axle/4-Tire Trucks	3	24.30	26,609	.0052	
2-Axle/6-Tire Trucks	372	24.30	3,299,454	.1890	
3 or More Axle Trucks	34	24.30	301,563	.1303	
All Single Unit Trucks					
3 Axle Tractor Semi-Trailers	19	24.30	168,521	.8646	
4 Axle Tractor Semi-Trailers	49	24.30	434,606	.6560	
5 + Axle Tractor Semi-Trailers	1,880	24.30	16,674,660	2.3719	
All Tractor Semi-Trailers					
5 Axle Double Trailers	103	24.30	913,559	2.3187	
6 + Axle Double Trailers	0	24.30			
All Double Trailer Combos.					
3 Axle Truck-Trailers	208	24.30	1,844,856	.0152	
4 Axle Truck-Trailers	305	24.30	2,705,198	.0152	
5 + Axle Truck-Trailers	125	24.30	1,108,688	.5317	
All Truck-Trailer Combos.					
All Vehicles	10,193		90,406,816	Design E.S.A.L.	

Figure 2-6.11. Worksheet from AASHTO Design Guide for Calculating 18-kip ESAL.

9.0 REFERENCES

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MODULE 2-7

VARIABILITY AND RELIABILITY IN PAVEMENT DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module provides information on the use of reliability/variability concepts as applied to pavement design.

Upon completion of the module, the participants will be able to accomplish the following:

1. Understand the basic principles of variability.

2. Define the following terms: mean, standard deviation, coefficient of variation, and distribution of variation.

3. Understand how variability in materials, traffic, climate and construction affects the adequacy of any design procedure.

4. Understand the use of reliabilty concepts as applied to structural design.

2.0 INTRODUCTION

One word that is practically synomous with pavements is "variability." There is variability in almost everything associated with pavement design, construction, performance, maintenance and rehabilitation. It is important for pavement engineers to have a basic understanding of the variations associated with pavements. This will help them to better understand pavement performance, the need for reduced variation in construction, and the impact of variation on design adequacy.

This section first presents a basic review of statistical variability concepts, then provides some interesting examples of pavement related variation, then describes how variability affects the adequacy of pavement design, and finally provides background information on design reliability concepts (as background for the sections on design reliability contained in the 1986 AASHTO Design Guide).

3.0 Basic Concepts of Variability

This section presents some very basic concepts on variability that must be mastered to gain an understanding of either pavement performance or design reliability. It is written for the practicing engineer who may not have had training in statistics.

3.1 Mean, Range, Standard Deviation, and Coefficient of Variation

The following example is used to illustrate the calculation of these basic statistical parameters. A section of concrete pavement that was all placed in one day (24 ft. by 1000 ft.) was cored and the specimens were tested for compression strength. The core locations were selected randomly through use of a coordinate system and random number table.

Test	Compressive Strength (psi)
1	3025
2	4489
3	3636
4	2601
5	3906

The mean compressive strength is calculated as follows:

$$MEAN = SUM\ (X_i)/n$$

$$= [\ 3025 + 4489 + 3636 + 2601 + 3906\ ]/5\ =\ 3531\ psi$$

The mean represents the average of all of the data points.

The range of the data is computed as follows:

$$RANGE = MAXIMUM - MINIMUM$$

$$=4489 - 2601 = 1888\ psi$$

The range provides information on the overall variation of the data.

The standard deviation is computed as follows:

$$STAND.\ DEV. = \{\ [\ (SUM\ X_i^2/n) - (SUM\ X_i)^2/n\ ]/(n - 1)\ \}^{0.5}$$

$$= \{\ [\ (3025^2 + 4489^2 + 3636^2 + 2601^2 + 3906^2\ )/5$$

$$- (3025 + 4489 + 3636 + 2601 + 3906)^2/5\ ]/(5-1)\ \}^{0.5}$$

$$=\ 740\ psi$$

The standard deviation is an index of the spread of the data from the mean. The more variable the data, the larger the standard deviation.

The coefficient of variation is computed as follows:

$$COEFFICIENT\ OF\ VARIATION = STANDARD\ DEVIATION/MEAN$$

$$= 740/3531 = 0.21,\ OR\ 21\ percent$$

The coefficient of variation is an excellent index of the amount of variability existing in the data, relative to the mean. For example, for concrete strength, a coefficient of variation for good quality control is generally 15 percent or less. Poor quality control is 20 percent or more. Each material property, deflection, thickness, etc. has a different normal coefficient of variation. Slab thickness has a very small coefficient of variation (e.g., 3 percent).

Sample versus Population: It is very important to realize that the mean or standard deviation of the five specimens represents only a sample <u>estimate</u> of the true mean of the entire section (which is called the population). The true population mean and standard deviation could only be determined by coring every piece of available concrete and testing many thousands of cores. The greater the sample size, however, the better the estimate of the true mean and standard deviation.

3.2 Distribution of Variation

The concrete core strength data shown in the previous section ranged from 2601 to 4489 psi. The five individual specimen strengths can be plotted in a histogram as illustrated in Figure 2-7.1a. Not much sense can be made out of this distribution. However, if say 100 cores were cut and tested, the distribution might look like Figure 2-7.1b., and all possible cores were cut and tested, the distribution might look like Figure 2-7.1c.

Many pavement material properties such as concrete strength approximately follow what is called a "normal" distribution, shown as dashed lines in Figure 2-7.1c. This distribution is widely used in quality control work and in studying the effects of variability on performance and design of pavements and other structures. Other distributions are considered to be more accurate in specific situations include the lognormal, gamma, uniform and beta (1).

Examples of the distributions of various pavement properties are given in the next section. Most of these distributions are approximately normal or lognormal.

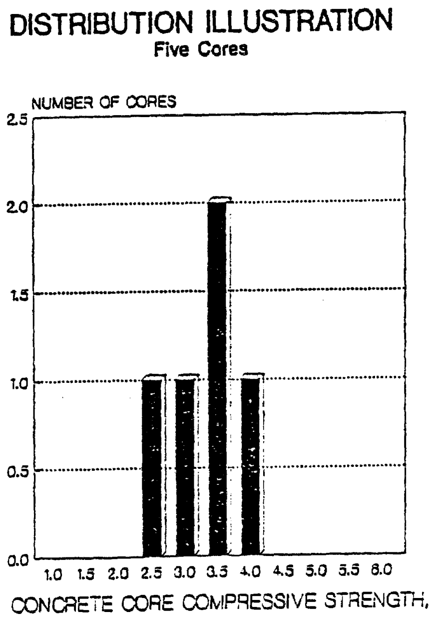
The bell-shaped normal distribution is defined by two parameters: the mean and the standard deviation. Figure 2-7.2 shows several normal distributions that have different means and standard deviations. The greater the standard deviation, the wider the distribution, the greater the range of the data and the greater the coefficient of variation.

The normal distribution (and the lognormal distribution) can be utilized for many pavement engineering applications. For example, consider the previous section of pavement. It is desirable to estimate the proportion of concrete pavement having a core strength less than 2500 psi as illustrated in Figure 2-7.3. The area under the curve is proportional to the probability that the strength lies from 0 to 2500 psi. This can be calculated using "standardized" normal distribution tables as follows.

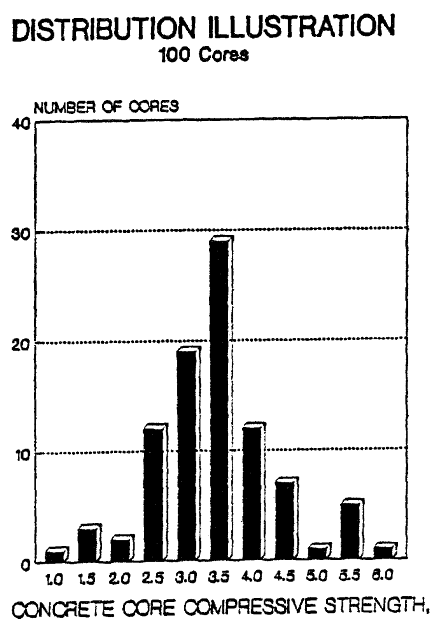
To use the standardized normal distribution tables, the actual strength data (mean and standard deviation) must be transformed to a scale where the mean is 0 and the standard deviation is 1 (rather than the actual mean of 3531 psi and the standard deviation of 740 psi).

The area under the standardized normal distribution curve is equal to 1.0. The area under the curve can be considered as probability. For example, one half of the area is less than the mean and one-half is greater than the mean. Tables have been developed to compute the area under the curve for any given value. The use of these tables and the normal distribution is illustrated in the following examples.

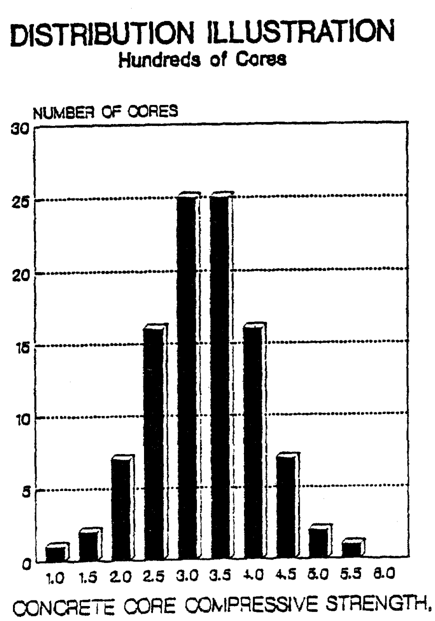
Example 1. Computation of the probability that concrete strength is less than 2500 psi. Assume that the mean and standard deviation of the sample is equal to that of the entire section of concrete pavement. The



(a)

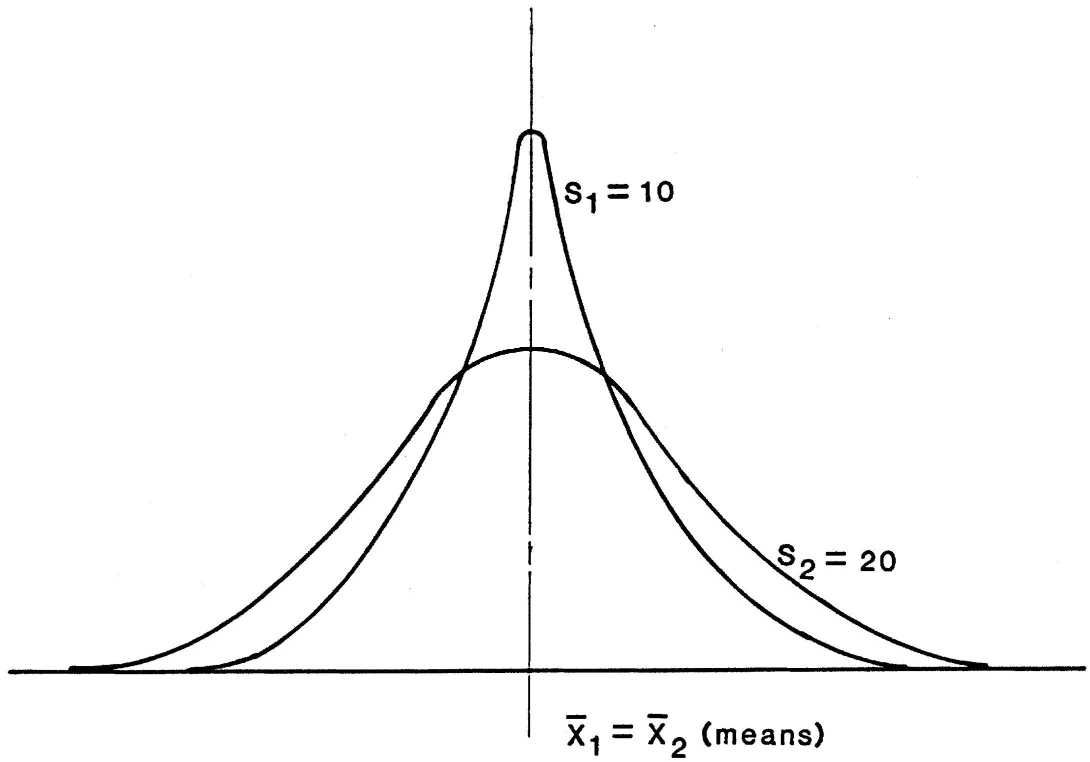


(b)

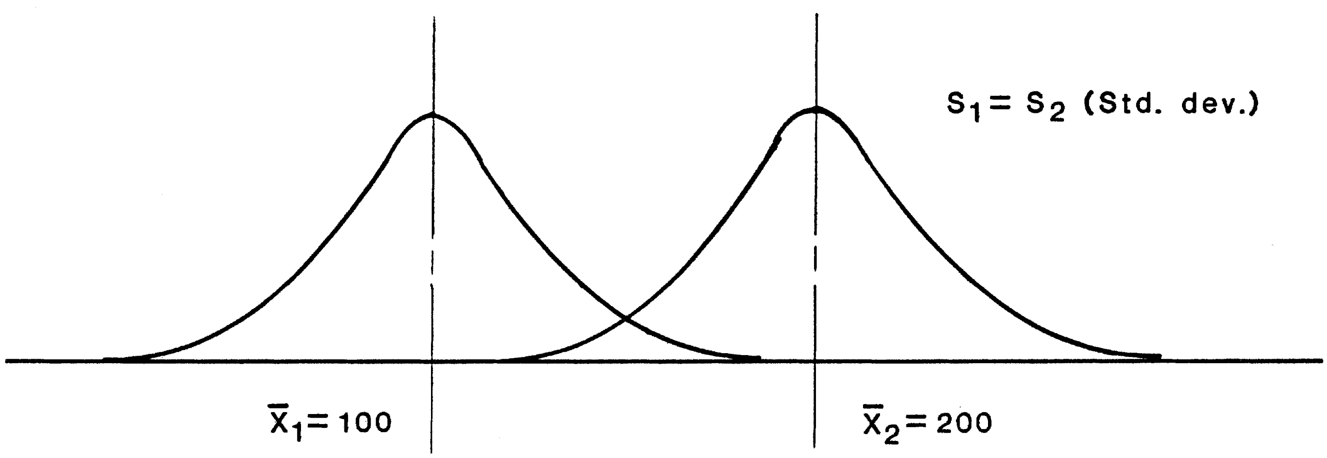


(c)

Figure 2-7.1. Example of Statistical Distribution.



(a) Distribution with equal means but different Std. dev.



(b) Distribution with equal Std. dev. and different means.

Figure 2-7.2. Sample Statistical Distributions.

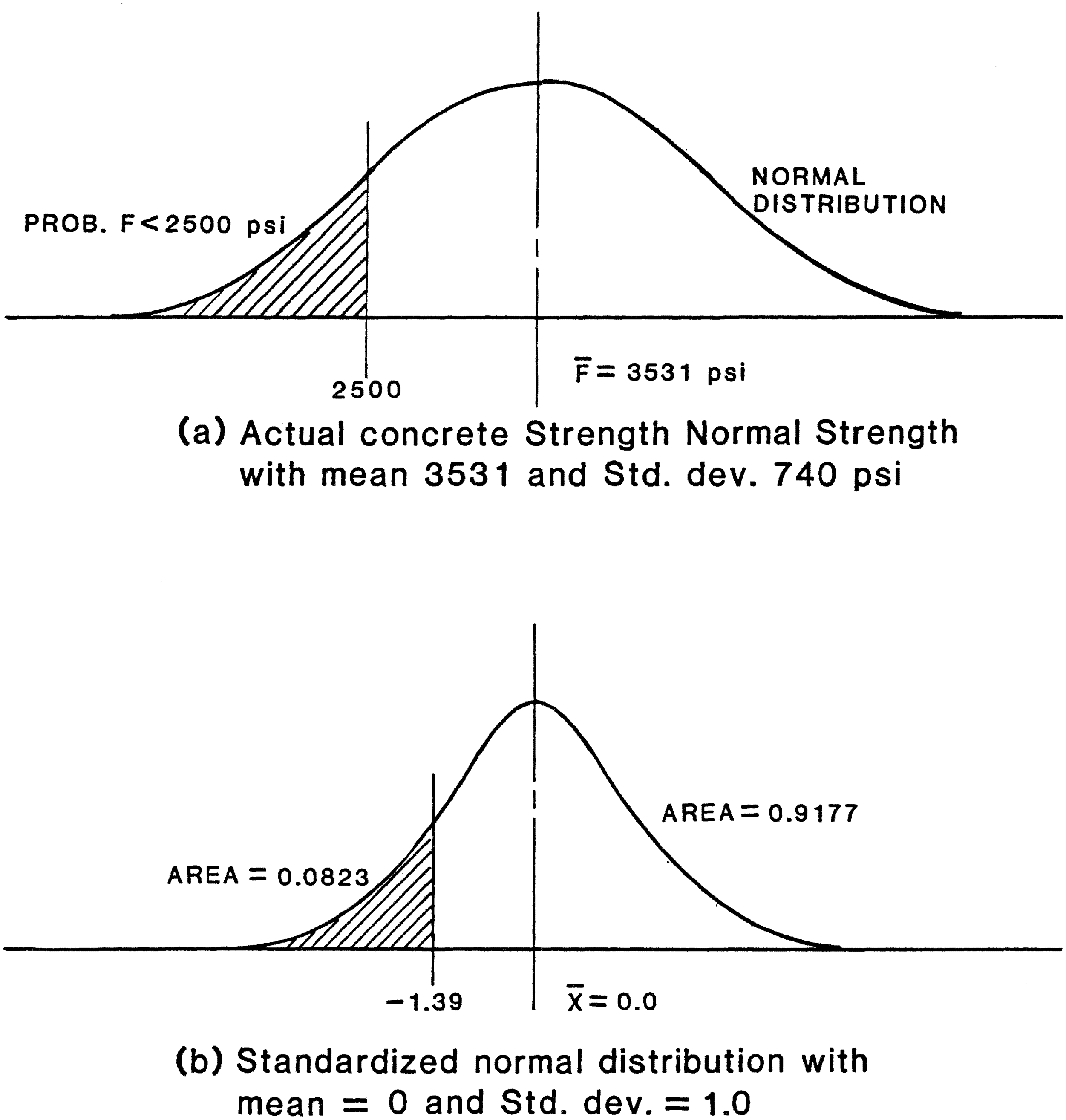


Figure 2-7.3. Illustration of the Estimation of the Proportion of Concrete Strength That is Less Than 2500 psi.

proportion of pavement that has a strength value less than 2500 psi is computed as follows:

First compute the standardized normal deviate:

$$z = [\,2500 - 3531\,]/740 = -1.39$$

The following values exist on each scale as shown in Figure 2-7.3:

Item	Actual Normal Distribution Scale	Standardized Normal Distribution Scale
Mean	3531 psi	0
Stand. Dev.	740 psi	1
Value of Interest	2500 psi	- 1.39

The probability that the concrete strength at any point in the slab is less than 2500 psi is then determined using Figure 2-7.4. Entering the table with a value of 1.39 (the minus sign is neglected in the table), an area of 0.9177 is obtained. This is the area from - infinity to + 1.39. By subtracting this value from 1.0, the probability of a strength value less than 2500 psi (or less than - 1.39) will be obtained.

$$\begin{array}{rr} & 1.0000 \\ - & \underline{0.9177} \\ & 0.0823 \end{array}$$

This means that 8.23 percent of the specimens are expected to have a strength less than 2500 psi, and 91.77 percent greater than 2500 psi.

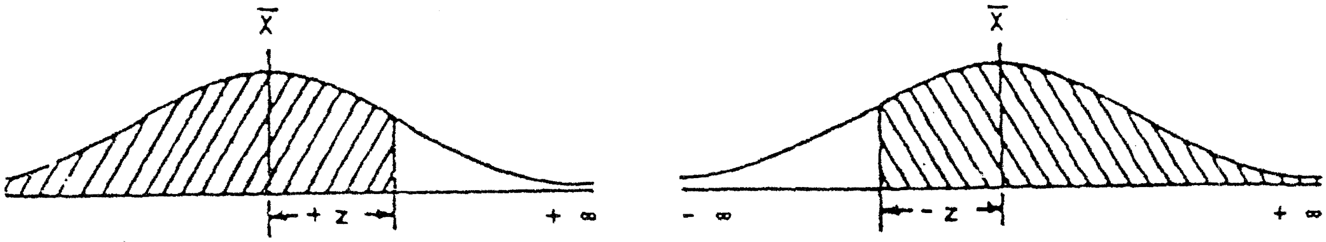
<u>Example 2. Computation of the probability that a given pavement section can sustain 1,000,000 18-kip equivalent single axle load applications.</u> Assume that the mean number of load applications that a pavement section can carry to a terminal serviceability index of 2.5 is 3,000,000 (say using the AASHTO Design Guide). Evidence exists that the number of load applications a pavement can carry to failure is log-normally distributed (simply the log to base 10 of the number of applications). Also assume that the standard deviation of the lognormal distribution is 0.45. The probability is computed as follows:

First compute the standardized normal deviate:

$$z = [\,\log(1{,}000{,}000) - \log(3{,}000{,}000)\,]/0.45 = -1.06$$

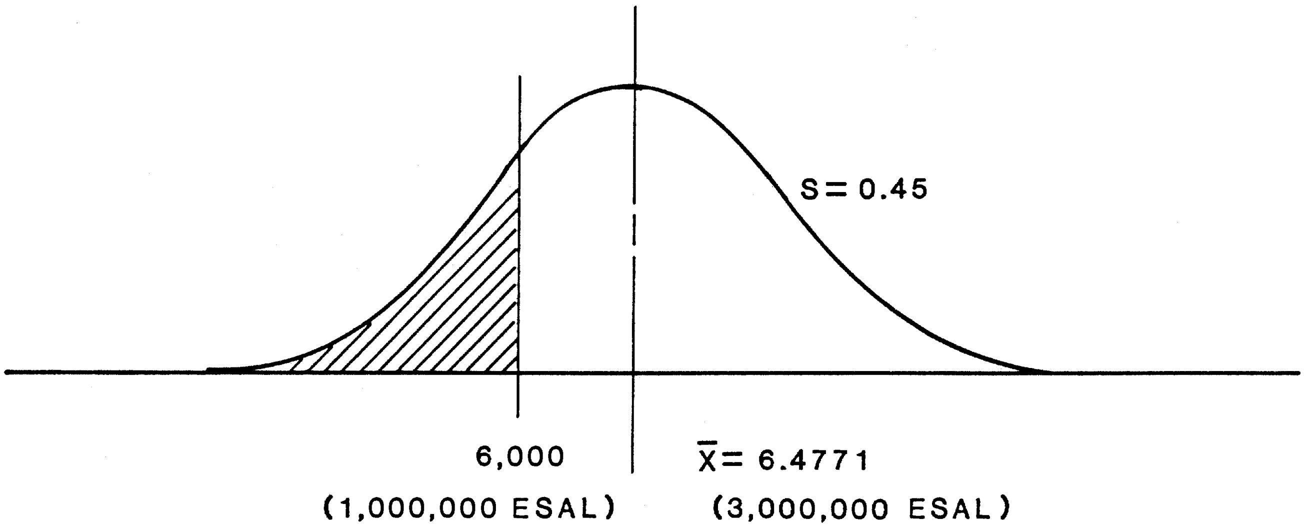
The following values exist on each scale as shown in Figure 2-7.5:

Item	Actual Log-Normal Distribution Scale	Standardized Normal Distribution Scale
Mean	6.4771	0
Stand. Dev.	0.45	1
Value of Interest	6.0000	- 1.06

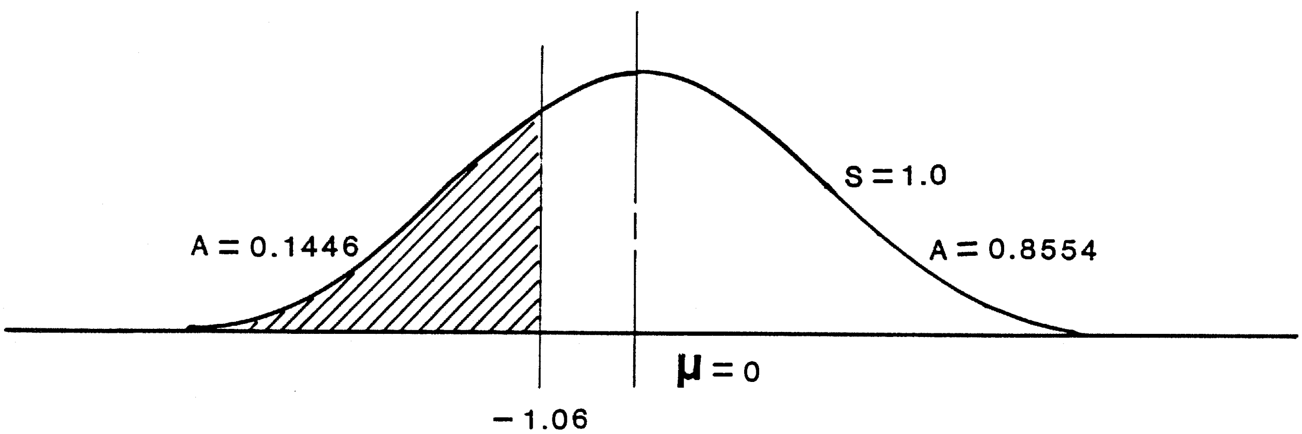


z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	.5000	.5040	.5080	.5120	.5160	.5199	.5239	.5279	.5319	.5359
0.1	.5398	.5438	.5478	.5517	.5557	.5596	.5636	.5675	.5714	.5753
0.2	.5793	.5832	.5871	.5910	.5948	.5987	.6026	.6064	.6103	.6141
0.3	.6179	.6217	.6255	.6293	.6331	.6368	.6406	.6443	.6480	.6517
0.4	.6554	.6591	.6628	.6664	.6700	.6736	.6772	.6808	.6844	.6879
0.5	.6915	.6950	.6985	.7019	.7054	.7088	.7123	.7157	.7190	.7224
0.6	.7257	.7291	.7324	.7357	.7389	.7422	.7454	.7486	.7517	.7549
0.7	.7580	.7611	.7642	.7673	.7704	.7734	.7764	.7794	.7823	.7852
0.8	.7881	.7910	.7939	.7967	.7995	.8023	.8051	.8078	.8106	.8133
0.9	.8159	.8186	.8212	.8238	.8264	.8289	.8315	.8340	.8365	.8389
1.0	.8413	.8438	.8461	.8485	.8508	.8531	.8554	.8577	.8599	.8621
1.1	.8643	.8665	.8686	.8708	.8729	.8749	.8770	.8790	.8810	.8830
1.2	.8849	.8869	.8888	.8907	.8925	.8944	.8962	.8980	.8997	.9015
1.3	.9032	.9049	.9066	.9082	.9099	.9115	.9131	.9147	.9162	.9177
1.4	.9192	.9207	.9222	.9236	.9251	.9265	.9279	.9292	.9306	.9319
1.5	.9332	.9345	.9357	.9370	.9382	.9394	.9406	.9418	.9429	.9441
1.6	.9452	.9463	.9474	.9484	.9495	.9505	.9515	.9525	.9535	.9545
1.7	.9554	.9564	.9573	.9582	.9591	.9599	.9608	.9616	.9625	.9633
1.8	.9641	.9649	.9656	.9664	.9671	.9678	.9686	.9693	.9699	.9706
1.9	.9713	.9719	.9726	.9732	.9738	.9744	.9750	.9756	.9761	.9767
2.0	.9772	.9778	.9783	.9788	.9793	.9798	.9803	.9808	.9812	.9817
2.1	.9821	.9826	.9830	.9834	.9838	.9842	.9846	.9850	.9854	.9857
2.2	.9861	.9864	.9868	.9871	.9875	.9878	.9881	.9884	.9887	.9890
2.3	.9893	.9896	.9898	.9901	.9904	.9906	.9909	.9911	.9913	.9916
2.4	.9918	.9920	.9922	.9925	.9927	.9929	.9931	.9932	.9934	.9936
2.5	.9938	.9940	.9941	.9943	.9945	.9946	.9948	.9949	.9951	.9952
2.6	.9953	.9955	.9956	.9957	.9959	.9960	.9961	.9962	.9963	.9964
2.7	.9965	.9966	.9967	.9968	.9969	.9970	.9971	.9972	.9973	.9974
2.8	.9974	.9975	.9976	.9977	.9977	.9978	.9979	.9979	.9980	.9981
2.9	.9981	.9982	.9982	.9983	.9984	.9984	.9985	.9985	.9986	.9986
3.0	.9987	.9987	.9987	.9988	.9988	.9989	.9989	.9989	.9990	.9990
3.1	.9990	.9991	.9991	.9991	.9992	.9992	.9992	.9992	.9993	.9993
3.2	.9993	.9993	.9994	.9994	.9994	.9994	.9994	.9995	.9995	.9995
3.3	.9995	.9995	.9995	.9996	.9996	.9996	.9996	.9996	.9996	.9997
3.4	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9997	.9998

Figure 2-7.4. Areas Under a Standard Normal Curve.



(a) Actual distribution of traffic loadings — 18 — kip ESAL



(b) Standardized normal distribution
(mean = 0, std. dev. = 1)

Figure 2-7.5. Statistical Distribution.

The probability that the pavement section can carry 1,000,000 18-kip ESAL is then determined using Figure 2-7.4. Entering the table with a value of 1.06 (the minus sign is neglected in the table), an area of 0.8554 is obtained. This is the area from - infinity to + 1.06. By subtracting this value from 1.0, the probability of the pavement carrying less than 1,000,000 applications (or less than - 1.06) will be obtained.

$$\begin{array}{r} 1.0000 \\ \underline{0.8554} \\ 0.1446 \end{array}$$

This means that there is a probability of 0.1446 or 14.46 percent that the pavement section cannot handle the 1,000,000 load applications. It also means that there is the corresponding probability of 85.56 percent that the pavement can handle the loads as illustrated in Figure 2-7.5.

4.0 INTERESTING EXAMPLES OF VARIABILITY

There are many sources of variability in pavement design, construction and performance. Some interesting examples are presented.

Design variability. The designer must make certain assumptions as to the inputs for the pavement design procedure such as future traffic loadings, future climatic conditions, material properties (that they will conform to the specifications) and roadbed soil properties. The values of these inputs that actually exist after construction or the traffic loads at the end of the design period may be greatly different than those assumed in design.

An illustration found in the literature is a comparison between in situ R-values taken from the grade after construction and design R-values assumed from samples taken prior to construction for a pavement project (11).

Parameter	(Pre-Construcion) Samples From Drilled Holes in Excavation Area	(Post-Construction) Samples From Top Subgrade
Mean	55	65
Range	13-70	35-75
Standard deviation	16	9

It is fortunate that on this project the mean and range of values are better for the in-situ pavement after construction than those assumed in design. Often the reverse of this is the case and premature failure occurs.

Another classical design uncertainty is the estimation of future traffic loadings. Figure 2-7.6 shows an illustration of the potential error in estimating the total accumulated 18-kip equivalent single axle (ESAL) loadings for a particular design project. This plot shows large variation between that assumed in design and what actually occurred. Many other projects have probably had much greater traffic estimation errors, usually underestimation.

Construction. Practically everything associated with pavement construction has associated variability and uncertainty. Common items where variability has been studied are shown in Figure 2-7.7 (PCC compressive strength), Figure 2-7.8 (PCC core thickness), Figure 2-7.9 (AC core

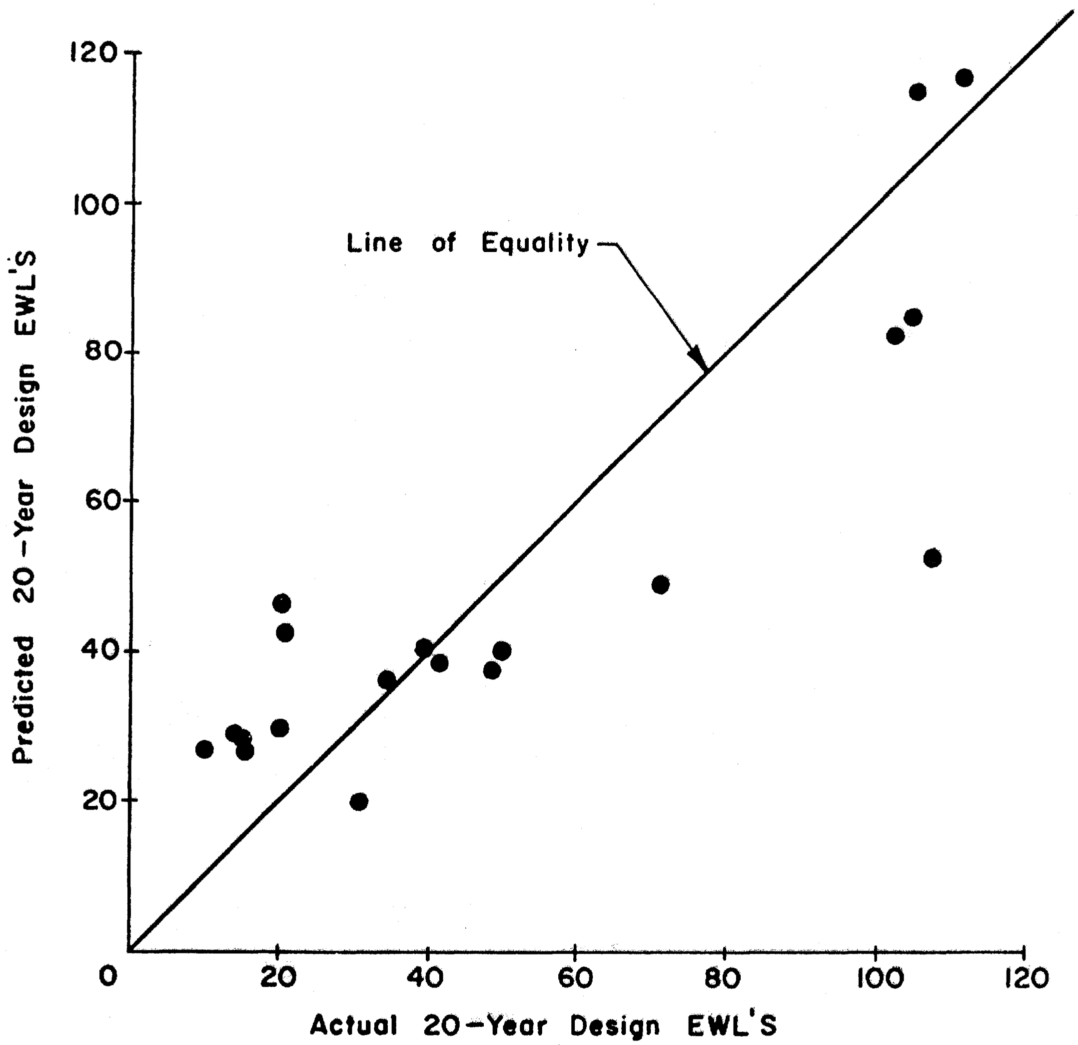


Figure 2-7.6. Predicted Versus Actual Kentucky EWL's for 20-Year Design Analysis Period (8).

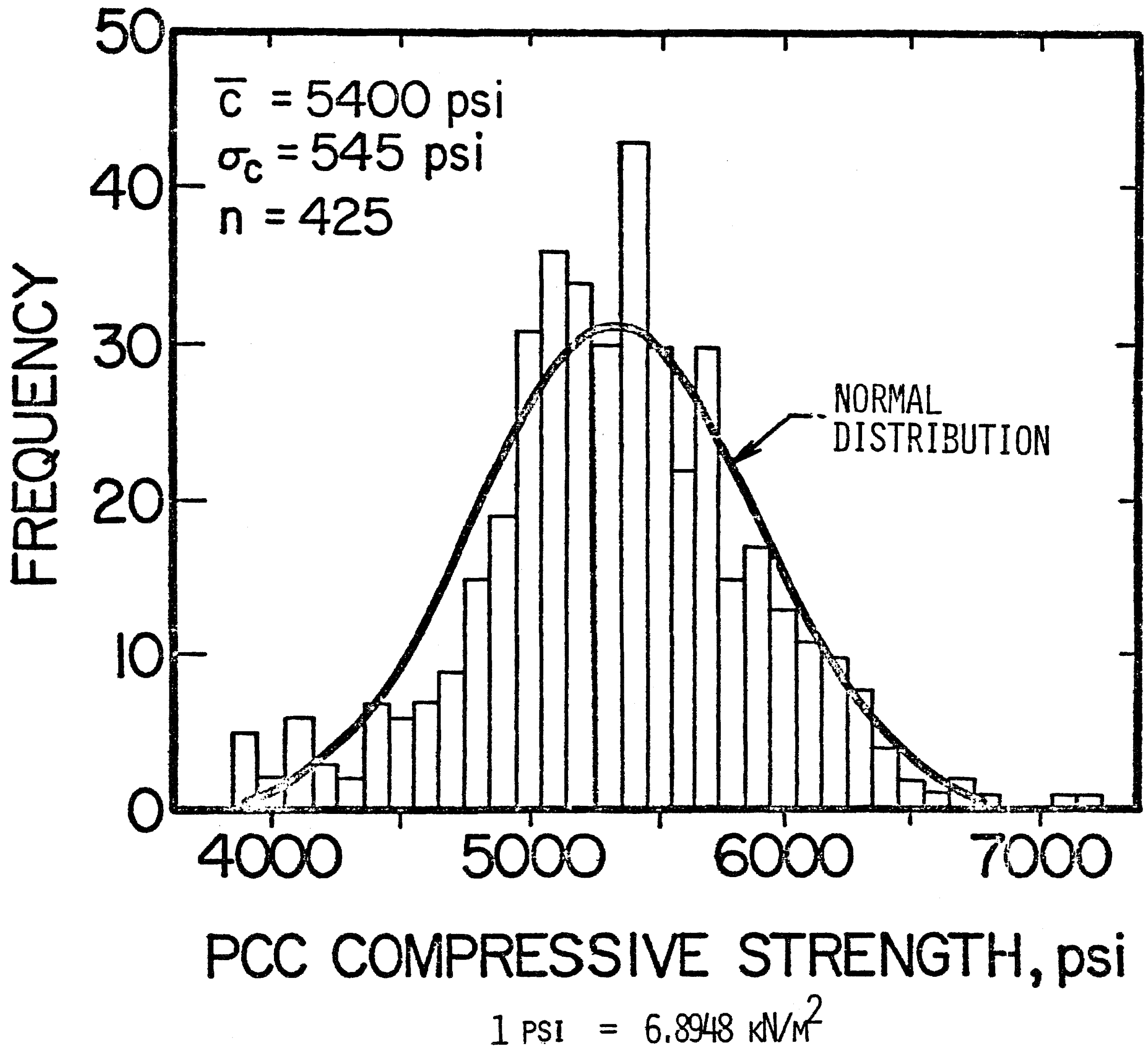


Figure 2-7.7. Variation of Compressive Strength of Cores Cut From a P.C.C. Slab Approximately 500 Feet (152 m) Apart (4).

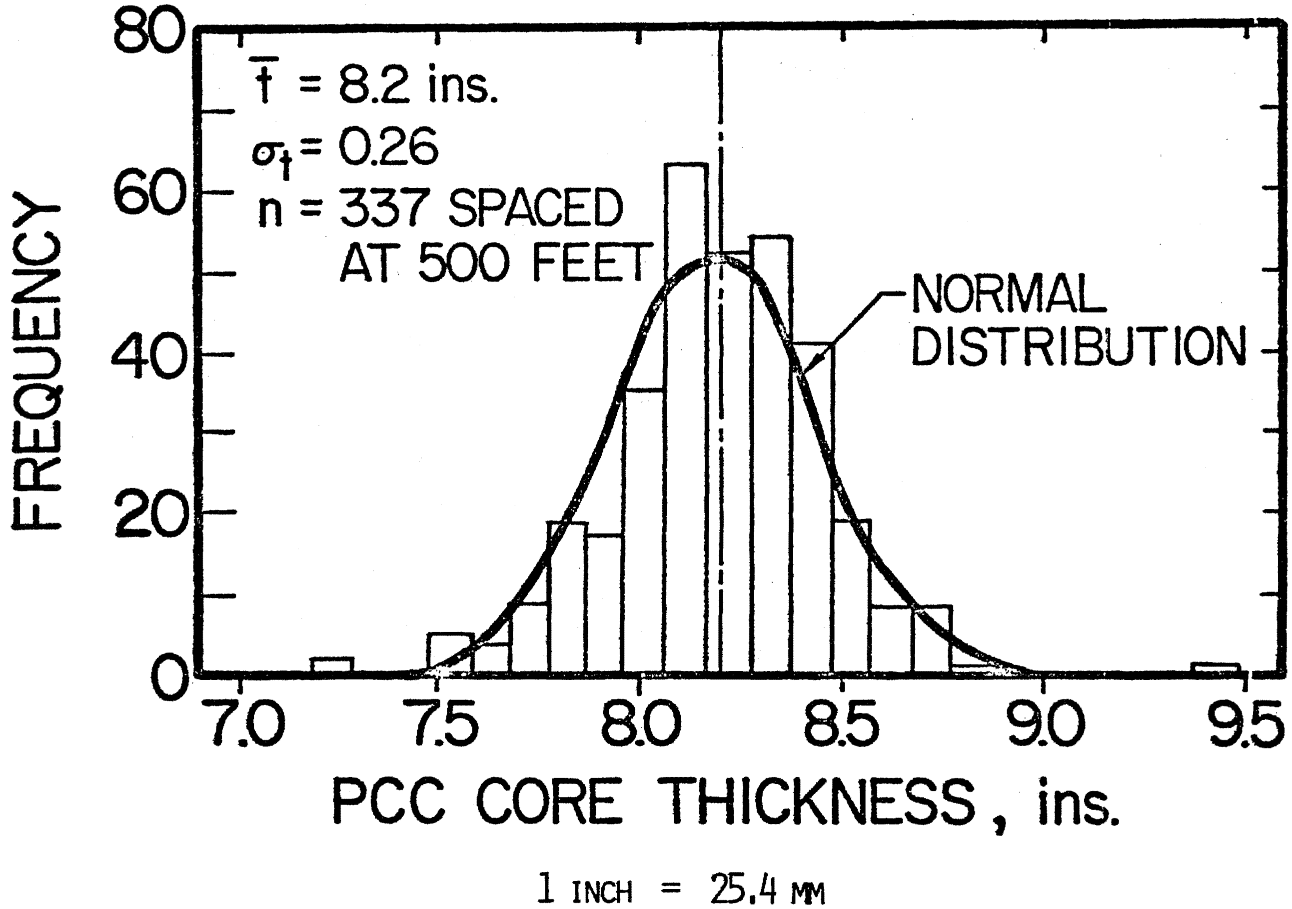


Figure 2-7.8. Variation of P.C.C. Slab Thickness Measured From Cores Cut Approximately 500 Fet (152 m) Apart (4).

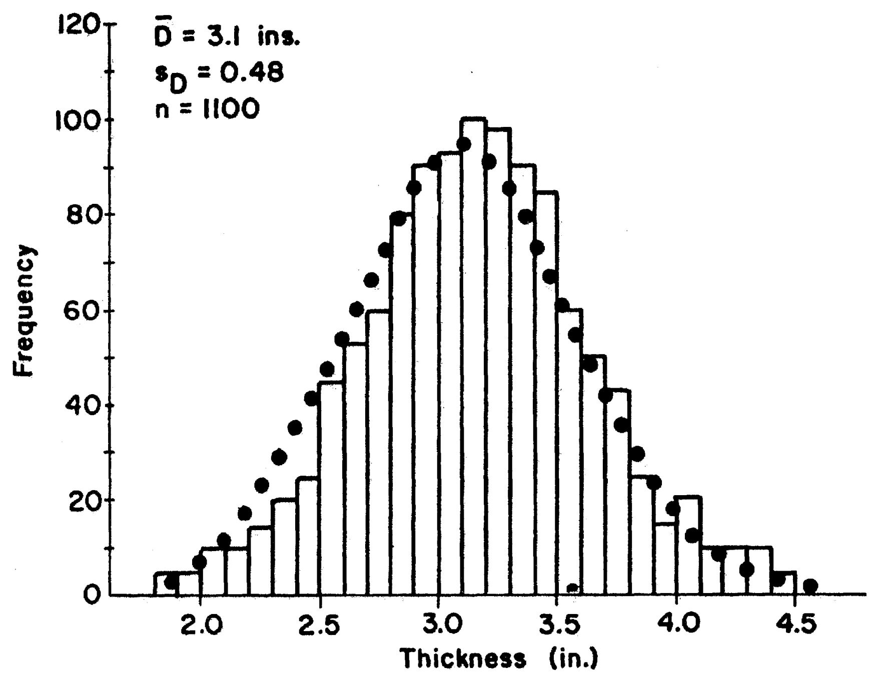


Figure 2-7.9. Thickness of Individual Cores for Asphalt Surfacing (8).

thickness), Figure 2-7.10 (density of embankments), and Figure 2-7.11 (initial serviceability index along a given project caused by construction profile variations). Material property variability is the most well understood source as it has been studied extensively. There is also variation in other construction areas such as asphalt content, moisture content, depth of reinforcement, and soil support. Many other variations are presented in References 4, 6, 8, 9, 11, 16, 17 and 19.

Any experienced construction engineer will tell you that there is a large difference in the way a pavement will perform, depending upon the "quality of construction" although this is very difficult to quantify.

One interesting way to observe the overall effects of construction/material variability is to observe the deflection profile of a pavement after construction of different layers and over time as shown in Figure 2-7.12. This variation is caused by several variables such as layer thickness, material properties, roadbed soil properties and drainage differences along the project.

This variation leads to variation in strains and stresses, and consequently in performance along the project. Anyone who walks along a pavement for some distance can see the varying localized deterioration of the pavement (e.g., alligator cracking, rutting, cracked slabs, deteriorated joints). This variation is caused largely from variations in material properties and other factors along the project.

Performance. As a result of the variabilities in design, construction and materials, there will be a large variability in the performance of pavement sections. This variation can be considered as follows:

1. Differences in performance along a given pavement project. If a pavement were divided into say 0.1 mile sections and the performance of each of these sections was measured in terms of cracking, rutting, faulting or PSI there would typically exist a large variation.

2. Differences in performance between seemingly identical projects that are located close to each other along the same highway and constructed under identical specifications.

A study of this variability was made during the development of the LTPP experimental design to determine the variability of performance of seemingly "identical" projects constructed adjacent to each along the same highway. Some results from one state from 12 similar projects are shown in Figure 2-7.13. For example, the PSR ranged from 4.2 to 2.5, the number of deteriorated joints/mile ranged from 0 to 18, etc. A coefficient of variation for major distress types ranged from 15 to 116 percent.

5.0 HOW VARIABILITY AFFECTS ADEQUACY OF DESIGN

The adequacy of a pavement design depends greatly on the variability associated with the pavement materials, traffic, climate, placement of dowels and reinforcement, etc. The following discussion provides some examples (1,2,4,6,10).

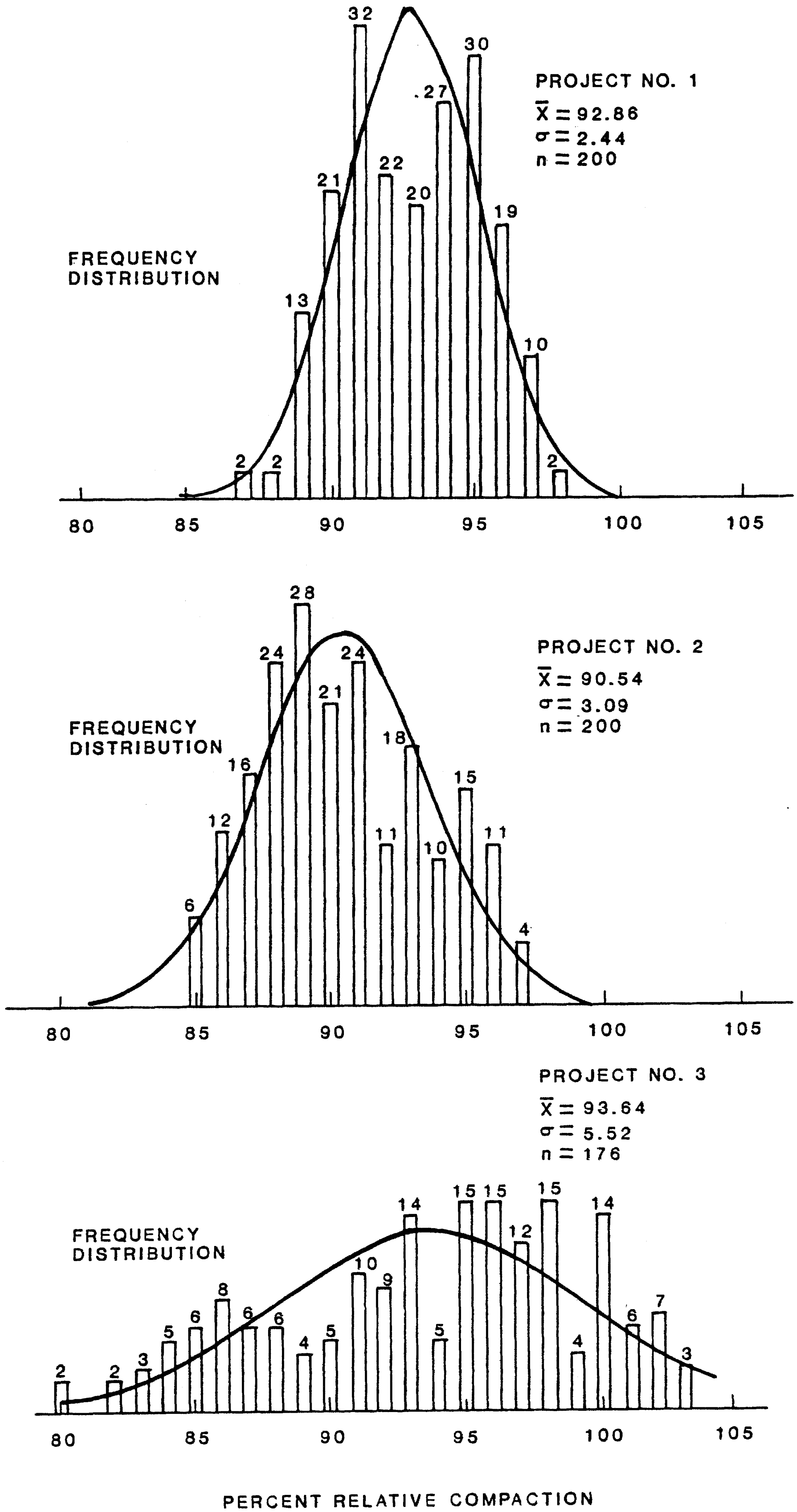


Figure 2-7.10. Variation in Density of Embankments (19).

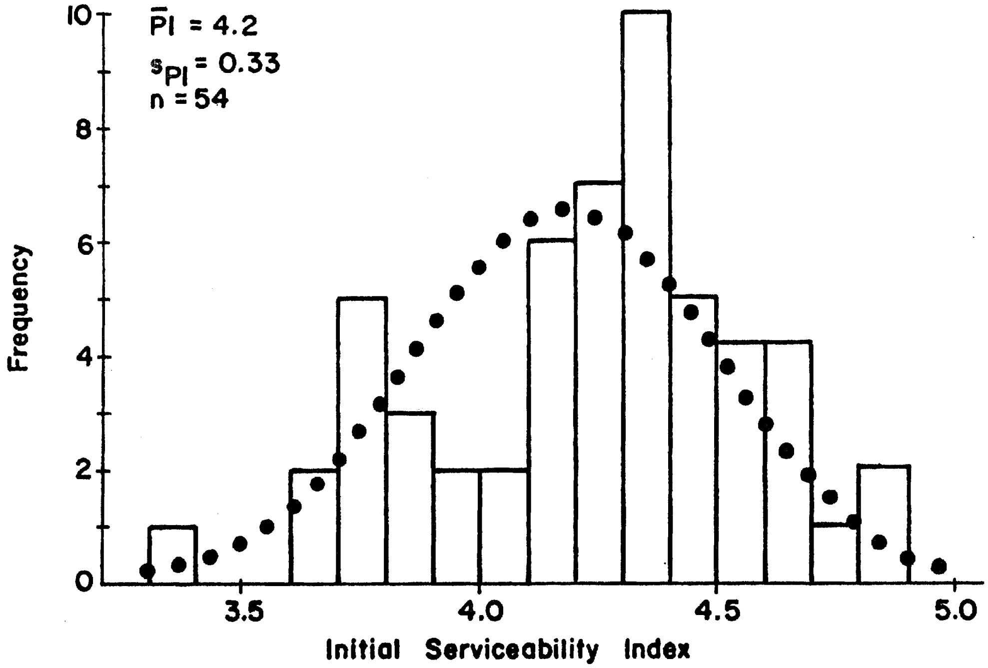


Figure 2-7.11. Histogram of Initial Serviceability Index of Newly Constructed Flexible Pavement (Measurements Taken on 0.2-mile Sections) (8).

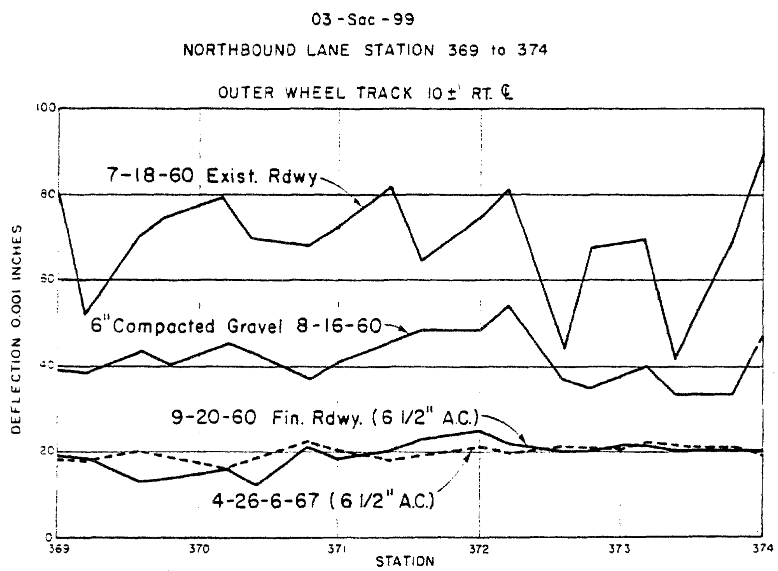


Figure 2-7.12. Effect of Age on Deflections (11).

Section	Age (years)	ESAL (millions)	PSR	Faulting (inches)	Cracking (feet/mile)	Deteriorated Joints (no./mile)
1	18	5	4.2	0.11	0	0
2	18	5	4.0	0.05	0	0
3	18	5	3.4	0.25	0	0
4	22	5	3.8	0.06	950	1
5	22	5	3.6	0.10	1162	0
6	19	8	3.1	0.26	1214	0
7	19	8	2.5	0.39	1478	0
8	22	6	3.3	0.24	106	5
9	22	7	3.3	0.16	106	9
10	20	6	3.8	0.19	0	0
11	17	7	3.2	0.33	106	9
12	18	8	2.6	0.32	1426	18
Mean	19.4	6.3	3.4	0.20	545	4
COV	10	20	15	54	116	115

Pavement: JRCP, 10-inch slabs, 100-foot joint spacing, fine-grained subgrade, no "D" cracking, age between 17 and 22 years, ESAL between 5 and 9 million in design lane.

Figure 2-7.13. Variation in Performance Between Seemingly Identical Projects in Illinois After 17 to 22 Years of Performance (18).

1. Variation in material properties along a pavement results in variation in the development of distress and roughness along the pavement. Localized failures resulting from "weak" areas result in a decrease in pavement life.

2. Variation in placement of such items as dowels at joints and depth of reinforcement result in variation in the development of distress and roughness along the pavement. Localized failures can occur causing a decrease in pavement life.

3. Variation between design and actual pavement design inputs can result in drastic shortening or lengthening of pavement design life. Some classic examples are traffic load estimation over a 20 year period, or climate conditions, or material strengths.

4. Design procedure inadequacy is another very important "variation" that affects pavement design. For example, a design procedure developed using limited data is commonly used for design situations beyond its range (such as other climatic zones). This could result in great variation in the predicted design life and the actual life achieved.

Perhaps many more examples could be described, but the point is clear, variability exists in almost everything associated with a pavement, and this variability has a great effect on pavement life. The next section describes the concepts of how to deal with variability in design to provide some assurance that the pavement will last its design life.

6.0 General Concept of Reliability of Design

The engineering design of any structure must consider the probability of failure, which of course is one minus the probability of success or design reliability. The consideration of the design reliability in engineering design has been underway since 1947 (14), particularly in the area of structural engineering.

6.1 Use of Structural Reliability Concepts

The consideration of reliability concepts in pavement design was first strongly advocated in the early 1970's and several papers were published on the subject (5,6,8,9,10,11,12,13,15,16,17).

Lemer and Moavenzadeh stated the need for consideration of reliability in design as follows:

> "Reliability is important in the pavement system because of the uncertainty involved in all aspects of the pavement process: planning, design, construction, operation and maintenance. Uncertainty arises from lack of information and inability to predict the future. It is embodied in the assumptions that must be made to derive analytical models, the limited amount of data available from tests, and the variable quality of the real-world environment." (15)

A reliability design factor was incorporated into the Texas Highway Department pavement design procedure for flexible pavements in 1973, and has been used successfully since that time (8,9,10). Reliability concepts were developed and incorporated into the AASHTO design procedures in 1973 by Kher and Darter (6). The same reliability concepts were then finally adopted into the AASHTO Design Guide in 1985 (2).

A general definition of pavement design reliability is as follows:

> "Reliability is the probability that the pavement system will perform its intended function over its design life and under the conditions (or environment) encountered during operation." (8)

Uncertainty in pavement design has traditionally been accommodated by the use of "safety factors." Judgment and experience must be used in assigning appropriate safety factors to the various design parameters. Using judgement, parameters of which the engineer is less certain are typically assigned higher safety factors. If the engineer is familiar with the design procedure, the most sensitive factors (for which small variations produce large changes in designs) can be assigned very conservative values.

Application of the traditional safety factor approach to pavement design can result in over-design or under-design, depending on the magnitudes of the safety factors applied and the sensitivity of the design procedures. A more realistic approach to addressing uncertainty is one which utilizes safety factors that reflect the amount of statistical variability associated with each of the parameters in the design process. The amount of uncertainty associated with the parameters and the relative importance of the parameters is critical to the design process. The use of design reliability concepts can accomplish the same thing as factors of safety.

6.2 Definitions of Reliability

As an example, the structural reliability of a simply supported beam can be defined in the following way:

$$R \text{ (percent)} = \text{Probability} [\text{ strength} > \text{stress}] \qquad (2\text{-}7.1)$$

The reliability of the beam not failing is the probability that strength will be greater than applied stress.

In pavement design, reliability can be defined in a similar way (2):

$$R \text{ (percent)} = \text{Probability} [N_t > N_T] \qquad (2\text{-}7.2)$$

where:

N_t = number of 18-kip ESAL applications to a terminal serviceability, as estimated by a predictive equation (analogous to strength)

N_T = number of 18-kip ESAL applications forecasted to be applied to the pavement over the design period (analogous to stress)

The reliability of the pavement design to perform its function (e.g., for the serviceability index to remain above an established terminal level) is the probability that the number of 18-kip ESAL that the pavement can handle to a terminal serviceability condition will be greater than the number of 18-kip ESAL applications forecasted.

Based upon this simple concept, the theory for expressing pavement design reliability can be developed. It is very similar to that used in structural engineering, but modified to consider repeated loadings (causing fatigue damage) which is predominant for pavements.

6.3 Example Reliability Design Using Stress/Strength

This section illustrates the computation of design reliability considering simple stress and strength of a concrete slab for structural reliability. This example will serve as the basis for understanding pavement design reliability (4).

Whenever the stress level at a point in a structure exceeds the strength at that point, a fracture occurs. The probability of fracture can be defined as

$$p_f = P\,[\text{ stress(S)} > \text{strength(F) }],$$

where P [ ] denotes the probability of occurrence of whatever is contained within the brackets. Conversely, the probability of no fracture, or the reliability (R), can be defined as $R = 1 - p_f$. The strength magnitude within a structure is a random variable in the sense that it varies from point to point in the structure. Applied stress magnitude is also a random variable which depends in part on loading conditions both from climatic factors and traffic loadings. Since both stress and strength are random variables, the p_f can be expressed as follows:

$$p_f = P\,[S > F] = P\,[d < 0]$$

where:

$d = F - S$
S = applied stress
F = strength

Assuming that both S and F are normally distributed, d will also be normally distributed as shown in Figure 2-7.14.

Using bars above the expressions to represent their mean values,

$$\overline{d} = \overline{F} - \overline{S}.$$

The standard deviation of d can be computed as s_d by the following expression

$$s_d = [(s_S^2 + s_F^2)]^{0.5}$$

where:

s_S = standard deviation of stress, S
s_F = standard deviation of strength, F

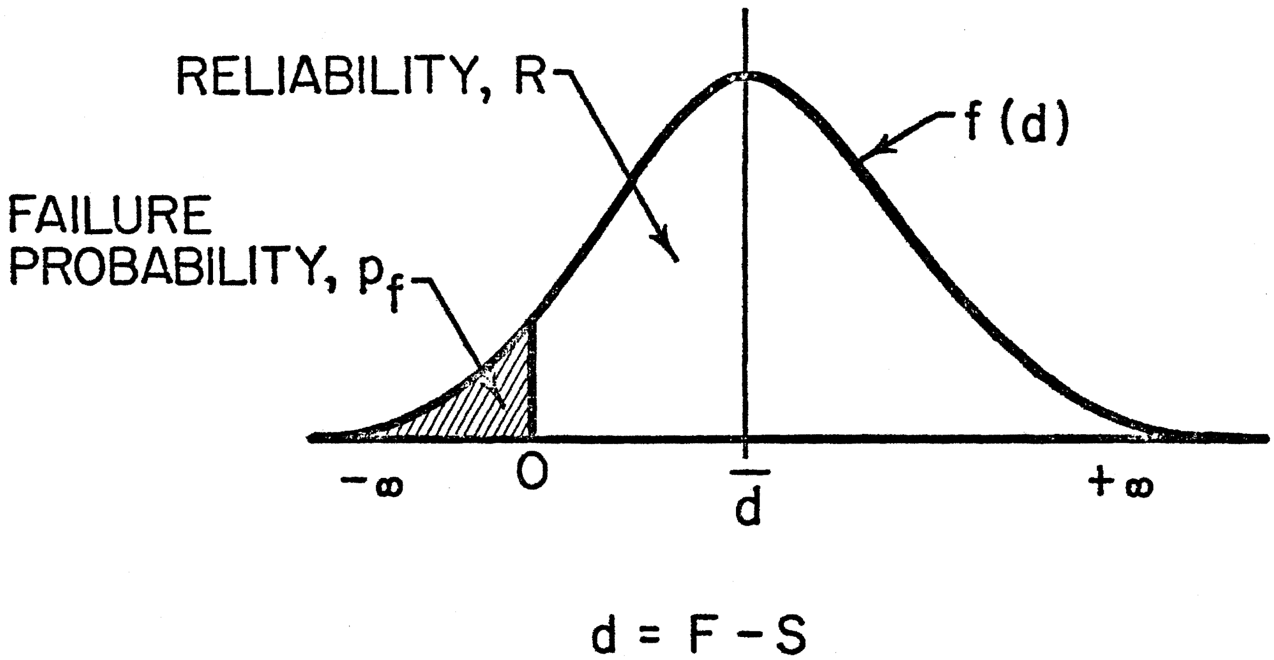


Figure 2-7.14. Distribution of d = F - S Showing Areas of Failure Probability and Reliability Probability (7).

As shown in Figure 2-7.14, the probability of fracture, p_f, is given by the area to the left of 0.

$$p_f = P\,[d < 0] = P\,[-\infty < d < 0]$$

Reliability is the area to the right of 0 as shown.

To calculate the p_f or R, normal distribution tables can be employed. For example, consider the following conditions for a given 7 inch thick slab.

S = 360 psi, mean tensile stress at the critical location
F = 690 psi, mean flexural strength of the slab
s_S = 48 psi, s_F = 125 psi

The parameter d must now be transformed into a normal variate with mean of zero and variance of 1.0 so that normal distribution tables can be used.

$$z = [\bar{d} - d]/s_d = [0 - d]/s_d = -330/115 = -2.64$$

The area under the normal curve from $-\infty$ to -2.64 is 0.0041, and therefore p_f = 0.41 percent. A graphic illustration of this area of failure is shown in Figure 2-7.15 where the actual distributions of the flexural strength and stress for the 7 inch thick slab are shown to overlap. This area of overlap is not the probability of failure but a function of the probability of failure.

The figure also shows the stress distribution for a 9 inch slab (the stress is much lower) where the probability of failure is very small.

This example has shown the basic calculations necessary to compute the probability of failure for a structure. The design reliability is just 1.0 minus the probability of failure, or 1.0 - 0.0041 = 0.9959.

6.4 Example Reliability Design Using Traffic Loadings

Reliability can be defined in the following way:

> "The reliability of a pavement design-performance concept is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period." (2)

The definition of pavement design reliability is given by:

$$R\text{ (percent)} = 100 * \text{Prob}\,(N_t \geq N_T)$$

where:

N_t = number of 18-kip ESALs the pavement can support before reaching terminal serviceability, as estimated by a predictive equation) (strength)

N_T = actual design period traffic (number of 18-kip ESALs forecasted to be applied to the pavement over the design period) (stress)

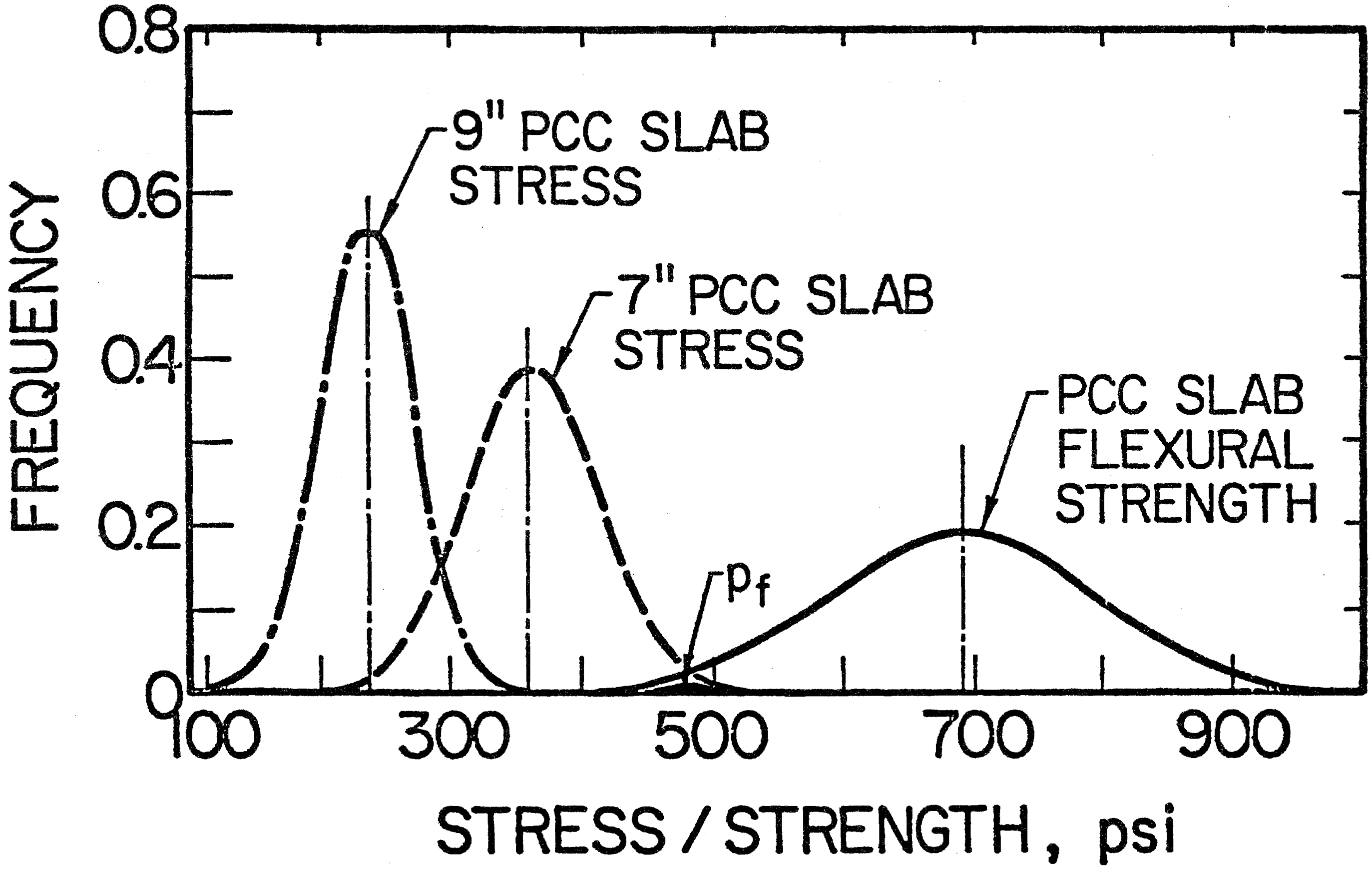


Figure 2-7.15. Illustration of the Distribution of Flexural Strength Along A P.C.C. Slab and the Estimated Distributions of Applied Stresses Using Westergaard Edge Loading Model for a 7 and 9 inch (178 and 229 mm) Slab. (Note: The Means and Standard Deviations are as follows: $\overline{F}$ = 690 psi, σ_F = 104 psi; $\overline{S}_{7''}$ = 360 psi, σ_s = 48 psi; $\overline{S}_{9''}$ = 237 psi, σ_s' = 30 psi (7).

The reliability level of a pavement design is the probability that the pavement's actual performance will equal or exceed the actual design period traffic, or in other words, the probability that the pavement will survive the design period traffic with a serviceability level greater than or equal to terminal serviceability. Figure 2-7.16 illustrates the concepts of these distributions and their overlap.

If it is assumed that the log of load applications to failure is normally distributed (a common fatigue phenomenon) and also that the log of the design period traffic estimate is normally distributed, then the above definition of reliability can be written as:

$$R \text{ (percent)} = 100 * \text{Prob} (\log N_t \geq \log N_T)$$

$$= 100 * \text{Prob} [(\log N_t - \log N_T) \geq 0]$$

$$= 100 * \text{Prob} (D \geq 0)$$

where:

$$D = \log(N_t) - \log(N_T)$$

Since $\log N_t$ and $\log N_T$ are both normally distributed, their difference D is also normally distributed. The relationship of their mean values can be written as:

$$\overline{D} = \overline{\log(N_t)} - \overline{\log(N_T)}$$

The standard deviation of D is s_D, which can be computed by the following equation:

$$s_D = [(s_{\log(Nt)})^2 + (s_{\log(NT)})^2]^{0.5}$$

where:

$s_{\log(Nt)}$ = the standard deviation of $\log N_t$ (the pavement life estimate)

$s_{\log(NT)}$ = the standard deviation of $\log N_T$ (the traffic estimate)

The transformation that relates D (with a mean of D and a standard deviation of s_D) and the standard normal variable Z (with a mean of 0 and a standard deviation of 1) is:

$$Z = (D - D)/s_D$$

For D = 0,

$$Z = Z_0 = - D/s_D = - \frac{\overline{\log N_t} - \overline{\log N_T}}{((s_{\log N_t})^2 + (s_{\log N_T})^2)^{1/2}}$$

For $D = \infty$,

$$Z = Z_\infty = \infty$$

The expression for reliability may then be written as:

$$R \text{ (percent)} = 100 * (Z_0 < Z < Z_\infty)$$

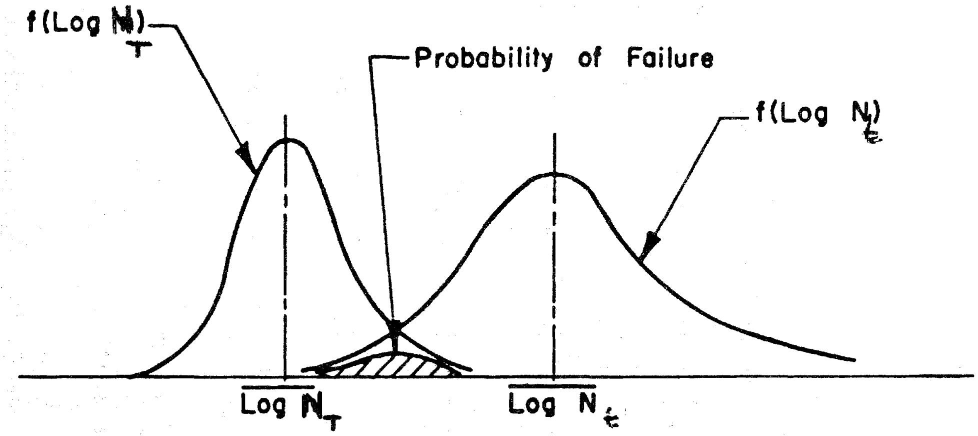


Figure 2-7.16. Illustration of Overlay of Distributions of Log N_T and Log N_t (8).

The reliability of a design can thus be determined as the area under the normal distribution curve between the limits of $Z = Z_0$ and $Z = \infty$. Figure 2-7.17 illustrates the concepts. Note that these concepts are identical to that illustrated with stress and strength.

6.5 Application of Reliability to the Design Process

The procedure described makes it possible to design at any predetermined level of reliability. The overall variance S_D of the pavement performance and the estimated design period traffic can be determined for the individual design problem if sufficient information is available, or the following default values provided in the AASHTO Design Guide can be used (2):

S_D = 0.34 for rigid pavements and
0.44 for flexible pavements,

for the case when variance of projected future traffic is NOT considered, and
= 0.39 for rigid pavements and
0.49 for flexible pavements,

for the case when variance of projected future traffic is considered. These values were developed from an analysis variance that existed at the AASHD Road test and future traffic prediction.

It is important to keep in mind that the reliability level is an adjustment to the performance of the pavement design as represented in terms of traffic, i.e., 18-kip ESALs to terminal serviceability. The consequence of this is that the pavement thickness is increased in order to delay the occurrence of load-associated deterioration. This approach is reasonable overall, but it clearly has some limitations. Serviceability loss is not only a function of traffic but also a function of roughness and other distresses, which are not entirely load-caused. The occurrence of distress types which are not predominantly load-associated (e.g., swelling soils) may not be affected by the reliability adjustment to thickness. Furthermore, increasing the pavement thickness to design for a specific reliability level may actually increase the rate of progression of some load-associated distresses, such as rutting in asphalt concrete layers.

6.6 Selection of Reliability Level

The selection of a reliability level for design of a pavement is dictated by the expected use of the pavement (i.e., its functional class) which affects the consequences of under-designing the pavement. Under-designing has more severe consequences for a pavement which is expected to carry a high volume of traffic (e.g., an urban expressway) is greater than it does for a pavement which is expected to carry a lower volume of traffic. This is primarily because the consequence of under-design in both cases will be higher levels of distress and lower levels of serviceability, but the difficulty and cost of rehabilitating or maintaining the heavily trafficked pavement will be much greater than for the less trafficked pavement. Figure 2-7.18 illustrates how slab thickness increases for different levels of reliability.

As the level of reliability selected increases, the initial design of the pavement can be expected to become more substantial (specifically, the thickness increases). This is accompanied by an increase in the cost of the initial design. However, a more substantial initial design, on the average,

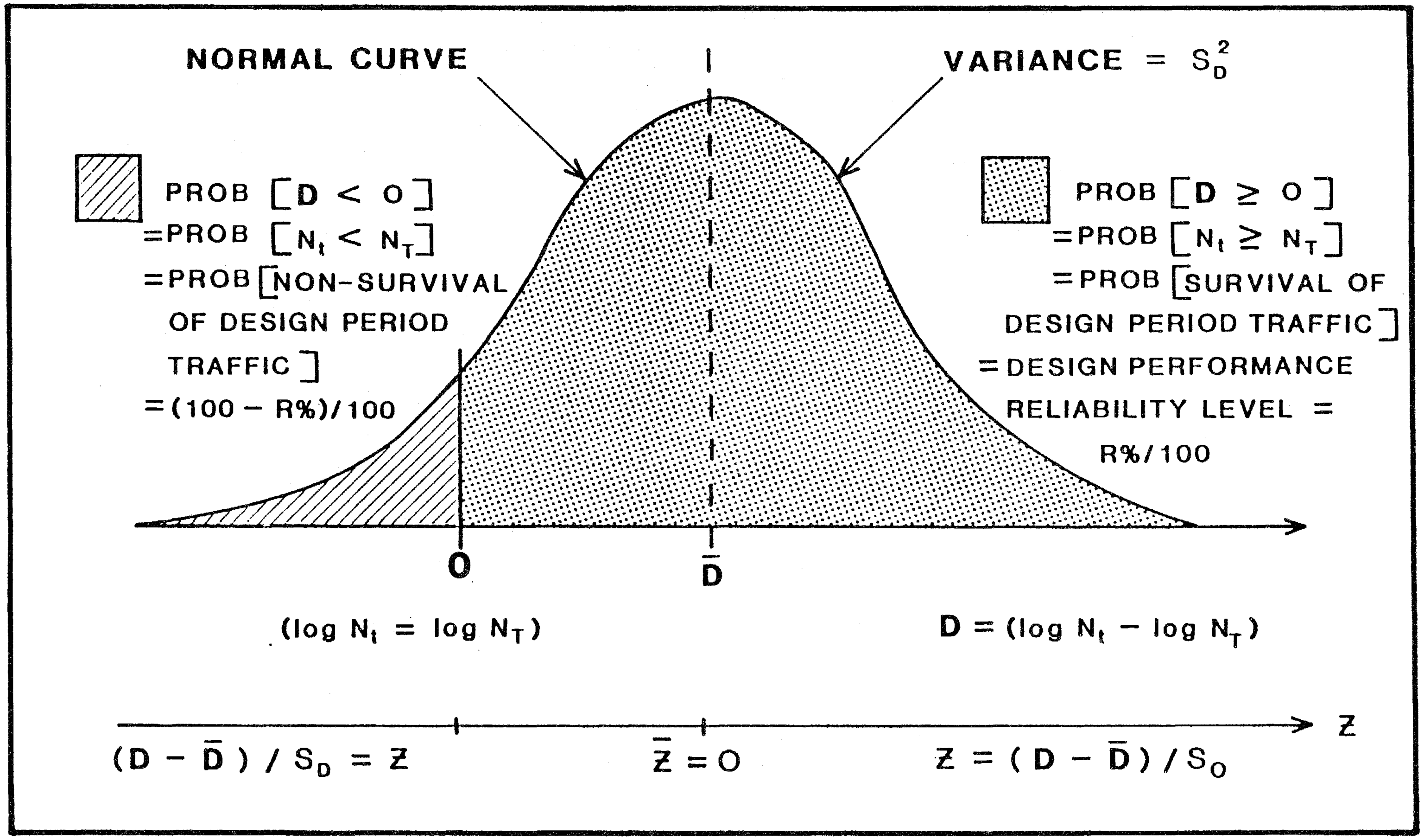


Figure 2-7.17. Illustration of Reliability on the Normal Distribution Curve.

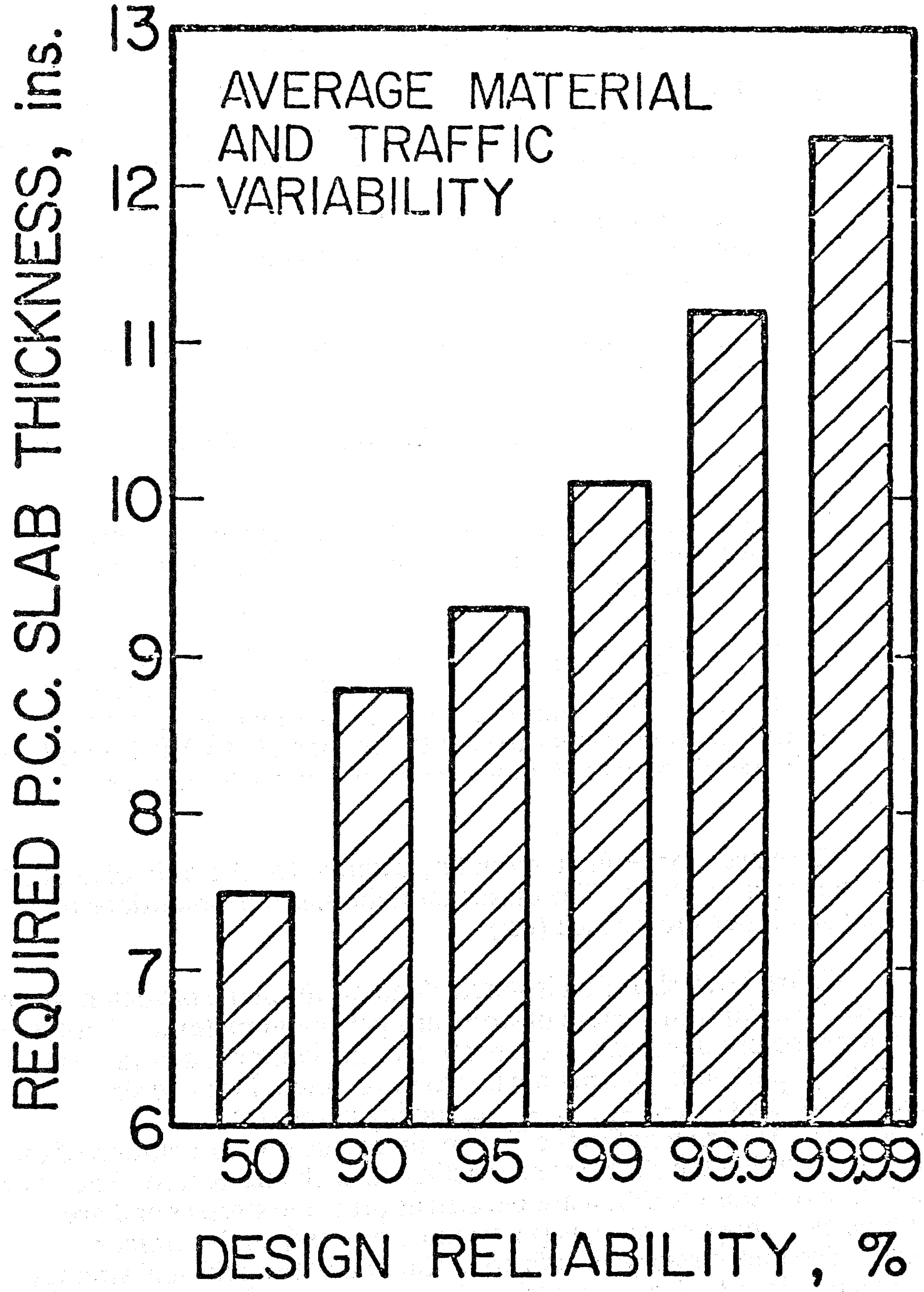


Figure 2-7.18. Design Reliability (for Traffic Loadings) Versus Required P.C.C. Slab Thickness for A Given Subbase and Subgrade (7).

will have a longer performance period; that is, for the same analysis period, it will have lower maintenance and rehabilitation costs. It follows that there exists some optimum level of reliability which will result in the lowest total cost over the analysis period. This concept is illustrated in Figure 2-7.19.

The 1986 AASHTO Design Guide (2) recommends design reliability levels for various functional classifications. These values were developed from a survey of state design procedures, in which the level of reliability inherent in each procedure was determined by functional class and location (i.e., rural or urban). Keep in mind that these values are generally applicable to all pavements of a particular functional class and location. The actual value of optimum reliability will depend on the variances associated with the individual design parameter for the specific pavement being designed.

6.7 Example REliability Design Calculations For Pavements

This section will illustrate the computation of design reliability considering 18-kip ESAL concept for pavements.

Example 1. For a particular pavement design, suppose:

1. $\log N_t = 7.1$ (i.e., predicted pavement ESAL = 12.6 million using the AASHTO performance equation)

2. $s_{\log(Nt)} = 0.4$

3. $\log N_T = 6.5$ (i.e., predicted traffic ESAL = 3.2 million)

4. $s_{\log(N_T)} = 0.2$

Then $Z_0 = -[7.1-6.5]/[(0.4)^2 + (0.2)^2]^{0.5}$

From a normal distribution table (Figure 2-7.4), the area from -1.342 to ∞ is 0.91. Therefore, the reliability level is 91 percent. Even though the estimated traffic is 3.2 million ESALs, the pavement is being designed for 12.6 million ESALs (or 3.98 times more) to provide for 91 percent reliability.

The procedure described above can be reversed to design for any selected reliability level.

Example 2. Suppose a reliability level of 95 percent is selected for the pavement in the previous example.

1. R = 95 percent

2. $Z_0 = -1.645$ (from normal distribution table)

$-[(\mathrm{Log}N_T)-6.5]/[(0.4)^2 + (0.2)^2]^{0.5}$

$\log N_t = 7.23$ (predicted performance = 17 million ESALs)

The higher value determined for $\log N_t$ in this example represents the increase that must be made in the design in order to provide the higher reliability level specified for the same actual design period traffic.

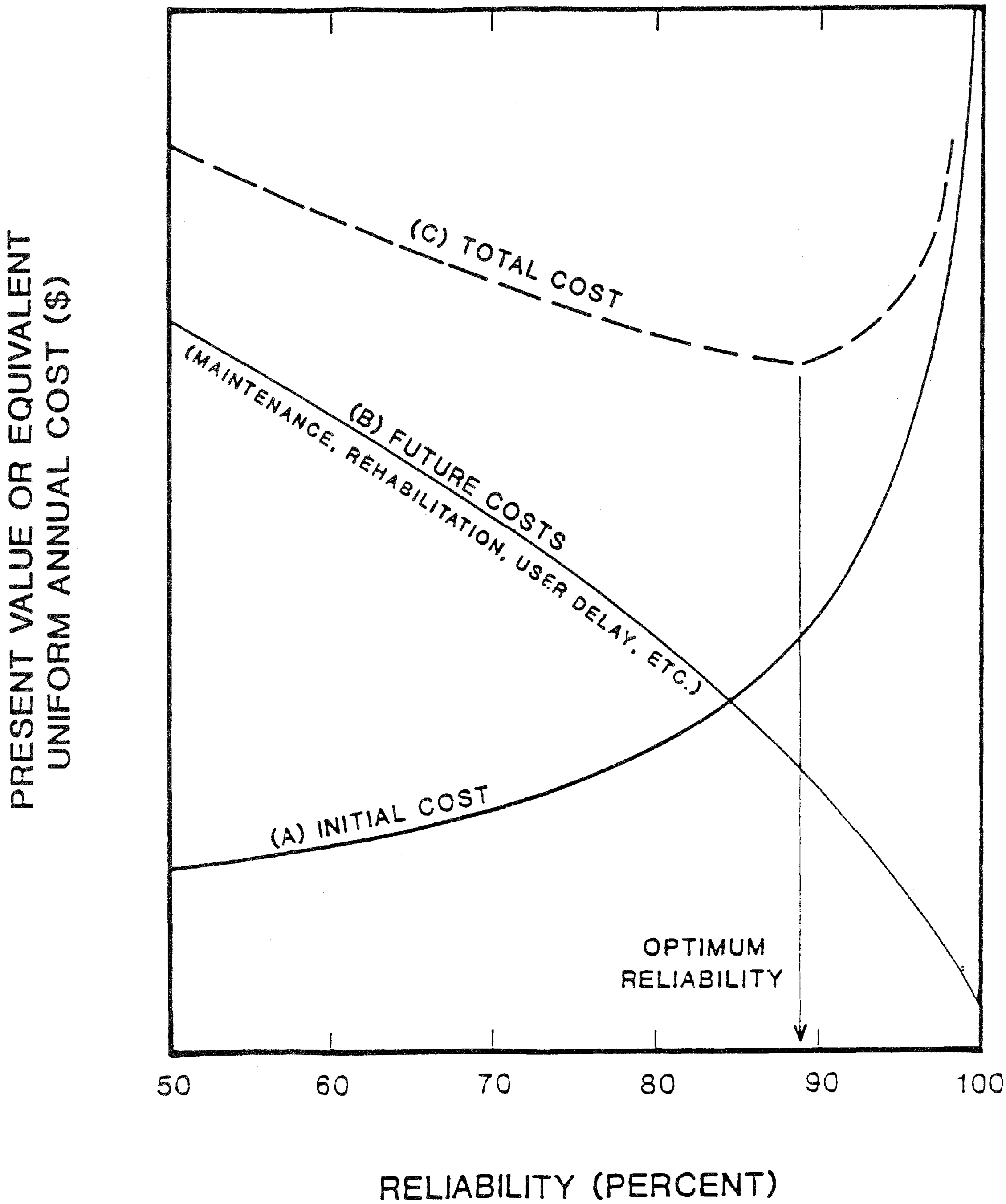


Figure 2-7.19. Optimum Reliability Corresponds to Minimum Total Cost (8).

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BLOCK 3

Flexible Pavement Design

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BLOCK 3 - FLEXIBLE PAVEMENT DESIGN

Modules

3-1 BASIC PRINCIPLES OF FLEXIBLE PAVEMENT DESIGN
3-2 AASHTO PROCEDURE FOR FLEXIBLE PAVEMENT DESIGN
3-3 OTHER PROCEDURES FOR FLEXIBLE PAVEMENT DESIGN
3-4 FLEXIBLE PAVEMENT SHOULDER DESIGN

This block introduces empirical and mechanistic design approaches and demonstrates how the inputs and other factors are considered in each design method. Steps for developing new design procedures are also described.

This block of instruction will show how each design procedure was developed, discuss its strengths and limitations, and will demonstrate the sensitivity of the resulting designs to variations in the inputs. In-depth familiarity with the AASHTO design procedure will be accomplished through the use of the AASHTO design computer program on microcomputers for "hands on" solution of example problems. The mechanistic approach will be discussed, and computer based programs will be used by the participants to calculate stresses and strains in actual pavement sections to develop a feeling of pavement responses in the design process.

Participants will be introduced to flexible pavement shoulder design considerations and concepts to round out the complete flexible pavement thickness design process.

Upon completion of this block the participant will be able to complete the instructional objectives listed for each module.

Module 3-1

BASIC PRINCIPLES OF FLEXIBLE PAVEMENT DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module introduces the participants to empirical and mechanistic-empirical design concepts as they apply to flexible pavement thickness design. Upon completion of this module the participants will be able to:

1. Differentiate between empirical and mechanistic-empirical design concepts.

2. Understand the steps required to develop empirical and mechanistic-empirical procedures for flexible pavement thickness design.

3. Describe the assumptions, advantages and limitations of each of the above design approaches.

4. List several common design procedures and identify their underlying design concepts (i.e., empirical or mechanistic empirical).

5. List the typical locations of flexible pavement critical stresses, describe methods of computing the magnitude of these stresses, and describe the distresses that they may cause.

6. Describe the effects of varying critical flexible pavement design inputs on pavement distress and performance.

2.0 INTRODUCTION TO THE DESIGN OF FLEXIBLE PAVEMENTS

Flexible pavement structural design is the determination of the thickness and vertical position of paving material elements which can best be combined to provide a serviceable roadway for predicted traffic over the selected pavement design life. These elements include the various subbase and base courses as well as the pavement surface and a suitable recognition of the roadbed soils. Each layer of the pavement structure can be designed and located to take advantage of the particular properties of that material. The goal is to use the most economical arrangement and minimum thickness of each material necessary to protect the underlying courses and the roadbed from distresses caused by imposed traffic loads.

The principle factors considered in flexible pavement structural design are the climate, traffic loads (magnitude and volume), roadbed soil characteristics, thickness and material properties of the surfacing and paving courses, and the required design reliability of the final structure.

Even though most of the critical pavement design factors have been recognized by pavement engineers for many years, various engineering agencies have adopted design procedures which have been developed or adapted to address local conditions based on experience and observed performance. Thus, it is not uncommon to obtain different design thicknesses from different design methods for identical input factors with different local procedures. Some of the differences between various design procedures can be attributed to the lack of a precise and quantitative description of what constitutes pavement failure. Others arise from the fact that differences were present in test methods, evaluation of environmental effects, and ways of handling traffic developed for each design procedure.

Two basic approaches are being actively developed to determine the required layer thicknesses for asphalt concrete pavement structures:

1. Empirical procedures.

2. Mechanistic-empirical procedures (which are often simply referred to as "mechanistic" or "rational" procedures).

2.1 Introduction to Empirical Design Approaches

Empirical procedures are derived from experience or observation alone, often without due regard for system behavior or pavement theory. Empirically derived relationships between performance, load, and pavement thickness for a given geographic location and climatic condition are the basis for many currently-used design methods. These models are generally used to determine required pavement thickness, the number of load applications to failure, or the occurrence of distresses as a function of pavement materials properties, subgrade type, climate, traffic, etc.

One advantage in using empirical models is that they tend to be simple and easy to use. Unfortunately they are usually only accurate for the exact conditions under which they were developed. They may actually be invalid outside of the range of variables used in the development.

The AASHTO, U. S. Army Corps of Engineers, Louisiana, and Utah designs are among a large family of pavement design techniques which were developed primarily on the basis of observed field performance.

2.2 Introduction to Mechanistic-Empirical Design Approaches

Pavement performance research in the United States is currently directed toward the mechanistic-empirical approach. Mechanistic-empirical design procedures utilize calculated stresses, strains, and deflections enhanced with distress or performance prediction models. The determination of pavement response (stresses, strains and deflections) to loads is accomplished using the techniques described later in this module. The distress and performance models are typically empirical relationships between distresses (such as rut depth and fatigue cracking) and design inputs such as material properties, stress repetitions, climate, etc. These models are developed from laboratory data and/or the observed performance of in-service pavements and can be used to estimate the maximum number of repetitions of a given level of stress, strain or deflection a pavement can withstand before reaching an unacceptable state of serviceability.

When the pavement response values have been determined at critical locations in the proposed pavement they can be compared with the maximum allowable values obtained from the distress and serviceability models. The pavement can then be designed iteratively by adjusting the different layer thicknesses and/or properties to obtain calculated stresses, strains, and deflections that are acceptable and that do not allow distresses to develop to an unacceptable level.

Mechanistic design offers the only direct analytical consideration of the numerous variables that influence pavement performance in a design procedure. The mechanistic approach has the potential to consider even very complex design factors such as the stress dependency of both the roadbed and base course, the time and temperature dependency of the asphaltic layers, and the interface conditions between layer components. While simplifying assumptions must be made in certain applications, the use of mechanistic theories offer the possibility of developing design procedures that are capable of addressing a wide range of climates, load conditions, pavement structures and material types.

Another important advantage to this design philosophy is the ability to analyze a pavement for several different failure modes such as cracking due to repeated loads or permanent deformation due to repetitive shear and compression stresses. The engineer can then adjust the design accordingly to produce a cost-effective pavement section that should not fail prematurely.

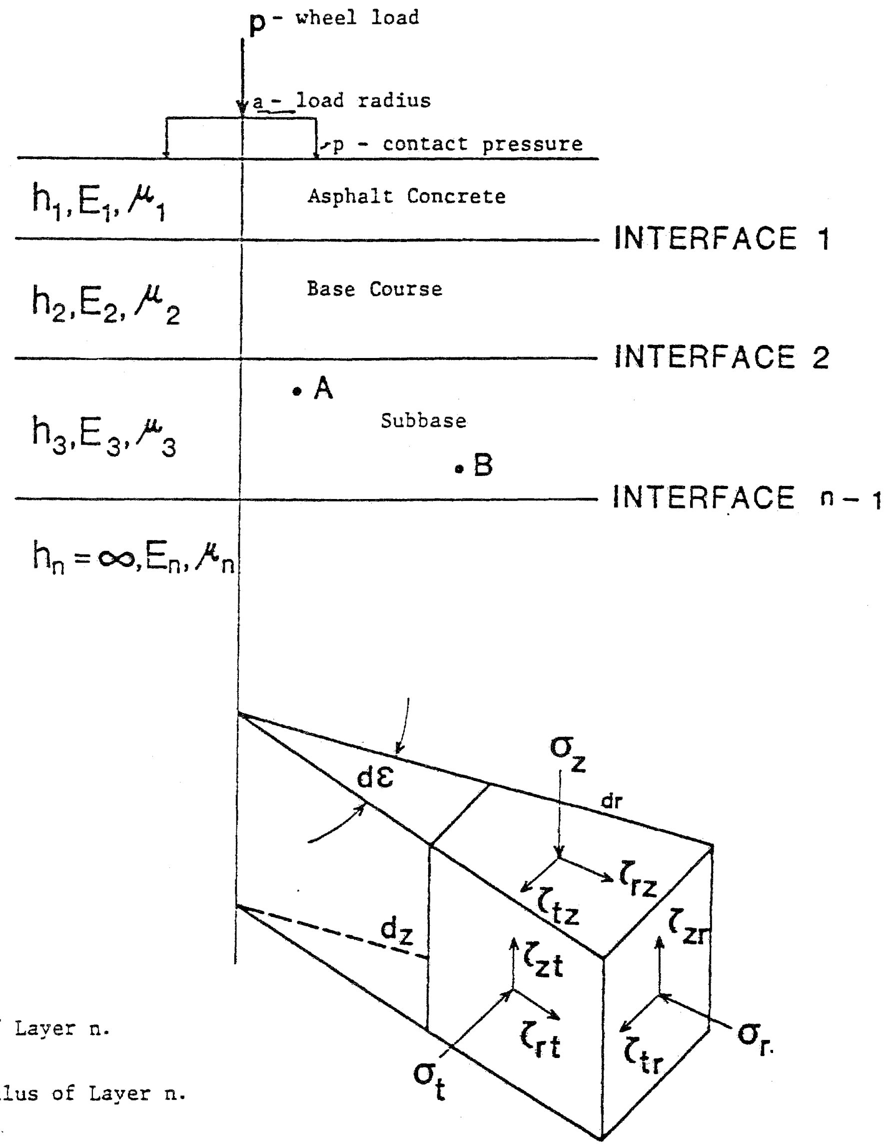
One disadvantage is that this approach typically requires more comprehensive and sophisticated data inputs than empirical design techniques. Extensive lab and field testing may be required to determine input parameters such as resilient modulus, creep compliance, dynamic modulus, Poisson's ratio, fracture toughness, etc. Another area of concern is that some of the simplifying assumptions that are commonly used in mechanistic approaches (linearly elastic materials and the validity of superimposing climatic and load related pavement responses) are not valid in all cases. In general, however, these assumptions do not produce large errors in the final thickness selection for the pavement design.

Current pavement and overlay design procedures that are based in part on mechanistic-empirical concepts include the Asphalt Institute, VESYS, BISAR, Shell, OAF and OAR procedures, PSAD, and, PDMAP.

3.0 STRESSES IN FLEXIBLE PAVEMENTS

When a wheel load is applied to a pavement structure, a stress is produced in each material. This stress also produces a corresponding strain in each material. These parameters are usually measured by recording the vertical deflection of the pavement under the load. The stresses, strains, and deflections are the pavement responses to the applied load. Stress is a force, load per unit area, (in pounds per square inch) and strain is the change in dimension per unit dimension (inches per inch). Pavement stresses, strains, and deflections are caused by traffic loading, daily or seasonal temperature and moisture changes, and by any changes in the conditions of pavement support. The magnitude of these responses determines the type and extent, and rapidity with which deterioration develops in the pavement system. The pavement structure and layer arrangement, as shown in Figure 3-1.1, have a dramatic influence on the stresses and strains. This figure will be explained further in this module.

The theory of elasticity is the fundamental procedure that has been used to calculate stresses in a continuous media subjected to a load. the



Where:

h_n = Thickness of Layer n.

E_n = Elastic Modulus of Layer n.

μ_n = Poisson's Ratio of Layer n.

σ_i = Stress Component in the i^{th} Direction.

τ_{ij} = Shear Stress Component in the i^{th} Direction in the Plane ij.

Figure 3-1.1 Generalized Multilayer Elastic System.

simplest configuration is a layer of one material. This situation was solved by Boussinesq, and is the starting point for elasticity analysis of layered structures.

3.1 Boussinesq Theory

A one-layer analysis can be used to calculate approximate stresses in actual multi-layered pavement structures, provided that the following requirements are met:

1. The pavement surface is less than 4 inches thick.

2. The ratio of pavement modulus (Ep) to that of the subgrade (Es) is less than or equal to one ($Ep/Es \leq 1$). Flexible pavements with a thick base and a very thin surface course meet this requirement.

3. The load radius (a) is relatively large as compared to pavement thickness (h) i.e., a/h is very large.

4. The pavement primarily consists of one layer such as the subgrade.

When a pavement structure meets the above requirements, Boussinesq showed that the vertical stress (σz) at any depth under a point load (P) can be calculated.

Since the tire imprint for a flexible pavement is considerably larger than a point load for which Boussinesq derived his equation, the load is assumed to be uniformly distributed on a circular tire imprint. Work by Newmark and others integrated the Boussinesq solutions to allow for circular loads through use of influence charts and tables. The variation of stress with depth for a circular load follows the same pattern as for the point load with only a difference in magnitude. Table 3-1.1 shows the elastic layer equations for calculating stress, strain, and deflection at any point in the one-layer flexible pavement system. Reference 1 provides the functions A through H in tabular form.

The significant conclusions of Boussinesq theory are:

1. Vertical stress decreases with depth and radial distance.

2. Maximum stress occurs directly under the load.

3. Stress is independent of material properties.

4. On any horizontal plane at depth z, the stress distribution is bell-shaped. The stress decreases with the distance away from the center of load.

3.2 Multi-Layer Theory

A flexible pavement is more realistically represented by three or more layers as shown in Figure 3-1.1. Calculating stresses and strains for a multi-layered system is a more involved procedure with more complicated assumptions and has not been practical before the introduction of the computer.

Table 3-1.1. Summary of One-Layer Elastic Equations. (Reference 1)

PARAMETER	GENERAL CASE	SPECIAL CASE ($\mu = 0.5$)
VERTICAL STRESS	$\sigma_z = p[A + B]$	(same)
RADIAL HORIZONTAL STRESS	$\sigma_r = p[2\mu A + C + (1 - 2\mu)F]$	$\sigma_r = p[A + C]$
TANGENTIAL HORIZONTAL STRESS	$\sigma_t = p[2\mu A - D + (1 - 2\mu)E]$	$\sigma_t = p[A - D]$
VERTICAL RADIAL SHEAR STRESS	$\tau_{rz} = \tau_{zr} = pG$	(same)
VERTICAL STRAIN	$\epsilon_z = \frac{p(1 + \mu)}{E_1}[(1 - 2\mu)A + B]$	$\epsilon_z \quad \frac{1.5p}{E_1} B$
RADIAL HORIZONTAL STRAIN	$\epsilon_r = \frac{p(1 + \mu)}{E_1}[(1 - 2\mu)F + C]$	$\epsilon_r = \frac{1.5p}{E_1} C$
TANGENTIAL HORIZONTAL STRAIN	$\epsilon_t = \frac{p(1 + \mu)}{E_1}[(1 - 2\mu)E - D)]$	$\epsilon_t = -\frac{1.5p}{E_1} D$
VERTICAL DEFLECTION	$\Delta_z = \frac{p(1 + \mu)a}{E_1}\left[\frac{z}{a}A + (1 - \mu)H\right]$	$\Delta_z = \frac{1.5pa}{E_1}\left(\frac{z}{a}A + \frac{H}{2}\right)$
BULK STRESS	$\theta = \sigma_z + \sigma_r + \sigma_t$	
BULK STRAIN	$\epsilon_\theta = \epsilon_z + \epsilon_r + \epsilon_t$	
VERTICAL TANGENTIAL SHEAR STRESS	$\tau_{zt} = \tau_{tz} = 0 \therefore [\sigma_t(\epsilon_t)$ is principal stress (strain)]	
PRINCIPAL STRESSES	$\sigma_{1,2,3} = \frac{(\sigma_z + \sigma_r) \pm \sqrt{(\sigma_z - \sigma_r)^2 + (2\tau_{rz})^2}}{2}$	
MAXIMUM SHEAR STRESS	$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2}$	

To use multilayer elastic theory, several assumptions must be made. They are:

1. Material properties of each layer are homogenous; i.e., the property of Point A is the same as Point B (see Figure 3-1.1).

2. Each layer has a finite thickness, except for the lower layer which extends to infinity. All layers are infinite in the lateral direction (no joints or cracks in the vicinity of the load).

3. Each layer is isotropic; i.e., the properties at a specific point such as A are the same in every direction or orientation (see Figure 3-1.1).

4. Full friction is developed between all layers at each layer interface.

5. Surface shearing forces (frictional forces) are not present at the surface.

6. The stress solutions are characterized by two material properties for each layer. They are Poisson's ratio (μ) and the elastic modulus (E).

Poisson's ratio is defined as the ratio of lateral strain to axial strain and is dimensionless. Poisson's ratio for compressible materials such as rubber, asphalt, or some cohesive soil is approximately equal to 0.5. For other material, the ratio ranges from 0.15 for concrete to 0.45 for asphaltic concrete. Poisson's ratios for common paving materials are presented in Table 3-1.2.

The elastic, or resilient modulus, is defined as the ratio of applied axial stress to axial strain. The elastic modulus has the dimensions of stress, pounds per square inch (psi).

Figure 3-1.1 shows an isolated element within the pavement structure to show the complicated state of stress in a pavement material. A total of nine stress components exist. These stresses are comprised of three normal stresses ($\sigma_z, \sigma_r, \sigma_t$) acting perpendicular to the element face and six shearing stresses ($\tau_{rt}, \tau_{tr}, \tau_{zr}, \tau_{rz}, \tau_{tz}, \tau_{zt}$) acting parallel to the face. At each point in the system, there exists an orientation of the element such that the shear stresses acting on each face is zero. The normal stresses under this condition are defined as principle stresses and are denoted as σ_1 (major stress), σ_2 (intermediate stress), and σ_3 (minor stress). The bulk stress (θ) is defined as the sum of the principal stresses at a given point (2). Given this triaxial stress state, strains are computed as follows:

$$\epsilon_z = (1/E)[\sigma_z - (\sigma_r + \sigma_t)]$$

$$\epsilon_r = (1/E)[\sigma_r - (\sigma_t + \sigma_z)]$$

$$\epsilon_t = (1/E)[\sigma_t - (\sigma_r + \sigma_z)]$$

Table 3-1.2. POISSON'S RATIOS FOR COMMOM PAVING MATERIALS

Material	Poisson's Ratio
Concrete	0.15
Asphaltic concrete	0.40
Granular base	0.37
Cement-treated base	0.20
Lime-stabilized soil	0.35
Subgrade	0.45

There are major limitations to the application of elastic layer theory:

1. Linear elastic theory assumes that all materials respond linearly over any stress range (see Figure 3-1.2(a)). Paving materials are "stress dependent," meaning, their response is a function of their stress state. Paving materials only respond linearly at very low stress states, stress states much lower than experienced by highway trafficking.

2. Linear elastic theory assumes that the material response is nonviscous (Figure 3-1.2(b)). Asphalt concrete is example of a complex viscoelastic material. It is time and stress dependent, and its properties can only be approximated using linear elastic theory.

3. Linear elastic theory assumes that all deformation is recoverable (see Figure 3-1.2(c)). In reality, paving materials require a great deal of time to fully recover strain. Most deformation can be considered to contain some plastic deformation. Generally, linear viscoelastic theory is used in predicting permanent deformation in pavement systems.

Fortunately, the stresses and strains calculated using elastic theory are reasonable, even with the limitations presented above.

3.3 Stresses and Strains

As mentioned, each loading whether load or environmental related will produce a pavement response, a stress or strain. It is these values that produce the deterioration in the pavement as seen in the distress appearing on the surface. The flexible pavement does not require extra appurtenance such as dowels or reinforcing steel which produce stresses and interact with stresses. The flexible pavement is analyzed as a continuum, and the stresses or strains are those produced by the load or the environment.

3.3.1 Vertical Stresses

The wheel load produces a vertical stress under the wheel as shown in the Boussinesq analysis. The vertical stress is responsible for compressing the pavement materials (rutting). Each layer in the pavement will have a different capability to resist this vertical compression.

3.3.2 Shear Stresses

The shear stress is produced by the load as it approaches a point in the pavement. The shear stress is most critical in the base course, and design procedures have been developed which minimize the shear in the base. Shearing stresses in the base produce unstable movements in the base as are commonly seen on low volume roads which are overloaded.

3.3.3 Radial Stresses

Under the wheel load, the pavement materials are deformed. This deformation is similar to the bending of a beam, and this typically produces a radial tensile stress at the bottom of the layers. The asphalt concrete

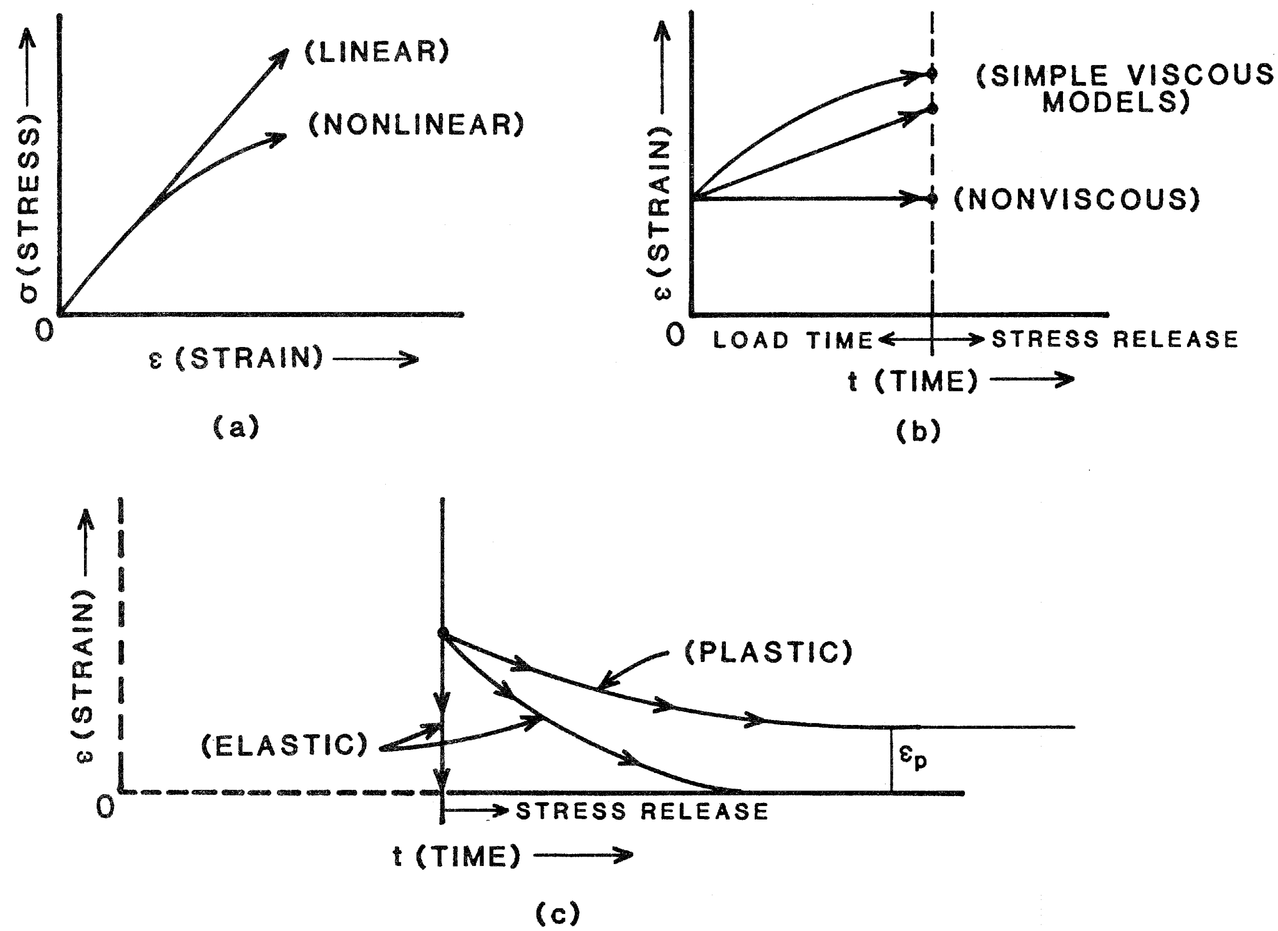


Figure 3-1.2. Material Characteristics (a) Linear Effects: (b) Viscous Effects; (c) Recoverable Effects (1).

surface layer develops considerable tensile stresses which lead to fatigue of the layer. Stabilized base layers also develop considerable tensile stresses which contribute to the cracking of stabilized layers.

3.4 Critical Stresses and Strains

The critical stress or strain is the maximum value of that stress or strain that occurs in the pavement system under the given loading conditions. Depending on the material combinations, the critical levels will occur in different materials for different pavements. Each material used in the pavement system has a specific tensile strength, compressive strength, or shear strength. When the strength of a material is exceeded, through repeated or single loading conditions, the material will fail. It is crucial to know precisely where stresses or strains will be at their maximum value, and what that maximum value will be.

The critical stresses for an asphalt surface over a granular base, a traditional flexible pavement, are shown in Figure 3-1.3. The critical tensile strain (Location 2) is located at the bottom of the asphalt concrete surface. With repeated traffic loading, the tensile stress at the bottom of the surface layer progressively damages the asphalt concrete until a crack begins. As the pavement is further trafficked, a fatigue crack propagates upward.

The vertical stress component at Location 1 is a compressive stress. With repeated traffic loading the compressive stress in the asphalt layer causes compaction of the surface layer which leads to rutting of asphaltic concrete.

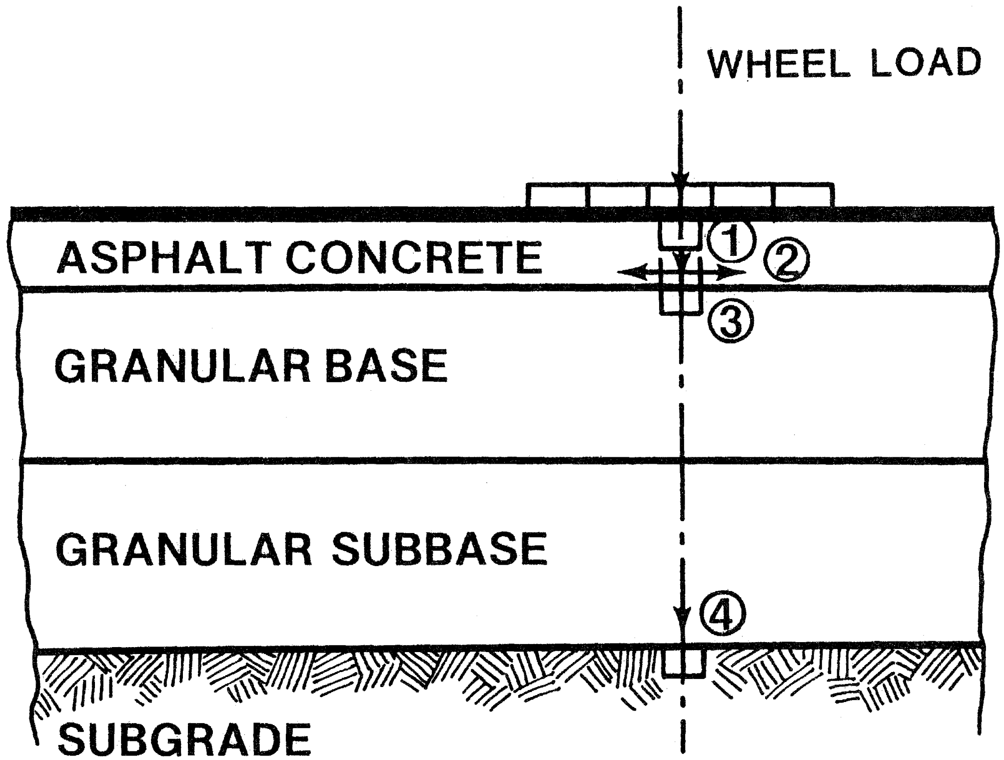
Unbound granular bases cannot resist tension (low cohesion in the Mohr failure envelop). Overstressing the base material can lead to pavement rutting and shoving as the base develops shear failure.

Pavement rutting results from repeated vertical stress on the top of subgrade (Location 4). Each application of a load produces some permanent deformation through consolidation of the roadbed material in every layer. The common distress termed rutting is typically most related to high vertical stress levels on the roadbed material.

Figure 3-1.4 shows the critical stress location for an asphalt surface over a stabilized base. The use of different materials in the pavement structure causes the critical radial strain to shift to the bottom of the stabilized layer. The remaining critical stresses are similar to the flexible pavement situation in Figure 3-1.3.

The magnitude of stresses and strains can be determined using graphical solutions or computer programs, although any calculation for more than one or two layers becomes very complicated. Many elastic layer computer programs have been developed to analyze flexible pavement systems, such as, ELSYM5, CHEVRON, BISAR, SDEL. Each program has its own unique attributes which allow different calculations to be performed:

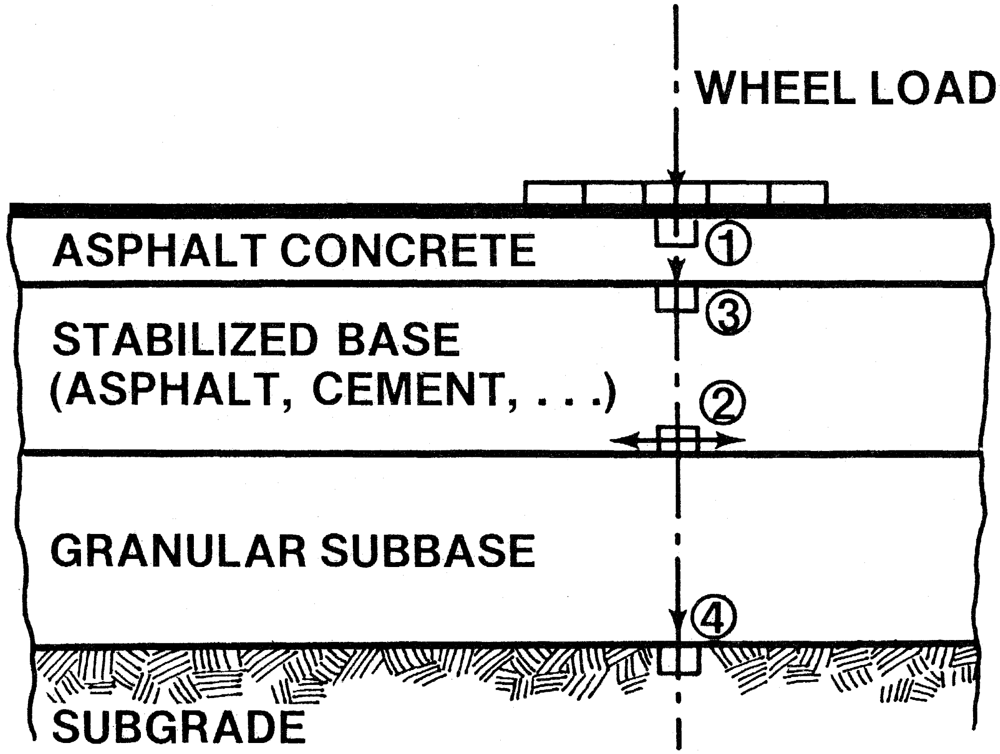
1. The CHEVRON elastic layer program is a strict application of elastic layer theory.



TYPICAL FLEXIBLE PAVEMENT WITH GRANULAR BASE

Figure 3-1.3. Typical Asphalt Pavement with a Granular Base Showinng the Critical Stress/Strain Locations.

1. Compressive Strain - Rutting.
2. Tensile Strain - Fatigue or Alligator Cracking.
3. Compressive Strain - Rutting.
4. Compressive Strain - Rutting, Depressions.



TYPICAL FLEXIBLE PAVEMENT WITH STABILIZED BASE

Figure 3-1.4. Typical Asphalt Pavement with a Stabilized Base Showing the Critical Stress/Strain Locations.

1. Compressive Strain - Rutting.
2. Tensile Strain - Transverse Reflective Cracking or Fatigue Cracking
3. Compressive Strain - Rutting.
4. Compressive Strain - Rutting, Depressions.

2. The BISAR program is based on linear elastic theory; however, an interface friction parameter may be specified (i.e. may have partial bonding between layers) and horizontal loading (i.e. braking of a vehicle) is possible.

3. SDEL is based on stress-dependent linear elastic theory (i.e. the response to a stress state by a material is a function of depth within the pavement layer).

4. ELSYM5 is an elastic layer program allowing multiple wheel loads and interface slip.

3.5 Finite Element Procedures

Finite element theory is a complex method of analyzing the elastic response of a pavement system. Each layer in the pavement is divided into many small "elements" as shown in Figure 3-1.5 The stress state in each element is calculated using theory of elasticity, assuming adjacent elements are dependent on each other, namely that they are connected at the nodes. Different formulations of finite element programs assume different levels of compatibility between the individual elements. Advanced applications can account for plasticity and non-linear responses between the elements at their nodes, and along the edges. Each element can take on different properties, depending on the storage capacity of the computer being used.

Several finite element programs have been developed to analyze flexible pavement systems. One of the more popular is ILLI-PAVE, a stress dependent finite element program developed at the University of Illinois. The finite element formulation allows the modulus of any particular element to be a function of its placement within the pavement layer (4).

Finite element programs are powerful tools that must be utilized with extreme caution. They require development of a finite element "mesh" with specific size requirements on element size for each loading condition and pavement cross-section. Figure 3-1.5 shows a finite element mesh developed for use with the ILLI-PAVE computer program. If the mesh is inaccurate, erroneous results will be produced. A thorough working knowledge of finite element concepts is recommended before using finite element programs.

3.6 Non-Load Related Stresses in Flexible Pavements

3.6.1 Shrinkage Cracking

Shrinkage cracking affects nearly all layers in a flexible pavement depending on the materials being used. Asphalt concrete undergoes continual volume changes under temperature cycling. Over time, the flexibility of the asphalt cement is reduced through oxidation under continued solar radiation, and the surface layer loses its ability to resist the stresses produced by each temperature cycle, and the pavement begins to crack. This pattern of cracking is typically called block cracking, and is not related to structural adequacy of the pavement. Over time, the continual volume changes will accumulate in the asphalt, and produce very wide cracking as the asphalt shrinks and grows brittle.

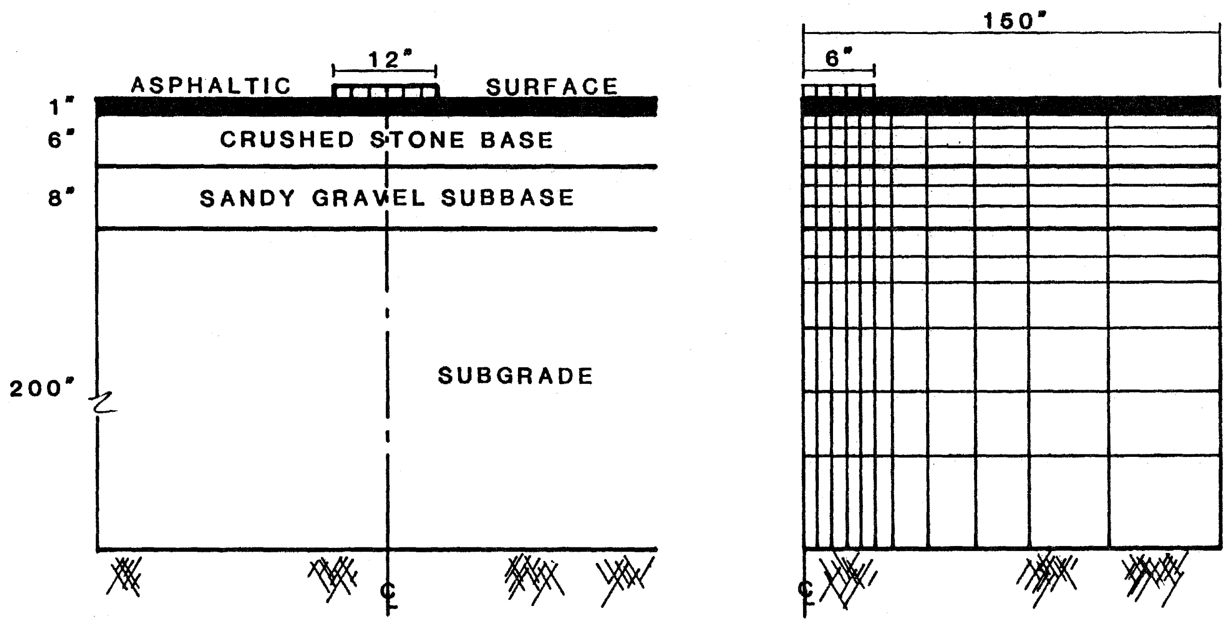


Figure 3-1.5. Example Finite Element Mesh.

Stabilized materials such as lime or cement stabilized materials undergo shrinkage in much the same manner as concrete as the cement material hydrates. When the shrinkage reaches a point, the material will crack under the shrinkage stress.

3.6.2 Temperature Induced Stresses

Thermal cracking of asphaltic concrete pavements has long been a problem for pavement designers in a variety of climatic areas. Thermal cracking is not only a problem in the colder climates such as Canada or the northern United States, but it can be found in a majority of the United States, when the proper combination of temperature and asphalt properties are present (3).

Low-temperature cracking results when tensile stresses develops in the asphalt concrete as the temperature drops to an extremely low value. This low value is a winter temperature and is generally represented by the lowest temperature occurring in one day. When these low values of temperature occur, the resulting tensile stress in the asphaltic concrete can become larger than the tensile strength of the mixture at that temperature. Should this occur, a tensile crack would form immediately.

Thermal fatigue cracking, however, develops under more moderate temperature extremes, typically under daily temperature cycles that may occur during any time of the year. Under daily temperature variations, the tensile stress will be highest during the night when the temperature drops from the warmer daytime level. Because the daily temperature cycling occurs at a level much higher than those present for low-temperature cracking, the stress is typically far below the tensile strength of the mixture at that temperature. Thus, failure does not occur immediately, but it develops over an extended time period similar to load-induced fatigue of the asphalt concrete; hence, the name thermal fatigue cracking.

Low temperature cracking is more prominent in the freeze areas (Northern United States and Canada). Thermal fatigue cracking can occur in any climatic zone which experiences large daily temperature fluctuations. It is an oversimplification to place the problem in one environmental region or another because of the complex relationship between climate and mixture properties that produce one form of cracking or the other. In areas of mild climate and moderately low temperatures during a winter period, the use of a hard grade asphalt cement may interact with these temperatures to produce low-temperature cracking similar to what occurs in the more northerly areas (3).

3.7 Sensitivity Analysis

A typical flexible pavement system shown in Figure 3-1.6 represents a cross-section very similar to that used in the AASHTO Road Test. This section was examined with the ELSYM5 elastic layer program to show the sensitivity of the pavement to input parameters. The results of the analysis are given in Table 3-1.3, from which the following conclusions can be drawn:

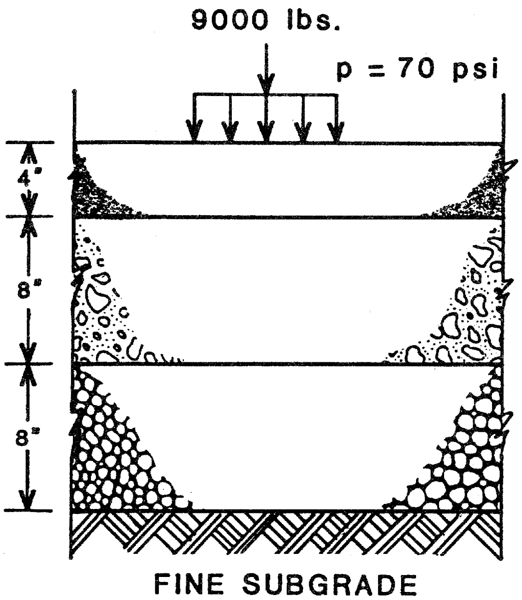
1. Tire pressure has a large effect on the stress and strain in the surface layer. As the tire pressure increases, the surface vertical stress, radial stress, and radial strain increase

Table 3-1.3. Sensitivity of Multi-Layer Elastic Theory of Various Input Parameters.

	STANDARD PAVEMENT	HIGH TIRE PRESSURE	STRONG SUBGRADE	$\mu = 0.5$	6" LIME STABILIZED	ASPHALT BASE	CEMENT BASE
Δ (TOP OF AC, in.)	0.034	0.036	0.021	0.034	0.029	0.023	0.017
σ_z (TOP OF AC, psi)	-70.0	-140.0	-70.0	-70.0	-70.0	-70.0	-70.0
σ_r (BOTTOM OF AC, psi)	200.0	291.0	191.0	270.0	188.0	-6.2	-38.4
ε_r (BOTTOM OF AC, $\varepsilon \times 10^6$)	296.0	429.0	284.0	294.0	280.0	22.0	-18.8
σ_z (TOP OF BASE, psi)	-26.5	-34.4	-27.3	-24.2	-27.4	-50.4	-58.2
σ_r (BOTTOM OF BASE, psi)	7.0	7.4	4.3	7.7	3.9	60.6	100.0
σ_z (TOP OF SUBBASE, psi)	-8.6	-9.3	-10.6	-8.4	-10.5	-3.7	-2.2
σ_r (BOTTOM OF SUBBASE, psi)	5.3	5.3	0.6	5.9	-1.0	2.2	1.1
σ_z (TOP OF SUBGRADE, psi)	-3.5	-3.5	-5.7	-3.5	-2.2	-1.7	-1.2
σ_z (36" SUBGRADE, psi)	-1.6	-1.6	-2.4	1.6	-1.4	-0.9	-0.7

NOTE: (+) TENSION
(−) COMPRESSION

STANDARD PAVEMENT



E1 = 500,000 psi
μ_1 = 0.30

E2 = 25,000 psi
μ_2 = 0.35

E3 = 15,000 psi
μ_3 = 0.35

E4 = 5000 psi
μ_4 = 0.45

SENSITIVITY TO:

TIRE PRESSURE = 140 psi

SUBGRADE STRENGTH -E4 = 15,000 psi

POISSON'S RATIO -μ = 0.5 FOR ALL LAYERS

LIME STABILIZE TOP 6" OF SUBGRADE - E4 = 50,000 psi

(SUBGRADE) -E5 = 5000 psi

ASPHALT STABILIZED BASE - E2 = 300,000 psi

CEMENT STABILIZED BASE - E2 = 1,000,000 psi

Figure 3-1.6. Standard Pavement for Sensitivity Analysis.

substantially. Because of the distance from the load, tire pressure has a smaller and smaller effect in the lower layers. This points up the major problem of increasing tire pressures, namely, it is the surface layer which will be overstressed.

2. The stiffness of the roadbed soil has a substantial effect on surface deflection and radial strain in the subbase. Since approximately 70% of the deflection results from the subgrade, increasing the subgrade stiffness substantially decreases the deflection. The radial stress in the subbase decreases with increasing subgrade strength due primarily to the increase in stiffness of the subgrade layer.

3. Poisson's Ratio has little or no effect on any of the pavement responses.

4. Stabilizing the subgrade to a depth of 6" with lime affects the surface deflection and the radial stresses of the granular layers because the lime will stiffen the soil. The radial stresses are reduced due to the confining of the granular layer between two "rigid" layers.

5. Stabilizing the base course, with either cement or asphalt, reduces surface deflection, decreases radial stress and strain in the surface, increases radial stress and vertical stresses in the base course, and reduces subgrade vertical stress. The critical radial stress and strain is shifted to the bottom of the stabilized layer due to the added stiffness of the stabilized layer.

This shifting of critical stresses indicates the importance which must be given to the modular ratio of the layers in a pavement. Materials cannot be placed in the pavement structure in a random fashion and perform in the same manner as a pavement with the same materials arranged in a different order.

3.8 The ELSYM5 Elastic Layer Program

This solution of the elastic layer theory provides the means to determine stresses, strains, and deflections in a pavement system with up to 5 different materials. Multiple wheel loads can be placed on the surface to model multiple axle configurations. This program has the added feature of inducing layer slip which may be useful in modelling material deficiencies in existing pavements.

3.8.1 Input Data

ELSYM5 is a user friendly menu driven program. The main menu screen is shown in Figure 3-1.7. Individual input screens are provided, and each one is related specifically to the data required. Figure 3-1.8 shows the input selection screen 1.2 for assistance in selecting the various parameters required to perform an elastic layered analysis. Figure 3-1.9 contains input screen 1.2.2 for the elastic layer data, primarily thickness, Poisson's ration and modulus of elasticity for each layer. Screen 1.2.3 shown in Figure 3-1.10 contains data describing the placement location of the load.

-ELSYM5-
Interactive Input Processor
Version 1.0, Released 10/85
Developed by
SRA Technologies, Inc.

Under contract to
Federal Highway Administration

MAIN MENU

1. Instructions
2. Create a New Data File
3. Modify an Existing Data File
4. Perform Analysis
5. Exit - Return to DOS

Selection:

Figure 3-1.7. ELSYM5, Elastic Layer Analysis Program, Main Menu Screen.

Screen 1.2

Create a New Data File Menu

1. Enter/Modify Run Title

2. Enter/Modify Elastic Layer Data

3. Enter/Modify Load Data

4. Enter/Modify Evaluation Location Data

5. Write Data to an Output File

6. Return to Main Menu

Selection:

Figure 3-1.8. ELSYM5 Data File Creation Screen.

Screen 1.2.2

ELASTIC LAYER DATA

Number of layers: 3

Layer Number (top to bottom)	Thickness (inches)	Poisson's Ratio	Modulus of Elasticity
1	4.00	.35	400000.00
2	8.00	.40	35000.00
3	.00	.35	5000.00

Note: Enter Zero thickness when bottom layer is semi-infinite.

Figure 3-1.9. ELSYM5 Elastic Layer Data Screen.

LOAD DATA Screen 1.2.3

Enter two of the following, the third is calculated.

Load: 9000 lbs Pressure: .00 psi Load Radius: 6 inches

Number of load locations: 1

Location number =	Coordinates X =	Y =
1	0	0

CR: To Next Data Field; F2: Skip to End of Screen

Figure 3-1.10. ELSYM5 Load Data Input Screen.

Multiple loads can be placed for analysis. Screen 1.2.4 shown in Figure 3.1-11 contains the description for the location of the points where stress or strain calculations are desired. The location is specified by the depth below the surface, and the distance from the origin used to place the load on the surface. At each step of the data entry, the user is prompted for entry of data until all data has been entered and the user is placed at the main menu and the analysis is performed.

3.8.2 Output Data

The stresses, strains, and deflections are calculated at each location specified in the input screen. Each layer is desired parameter is displayed. The selection menu is shown in Figure 3-1.12. This figure also contains the output for the displacements on the surface of the pavement structure shown in the preceding input screens. Under the wheel load (0,0) the deflection in the x and y directions are zero, while the vertical deflection (UZ) is .0383 inches. Figure 3-1.13 contains the selection screen for layer 2, and the output for stresses at a depth of 3 inches, which is the bottom of the asphalt concrete layer. Of importance to the pavement designer are the normal stresses in the X, Y, and Z directions. Directly under the wheel load, the SXX and SYY stresses are the radial tensile stresses which develop fatigue in the asphalt concrete layer. The sign convention in this program has compressive stresses with negative signs and tension stresses with positive signs. The strains could be printed in the same manner.

3.9 Design Principles

Figure 3-1.14 and Figure 3-1.15 illustrate the sensitivity of pavement responses to the various input parameters for a two-layer elastic system. Each of these charts illustrates how changes in one parameter of the pavement structure will alter the response:

The principal objectives in designing a flexible pavement with these analytical programs are to minimize critical stresses or strains. These stress or strain values can be reduced through a judicious choice of material thicknesses and modulus values. The most efficient ways of reducing tensile stresses and strains in the upper layers are:

1. Reduce the modulus ratio between the upper layers (i.e., decrease E1/E2).

2. Increase the thickness ratios of the upper layers (i.e., increase h1/h2).

The most efficient way of reducing subgrade compressive stresses and strains is to reduce the ratio of load radius to base thickness (i.e decrease a/h2). Another method of reducing subgrade stresses is to increase the rigidity of the upper pavement layers, particularly the layer directly above the subgrade. The capabilities of the mechanistic approach will be highlighted by comparison with the empirical approach.

4.0 EMPIRICAL THICKNESS DESIGN CONCEPTS FOR FLEXIBLE PAVEMENTS

Empirical procedures are those that rely largely on engineering experience and judgment, mathematical performance or distress models based on

Screen 1.2.4

EVALUATION LOCATION DATA

Results are evaluated for all combinations of X-Y coordinates and Depths of Z.

Number of X-Y positions: 0

Number of Z position : 0

Position	X	Y	:	Position	Z
			:		
			:		
			:		
			:		
			:		
			:		
			:		
			:		
			:		
			:		

CR: To Next Data Field; F2: Skip to End of Screen

Figure 3-1.11. ELSYM5 Evaluation Location Data Screen.

```
                    Displacements

   XP       YP       UX          UY          UZ
  .00      .00   .000E+00    .000E+00    .383E-01

            RESULTS MENU FOR ELSYM5
            ---------------------------
            LAYER =  1        Z =       .00

1. -  Stresses   Normal & Shear & Principal
2. -  Strains    Normal & Shear & Principal
3. -  Displacements
4. -  Return or Continue with Next Layer

  Selection ==>
```

Figure 3-1.12. ELSYM5 Output Screen for Displacements on Surface Under Center on Load.

		Normal Stresses			Shear Stresses		
XP	YP	SXX	SYY	SZZ	SXY	SXZ	SYZ
.00	.00	.668E+02	.668E+02	-.425E+02	.000E+00	.000E+00	.000E+00

		Principal -- Stresses			Shear Stresses		
XP	YP	PS1	PS2	PS3	PSS1	PSS2	PSS3
.00	.00	.668E+02	.668E+02	-.425E+02	.547E+02	.000E+00	.547E+02

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = 3.00

1. - Stresses Normal & Shear & Principal
2. - Strains Normal & Shear & Principal
3. - Displacements
4. - Return or Continue with Next Layer

Selection ==>

Figure 3-1.13. ELSYM5 Output for Stresses at the Bottom of the Asphalt Concrete Surface Layer.

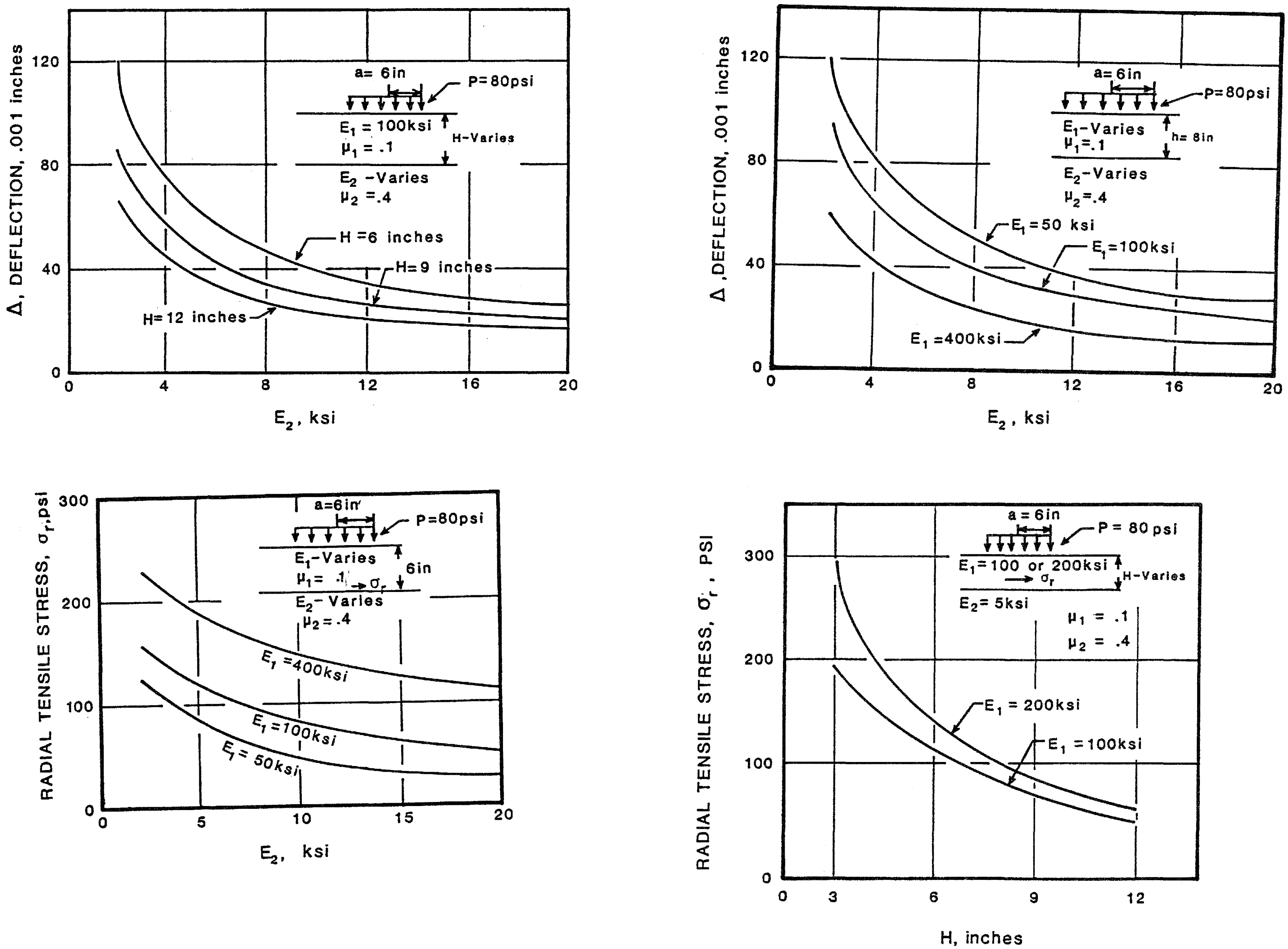


Figure 3-1.14. Sensitivity of Radial Tensile Stress and Vertical Deflection to Modulus and Thickness of the Base Layer.

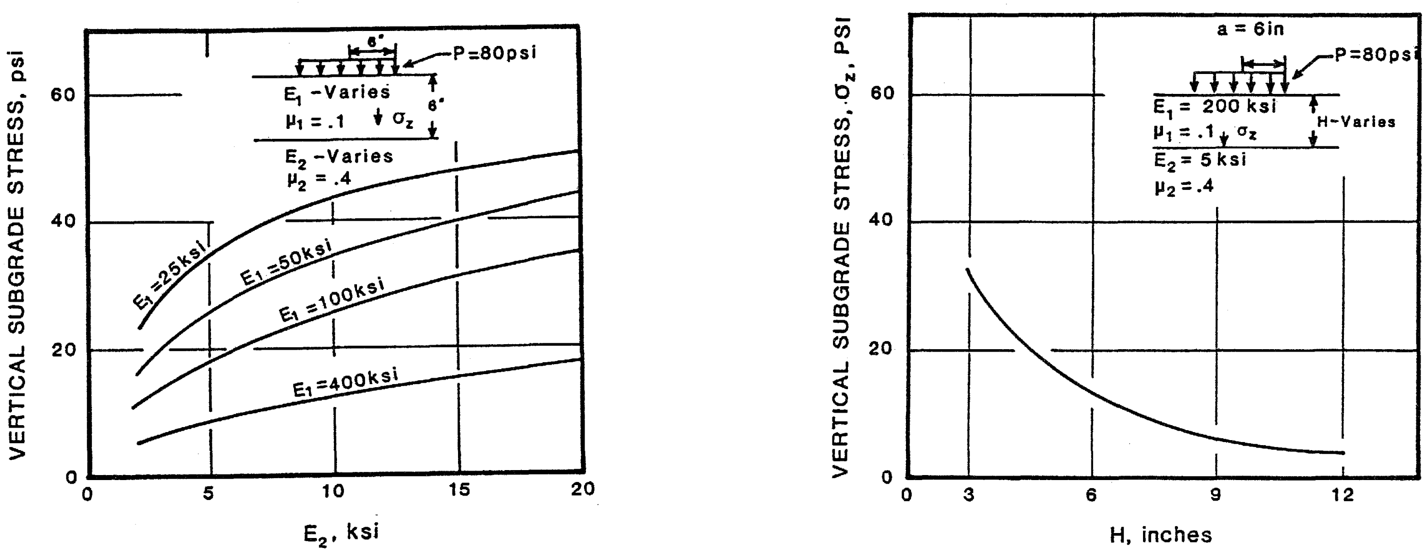


Figure 3-1.15. Sensitivity of Vertical Subgrade Stress to Modulus and Thickness of Base Course.

measurements of field performance, or some combination thereof, often without consideration of structural theory.

Performance models typically take the following form:

$$y = a + b*x1^{c} + d*x2^{e} + ...$$

where:

y = the predicted performance variable, such as rutting, cracking, serviceability, etc.

x1, x2, ... = independent design variables, such as traffic volume and composition, climate, material properties, layer thicknesses, etc.

a, b, c, ... = constants.

Examples of empirical models might include:

1. Estimation of predicted loss of serviceability for a given pavement design, traffic, and climatic conditions over a period of time.

2. Prediction of the rutting that will be found on a particular pavement given traffic volumes and compositions, pavement materials properties, subgrade type, climate, etc.

3. Predicting the number of 18-kip ESALs that a pavement can withstand before fatigue cracking reaches an unacceptable level.

Empirical models can be developed for <u>any</u> distress or performance parameter that can be measured provided there exists a correlation with other measurable or observable conditions or design inputs.

Many empirical procedures deal with the concept of loss of serviceability, or functional failure, rather than structural failure. This can also be considered a plus from the user's standpoint because designs that are based on structural integrity may provide unacceptable service conditions without failing structurally (e.g., faulting of rigid pavement joints). Designs based on roughness or serviceability should satisfy the users without sacrificing structural integrity.

Empirical models are subject to certain limitations. They are usually accurate only for the exact conditions and ranges of independent variables under which they were developed and may actually be invalid outside of these ranges. Thus, empirical procedures allow the user to confidentially design only pavement sections that are similar to the sections that were used to develop the design procedure. This applicability may represent a narrow range of conditions and designs and may hamper the consideration of many important design factors that could be considered using mechanistic procedures. The use of empirical design procedures in the development and analysis of <u>new and unique designs</u> can be a questionable proposition.

4.1 A Framework for the Development of Empirical Design Procedures

A general approach to the development and use of empirical design procedures is illustrated in Figure 3-1.16.

The first step in the development of an empirical pavement design procedure is to establish a base of experience upon which to draw conclusions regarding the effects of various design inputs on pavement performance. This experience may include the opinions of "experts" who can provide information concerning the design and performance history of both existing and inaccessible (distant or reconstructed) pavements, but should include measured and directly observed data as well. It is very important that the database include pavement sections that represent variations in all of the inputs that are desired in the design models (e.g., varying layer thicknesses, pavement support, traffic levels, climatic conditions, etc.). While the inclusion of sufficient cases to complete a replicated factorial design matrix is desirable to assure the development of statistically significant models it is practically impossible to achieve in practice. Experience has shown that very good models can be developed using an incomplete design matrix if the database includes a sufficient number of cases representing variations in the design input parameters.

After an appropriate database has been established, preliminary performance and distress models must be developed using statistical analysis procedures. One approach to developing these models is to determine which independent (design or input) variables appear to correlate well with the dependent (performance or distress) variable. This can be accomplished by many commercially available computer programs (10, 11) or by simply plotting data pairs and visually assessing the degree of correlation. Linear and nonlinear regression analysis techniques can then be used to develop models using only those variables or combinations of variables that appear to impact the dependent variable being modelled. References concerning modelling techniques are presented at the end of this module (12, 13).

The initial models must be analyzed and verified to determine their behavior within the valid ranges of the independent variables because it is possible to develop models that appear to explain much of the variability in the dependent variable (models with high values of R squared) but that make little sense practically (for example they may suggest higher serviceability or lower distress with increasing traffic). The models should also be able to use actual case study input data from the database to predict with reasonable accuracy the actual performance and distress levels found in the pavements that were used to develop the models. If the initial models are inaccurate or behave poorly they should be discarded and new models should be developed using different analysis techniques.

Acceptable models should be further analyzed to identify weaknesses or potential constraints on their use. Failure criteria must also be established. Sensitivity analyses can be conducted to determine the effects of each of the input parameters on the performance or distress being predicted. These analyses will assist the user in determining which design factors should be modified to produce a pavement with the desired performance characteristics.

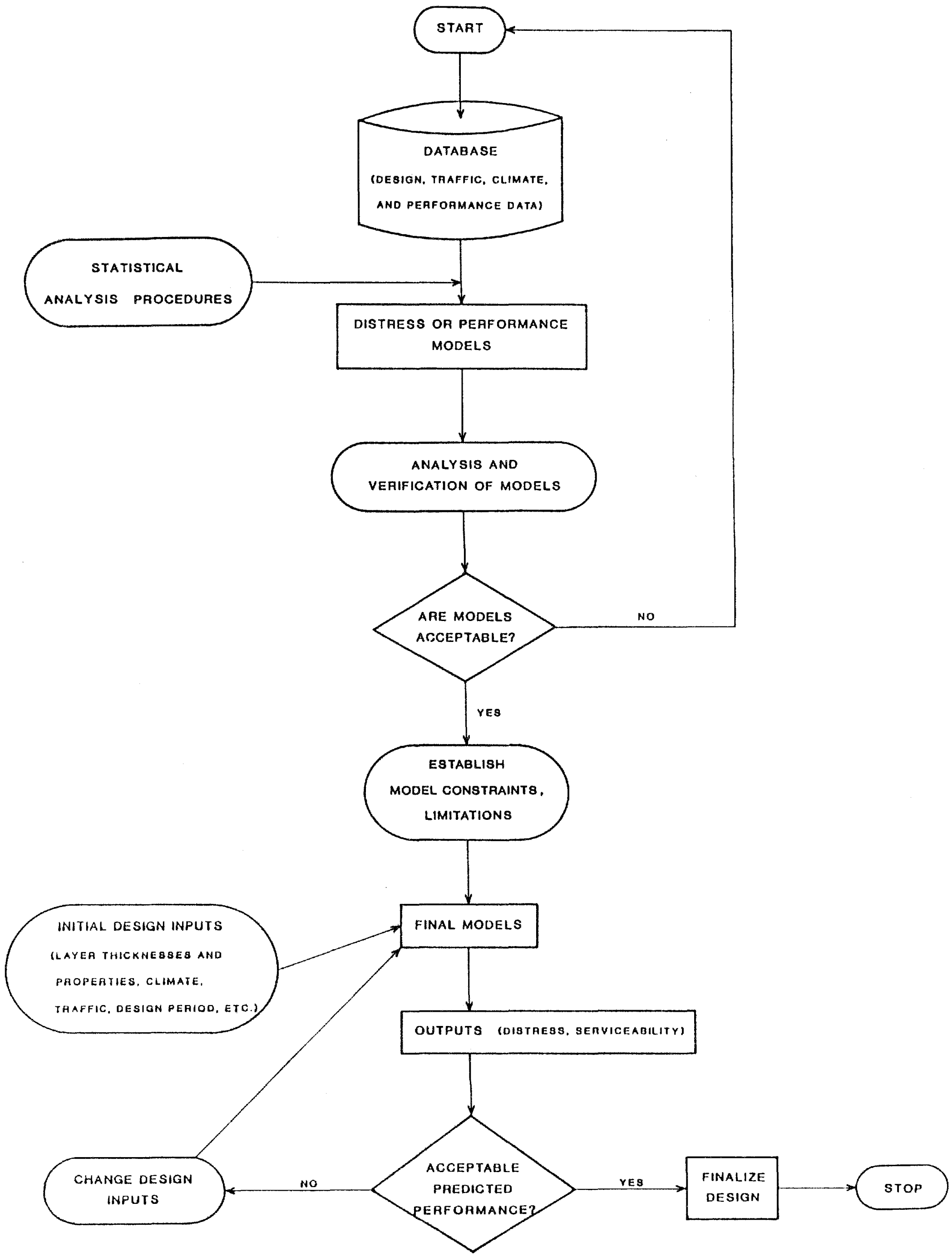


Figure 3-1.16. General Flow Diagram for the Development and Use of an Empirical Design Procedure.

The following section briefly describes the development of one of the most commonly-used empirical design procedures, the AASHTO Thickness Design Procedure for Asphalt Concrete Pavements.

4.2 AASHTO Thickness Design Procedure for Asphalt Concrete Pavements

The AASHTO flexible pavement design method is based on results obtained from the AASHO Road Test conducted in the late 1950's and early 1960's in northern Illinois (1). It is an empirical procedure that relates pavement performance measurements (loss of serviceability) directly to the traffic volume and loading characteristics, roadbed soil strength, pavement layer material characteristics, and environmental factors actually present during the Road Test. The design equations have been generalized to make them applicable to broader sets of input variables. The original flexible pavement design equations were issued in October, 1961. The 1986 AASHTO Guide for the Design of Pavement Structures follows the same basic design approach, although improvements to the flexible pavement design procedures have been included.

The AASHTO procedure is based on providing enough strength in the pavement layers to prevent overloading of the subgrade soil by the applied loads. The pavement performance is measured by a Present Serviceability Index, which is a function of the mean slope variance in the two wheel paths, the amount of cracking and patching in the pavement surface, and the depths of rutting in the wheel paths. The details of this procedure are described in Module 3-2.

About 500 flexible pavement test sections (160 feet in length) were constructed over a single subgrade soil and in a single climate. The thickness and composition of the pavement layers was varied within the ranges described below:

Pavement Layer	Material	Trial Thicknesses (in)
Surface	Asphalt Concrete	1, 2, 3, 4, 5, 6
Base	Crushed Stone	0, 3, 6, 9
Subbase	Gravel	0, 4, 8, 12, 16

A complete replicated factorial experimental design was used to provide information on the performance associated with all possible combinations of the above materials and thicknesses.

The sections were constructed in a series of loops and were subjected to 1.1 million single- and tandem-axle load applications that ranged from 2000 pounds to 48000 pounds, with each section being exposed only to axle loads of one particular size and configuration. In this way the effects of different loads could be assessed together with the variations in pavement structure. All load applications were completed over a two-year period.

Roughness and distress (cracking, rutting and patching) were measured periodically over the two-year period and were used to compute the present serviceability index (PSI is related to PSR) of each section. Thus, the present serviceability index of each pavement test section was related to both time and number of traffic loadings.

The performance data have been used to develop an empirical model (equation) using regression techniques. A "structural number" (SN) was determined for each pavement section by assigning relative structural strength coefficients (a_i) to a unit thickness of each material that would allow the substitution of a certain thickness of one type of material for another (in proportion to the ratios of their strength coefficients) with the same resulting load carrying capacity. The structural number of a pavement section was defined as the sum of the products of thickness and layer coefficient for each of the pavement layers ($SN = a_1D_1 + a_2D_2 + ...$). Serviceability loss was then related to the applied number of 18-kip ESALs, effective roadbed resilient modulus (M_R) and the pavement structural number. This model could be used to predict serviceability loss for a given set of design inputs (layer thicknesses, materials properties, traffic, etc.) or to produce a required structural number given traffic and serviceability requirements.

The performance model was then enhanced to include consideration of required design reliability and the variability of materials and construction quality for a given project. The performance period can also be corrected for environmentally-induced losses of serviceability (frost heave, etc.).

This model is deficient because it is directly applicable only to the northern Illinois climate and the specific subgrade and materials used for the pavement/subgrade structure. It also is based on an accelerated procedure for accumulating traffic, which considers only two years of environmental effects in conjunction with many years worth of traffic. These liabilities have been reduced somewhat by the incorporation of the experiences of several agencies with pavements in other climates, using different materials and actual traffic/climate conditions.

The AASHO Road Test brought forth many important concepts, including demonstration of the major influences of traffic loads and repetitions upon design thickness. Equally important was the development of the serviceability-performance method of analysis, which provided a quantifiable way of defining "failure" conditions based on a user-oriented definition rather than one based primarily on structural failure.

Many other performance and distress prediction models have been developed for flexible pavements. It should be recognized that each model was developed using a specific pavement database and model development techniques and is, therefore, subject to limitations and is probably generally applicable only for specific conditions. These models are often valuable, however, because a study of their functional form, included factors, and sensitivity of the models to these factors may show many important effects of various design inputs on the performance of flexible pavements. The user should also remember to consider key design factors that are not included in all empirical models (e.g., design reliability, climatic effects, etc.).

5.0 MECHANISTIC-EMPIRICAL DESIGN CONCEPTS FOR FLEXIBLE PAVEMENTS

5.1 Introduction

The basic components of mechanistic-empirical or rational methods are:

1. Structural analysis.
2. Distress or performance functions.

Structural analysis refers to the calculation of stress, strain, or deflection in a pavement which has been subjected to external loads or the effects of temperature or moisture. Once these values are determined at critical locations in the proposed pavement, they can be compared with values known from experimental or theoretical studies to be the maximum allowable, based on predictions of pavement performance (physical distress, such as cracking or rutting, or roughness). The pavement can then be designed by adjusting the different layer thicknesses so that the calculated stresses, strains, and deflections are less than the allowable maximum values.

The use of analytical methods to estimate the stress, strain or deflection state of pavements is not new. For asphalt concrete pavements, the publications of Burmeister (14), McLeod (15), Acum and Fox (16), and Palmer (17), beginning in 1940, have provided some of the basic theories applicable to this type of pavement. Mechanistic flexible pavement design procedures are typically based on the assumption that a pavement can be modelled as a multi-layered elastic or visco-elastic structure on an elastic or visco-elastic foundation. Assuming that pavements can be modelled in this manner, it is possible to calculate the stress, strain, or deflection (due to traffic loadings and/or environments) at any point within or below the pavement structure. However, researchers recognize that pavement performance will likely be influenced by a number of factors which will not be precisely modelled by mechanistic methods. It is, therefore, necessary to calibrate the models with observations of performance, i.e., empirical correlations. Thus the procedure is referred to as a mechanistic-empirical design procedure.

Although current methods of design for flexible pavements make no direct use of mechanistic design procedures, there are a few exceptions. For example the Kentucky Department of Transportation (18), the Asphalt Institute (19) and Shell International (20) have all developed such procedures for general application to a variety of design considerations. FHWA-VESYS and BISAR computer programs are also based on mechanistic equations that relate stresses, strains and deflections to such distress manifestations as fatigue and rutting.

5.2 Benefits of Mechanistic-Empirical Design Procedures

Researchers hypothesize that mechanistic-empirical design procedures will model a pavement more correctly than empirical equations. The primary benefits which could result from the successful application of mechanistic-empirical procedures include:

1. The ability to more accurately model the behavior of pavement sections.

2. The ability to extrapolate general pavement performance from limited field and laboratory results.

3. the ability to predict the occurrence of specific types of distress.

4. Improved design reliability.

Several important design factors that cannot be accurately addressed using empirical techniques can be considered using mechanistic techniques. Among those factors are the stress dependency of both the subgrade and base course, the time and temperature dependency of the asphaltic layers, the interface conditions between layer components, and modeling of the major distress modes of failure (rutting and fatigue cracking) by distress functions derived from the laws of mechanics. Mechanistic methods offer the only ways to incorporate the numerous variables that influence pavement performance into the design procedure and the use of mechanistic theories offers the possibility of universal (i.e., capable of including the effects of wide ranges of conditions) designs, which cannot be said of empirical methods.

One of the most significant benefits is the ability to structurally analyze and extrapolate the predicted performance of practically any flexible pavement design from limited amounts of field or laboratory data before attempting full-scale construction projects. This offers the potential to save time and money by eliminating from consideration those concepts that have been analyzed and are judged to have little merit.

Pavement management systems require the ability to predict the occurrence of distress in order to minimize the costs of maintenance and rehabilitation. Mechanistic-empirical procedures offer the best opportunity to meet this requirement by predicting the time of occurrence and density of specific distress types.

Uncertainty and variability in performance that result from the application of a general design procedure to site-specific conditions influence the amount of conservatism included in the design (overdesign). Reduction of this conservatism by improving design reliability provides for more efficient use of limited funds and allows the construction or rehabilitation of more projects with a fixed amount of money.

A subset of benefits which could result from the development of mechanistic procedures are summarized below:

1. Estimates of the consequences of new loading conditions can be evaluated. For example, the damaging effects of increased loads, high tire pressures, multiple axles, etc., can be modelled using mechanistic processes.

2. Better utilization of available materials can be accomplished by simulating the effects of varying the thickness and location of layers of stabilized local materials.

3. Better diagnostic techniques can be developed to evaluate the development of premature distress or why some pavements exceed their design expectations.

4. The effects of aging materials (e.g., the hardening of asphalts over time) can be included in performance estimates.

5. Seasonal effects, such as thaw-weakening, can be included in performance estimates.

6. Methods can be developed to better evaluate the long-term benefits of providing improved drainage in the roadway section.

One of the biggest drawbacks to the use of mechanistic design methods is that they require more comprehensive and sophisticated (and, therefore, more expensive and difficult-to-obtain) data inputs than typical empirical design techniques. Modulus of resilience, creep compliance, dynamic modulus, Poisson's ratio, fracture toughness, etc., have replaced arbitrary terms for subgrade and material strength used in earlier empirical techniques. However, the potential benefits are believed to far outweigh the drawbacks. In summary, mechanistic-empirical design procedures offer the best opportunity to improve pavement design technology for the next several decades.

5.3 A Framework for the Development of Mechanistic-Empirical Design Procedures

The development of a mechanistic-empirical pavement design procedure requires that consideration be given to the following items:

1. Determination of types of design considerations (i.e., cracking, rutting, roughness, etc.).

2. Development of a plan to obtain accurate input information.

3. Development and calibration of prediction models.

4. Testing.

Figure 3-1.17 illustrates a framework for the development of mechanistic-empirical design procedures for flexible pavements. This framework has been used by researchers to develop mechanistic design procedures and can be applied by user agencies (state highway departments) as a guide for in-house development. Mechanistic design procedures can be applied to a wide variety of pavement performance measures. The dominant types of performance measures which can be predicted by mechanistic design procedures relate to physical distresses caused by traffic loadings or environment. Only one mechanistic design procedure directly predicts ride quality. The VESYS program developed for the FHWA has the capability to predict Present Serviceability Index (PSI), but requires careful calibration. Most other agencies have developed empirical methods using calculation of stress, strain or deformation as independent variables for correlating with field performance observations.

Design considerations are best suited to physical distress; only those distress types which control performance or trigger some kind of maintenance or rehabilitation need to be considered. For example, for asphalt concrete-surfaced pavements, fatigue cracking, rutting, and possibly low temperature cracking would be likely candidates. If one or more of these is not a problem for the developing agency, it can be eliminated from their design process.

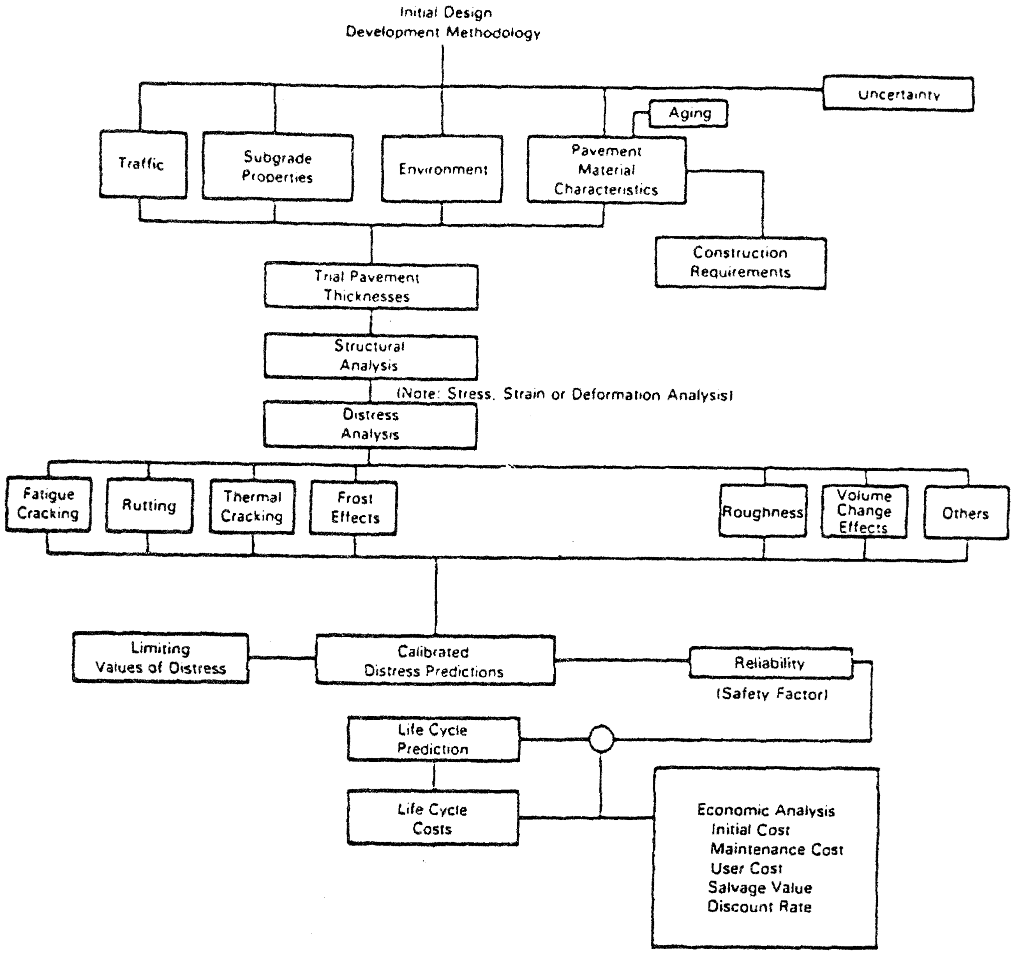


Figure 3-1.17. A Framework for the Development of Mechanistic-Empirical Design Procedures.

The development process also requires the selection of a series of trial pavement sections which are considered to include a range of thicknesses and materials appropriate to the design problem. A structural analysis is made for each trial section to calculate the stress, strain and deflection at specific locations, which are determined by the distress criteria.

Distress criteria must be developed for each of the distress types to be predicted by the design procedure. It must be decided what types of distress should control design and threshold values for each type must be established (e.g., how much cracking or rutting is considered acceptable before maintenance and rehabilitation costs become excessive). For example, for flexible pavements, one distress criteria is based on the development of alligator (fatigue) cracking, which is caused by excessive load repetitions causing tensile strain in the asphalt concrete. Similar criteria must be developed for each of the distress types shown in Figure 3-1.17. The "others" refer to future developments which can be or will be developed.

The most important step in the development and implementation of a mechanistic-empirical design procedure is the field testing and calibration of the predictive models that are utilized in the mechanistic design procedure. Even though a mechanistic design procedure is developed using basic material properties and structural analysis techniques, there are still numerous assumptions and simplifications that must be made in its development. In fact, most mechanistic procedures actually include a combination of mechanistic and empirical predictive models that are used in the design process. For example, there is the problem of climate, which is so complex that it will never be completely modelled mechanistically. Thus, climate, aging, and other factors are typically considered empirically.

It is necessary to ensure that the predictive models used in the mechanistic design procedure actually give reasonable predictions for the geographic regions under consideration. Thus, climate, materials, thickness combinations, and traffic should be included in the experiment design for verification/calibration. If this verification/calibration testing is not accomplished, there is a risk that the mechanistic design procedures will not provide accurate or acceptable results.

An example calibration is summarized as follows:

1. Obtain data from at least 20 actual field test sections with known design, materials, traffic, and climate data. The distress within the sections should range from extensive to very little.

2. The inputs to the structural analysis model should be obtained as specified in the design procedure (e.g., strain, stress, strength, resilient modulus, number of applied traffic loads, climate, etc.) for each of the field pavement sections.

3. Distress estimates should be computed for each section using the appropriate outputs from the structural analysis combined with damage prediction models.

4. The distress estimates are compared with actual field observations of distress to determine calibration factors.

A calibration procedure such as this will result in realistic pavement designs and will provide the needed confidence and credibility for the mechanistic approach.

After the calibration process has been completed, the prediction models developed for each distress type should be tested on a wide range of projects for which performance information is available. Some final adjustments in the distress models may be necessary as part of this final step. The agency should maintain an ongoing program of data acquisition to continually improve the system.

At the completion of this final testing, the agency will have a verified, reliable mechanistic-empirical design system with capabilities beyond the usual empirical design methods.

The implementation of mechanistic procedures could take one of several forms.

1. The procedures could be used to develop design curves similar to those developed by The Asphalt Institute, Shell International or the Kentucky Department of Transportation. In this form, the analyst will pre-solve a large number of problems sufficient to develop design curves. The user is not required to do any analytical work in order to prepare design recommendations. A relatively simple step-by-step procedure can be specified for design.

2. The procedures could be used in site-specific cases to predict performance when conditions exceed normal design criteria (e.g., excessive loads on standard vehicles or any load on nonstandard vehicles).

3. The procedures could be used to answer "what if" questions (e.g., "What would be the effect of increasing the legal axle load on performance?" or "What would be the effect of increased tire pressure?")

Once a user agency has the capability to use mechanistic design procedures, it can be anticipated that many additional applications will be found.

6.0 BASIC DESIGN CONSIDERATIONS FOR MINIMIZING FLEXIBLE PAVEMENT DISTRESSES

6.1 Critical Stresses in Flexible Pavement Systems

6.1.1 Definition

Critical stresses and strains in pavement applications can be defined as those material responses which, through a single or repeated occurrence, will result in structural deterioration of the pavement. Critical stresses and strains can develop at more than one location in multi-layered pavement structures because of the varying material properties and location within the structure of each layer. Different critical stress locations generally lead to different distress manifestations. Thus, pavement structures must be designed with consideration for reducing stresses at various critical locations to acceptable levels. The typical location of flexible pavement critical stresses and strains are discussed in the following paragraphs.

6.1.2 Location of Critical Stresses in Flexible Pavement Systems

Typical critical stress/strain locations for a flexible pavement with a granular base were shown in Figure 3-1.4.

The vertical stress component directly beneath the applied load at location 1 is a compressive stress. Compaction and densification of the asphalt concrete surface layer can result from the repeated application of heavy traffic loadings, which would show up as rutting at the surface. The contribution of the asphalt concrete layer to rutting is generally negligible, however, since this layer is usually compacted to very near its theoretical maximum achievable density during original construction.

The tensile strain at the bottom of the asphalt concrete surface (location 2) can result in the initiation of cracking. This cracking reduces the pavement section resisting the tensile stress and results in rapid propagation of the crack to the surface under repeated traffic loads, where it shows up as "alligator" or fatigue cracking. This cracking allows surface moisture to infiltrate the pavement structure and soften the lower layers, resulting in further deterioration of the pavement. The consideration of this tensile strain should be included in all flexible pavement designs.

Since unbound granular bases cannot resist any tension, the critical stress and strain components at location 3 are associated with consolidation of the granular material, which can contribute significantly to pavement rutting.

The largest contribution (70-95 percent) to flexible pavement rutting is generally caused by permanent deformation of the subgrade due to repeated excessive vertical stress within the subgrade (location 4). Subgrade deformation causes settlement of the upper layers, which generally results in rutting and various forms of cracking.

Figure 3-1.5 identifies the critical stress locations for a flexible pavement placed over a stabilized base. The critical radial strain (location 2) is shifted to the bottom of the stabilized base layer, which is now capable of resisting tensile forces. The failure of this layer in tension may now result in either transverse reflection cracking (cement stabilized bases) or "alligator" or fatigue cracking (asphalt stabilized bases). The remaining critical stress locations and resulting distresses are similar to those occurring in the flexible pavement over a granular base, although stresses in the lower pavement layers are generally greatly reduced by the presence of the stabilized base layer.

6.1.3 Computation of Critical Stresses in Flexible Pavement Systems

The computation of flexible pavement stresses and strains can be accomplished easily using graphical solutions or the elastic layer computer programs presented in this module.

6.1.4 Relationships Between Pavement Response and Pavement Performance

Every stress repetition causes structural damage and brings the pavement closer to failure. The closer the stress level is to the strength or failure stress of the material, the greater the amount of damage done by each repetition at that stress level. If we assume that the amount of damage required to produce failure is constant, the number of load repetitions required to produce failure in a particular material will vary inversely with the resulting stress. These fatigue relationships are generally derived empirically from field and laboratory performance data and vary widely with the materials and modes of failure (i.e., tension, compression, etc.) in question.

Figure 3-1.18 presents a relationship between asphalt concrete radial tensile strain and the predicted number of allowable stress repetitions before a particular pavement structure will fail due to fatigue or alligator cracking. Figure 3-1.19 presents a similar relationship between subgrade compression stress and rutting failure for a specific pavement structure. Fatigue or performance curves of this type can be used to determine the maximum allowable stress or strain levels that will allow the pavement to carry the design traffic loads over the proposed pavement design life. Each trial thickness design must be analyzed to determine whether the resulting critical stresses exceed these maximums.

EXAMPLE:

A flexible pavement must be designed to sustain 10 million applications of the design load over its design life. It has been determined that the relationships depicted in Figures 3-1.18 and 3-1.19 apply to this pavement structure. Thus, the allowable levels of tensile radial strain in the asphalt concrete and vertical compressive strain in the subgrade are estimated as 80×10^{-6} in/in and 400×10^{-6} in/in, respectively.

A structural analysis is accomplished and strains of 100×10^{-6} in/in and 350×10^{-6} in/in are computed for the asphalt concrete and subgrade, respectively. Thus, the subgrade strain is acceptable, but the asphalt concrete strain must be reduced using the approaches described in this module.

6.4 Design Considerations to Minimize Distress

6.4.1 Goals for Subgrade and Wearing Surface Protection

The principal objectives in designing a flexible pavement system are to avoid:

1. Critical vertical stresses in the lower pavement layers (permanent subgrade deformation) which are one of the causes of rutting.

2. Critical tensile radial stresses in the upper layers which result in fatigue or reflective cracking of the surface or fatigue cracking of the stabilized base.

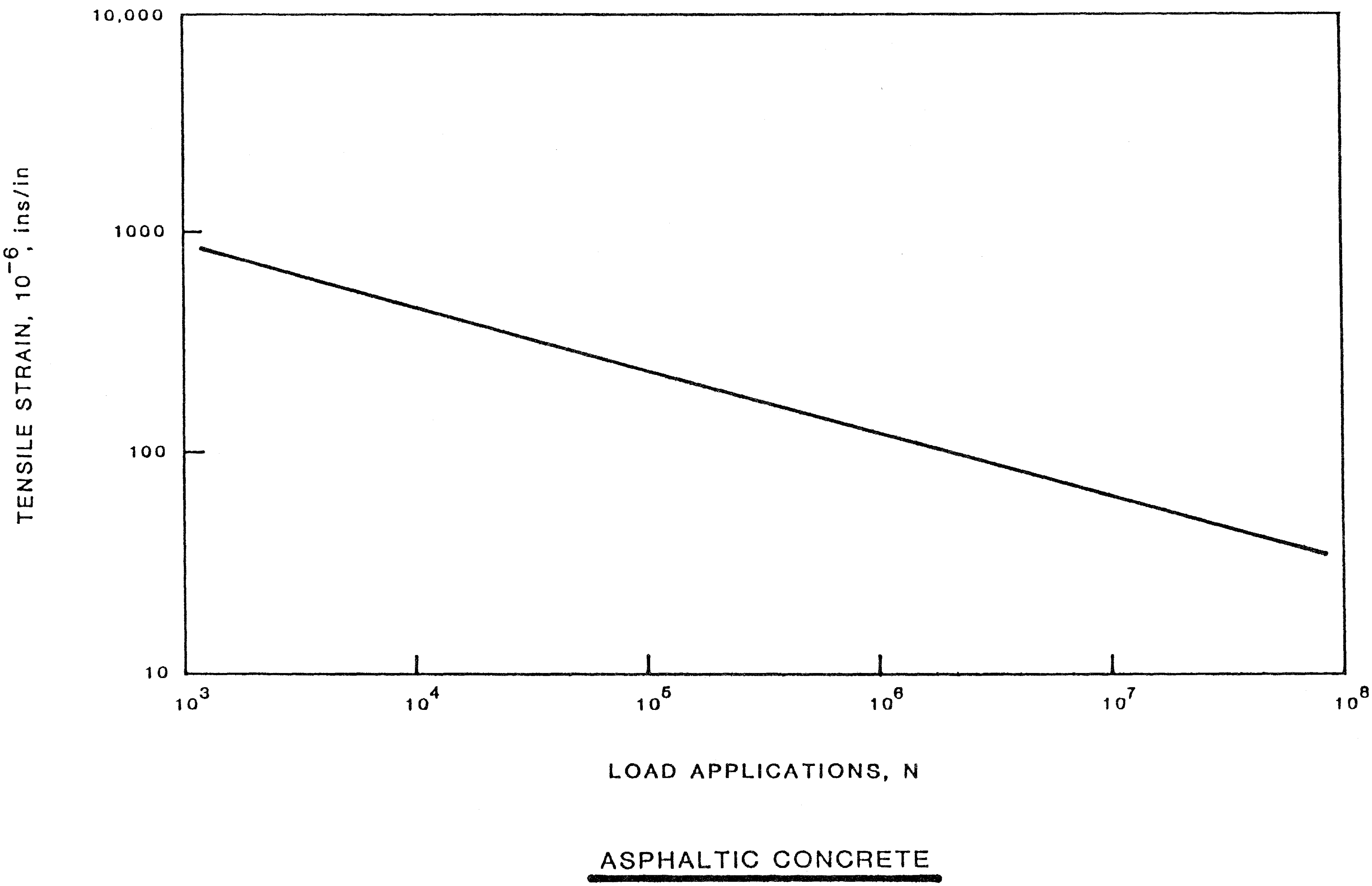
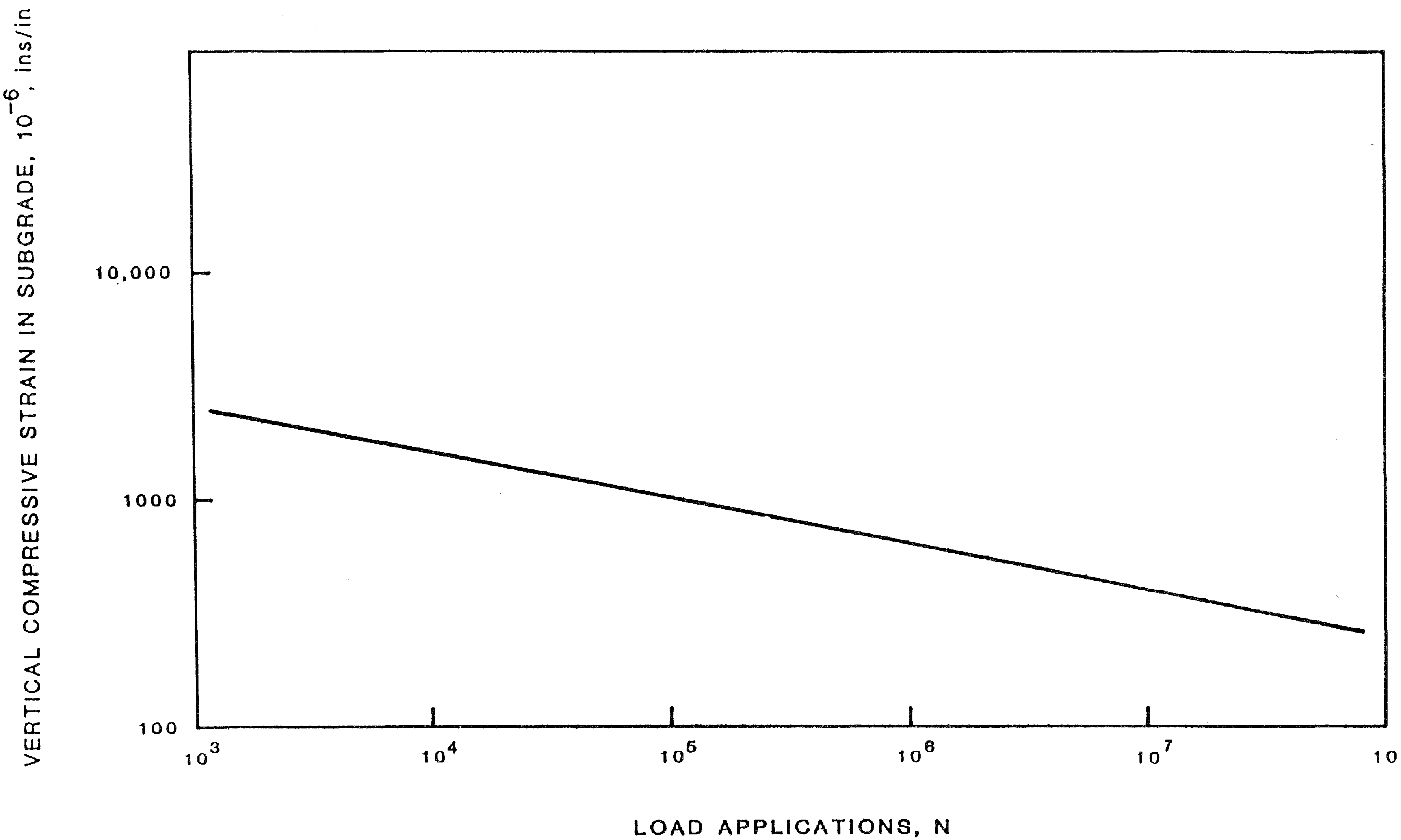


Figure 3-1.18. Typical Fatigue Curve for Tensile Strain in Asphalt Concrete.



SILTY CLAY SUBGRADE

Figure 3-1.19. Typical Fatigue Curve for Permanent Deformation in a Silty-Clay Subgrade.

In addition, each layer must be designed (thickness and material properties) so that each of the underlying layers is not overstressed. The total pavement thickness constructed over the subgrade should also be designed to consider the depth required to restrain soil swelling pressures and the depth required to provide frost protection.

In addition to the problem of providing adequate cover for the subgrade and surfacing materials, there is the attendant design problem of providing an adequate wearing surface. This must be a highly cohesive material capable of withstanding not only the high shearing and compressive stresses caused by heavy wheel loads, but it must also be resistant to abrasion and hold its shape. The design thickness of this layer is a function of the level of the anticipated traffic volumes and composition, with increased thicknesses for higher volumes of heavier vehicles.

6.4.2 Structural Design to Minimize Distress

Pavement layers generally attempt to resist portions of the load that are directly proportional to their stiffnesses. That is, increasing the modulus of elasticity for a given layer will make that layer stronger, but it will also force the layer to assume a greater stress. If the increased stiffness is not accompanied by an increase in layer thickness, the increase in stress could be greater than the increase in strength, particularly in thinner pavements. Thus, the most efficient ways to reduce tensile stresses/strains in the upper layers are to reduce the elastic modulus of the critical layer and/or increase its thickness. Note that when the surface is very thin, has a reduced stiffness, and/or a stiff stabilized base is present (e.g., with thin surface courses and surface treatments) the surface course may actually go into radial compression directly under the load.

There are several ways to reduce the vertical stresses transmitted to the subgrade. Stresses in the subgrade decrease rapidly with depth. At depths z such that $z/a \geq 3$ (where a = radius of the loaded area), the influence of the stiffness of the upper pavement layers becomes insignificant. Thus, the most efficient way to reduce subgrade compressive stresses is to thicken the pavement structure, usually in the subbase or the layer that provides the most stiffness for the least cost. There is some benefit to increasing the thickness of the asphalt concrete surface, although this is not usually as efficient or cost-effective unless the pavement structure is relatively thin and the surface layer is very stiff. Subgrade compressive stresses can also be reduced sharply by increasing the stiffness of the upper pavement layers, particularly the layer directly above the subgrade.

Since most of the surface deflection results from compression of the subgrade (70-95%), the same designs that reduce subgrade vertical compression stress will also be most effective for reducing overall pavement deflection.

6.5 Other Design Considerations

6.5.1 Frost Protection

Pavement damage due to the freezing of water in the subgrade can take two forms. The first is the development of "ice lenses" in the subgrade, which cause the pavement to heave and develop serious structural cracking.

Fatigue damage may also result from the loss of support that accompanies the spring thaw when ice in the upper portions of the pavement structure melts but cannot drain because of the ice below.

The development of frost damage requires that three factors be present:

1. A frost-susceptible soil.
2. Appropriate temperature conditions.
3. A supply of water.

There are very few locations in the United States where these conditions cannot be found. The Corps of Engineers have categorized materials by gradation to identify their degree of frost-susceptibility. Category F1 is considered least susceptible and Category F4 is considered most susceptible. These categories are defined as follows:

Group	Description
F1 -	Gravelly soils containing between 3 and 20 percent finer than 0.02 mm by weight.
F2 -	Sands containing between 3 and 15 percent finer than 0.02 mm by weight.
F3 -	(a) Gravelly soils containing more than 20 percent finer than 0.02 mm by weight, and sands, except fine sands, containing more than 15 percent finer than 0.02 mm by weight. (b) Clays with plasticity indices of more than 12, except (c) varved clays existing within uniform conditions.
F4 -	(a) All silts including sandy silts. (b) Fine silty and containing more than 15 percent finer than 0.02 mm by weight. (c) Lean clays with plasticity indices of less than 12. (d) Varved clays with nonuniform subgrade.

A frost design can be accomplished using one of several approaches, including the use of nonfrost-susceptible materials, insulation, and drainage. These general approaches can be used separately or in combination to produce full protection or reduced frost effects (because economics rarely permit design for full-protection). The design that is chosen is generally a trade-off between initial construction and maintenance costs.

The first approach to frost protection is to desensitize the subgrade materials by stabilizing them with an appropriate material (asphalt, Portland cement, lime, etc.) to some depth.

A second approach is to protect the subgrade by providing a cover of nonfrost-susceptible paving materials equal in thickness to <u>at least</u> one-half the maximum frost penetration depth in the project area. This can be accomplished by constructing a very thick pavement on top of the subgrade or by excavating the subgrade as required and backfilling with the nonfrost-susceptible materials. Suitable nonfrost-susceptible materials include open-graded (less than 10 percent passing the #200 sieve) aggregate and treated bases, Portland cement concrete and asphalt concrete.

Insulating layers of foamed plastic have been used successfully in various parts of the world for reducing frost penetration (3). These foam materials have low coefficients of thermal conductivity, thereby reducing frost penetration. Such insulating layers can be used in lieu of thick base courses.

The installation of drains is another possible alternative to constructing a thickened pavement for frost protection. The removal of water from the frost-susceptible materials will reduce or eliminate frost damage.

More detailed discussions of frost penetration and pavement damage are presented in Block 2 and Reference 2.

6.5.2 Design Reliability

One input that is often poorly addressed in many pavement design procedures is that of design reliability. A good design process provides some means of incorporating a degree of certainty that the various design alternatives will last for the analysis period, taking into account random variations in both traffic prediction and performance prediction.

Designs that utilize average values for all inputs (including projected traffic and performance) have a reliability of 50 percent -- that is, approximately one-half of the constructed mileage of such pavements can be expected to perform satisfactorily while the other half fails prematurely. While such minimal reliability may be acceptable or even desirable for the construction of rural local roads where the consequences of interrupting traffic flow for repair work are relatively small and budgets for new construction are small, higher levels of performance are generally expected by the travelling public for more heavily-travelled pavements.

The new AASHTO Design Guide (6) provides for control of design performance reliability through the use of a reliability factor (F_R) that is multiplied times the design period traffic prediction (w_{18}) to produce design load applications (W_{18}) for input in the design equations. For a given reliability level (R), the reliability factor is a function of the overall standard deviation (S_o) that accounts for random variation in the traffic prediction and normal variation in the performance prediction for a given W_{18}. This selected level of reliability and overall standard deviation will account for the combined effect of the variation of all of the design variables, so the designer should use the best estimates of the mean value for all other input values, rather than "conservative" values.

Table 3-1.4 is taken from the new AASHTO Design Guide (6) and presents suggested levels of reliability for pavements of various functional classification. This table is based on a survey of the AASHTO Pavement Design Task Force concerning the inherent reliability of many current pavement design procedures. Although these numbers can be justified based on a large amount of past experience, consideration of specific project parameters will likely yield a more accurate assessment of the required reliability.

Figure 3-1.20 provides a graph illustrating one means of identifying an optimum level of reliability for a particular design project. Three curves are shown in the figure. The first curve (A) represents the effects of

Table 3-1.4. Suggested Levels of Reliability for Various Functional Classifications (6).

FUNCTIONAL CLASSIFICATION	RECOMMENDED LEVEL OF RELIABILITY	
	URBAN	RURAL
INTERSTATE AND OTHER FREEWAYS	85 - 99.9	80 - 99.9
PRINCIPAL ARTERIALS	80 - 99	75 - 95
COLLECTORS	80 - 95	75 - 95
LOCAL	50 - 80	50 - 80

NOTE: RESULTS BASED ON A SURVEY OF THE AASHTO PAVEMENT DESIGN TASK FORCE

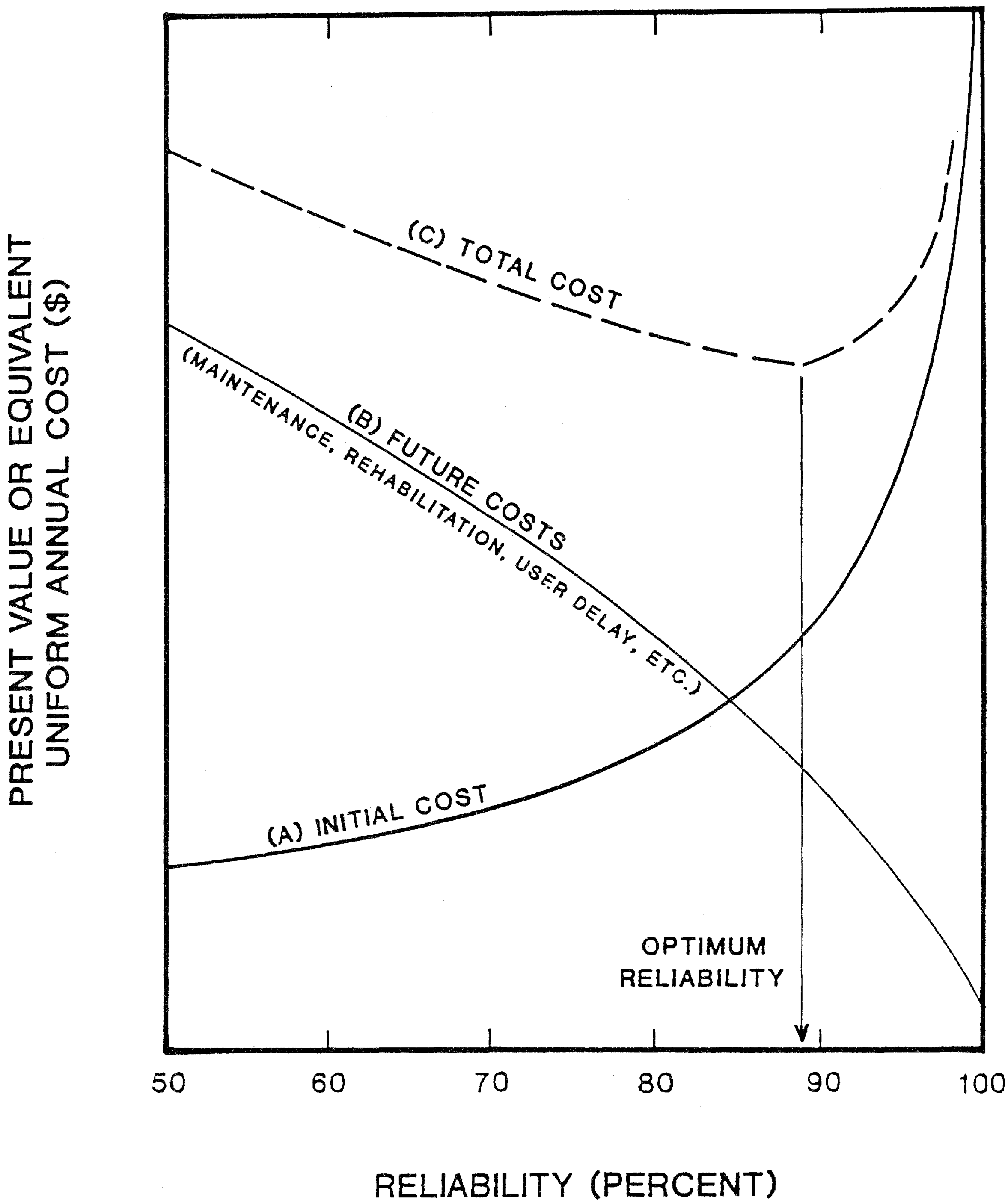


Figure 3-1.20. Illustration of an approach to identifying the optimum reliability level for a given facility.

reliability on the cost (expressed in net present value or equivalent uniform annual cost) of the initial pavement structure. As the design reliability increases, so does the required initial pavement thickness and its associated cost. The second curve (B) represents the effects of reliability on the future distress-related costs (maintenance, rehabilitation, user delay, etc.) The third curve (C) represents the sum of the first two curves. Since the objective is to minimize the total overall project cost, the optimum reliability for a given project corresponds to the minimum value on curve (C). It should be recognized that this optimum reliability is applicable only to the level of usage and consequences of failure associated with a particular project. Although other projects may have the same level of usage, varying soil and environmental conditions may affect the level of risk and, therefore, the optimum reliability.

The selected standard deviation should be representative of local conditions, with lower values being selected in areas where good performance models and traffic data are available and higher values where the opposite situation exists. The new AASHTO Design Guide suggests values of 0.40 - 0.50 for flexible pavements and 0.30 - 0.40 for rigid pavements.

Reliability concepts and the use of the reliability factor and overall standard deviation were introduced in Block 2 and are presented in detail in Appendix EE and Part I of Chapter 4 of the new AASHTO Design Guide (6).

6.5.3 Stage Construction

To obtain the maximum economy in pavement design, provision is often made to construct the pavement in stages, applying successive layers of asphalt concrete according to design and a predetermined time schedule. The economy of this design theory develops from the fact that all pavements, regardless of their structural characteristics, suffer some degree of surface distress and attrition requiring treatment before the pavements' useful life is consumed. There are also many cases where, because of a poor foundation, considerable uneven settlement of the roadway can be expected for several years. The most economical solution in these cases may be to design the initial pavement for a shorter-than-usual period to allow time to realize the majority of settlement and then place the second stage pavement, which would include preleveling to reestablish a smooth grade. The second stage pavement, depending upon conditions, could be designed for the remainder of the original design period or for an extended period. The design of planned stage construction should not be confused with the design of major maintenance or rehabilitation of existing pavements.

There are many types of projects for which stage construction of the pavement should be considered, including the following:

1. If a road is expected to accommodate substantially heavier traffic at a known point in the future, the pavement can be initially designed for a short design period and later thickened to accommodate the heavier traffic for a longer design life.

2. When there is not quite enough money available to construct the full thickness required for the original design period, the pavement might be designed for construction in two stages, with the first stage being designed for a shorter design period. It is

important to be sure that the funding required for the second stage will be available when needed, however.

The method of design for stage construction is based on the concept of "remaining life." Since the procedure involves "planned" stage construction, the pavement is designed on the presumption that the planned overlay will be placed before the pavement has used up all of its fatigue life. To ensure that this is the case, certain modifications are typically made in the design inputs that would be used if planned stage construction was not being used. One important consideration is the effect of "compound reliability," wherein the overall reliability of a two-stage strategy with each stage designed for a 90 percent reliability level would be 0.90 x 0.90 = 0.81 or 81 percent rather than 90 percent. One method to achieve a given overall level of reliability is to design each stage with the following level of reliability:

$$R_{(stage)} = R_{(overall)}^{1/n}$$

where n is equal to the number of stages including that of the initial pavement structure.

References 5 and 6 discuss stage construction design concepts in more detail.

7.0 SUMMARY

Two basic approaches are currently being used to determine the required layer thicknesses for asphalt concrete pavement structures: empirical procedures and mechanistic-empirical or rational procedures.

Empirical procedures are derived from experience or observation alone, often without due regard for system behavior or pavement theory. These models are generally used to determine required pavement thickness, the number of load applications to failure, or the occurrence of distresses as a function of pavement materials properties, subgrade type, climate, traffic, etc. The advantages to using empirical models is that they tend to be simple and easy to use. Unfortunately, they are usually only accurate for the exact conditions under which they were developed. The AASHTO, U. S. Army Corps of Engineers, Louisiana, and Utah designs are among a large family of pavement design techniques which were developed primarily on the basis of field performance.

The development of an empirical design procedure begins with the establishment of a database of experience from which to observe relationships between design inputs and performance. Preliminary distress and serviceability models can then be developed using statistical analysis procedures. These models must be analyzed and verified to determine their behavior within the valid ranges of the independent variables before they can be accepted for use in a design procedure.

Mechanistic-empirical design procedures utilize calculated stresses, strains, and deflections and distress or performance prediction models. The determination of pavement response to loads is accomplished using the techniques described in Module 3-1. distress and performance models are developed from laboratory data or the observed performance of in-service pavements and can be used to estimate the maximum number of repetitions of a given level of stress, strain or deflection a pavement can withstand before reaching an unacceptable state of serviceability. Mechanistic approaches offer the only ways that the numerous variables that influence pavement performance can be incorporated directly into a design procedure. They also offer the ability to analyze a pavement for several different failure modes. One disadvantage to mechanistic design approaches is that they typically require more comprehensive and sophisticated data inputs than empirical design techniques.

The development of a mechanistic design procedure begins with the selection of a pavement analysis model and the determination of critical pavement responses. The critical responses of appropriate pavement sections is then predicted using the selected analysis model, and empirical performance relationships are used to relate the predicted pavement responses to predicted distress development and performance. Calibration of the mechanistic-empirical models is accomplished using design and performance data from in-service pavements. Critical levels of distress/performance must be established to complete the design procedure.

The principal objectives in designing a flexible pavement system are to avoid: 1) permanent deformation in the lower pavement layers which cause rutting, and 2) critical radial stresses in the upper layers which cause alligator or fatigue cracking. The most efficient way to reduce subgrade vertical compression stresses is usually to thicken the entire pavement structure. Reduction of radial strains in the upper layers can be accomplished by reducing the stiffness of the layers or increasing their thickness.

Other considerations in the flexible pavement design process include frost protection, prevention of thermal cracking, design reliability, and stage construction.

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MODULE 3-2

AASHTO PROCEDURE FOR FLEXIBLE PAVEMENT THICKNESS DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module will familiarize the participants with the historical development of the AASHTO flexible pavement thickness design procedure and provide guidance on the use of the current 1986 procedure. Upon completion of this module the participants will be able to accomplish the following:

1. Describe the development of the AASHTO flexible pavement design equations.

2. Identify the basic assumptions required for the development of these equations.

3. Discuss procedures and considerations for determining design input variables, including layer coefficients, effective roadbed soil resilient modulus, design serviceability loss, design reliability, overall standard deviation, and structural numbers.

4. Design flexible pavements for a variety of traffic and roadbed soil conditions, environmental effects, types of bases and surface materials.

5. Compare the cost-effectiveness of alternate designs for a given project.

6. Perform sensitivity analyses to determine the effects of input variables on pavement design.

7. Identify the strengths, weaknesses and limitations of the AASHTO flexible pavement thickness design procedure.

2.0 INTRODUCTION

One of the major objectives of the AASHO Road Test was to provide information that could be used to develop pavement design criteria and procedures. This objective was met with the development and circulation of the "AASHO Interim Guide for the Design of Rigid and Flexible Pavements" (1) in 1961, which contained design procedures based on empirical models derived from data collected at the AASHO Road Test. After the Guide had been used for several years, the AASHTO Design Committee evaluated and revised the Interim Guide in 1972 (2) and again (for rigid pavement applications) in 1981 (3).

Further evaluations of the Guide were undertaken in 1983, and it was determined that although the Guide was still serving its main objectives, some improvements could be made to incorporate advances in pavement design and analysis technology that had been made since 1972. Thus, in 1984-85 the Subcommittee on Pavement Design and a team of consultants revised the existing guide under NCHRP Project 20-7/24 and issued the current version entitled "AASHTO Guide for the Design of Pavement Structures -- 1986" (4).

The current guide retains the modified AASHO Road Test performance prediction equations as the basic models for use in pavement design. Major flexible pavement design procedure changes have been made in several areas, including the following:

1. Incorporation of a design reliability factor (based on a shift in the design traffic) to allow the designer to use the concept of risk analysis for various classes of highways.

2. Replacement of the soil support number with the resilient modulus (AASHTO test method T274) to provide a rational testing procedure for defining materials properties.

3. Use of the resilient modulus test for assigning layer coefficients to both stabilized and unstabilized material.

4. Provision of guidance for the construction of subsurface drainage systems and modifications to the design equations to take advantage of improvements in performance that result from good drainage.

5. Replacement of the subjective regional factor with a rational approach to the adjustment of designs to account for environmental considerations such as moisture and temperature climate considerations, including thaw-weakening and other seasonal variations in material properties.

The new Guide also includes recommendations and guidelines for conducting economic analysis of alternative designs and a summary of the latest concepts concerning the development and use of mechanistic-empirical design procedures.

3.0 DESIGN ASSUMPTIONS AND PROCEDURES

The general applicability and accuracy of any empirical pavement design approach is governed by the selection of the independent variables and their ranges in the experimental design, field conditions and random variability, simplifying assumptions used in the analysis procedures, and the analysis techniques themselves. The AASHTO procedure for flexible pavement thickness design is no exception.

3.1 Specific Conditions of the AASHO Road Test

The location of the AASHO Road Test was near Ottawa, Illinois, about 80 miles southwest of Chicago. The facility was constructed along the alignment of Interstate Route 80. This site was chosen because:

1. The soil within the area was uniform and of a type representative of that found in large areas of the country.

2. The climate was typical of that found throughout much of the northern United States.

3. The pavement construction work could ultimately be incorporated in the construction of sections of Interstate 80.

The climate of the Road Test area is temperate with an average annual precipitation of about 34 inches. The average depth of frost penetration is about 28 inches. The AASHTO designation A-6/A-7-6 soils (CBR = 2 - 4) found at the site are generally poorly drained and typically retain more precipitation than can evaporate, thus yielding a positive Thornthwaite Moisture Index of about 30. The modulus of subgrade reaction (k) after the spring thaw is typically about 45 psi/in.

The test facilities consisted of six two-lane test loops, located as shown in Figure 3-2.1. The north tangent of each loop was constructed of flexible pavement sections and the south tangent was constructed of rigid pavement sections. Most of the 234 flexible pavement structural design sections (468 test sections, 160 feet in length) comprised a complete replicated factorial experiment investigating the effects of varying thicknesses of surfacing (1, 2, 3, 4, 5, and 6 inches), base course (0, 3, 6, and 9 inches), and subbase (0, 4, 8, 12, and 16 inches). Several additional studies were also conducted to evaluate surface treatments, shoulders and four different types of base course (crushed stone, gravel, cement-treated gravel and bituminous-treated gravel).

Although conventional construction techniques were used, construction was of extremely high quality because of close supervision and on-site materials testing by the Illinois Division of Highways and the Highway Research Board staff. In addition, an extraordinary effort was put forth to insure uniformity of all pavement components. For example, no construction equipment other than that necessary for compaction was permitted to operate in the center 24-foot width of the roadway, and all turning operations on the grade were limited to specially designated transition areas. Therefore, variations in concrete, aggregates, moisture, density, subgrade soil properties, etc., were much lower than can be expected in most normal highway construction.

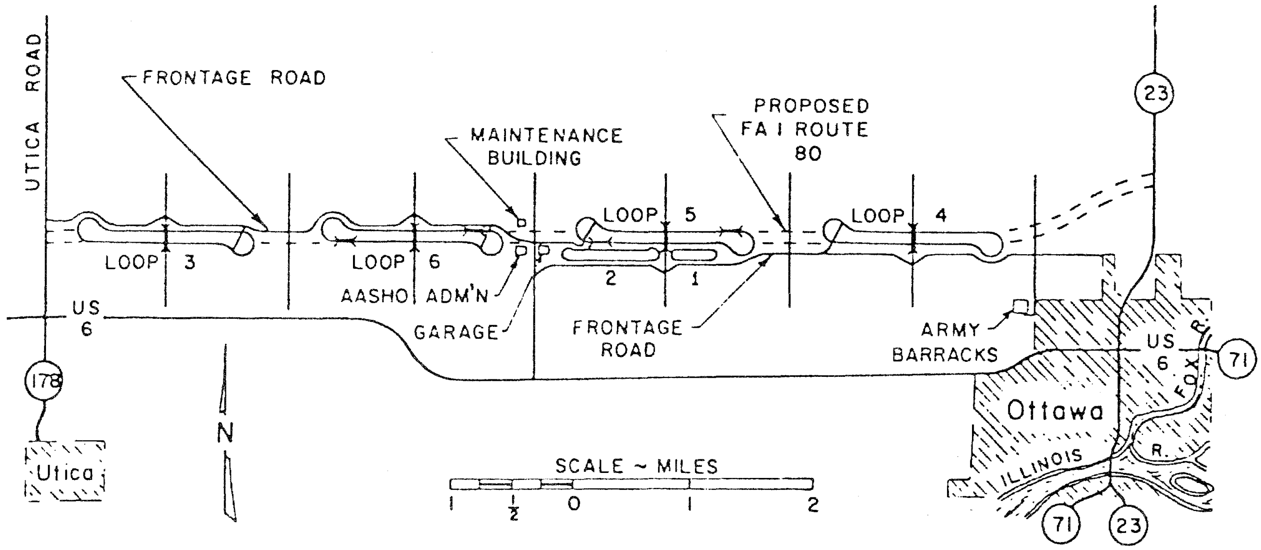
No traffic operated over Loop 1. All vehicles assigned to any one traffic lane in loops 2 through 6 had the same axle arrangement - axle load combinations, as described in Table 3-2.1. Tire pressures and steering axle loads were representative of normal practice for the time. The test was conducted over a two-year period, which was probably too short for the complete evaluation of environmental and aging effects, but was sufficient to allow the application of 1,114,000 load applications to each loop.

Performance measurements were taken at regular intervals to provide information concerning the roughness and visible deterioration over time of the surfacing of each section. These measurements included transverse pavement profiles (rutting), cracking, patching, deflections, strains, layer thicknesses and temperatures and numerous other measurements. This information was used directly in the development of the performance models that eventually became the basis for the current AASHTO design procedure.

4.0 THICKNESS DESIGN PROCEDURE

4.1 Design Inputs

This section describes the design inputs that are required to use the AASHTO Guide for Design of Pavement Structures--1986. These inputs are classified under five separate categories, as described herein.



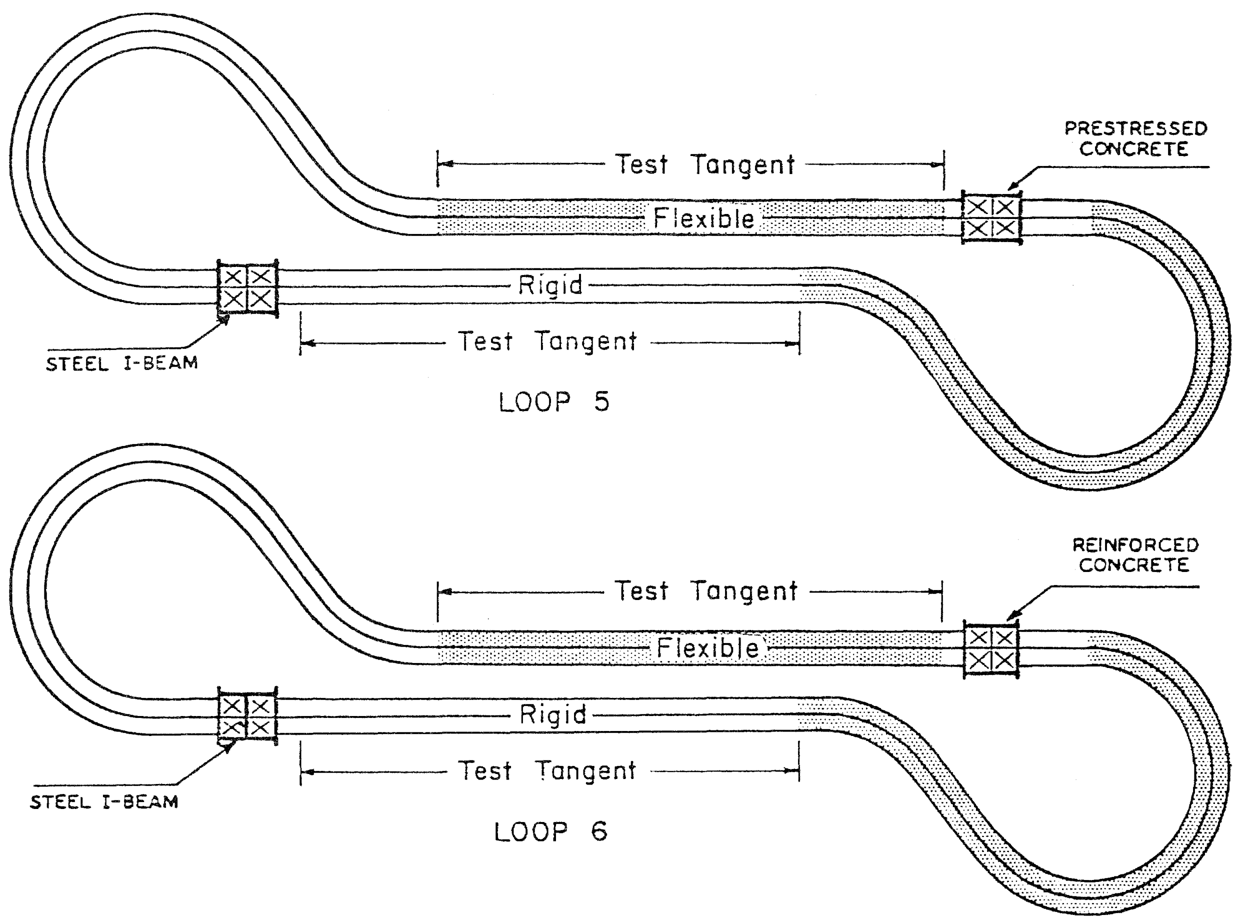
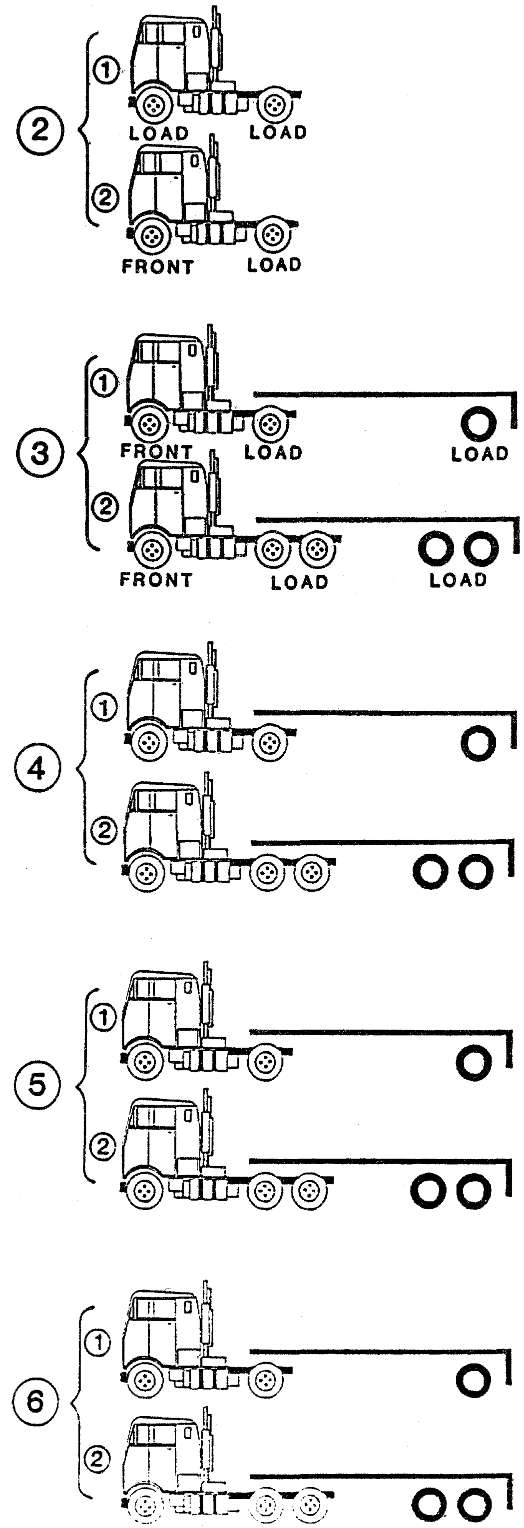


Figure 3-2.1 Layout of the AASHO Road Test.

Table 3-2.1. Axle Weights and Distributions Used on Various Loops of the AASHO Road Test (5).



LOOP	LANE	WEIGHT IN KIPS FRONT AXLE	LOAD AXLE	GROSS WEIGHT
2	1	2	2	4
2	2	2	6	8
3	1	4	12	28
3	2	6	24	54
4	1	6	18	42
4	2	9	32	73
5	1	6	22.4	51
5	2	9	40	89
6	1	9	30	69
6	2	12	48	108

4.1.1 General Design Variables

General design variables are those that must be considered in the design and construction of any pavement surface. Included in this category are time constraints (such as selected performance and analysis periods), traffic, design reliability, and environmental effects (such as roadbed swelling and frost heave).

Time Constraints

The selection of various performance and analysis periods forces the designer to consider design strategies that range from a low-maintenance structure that lasts for the entire analysis period to stage construction alternatives that require an initial structure and planned maintenance/overlays.

The performance period is the period of time that elapses as a new or rehabilitated pavement structure deteriorates from its initial serviceability to its terminal serviceability and requires rehabilitation. The designer must select minimum and maximum allowable bounds. The selection of these values is impacted by such factors as pavement functional classification, the public's perception of how long a "new" surface should last, funds available for initial construction, life-cycle costs, and other engineering considerations.

The analysis period is the period of time that any design strategy must cover. The analysis period may be identical to the selected performance period. However, realistic practical performance limitations for some pavement designs may necessitate the consideration of stage construction or planned rehabilitation to achieve the desired analysis period.

In the past, pavements were typically designed and analyzed for a 20-year performance period. It is now recommended that consideration be given to longer periods, since these may be better suited for the evaluation of alternative long-term strategies based on life-cycle costs. In any event, it is recommended that the analysis period should be selected to include at least one rehabilitation of the pavement. Guidelines for the selection of appropriate analysis periods for various highway types are tabulated in Block 6 and Reference 4.

Traffic

The AASHTO flexible pavement thickness design procedures are based on cumulative expected 18-kip equivalent single-axle loads (ESAL) during the analysis period (w_{18}). The computation of cumulative ESAL is described in Module 2-6 and should be studied thoroughly.

Reliability

Design reliability refers to the degree of certainty that a given design alternative will last the analysis period. As described in Module 2-7, the AASHTO design-performance reliability is controlled through the use of a design reliability factor (F_r) that is multiplied by the design period traffic prediction (w_{18}) to produce design traffic applications (W_{18}) for use in the design equation. For a given reliability level (R), the reliability factor is a function of the overall standard deviation (S_o) that accounts for standard variation in materials and construction, the chance variation in the traffic prediction, and the normal variation in pavement performance for a given W_{18}.

The following table provides appropriate levels of design reliability for pavements with various functional classifications recommended by AASHTO:

	Recommended Level of Reliability, R (%)	
Functional Classification	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

The selected standard deviation must be representative of local conditions. The following values are recommended for general use, but should be evaluated for local usage:

Design Condition	Standard Deviation
Variation in pavement performance prediction without traffic error	0.35 flexible 0.25 rigid
Total variation in pavement performance prediction and in traffic estimation	0.45 flexible 0.35 rigid

When stage construction is to be considered, it is important to recognize the need to compound the reliability of each individual stage of the strategy to achieve the desired overall reliability. The design reliability of each stage can be expressed as:

$$R_{stage} = (R_{overall})^{1/n}$$

where n is the number of stages being considered. For example, if three stages are to be constructed and the desired level of overall reliability is 95%, the reliability of each individual stage must be $(0.95)^{1/3}$ or 98.3%.

Environmental Impacts

Temperature and moisture changes have an effect on the strength, durability, and load-carrying capacity of the pavement and roadbed materials through the mechanics of swelling soils, frost heave, and other phenomena. Block 2 discusses the treatment of pavement materials to minimize these problems and provides criteria for quantifying the input requirements for evaluating roadbed soils.

If a swelling clay or frost heave potential exists and the pavement design does not take steps to prevent adverse effects, the loss of serviceability over the analysis period should be estimated and added to that resulting from cumulative axle loads. A conceptual example of environmental serviceability loss versus time is presented in Figure 3-2.2.

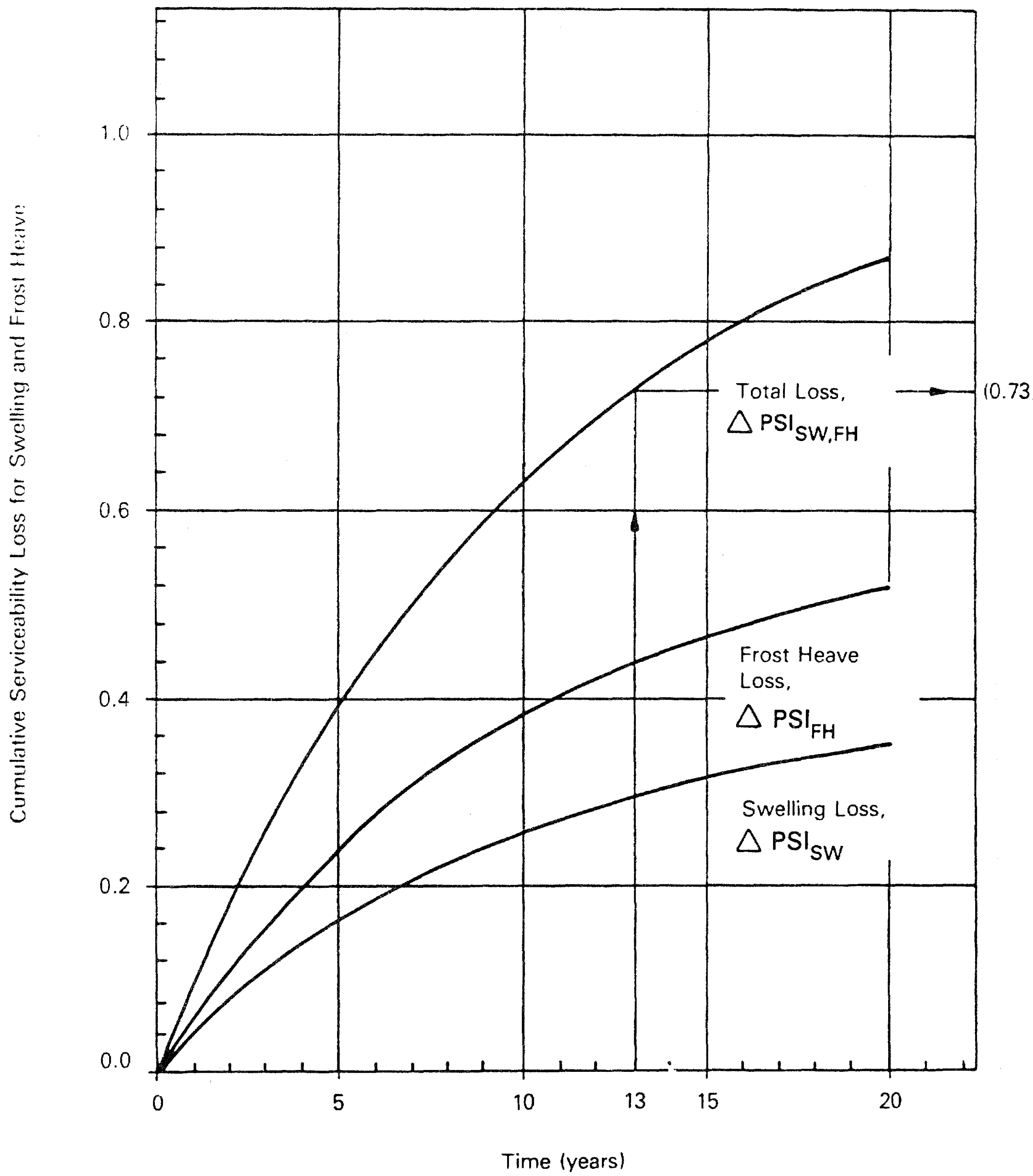


Figure 3-2.2 A Conceptual Example of the Environmental Serviceability Loss Versus Time Graph That May be Developed for a Specific Location (4).

4.1.2 Performance Criteria

The serviceability of a pavement is defined as its ability to serve the type of traffic which uses the facility. The primary measure of serviceability used by the AASHTO procedures is the Present Serviceability Index (PSI), which ranges from 0 (impassible) to 5 (perfect).

Initial and terminal serviceability indexes must be established to compute the total change in serviceability that will be input to the design equations. Initial serviceability index (p_o) is a function of pavement design and construction quality. Typical values from the AASHO Road Test were 4.2 for flexible pavements and 4.5 for rigid pavements. Terminal serviceability index (p_t) is the lowest index that will tolerated before rehabilitation, resurfacing or reconstruction becomes necessary and generally varies with the importance or functional classification of the pavement. Recommended terminal serviceability indexes are often 2.5 or higher for major highways and 2.0 to 2.5 for less important pavements.

The input required to the AASHTO flexible pavement thickness design procedure is $\Delta PSI = p_o - p_t$.

4.1.3 Determination of Materials Properties for Structural Design

As described earlier, the basis for materials characterization in the AASHTO Design Guide (4) is elastic or resilient modulus. The use of these properties in determining flexible pavement design inputs is described below.

Effective Roadbed Soil Resilient Modulus - The AASHTO flexible pavement design procedure requires the input of an effective roadbed soil resilient modulus, which is equivalent to the combined effect of all seasonal modulus values. The computation of the effective modulus is described below. This method should be used only for estimating the modulus of soils under flexible pavements that are to be designed using serviceability criteria.

1. Seasonal resilient modulus values must be determined to quantify the relative damage a pavement is subjected to during each season of the year and include this damage in the overall design. These values can be estimated in any of the following ways:

    a. Perform laboratory resilient modulus tests (AASHTO T274, see Module 2-2) on representative soil samples in stress and moisture conditions simulating those of the primary moisture seasons (i.e., those seasons during which a significantly different resilient modulus will be obtained). This will establish a laboratory relationship between resilient modulus and moisture content which can be used with estimates of in situ moisture content of the soil beneath the slab during various seasons to generate resilient modulus values for those seasons.
    b. Back calculate the resilient modulus for different seasons using deflections measured on in-service pavements.

c. Estimate "normal" or summer resilient modulus values from known relationships between resilient modulus and known soil properties (e.g., clay content, plasticity index, etc.) and use empirical relationships to estimate seasonal variations (e.g., spring thaw modulus = 20 - 30 percent of "normal" or summer modulus and frozen subgrade modulus = 20000 - 50000 psi).

2. Separate the year into time intervals during which the different seasonal moduli are effective. All of the "seasons" must be definable in terms of the selected time interval. It is suggested that the one-half month should be the shortest time interval used. Figure 3-2.3 presents a chart for estimating effective roadbed soil resilient modulus that provides for entry of seasonal roadbed soil moduli at half-month intervals.

3. The relative damage value (u_f) corresponding to each seasonal modulus must be estimated using the vertical scale or corresponding equation shown at the right of Figure 3-2.4. For example, the relative damage corresponding to a roadbed soil resilient modulus of 4000 psi is 0.51. Each damage value is entered in the appropriate box adjacent to the corresponding resilient modulus.

4. The relative damage values should all be added together and divided by the number of seasonal increments (in this case, 24) to determine the average relative damage.

5. The effective roadbed soil resilient modulus (M_R) is estimated as the value corresponding to the average relative damage on the M_R - u_f scale. Figure 3-2.4 provides an example of the effective M_R estimation process.

If the procedures described above cannot be accomplished, Figure 3-2.5 and Table 3-2.2 provide guidelines (intended for use on low-volume roads) for assigning effective roadbed soil resilient modulus values based on climate zone and relative quality of subgrade soil.

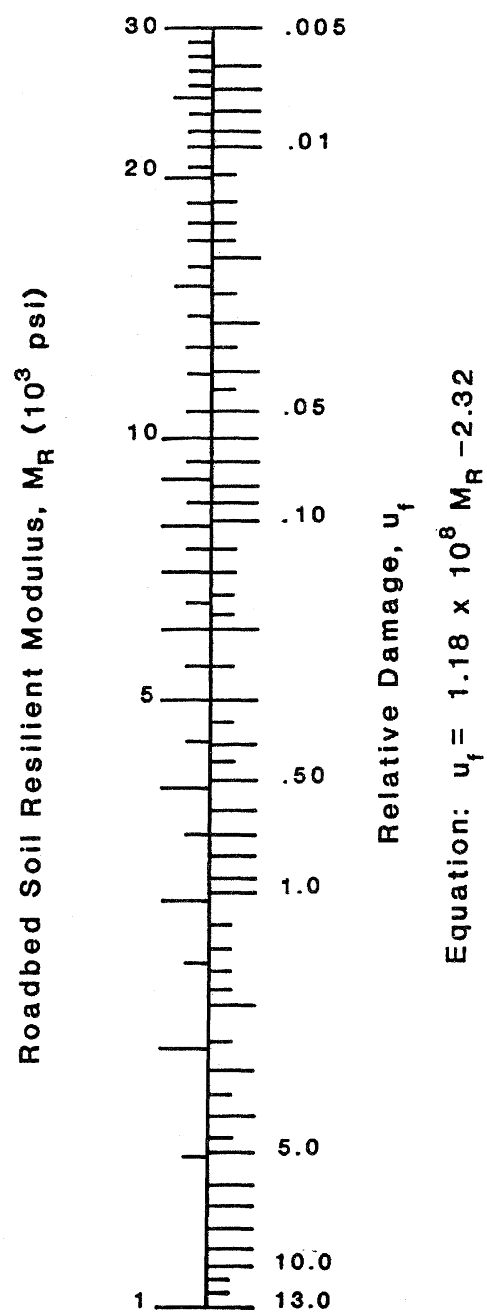
Pavement Layer Materials Characterization

Although the concept of layer coefficients is still central to the AASHTO flexible pavement design procedure, the 1986 AASHTO Design Guide relies more heavily on the determination of materials properties for the estimation of appropriate layer coefficient values. The preferred tests are the resilient modulus (AASHTO Method T274) for subbase and unbound granular materials and elastic modulus (ASTM D4123 or ASTM C469) for asphalt concrete and other stabilized materials. Details concerning these tests are presented in Block 2.

Layer Coefficients

The AASHTO flexible pavement layer coefficient (a_i) is a measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement. For example, two inches of a material with a layer coefficient of 0.20 is assumed to provide the same structural

Month	Roadbed Soil Modulus, M_R (psi)	Relative Damage, u_f
Jan.		
Feb.		
Mar.		
Apr.		
May		
June		
July		
Aug.		
Sept.		
Oct.		
Nov.		
Dec.		
	Summation: $\sum u_f =$	



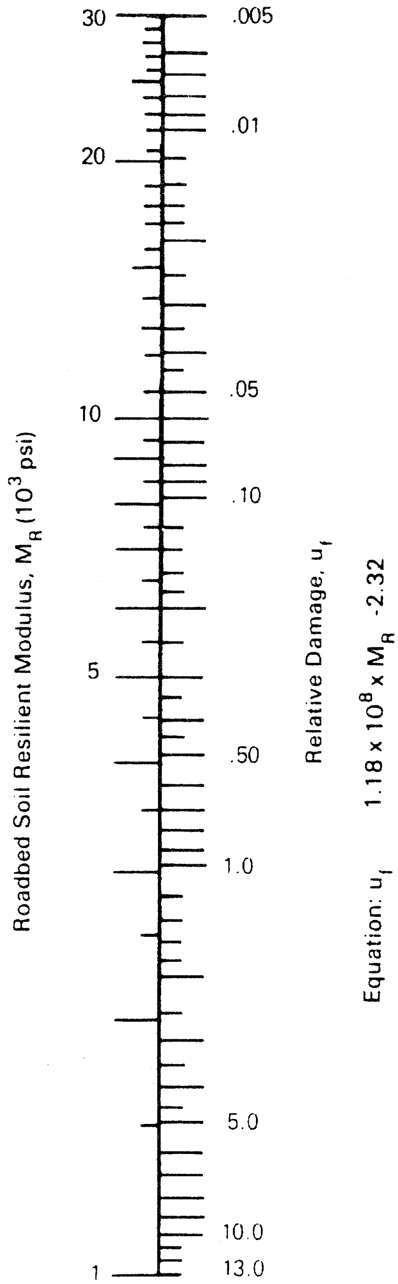
Average: $\bar{u}_f = \dfrac{\sum u_f}{n} =$ _______

Effective Roadbed Soil Resilient Modulus,

M_R (psi) = ______ (corresponds to $\bar{u}_f$)

Figure 3-2.3 Chart for Estimating Effective Roadbed Soil Resilient Modulus (4).

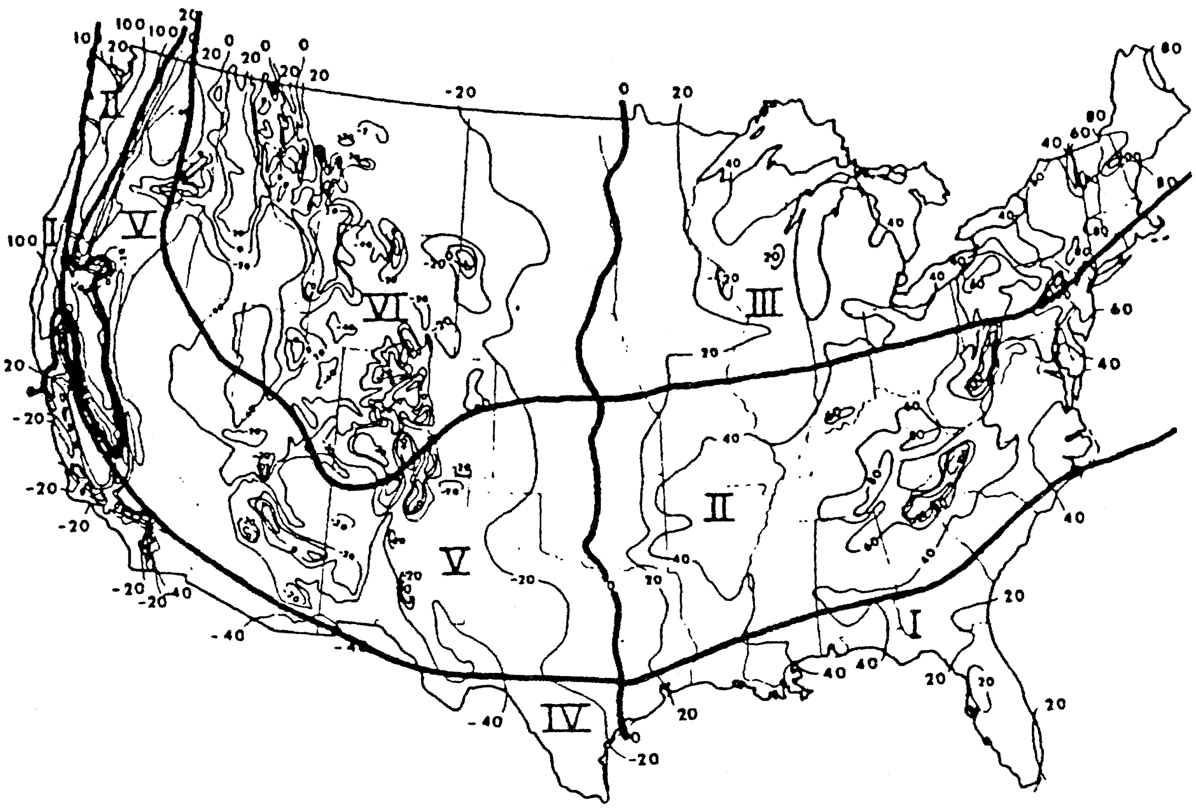
Month	Roadbed Soil Modulus, M_R (psi)	Relative Damage, u_f
Jan.	20,000	0.01
Feb.	20,000	0.01
Mar.	2,500	1.51
Apr.	4,000	0.51
May	4,000	0.51
June	7,000	0.13
July	7,000	0.13
Aug.	7,000	0.13
Sept.	7,000	0.13
Oct.	7,000	0.13
Nov.	4,000	0.51
Dec.	20,000	0.01
	Summation: Σu_f =	3.72



Average: $\bar{u}_f = \frac{\Sigma u_f}{n} = \frac{3.72}{12} = 0.31$

Effective Roadbed Soil Resilient Modulus, M_R (psi) = 5,000 (corresponds to $\bar{u}_f$)

Figure 3-2.4 Chart for Estimating Effective Roadbed Soil Resilient Modulus for Flexible Pavements Designed Using the Serviceability Criteria (4).



REGION	CHARACTERISTICS
I	Wet, no freeze
II	Wet, freeze - thaw cycling
III	Wet, hard-freeze, spring thaw
IV	Dry, no freeze
V	Dry, freeze - thaw cycling
VI	Dry, hard freeze, spring thaw

Figure 3-2.5 The Six Climatic Regions in the United States.

Table 3-2.2. Effective Roadbed Soil Resilient Modulus Values, M_R (psi), that may be Used in the Design of Flexible Pavements for Low-Volume Roads. Suggested Values Depend on the U.S. Climatic Region and the Relative Quality of the Roadbed Soil.

U.S. Climatic Region	Relative Quality of Roadbed Soil				
	Very Poor	Poor	Fair	Good	Very Good
I	2,800*	3,700	5,000	6,800	9,500
II	2,700	3,400	4,500	5,500	7,300
III	2,700	3,000	4,000	4,400	5,700
IV	3,200	4,100	5,600	7,900	11,700
V	3,100	3,700	5,000	6,000	8,200
VI	2,800	3,100	4,100	4,500	5,700

*Effective Resilient Modulus in psi

contribution as one inch of a material with a layer coefficient of 0.40. The development of the concept of layer coefficients was presented earlier in this module.

The determination of appropriate layer coefficients can be accomplished by deriving them from test roads or satellite sections (as was done at the AASHO Road Test) or by using predetermined relationships based on materials properties such as resilient or elastic modulus. A discussion of the estimation of layer coefficients for five categories of materials is presented below.

Asphalt Concrete Surface Course

Figure 3-2.6 presents a chart that can be used to estimate the structural layer coefficient of a dense-graded asphalt concrete surface course based on its elastic (resilient) modulus (E_{AC}) at $68^{o}F$. Caution is recommended in the selection of layer coefficients for asphalt concretes with modulus values exceeding 450,000 psi because their increase in stiffness is accompanied by increased susceptibility to thermal and fatigue cracking.

Granular Base Layers

Figure 3-2.7 presents a chart that may be used to estimate a structural layer coefficient for a granular base material (a_2) based on one of four different laboratory test results, including base resilient modulus, E_{BS}. The following relationship may also be used in lieu of Figure 3-2.7 to estimate the layer coefficient, a_2, for a granular base material from its elastic (resilient) modulus, E_{BS} (6):

$$a_2 = 0.249(\log_{10}E_{BS}) - 0.977$$

Granular Subbase Layers

Figure 3-2.8 presents a chart that may be used to estimate a structural layer coefficient for a granular subbase material (a_3) based on one of four different laboratory test results, including base resilient modulus, E_{SB}. The following relationship may also be used in lieu of Figure 3-2.8 to estimate the layer coefficient, a_3, for a granular base material from its elastic (resilient) modulus, E_{SB} (6):

$$a_3 = 0.227(\log_{10}E_{SB}) - 0.839$$

Cement-Treated Bases

Figure 3-2.9 provides a chart that may be used to estimate the structural layer coefficient, a_2, for a cement-treated base material from either its elastic modulus, E_{BS}, or alternatively, its 7-day unconfined compressive strength (ASTM D1633).

Bituminous-Treated Bases

Figure 3-2.10 presents a chart that can be used to estimate the structural layer coefficient, a_2, for a bituminous-treated base material from either its elastic modulus, E_{BS}, or alternatively, its Marshall stability (AASHTO T245, ASTM D1559).

4.1.4 Pavement Structural Characteristics

Drainage

The AASHTO flexible pavement design procedure provides a means to adjust layer coefficients to take into account the effects of certain levels

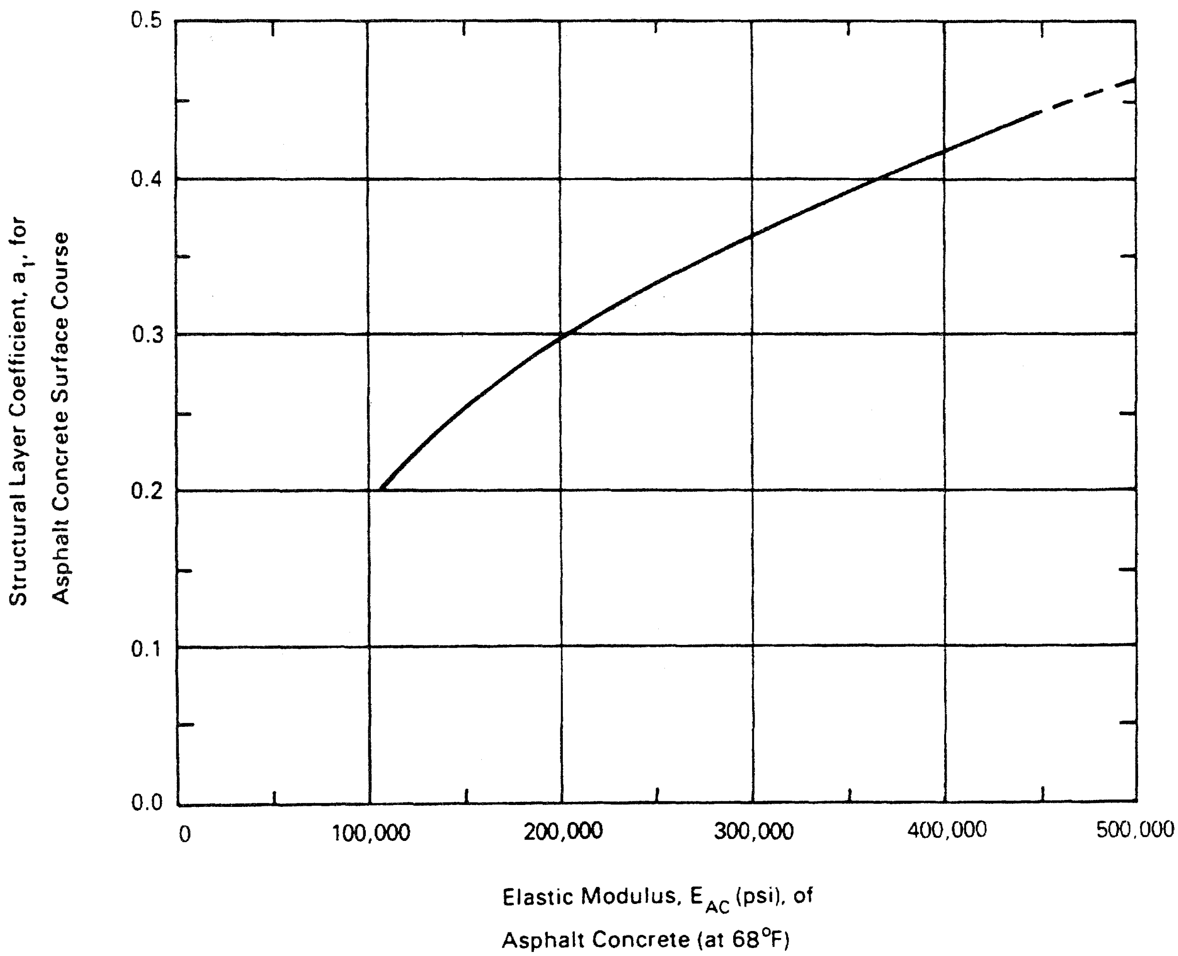
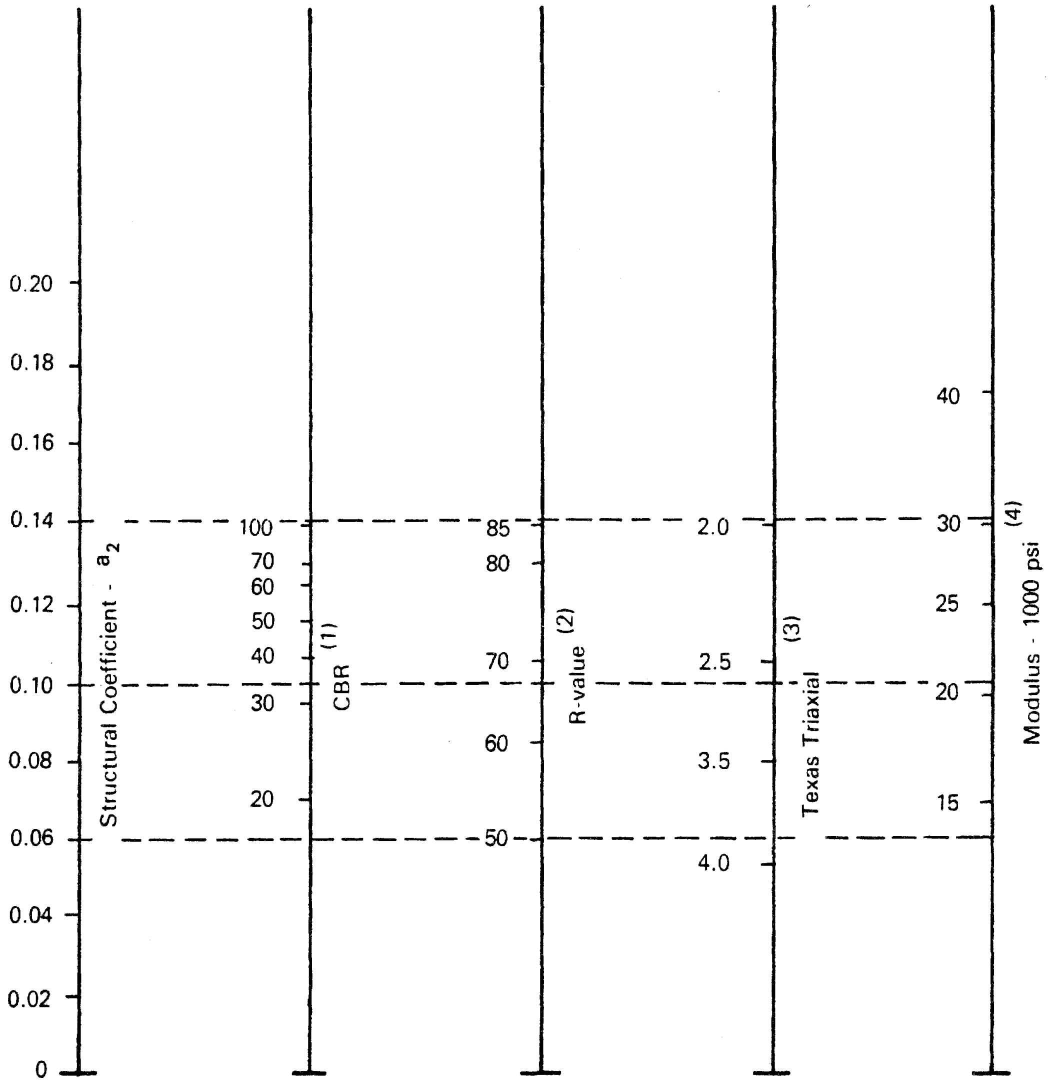
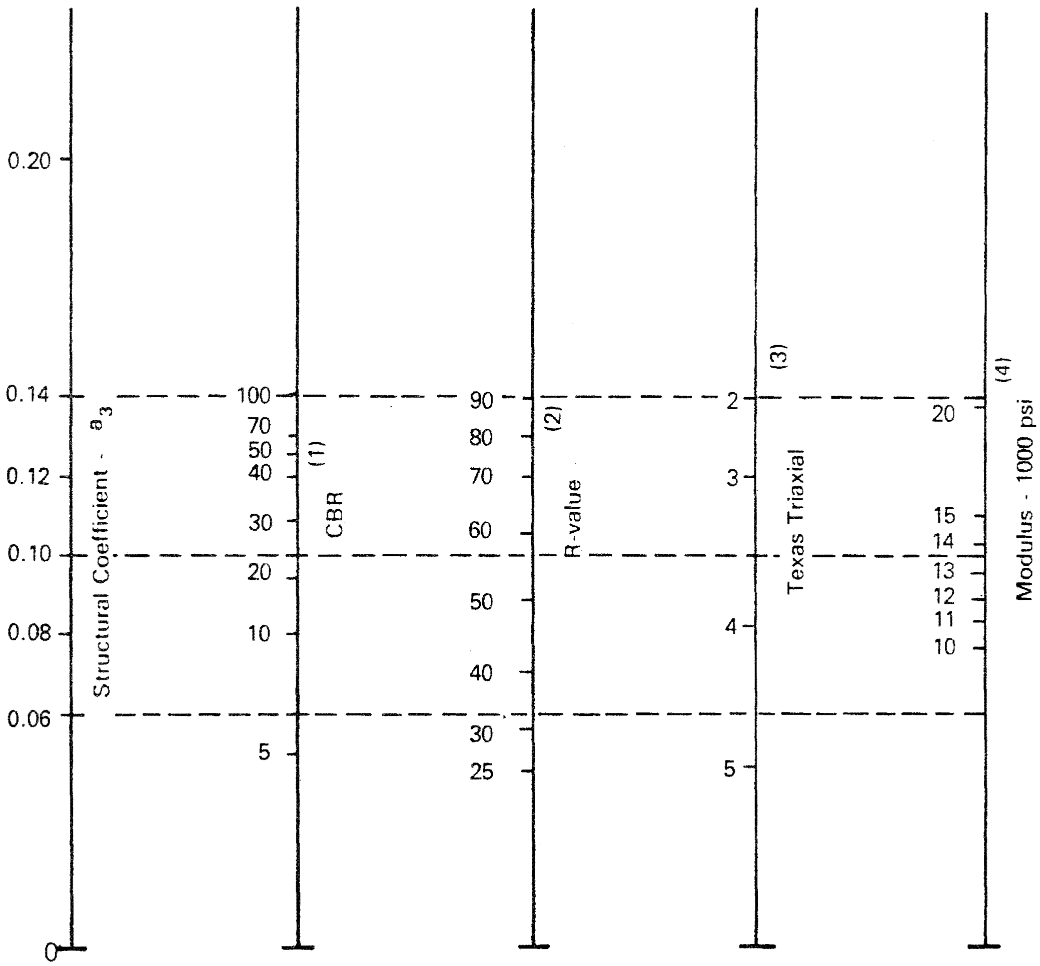


Figure 3-2.6 Chart for Estimating Structural Layer Coefficient of Dense-Graded Asphalt Concrete Based on the Elastic (resilient) Modulus (4).



(1) Scale derived by averaging correlations obtained from Illinois.
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.
(4) Scale derived on NCHRP project (3).

Figure 3-2.7 Variation in Granular Base Layer Coefficient (a_2) with Various Base Strength Parameters (4).



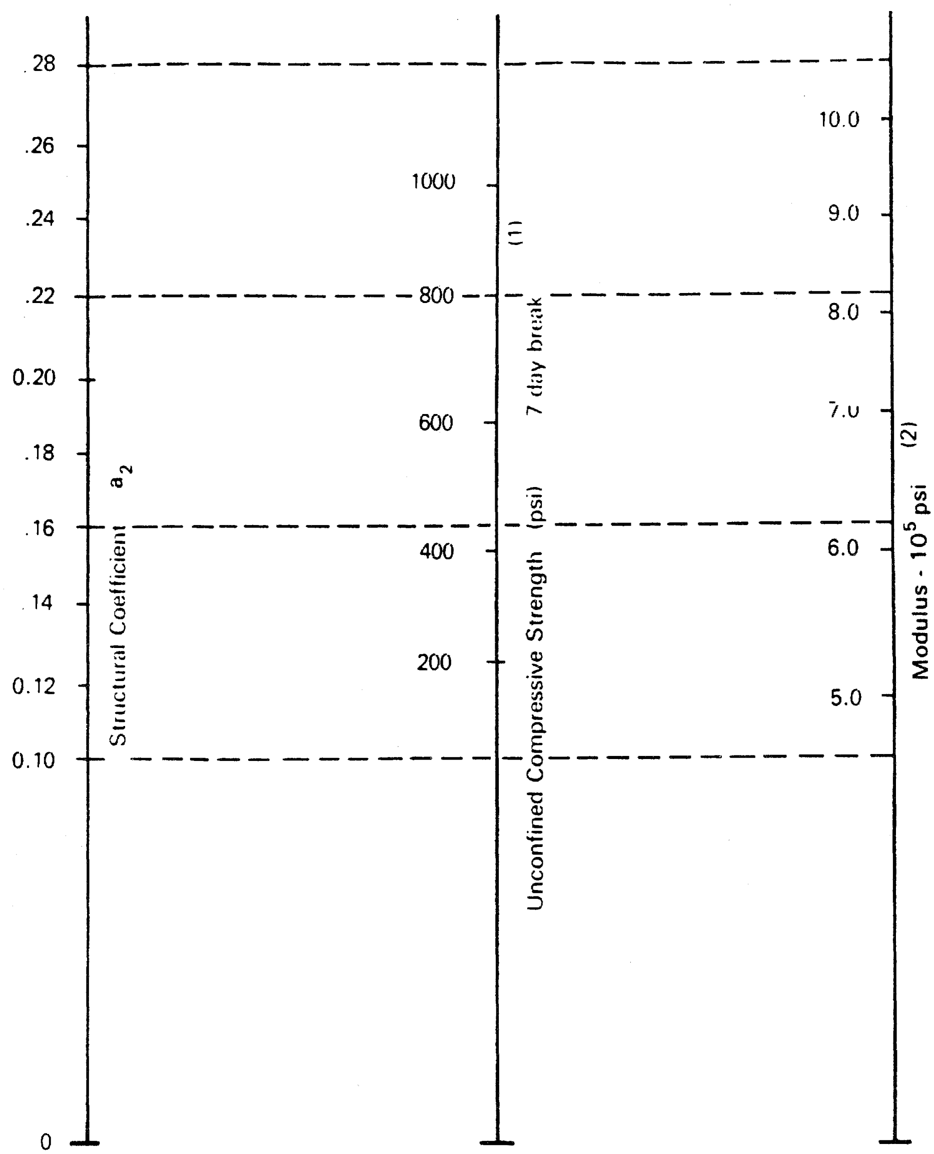
(1) Scale derived from correlations from Illinois.

(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.

(3) Scale derived from correlations obtained from Texas.

(4) Scale derived on NCHRP project *(3)*.

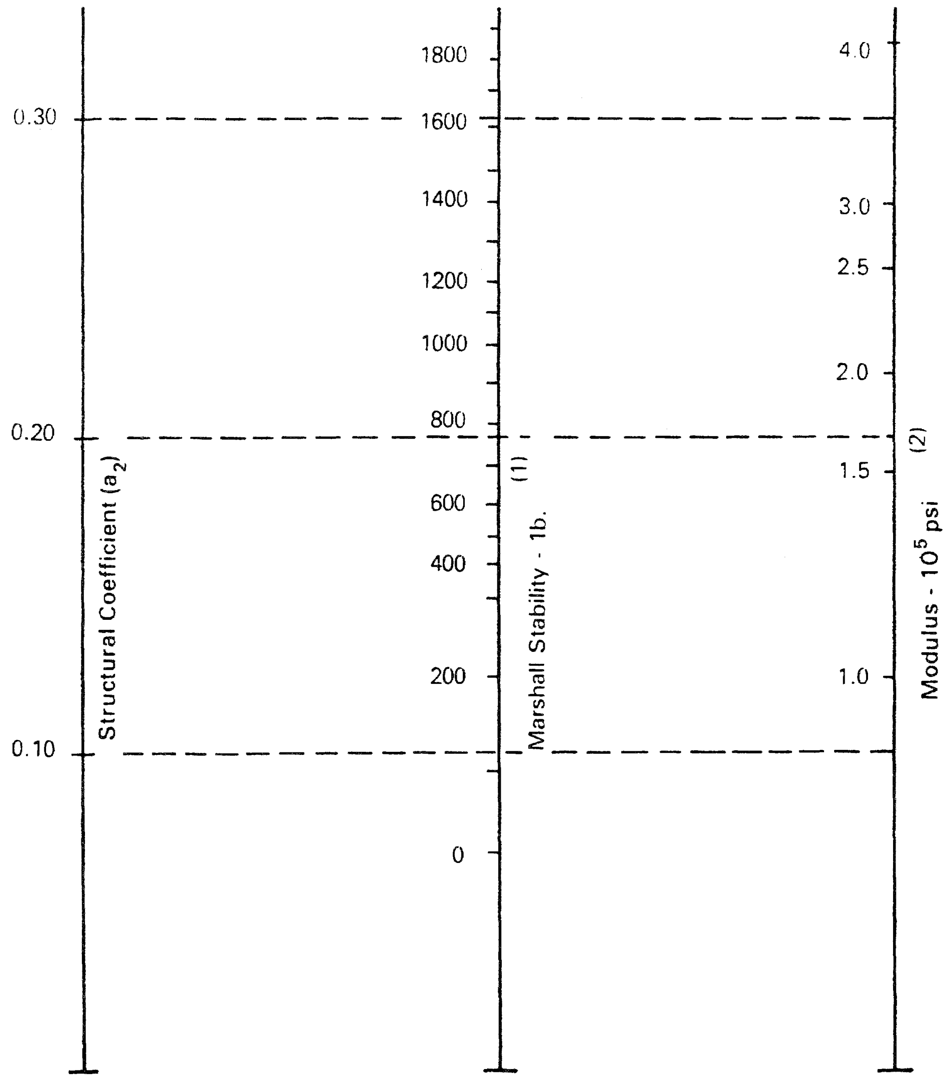
Figure 3-2.8 Variation in Granular Subbase Layer Coefficient (a_3) with Various Subbase Strength Parameters (4).



(1) Scale derived by averaging correlations from Illinois, Louisiana and Texas.

(2) Scale derived on NCHRP project

Figure 3-2.9 Variation in "a" for Cement-Treated Bases with Base Strength Parameter (4).



(1) Scale derived by correlation obtained from Illinois.

(2) Scale derived on NCHRP project

Figure 3-2.10 Variation in a_2 for Bituminous-Treated Bases with Base Strength Parameter (4).

of drainage on pavement performance. Guidance concerning the design or effectiveness of various drainage approaches is not provided; the design engineer must identify the level or quality of drainage that is achieved under a specific set of drainage conditions.

The following definitions were described in Module 2-4 for various levels of drainage quality:

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	water will not drain

The effect of drainage of all untreated layers below the surface is considered by multiplying the layer coefficients, a_i, by a modifying factor, m_i. This factor was shown in Module 2-4 and can be obtained from Table 3-2.3 and is a function of the drainage characteristics of the roadbed soil (as categorized above) and the amount of time the soil is in a saturated condition. The structural number equation modified for drainage becomes:

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

where:

a_i = layer coefficient of layer i
D_i = thickness of layer i, inches
m_i = drainage modifying factor for layer

The drainage conditions at the AASHO Road Test are assumed to be "fair" and the m_i values there are assumed to be 1.0, regardless of the material. It should be noted that these values are assumed because the structural models should not require adjustment for the conditions at the Road Test. However, these same materials would probably receive drainage modifying factors of less than 1.0 for a new construction project, and the designer should select appropriate values to reduce the possibility of a poor design.

The values in Table 3-2.3 apply only to the effects of drainage on untreated base and subbase layers. Although the effects of drainage are certainly beneficial for stabilized layers as well, the effects (for flexible pavements) are not so pronounced as for unbound materials.

4.2 Computation of Required Pavement Thickness

The AASHTO flexible pavement design process can be accomplished using the design inputs and design equations discussed in this module. These equations can be solved manually, using a series of nomographs, or recently-developed computer software (7). The complexity of the design procedure can make the manual solution a tedious process. Although the nomographs simplify the process, they include some inherent assumptions that make their solutions somewhat less precise than those provided by the manual and computer solutions. The computerized approach allows easy consideration of all design factors (including stage construction, frost heave, swelling soil and cost considerations) and provides accurate solutions to the design equations. The use of the nomographs is described in this module because it is anticipated that they will be widely used. The DNPS86 AASHTO computer program will be described separately in this module.

Table 3-2.3. Recommended m_i Values for Modifying Structural Layer Coefficients of Untreated Base and Sub-Base Materials in Flexible Pavements (4).

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.40 - 1.35	1.35 - 1.30	1.30 - 1.20	1.20
Good	1.35 - 1.25	1.25 - 1.15	1.15 - 1.00	1.00
Fair	1.25 - 1.15	1.15 - 1.05	1.00 - 0.80	0.80
Poor	1.15 - 1.05	1.05 - 0.80	0.80 - 0.60	0.60
Very Poor	1.05 - 0.95	0.95 - 0.75	0.75 - 0.40	0.40

The basic AASHTO design process for flexible pavements begins with the determination of required structural number (SN) based on the level of traffic. Trial pavement designs are then identified by using different layer thicknesses that provide the required structural number, meet minimum layer thickness criteria, and provide adequate protection for underlying materials. The associated performance period is then corrected for losses of serviceability due to environmental considerations. Stage construction options should then be considered to allow planned rehabilitation for environmental or economic reasons. Finally, life-cycle cost economic analyses must be conducted to compare the alternate pavement designs and assist in the selection of the final pavement design.

4.2.1 Determination of the Required Structural Number

Figure 3-2.11 presents the recommended nomograph for determining the design structural number (SN) required for specific conditions. This nomograph solves the following equation:

$$\log_{10} W_{18} = Z_R * S_o + 9.36 * \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\dfrac{\Delta PSI}{4.2 - 1.5}\right]}{0.40 + \dfrac{1094}{(SN+1)^{5.19}}} + 2.32 * \log_{10} M_R - 8.07$$

The required inputs are:

1. The estimated future traffic, W_{18}, for the performance period.
2. The reliability, R, which assumes that average values are used for all inputs.
3. The overall standard deviation, S_o.
4. The effective roadbed soil resilient modulus, M_R.
5. The design serviceability loss, $\Delta PSI = p_o - p_t$.

4.2.2 Selection of Trial Pavement Thickness Designs

Once the design structural number for an initial pavement structure has been determined, the designer must identify a set of pavement layer thicknesses which will provide the required load-carrying capacity that corresponds to the design structural number. The following equation, which was presented earlier in this module, provides the means for converting the structural number into actual thicknesses of surfacing, base and subbase materials:

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

where:

a_i = layer coefficient of layer i
D_i = thickness of layer i, inches
m_i = drainage modifying factor for layer

NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R * S_O + 9.36 * \log_{10}(SN + 1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2 - 1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 * \log_{10} M_R - 8.07$$

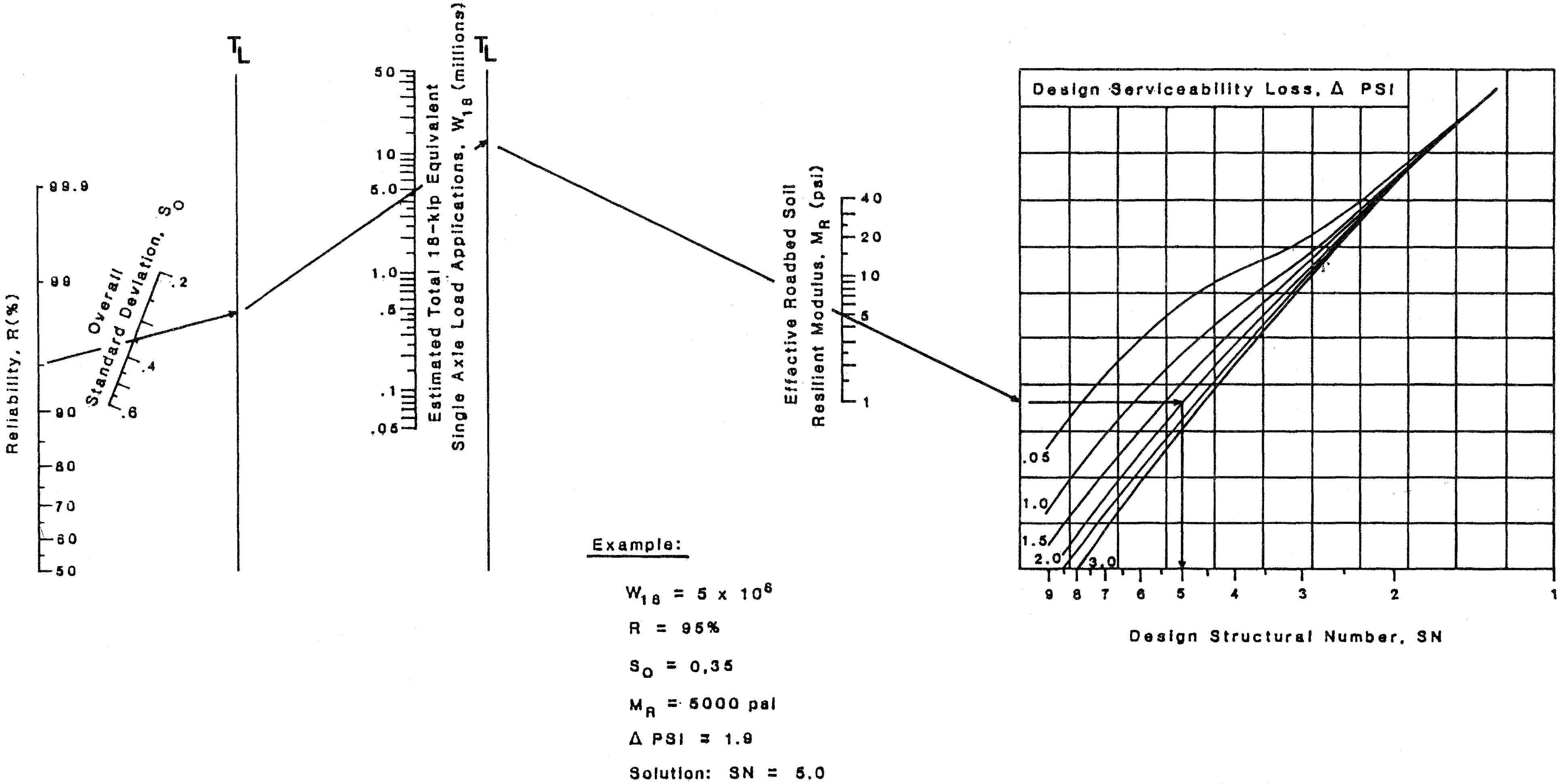


Figure 3-2.11 AASHTO Flexible Pavement Thickness Design Nomograph (4).

This equation does not have a single unique solution; there are many combinations of layer thicknesses that can be used to achieve a given structural number. There are, however, several design, construction and cost constraints that can be applied to reduce the number of possible layer thickness combinations and to avoid the possibility of constructing an impractical design. Some of these constraints are presented below.

4.2.3 Layered Design Analysis

Flexible pavement structures are layered systems and should be designed accordingly. Each unbound or aggregate layer must be protected from excessive vertical stresses that could result in permanent deformation. This requires that minimum thickness values be observed depending on the traffic requirements. The procedure to accomplish this is shown in Figure 3-2.12 and is described below.

The design nomograph presented in Figure 3-2.11 can be used to determine the design structural numbers required for the protection of any unbound layer by substituting the resilient modulus of that layer for the roadbed resilient modulus in the nomograph. The minimum required surface course thickness, D_1, is then determined by dividing the structural number required to protect the base course by the layer coefficient of the surface course (SN_1/a_1). The selected surface course thickness, D^*_1, must be greater than or equal to this minimum thickness.

$$D^*_1 \geq SN_1/a_1 = D_1$$

The actual structural number provided by the selected surface course thickness for the protection of the base course is computed as:

$$SN^*_1 = a_1D^*_1$$
$$\geq SN_1$$

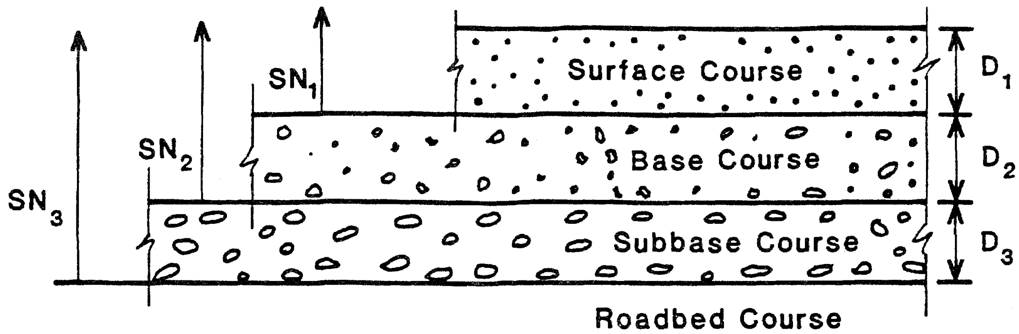
The minimum required base course thickness is determined in a similar manner. The structural number provided by the thickness of surface provided ($a_1D^*_1 = SN^*_1$) is subtracted from the structural number required to protect the subbase course, SN_2. This quantity is divided by the product of the base course layer coefficient and drainage modifying factor to determine the minimum required base course thickness, D_2. The selected base course thickness, D^*_2, must be greater than D_2.

$$D^*_2 \geq (SN_2 - SN^*_1)/a_2m_2$$

The actual structural number provided by the surface and base course for the protection of the subbase course is computed as:

$$SN^*_1 + SN^*_2 = a_1D^*_1 + a_2m_2D^*_2$$
$$\geq SN_2$$

Finally, the minimum required subbase thickness is determined. The structural number provided by the selected surface and base course thicknesses (SN^*_2) is subtracted from the structural number required to protect the roadbed soil, SN_3. This quantity is divided by the product of



$$D^*_1 \geq \frac{SN_1}{a_1}$$

$$SN^*_1 = a_1 D^*_1 \geq SN_1$$

$$D^*_2 \geq \frac{SN_2 - SN^*_1}{a_2\, m_2}$$

$$SN^*_1 + SN^*_2 \geq SN_2$$

$$D^*_3 \geq \frac{SN_3 - (SN^*_1 + SN^*_2)}{a_3\, m_3}$$

1) a,D,m, and SN are as defined in the text and are minimum required values.

2) An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value.

Figure 3-2.12 Design of Flexible Pavement Layer Thicknesses Using Layer Analysis Concepts.

the subbase layer coefficient and drainage modifying factor to determine the minimum required subbase thickness, D_3. The selected base course thickness, D^*_3, must be greater than D_3.

$$D^*_3 \geq [SN_3 - (SN^*_1 + SN^*_2)]/a_3m_3$$

The actual structural number provided by the three upper layers for the protection of the roadbed soil is computed as:

$$SN^*_1 + SN^*_2 + SN^*_3 = a_1D^*_1 + a_2m_2D^*_2 + a_3m_3D^*_3$$
$$\geq SN_3$$

This approach provides trial designs that meet the structural requirements for the protection of each layer in the pavement structure. This procedure should not be applied to determine the required layer thickness above materials having a resilient modulus of greater than 40,000 psi. Layer thickness above such "high modulus materials" should be established based on cost-effectiveness and minimum practical thickness considerations.

An example of thickness selection follows:

Given:

Material	Modulus (psi)	a_i	m_i
AC	400,000	0.42	----
Crushed Stone Base	30,000	0.14	0.80
Gr. Subbase	14,000	0.10	0.70
Roadbed Soil	5,000	----	----

Design Reliability = 90%

Overall Standard Deviation = 0.35

W_{18} = 10 million

Design Serviceability Loss = 2.0

Computations:

From Figure 3-2.12:

SN_1 (protect base) = 2.8
SN_2 (protect subbase) = 3.8
SN_3 (protect roadbed) = 5.4

Minimum Required Surface Thickness = D_1
= 2.8/0.42 = 6.7 in

Use D^*_1 = 7.0 in AC surface
SN^*_1 = 7.0*0.42 = 2.94 > 2.8 -- ok

Minimum Required Base Thickness = D_2
= (3.8 - 2.94)/0.14 = 6.1 in

Use D^*_2 = 7.0 in granular base
SN^*_2 = 7.0*0.14 = 0.98

$SN^*_1 + SN^*_2 = 2.94 + 0.98 = 3.92 > 3.8$ -- ok

Minimum Required Subbase Thickness = D_3

$$= (5.4 - 3.92)/0.10 = 14.8 \text{ in}$$

Use D^*_3 = 15.0 in granular subbase

$SN^*_3 = 15.0*0.10 = 1.50$

$$SN^*_1 + SN^*_2 + SN^*_3 = 2.94 + 0.98 + 1.50$$
$$= 5.42 > 5.40 \text{ -- ok}$$

4.2.4 Stability and Constructability

It is generally impractical and uneconomical to place layers of material that are less than some minimum thickness. Also, traffic considerations may dictate the placement of certain minimum layer thicknesses for stability and cohesion. The following values are suggested minimum thicknesses for surface and base layers for various traffic conditions, which should be modified for local conditions (1):

	Minimum Thickness (inches)	
Traffic, ESAL's	Asphalt Concrete	Aggregate Base
Less than 50,000	1.0 (or surface treatment)	4
50,000 - 150,000	2.0	4
150,000 - 500,000	2.5	4
500,000 - 2,000,000	3.0	6
2,000,000 - 7,000,000	3.5	6
Greater than 7,000,000	4.0	6

Individual agencies should also establish the effective thicknesses and layer coefficients for surface treatments. While the thicknesses of such layers are usually negligible in computing structural numbers, they may have large effects on base and subbase properties because they reduce the entry of surface water into the pavement structure.

4.2.5 Cost Considerations in Selection of Layer Thicknesses

After minimum layer thicknesses have been established according to the procedures outlined above and checked against construction and maintenance constraints, the initial cost of the pavement can be minimized to provide one alternate design for each combination of materials being considered.

One approach to minimizing the pavement materials cost is to calculate the cost per unit contribution to the structural number for each layer in the pavement system. This number is the unit cost (typically expressed in dollars per square yard-inch or similar units) to achieve a contribution of 1.0 to the structural number. It can be calculated as the Cost per unit of effective strength which equals the Unit Material Cost/$a_i m_i$

As an example, the following numbers can be obtained:

Material	$/sq.yd.-in	a_i	m_i	$/Unit Structural Number
Crushed Stone	$0.40	0.16	0.80	$3.12
Gravel	$0.32	0.10	0.95	$3.37
Asphalt Concrete	$1.50	0.37	1.00	$4.05

Another way to consider this information is to take the reciprocal of these costs, which tells the designer how much structural number can be achieved per dollar spent for each material.

Material	$/Unit Structural Number	Structural Number/$
Crushed Stone	$3.12	0.32
Gravel	$3.37	0.30
Asphalt Concrete	$4.05	0.25

The most cost-effective design for a given set of materials will maximize the thickness of the material that provides the most structural contribution for each dollar spent (in this case, crushed stone) and minimize the thickness of the least "structurally cost-effective" material (in this case, asphalt concrete) within the thickness constraints described earlier.

4.3 Consideration of Serviceability Losses Due to Environmental Sources

Roadbed swelling and frost heave are important environmental considerations because of their potential effect on the rate of serviceability loss. If either swelling or frost heave are to be considered for their effects on loss of serviceability or need for future overlays (stage construction), the following procedure should be used to determine the performance period that will be achieved considering the effects of both traffic and environmental effects (refer to Figure 3-2.13):

1. Select an appropriate structural number for the initial pavement structure using the procedures described previously. Because the structural number selected has very little effect on the loss of serviceability due to environmental causes, it is recommended that the initial structural number be no more than that which would be required for conditions with no swelling or frost heave problems. For the example shown in Figure 3-2.13, the initial structural number was assumed to be 4.4 ($R = 0.95$, $M_R = 5000$, $p_o = 4.4$, $p_t = 2.5$, traffic = 5 million 18-kip ESAL, 15 year performance period). Any structural number less than 4.4 may be an appropriate starting point (depending on stage construction considerations) provided minimum performance period considerations are not violated.

2. Select a trial performance period that might be expected under the swelling/frost heave conditions anticipated and enter this in Column 2. A performance period of 13 years was estimated in Figure 3-2.13. This number should be less than the performance period expected for no frost or swelling conditions. Shorter periods should be estimated for more severe environmental conditions.

3. Use the graph of cumulative environmental serviceability loss (prepared as described earlier, see Figure 3-2.12) to estimate the total serviceability loss that could be expected due to environmental conditions during the performance period selected in Column 2 and enter this value in Column 3.

4. Subtract the environmental serviceability loss (Column 3) from the desired total serviceability loss (4.4 - 2.5 = 1.9 is used in this example) to establish the corresponding traffic serviceability loss and enter the result in Column 4.

Figure 3-2.13 Example of Process Used to Predict the Performance Period of an Initial Pavement Structure Considering Swelling and/or Frost Heave (4).

Initial SN 4.4
Maximum Possible Performance Period (years) 15
Design Serviceaility Loss, Δ PSI = $p_o - p_t$ = 4.4 - 2.5 = 1.9

(1) Iteration No.	(2) Trial Performance Period (Years)	(3) Total Serviceability Loss Due to Swelling and Frost Heave $\Delta PSI_{SW,FH}$	(4) Corresponding Serviceability Loss Due to Traffic ΔPSI_{TR}	(5) Allowable Cumulative Traffic (18-kip ESAL)	(6) Corresponding Performance Period (Years)
1	13.0	0.73	1.17	2.0×10^6	6.3
2	9.7	0.63	1.27	2.3×10^6	7.2
3	8.5	0.56	1.34	2.6×10^6	8.2

Column No.	Description of Procedures
2	Estimated by the designer (Step 2).
3	Using estimated value from Column 2 with Figure 2.2, the total serviceability loss due to swelling and frost heave is determined (Step 3).
4	Subtract environmental serviceability loss (Column 3) from design total serviceability loss to determine corresponding serviceability loss due to traffic.
5	Determined from Figure 3.1 keeping all inputs constant (except for use of traffic serviceability loss from Column 4) and applying the chart in reverse (Step 5).
6	Using the traffic from Column 5, estimate net performance period from Figure 2.1 (Step 6).

5. Use the design nomograph to estimate the allowable cumulative 18-kip ESAL traffic that corresponds to the serviceability loss entered in Column 4. Enter this value in Column 5. Note that the reliability, effective roadbed soil resilient modulus and initial structural number used in determining the initial structural number must be used here as well.

6. Estimate the number of years it will take for the projected traffic to accumulate the ESAL entered in Column 5. This projected performance period should be entered in Column 6.

7. Compare the trial performance period in Column 2 with the projected performance period in Column 6. If the difference is greater than 1 year, average the two and use this number for the trial performance period in the next iteration. If the difference is less than 1 year, convergence is reached and the average is said to be the predicted performance period of the initial pavement structure corresponding to the selected initial structural number. For this example, convergence was reached after three iterations and the predicted performance period is about 8 years.

5.0 SENSITIVITY ANALYSES

The nomographs described in the text can be used to effectively design any flexible pavement situation encountered in this course, and in hopefully practice. With the increased availability of microcomputer programs the ability to reduce mundane calculations and consider more options in a design should provide the design engineer with the ability to examine more alternatives and develop a more comprehensive design. The DNPS86 computer program developed by AASHTO (10) will be used to evaluate the influence of design variables on the final thicknesses, and will also be used in the workshop problems.

5.1 DNPS86

The AASHTO computer program to implement the AASHTO Design Procedure presents screens to the user that develop the data required for a pavement design. As each screen is completed, the next screen is presented and when all input data has been entered, the thicknesses can be calculated. Figure 3-2.14 is the copyright screen. The next screen activated is shown in Figure 3-2.15 which calls for general design input requirements. Figure 3-2.16 contains the input screen for the effective resilient modulus calculations described in Module 2-2. Up to 24 seasonal values can be input for the calculation. Figure 3-2.17 shows the next two screens which allow for selection of pavement type, and input of reliability, serviceability, and frost heave data.

The traffic inputs are shown in Figure 3-2.18. This screen has several limitations which were described in detail in Module 2-6. The calculation scheme illustrated in Figure 3-2.18 does not allow for separate consideration of growth rates for the different vehicle classifications. ESALCALC should be used to determine traffic, and the numbers on this input screen should be selected to produce the same design ESAL values. Figure 3-2.19 shows the input screen for layer structural coefficient values and the associated costs for materials and maintenance. The structural layer coefficients are used in

AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
Version 1 - September 1986

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000000 00  00 000000 000000  000000 000000
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000000 00  00 00     000000  000000 000000
00000  00  00 00      0000    0000   0000
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American Association of State Highway
and Transportation Officials

444 N. Capitol Street, N.W., Suite 225
Washington, D.C. 20001

Press Any Key to Continue ...

Figure 3-2.14 Copyright Screen for DNPS86 AASHTO Program.

DNPS86 (1) No.

* * * GENERAL DESIGN INPUT REQUIREMENTS * * *

ANALYSIS PERIOD (YEARS)	20.0
DISCOUNT RATE (PERCENT)	0.00
NUMBER OF TRAFFIC LANES (ONE DIRECTION)	1
LANE WIDTH (FEET)	12.00
COMBINED WIDTH OF SHOULDERS (FEET, ONE DIRECTION)	0.00

Figure 3-2.15 General Design Input Screen.

* * * ROADBED SOIL RESILIENT MODULI * * *

Season No.	Resilient Modulus (psi)	Season No.	Resilient Modulus (psi)
1	5000	13	0
2	0	14	0
3	0	15	0
4	0	16	0
5	0	17	0
6	0	18	0
7	0	19	0
8	0	20	0
9	0	21	0
10	0	22	0
11	0	23	0
12	0	24	0

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

Figure 3-2.16 Input Screen for Effective Resilient Modulus.

DNPS86 (1) No.

ROAD SURFACE
(P)aved or (A)ggregate . . . . P

DNPS86 (1) No.

* * * DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS * * *

DESIRED LEVEL OF RELIABILITY (PERCENT) 90.00

DESIGN TERMINAL SERVICEABILITY 2.50

ROADBED SOIL SWELLING AND/OR FROST HEAVE
Consider? (Y)es or (N)o . . . . N

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

Figure 3-2.17 Pavement Selection and Reliability Input Screens.

* * * FLEXIBLE PAVEMENT DESIGN INPUTS * * *

PERFORMANCE PERIOD FOR INITIAL PAVEMENT (YEARS) .	20.0
SERVICEABILITY INDEX AFTER INITIAL CONSTRUCTION .	4.50
TRAFFIC	
Growth Rate (percent per year)	0.00
(S)imple or (C)ompound Growth	C
Initial Yearly 18-kip ESAL (both directions) .	100000
Directional Distribution Factor (percent) . . .	50
Lane Distribution Factor (percent)	100
Calculated Total 18-kip ESAL During the Analysis Period (in the design lane)	1000000
OVERALL STANDARD DEVIATION (LOG REPETITIONS) . .	0.490

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

Figure 3-2.18 Traffic Input Screen.

DNPS86 (1) No.

* * * ADDITIONAL FLEXIBLE PAVEMENT DESIGN INPUTS * * *
AND ASSOCIATED COSTS

PAVEMENT LAYER CHARACTERISTICS, MATERIAL PROPERTIES & COSTS

No.	Description	Spcfd Thick (in.)	Layer Coef	Elastic Modulus (psi)	Drain Coef	Unit Cost ($/CY)	Salv Value (%)
1		0.00	0.00	0	1.00	0.00	0
2		0.00	0.00	0	1.00	0.00	0
3		0.00	0.00	0	1.00	0.00	0
4		0.00	0.00	0	1.00	0.00	0
5		0.00	0.00	0	1.00	0.00	0

OTHER CONSTRUCTION RELATED COSTS

Shoulders, If Not Full Strength ($/linear ft) .	0.00
Drainage ($/linear foot)	0.00
Mobilization and Other Fixed Costs ($/lin ft) .	0.00

MAINTENANCE COST

Initial Year Costs Begin to Accrue	0
Yearly Increase ($/lane mile/year)	0.00

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

Figure 3-2.19 Structural Layer Coefficients and Material Costs.

selecting thicknesses according to the procedure outlined in this module, and for calculating total costs and performing simple cost comparisons.

When all screens have been filled, the final menu screen shown in Figure 3-2.20 will appear. This screen allows the analysis to be performed, results saved or printed, and values to be changed for running comparative designs. A typical output is shown in Figure 3-2.21

5.2 Results

The following table of design inputs was used to determine the sensitivity of the AASHTO flexible pavement thickness design procedure (computer program solutions) to effective roadbed soil resilient modulus, terminal serviceability, overall standard deviation, design reliability and initial traffic.

Design Input	Low Value	Nominal Value	High Value(s)
Initial Annual Two-Way Design Traffic (ESAL*10^6)	1.33	2.67	4
Effective Roadbed Soil Resilient Modulus (psi)	2500	5000	10000
Terminal Serviceability	2.0	2.5	3.0
Design Reliability	50 and 70	90	98
Overall Standard Deviation	0.30	0.45	0.60

The following design inputs were held constant throughout analyses:

Design and analysis periods . . . . . . 20 years
Environmental effects . . . . none
Initial serviceability index 4.2
Truck traffic growth . . . . 4% compounded annually
Traffic directional distribution factor . 50%
Truck lane distribution factor 70%
Layer coefficients:
- AC 0.36
- Base 0.10
- Subbase 0.10

Drainage modifying factors . . . . 1.00 for all layers
Modulus values:
- AC . . . . 300,000 psi
- Base . . . . 25,000 psi
- Subbase . . . . 13,500 psi

The following table summarizes the combinations of design inputs that were entered into the AASHTO computer design program and the pavement thickness designs that resulted. All inputs were varied one at a time, holding all other inputs constant at their nominal values. These results are presented graphically in Figures 3-2.22 through 3-2.26.

* * * PERFORM ANALYSIS, PRINT RESULTS OR EXIT * * *

OPTIONS

1. Perform Analysis
2. Perform Analysis and Print Results
3. Print Previous Results
4. Return to Edit Session
5. Exit

Enter Desired Option . . . .

Figure 3-2.20 Selection Menu.

```
DNPS86 (1)                                                        No.
        * * *  SOLUTION FOR INPUT DATA FILE: test-ff           * * *

 FLEXIBLE PAVEMENT STRUCTURAL DESIGN           LIFE CYCLE COSTS ($/SY)

   Performance Life (yrs)                        Initial Pavement
   18-kip ESAL Repetitions                         Construction
                                                   Maintenance
   Layer        Layer          Required            Salvage Value
    No.      Description       Thickness
                               (inches)          First Overlay
     1                                             Construction
     2                                             Maintenance
     3                                             Salvage Value
     4
     5                                           Second Overlay
                                                   Construction
 DESIGN FOR PROJECTED FUTURE OVERLAY(S)            Maintenance
                                                   Salvage Value
                          First      Second                       ----------
   Overlay Type          (none)      (none)      Net Present Value
   Req'd Thick (in)
   Perf Life (yrs)
   18-kip ESAL Reps                              Press Any Key to Continue ...
```

Figure 3-2.21 Output Screen.

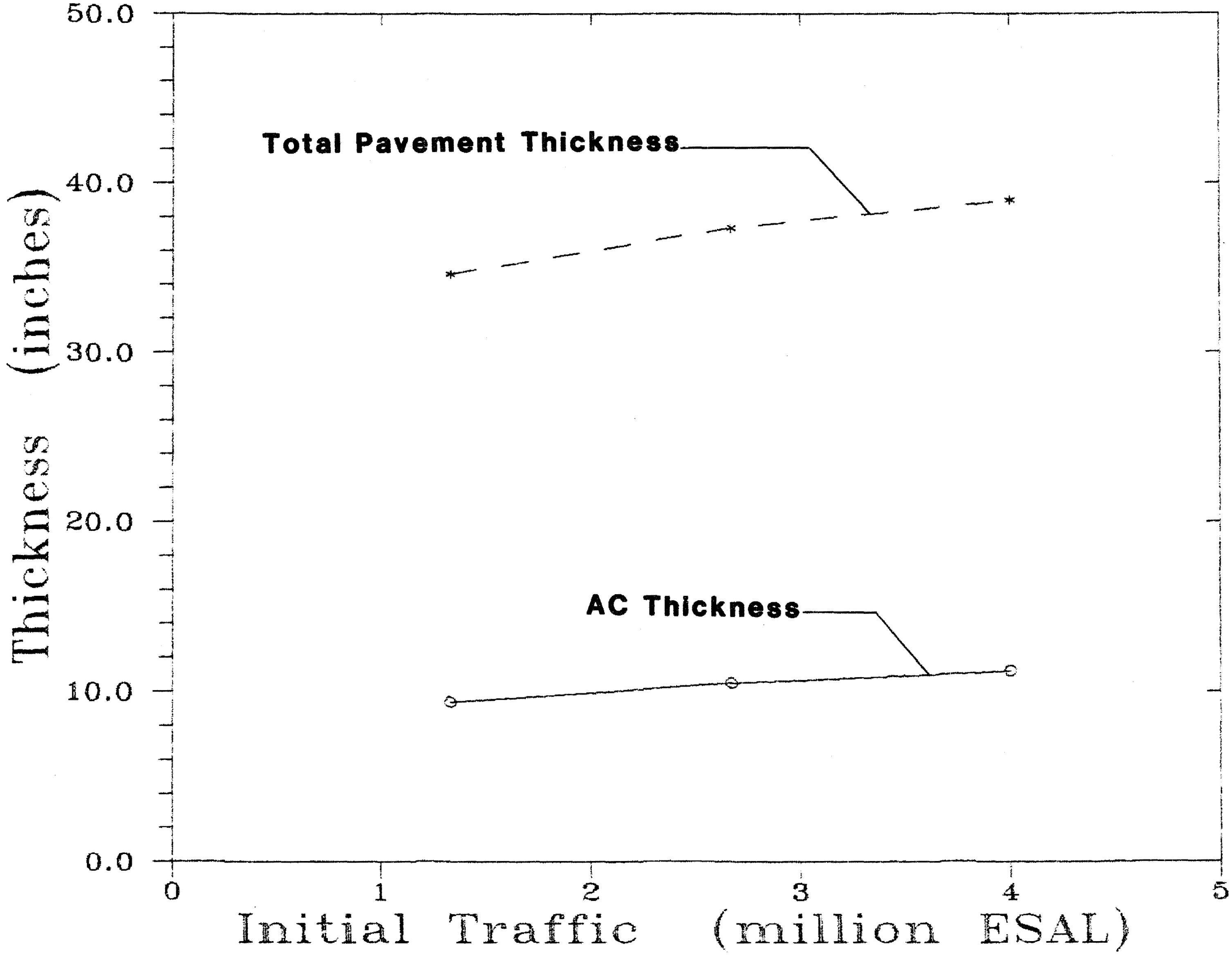

Figure 3-2.22 Illustration of the Sensitivity of the AASHTO Flexible Pavement Thickness Design Procedure to Variation in Traffic.

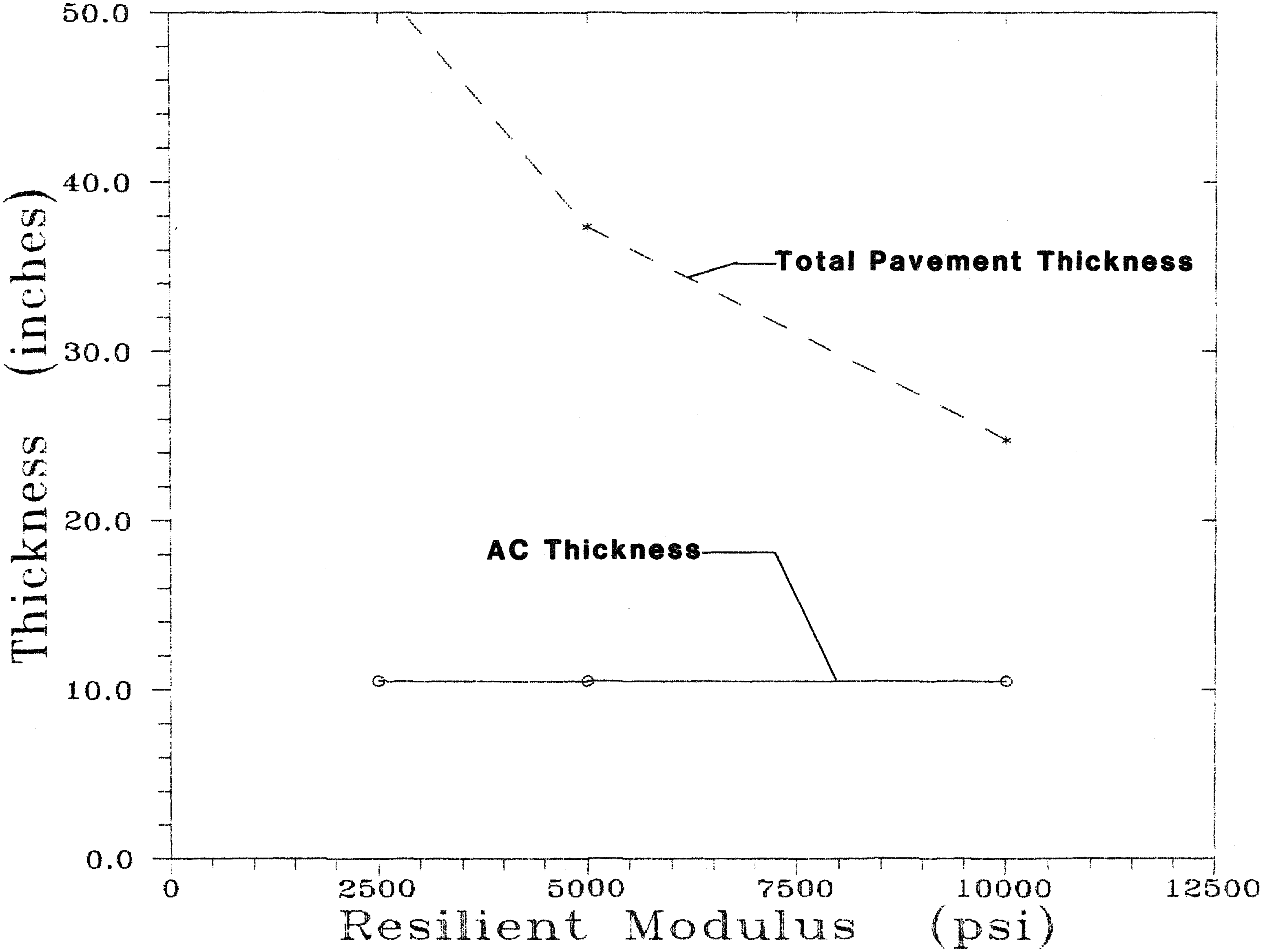

Figure 3-2.23 Illustration of the Sensitivity of the AASHTO Flexible Pavement Thickness Design Procedure to Variation in Roadbed Soil Resilient Modulus.

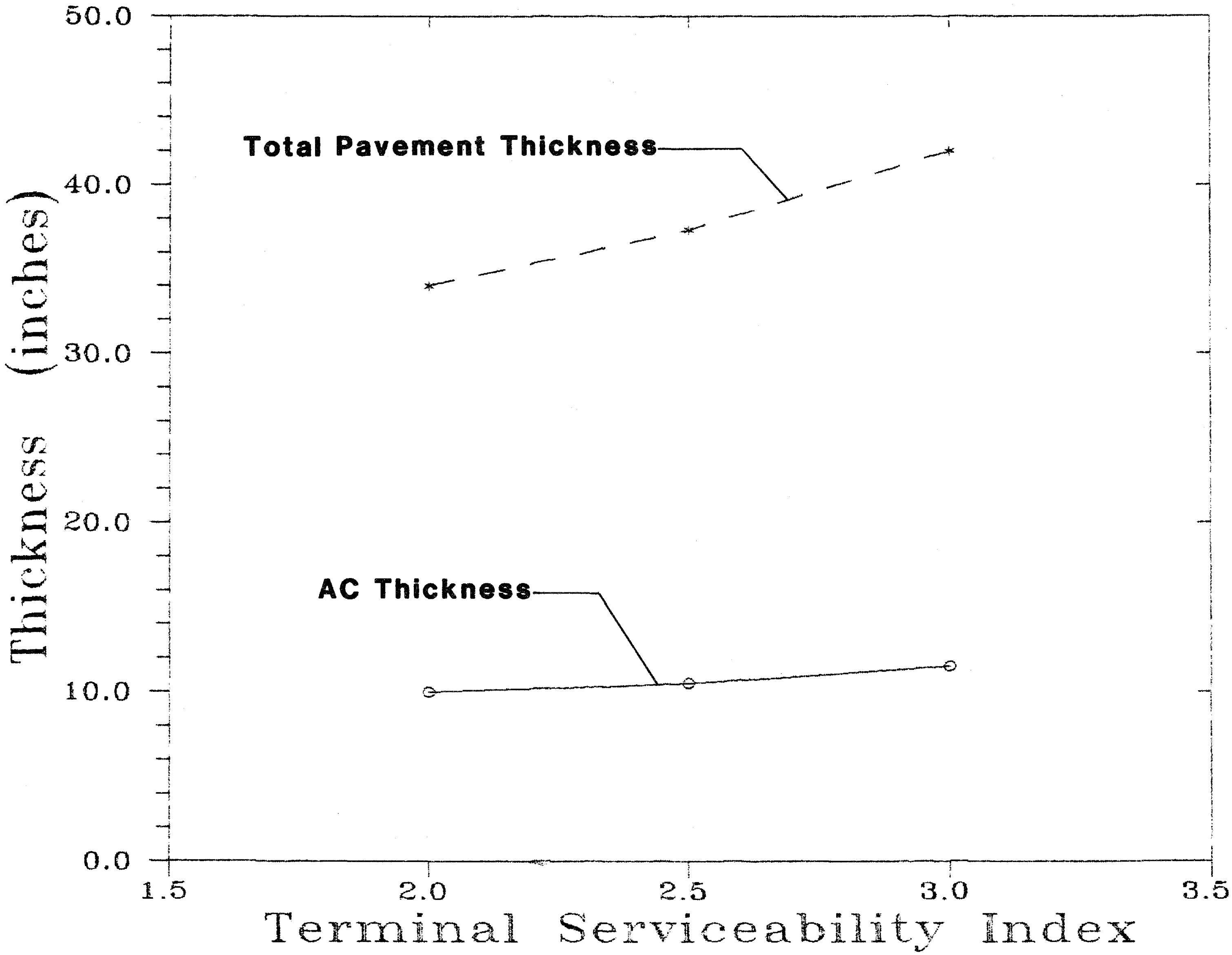

Figure 3-2.24 Illustration of the Sensitivity of the AASHTO Flexible Pavement Thickness Design Procedure to Variation in Selected Terminal Serviceability.

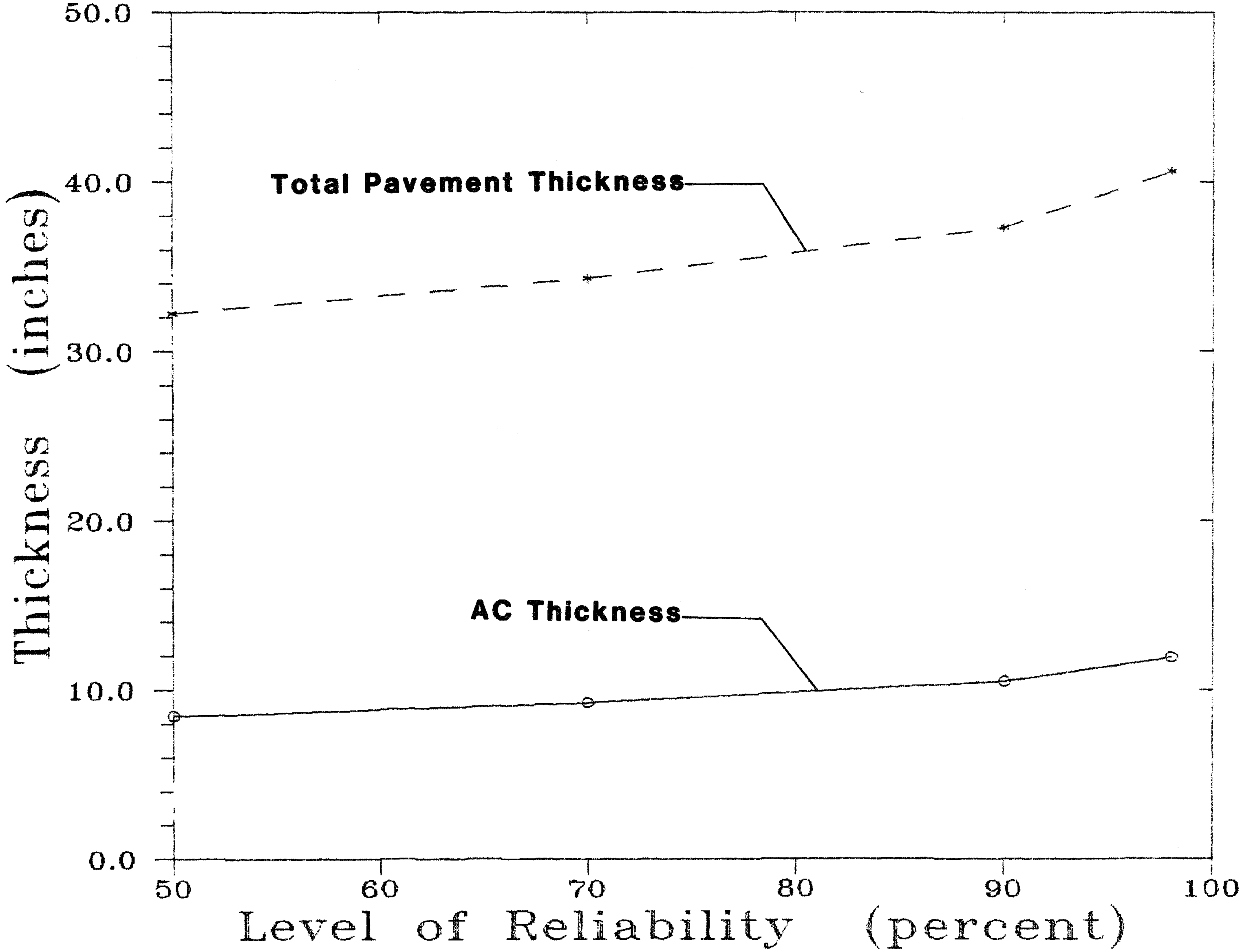

Figure 3-2.25 Illustration of the Sensitivity of the AASHTO Flexible Pavement Thickness Design Procedure to Variation in Selected Level of Reliability.

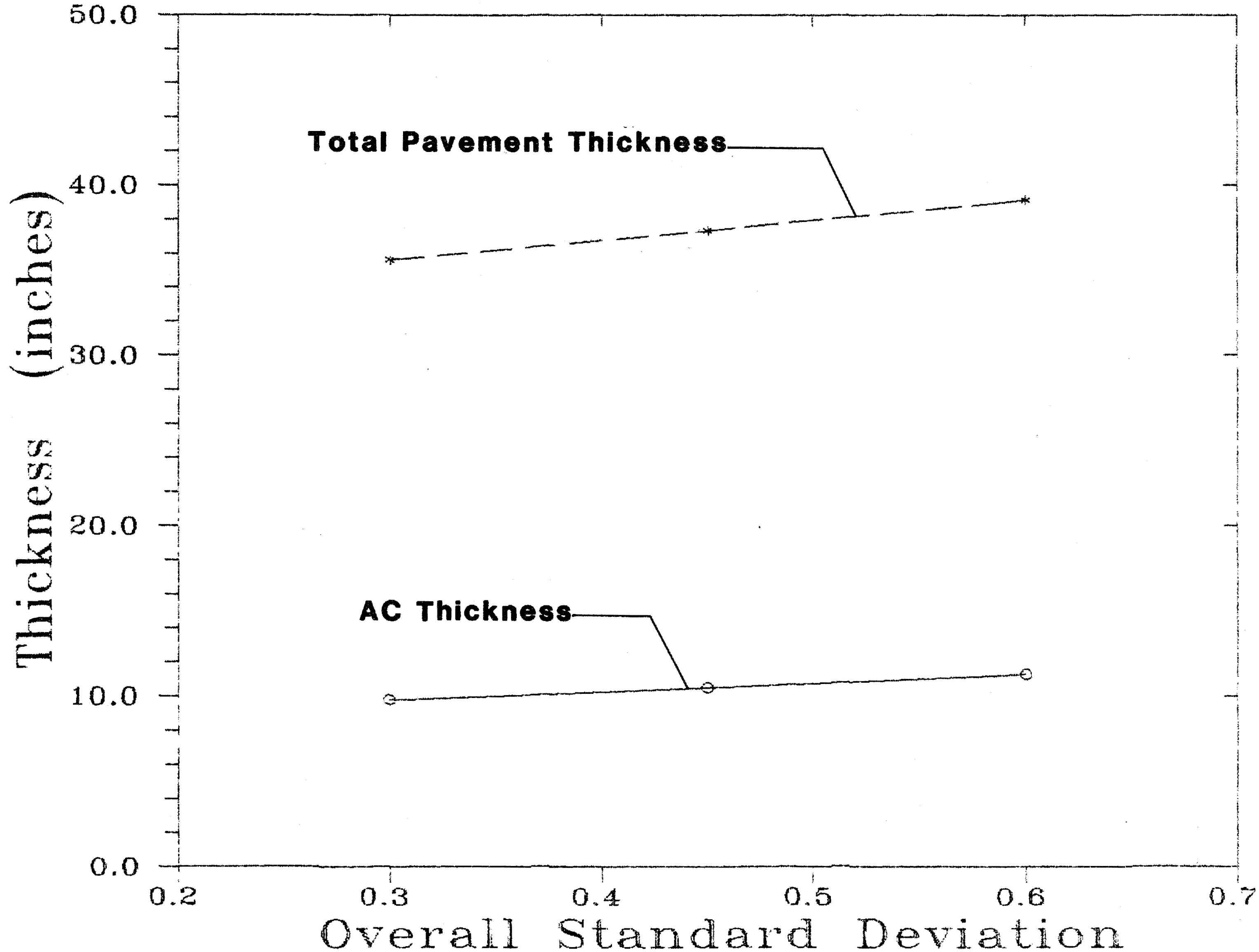

Figure 3-2.26 Illustration of the Sensitivity of the AASHTO Flexible Pavement Thickness Design Procedure to Variation in Selected Overall Standard Deviation.

Input Description	SN	AC Thickness (in)	Total Thickness (in)
All Nominal Values	6.3	10.5	37.4
Low Initial Traffic	5.6	9.4	34.6
High Initial Traffic	6.7	11.2	39.0
Low Roadbed Resilient Modulus	7.7	10.5	51.8
High Roadbed Resilient Modulus	4.9	10.5	24.8
Low Terminal Serviceability	5.9	10.0	34.0
High Terminal Serviceability	6.7	11.5	42.0
Very Low Design Reliability	5.2	8.5	32.2
Low Design Reliability	5.7	9.3	34.3
High Design Reliability	7.2	11.9	40.6
Low Overall Standard Deviation	5.8	9.8	35.6
High Overall Standard Deviation	6.6	11.3	39.1

A visual review of the outputs confirms the effect of each design factor on overall pavement thickness and structural number. Increases in required structural number and overall pavement thickness accompany increases in traffic, terminal serviceability, design reliability and overall standard deviation and decreases in roadbed resilient modulus.

Figure 3-2.22 illustrates the effect of traffic on thickness design. For the traffic input values considered, a 300 percent increase in initial traffic is accompanied by only a 19 percent increase in surface thickness and a 13 percent increase in total pavement thickness. This is probably due to the fact that all of the traffic input values considered represent relatively high volumes of traffic and all of the designs are quite thick. The effect of traffic on thickness design is more pronounced for lower levels of traffic and thinner pavement sections. In such cases, substantial increases in traffic result in more significant increases in design thickness.

Figure 3-2.23 summarizes the effect of varying the effective roadbed soil resilient modulus on the pavement thickness design. Softer roadbed soils require more total pavement thickness to reduce vertical stresses and prevent permanent deformation. This increase in pavement thickness is generally accomplished most efficiently and inexpensively by increasing the base and subbase layer thicknesses. It is not necessary to increase the surface thickness unless radial strains at the bottom of the asphalt concrete layer are also excessive. Note that the largest changes in required overall pavement thickness occur around relatively small effective resilient modulus values (<5000 psi) and that further decreases in pavement thickness are relatively small for increases in effective resilient modulus above 10,000 psi.

Figure 3-2.24 shows the effect of selecting various terminal serviceability indexes (varying the allowable change in serviceability) on pavement thickness design. Fairly linear increases in both surface thickness and overall pavement thickness accompany increases in terminal serviceability. Since increases in overall pavement thickness affect the development of subgrade deformation and increases in surface thickness are most effective in reducing fatigue cracking, both thicknesses must be increased to effectively maintain a higher level of serviceability over the performance period. The increase in overall pavement thickness per unit increase in terminal serviceability is greater than the increase in surface thickness, which reflects the fact that small increases in surface thickness can effectively reduce the development of fatigue cracking while larger increases in overall pavement thickness are necessary to reduce vertical stresses in the roadbed soil.

Figure 3-2.25 presents the effect of varying the level of design reliability on flexible pavement thickness design. This figure shows that relatively small increases in overall pavement thickness and surface thickness will provide large increases in design reliability up to about 90 percent. For example, improving the design reliability from 50 to 90 percent results in surface and total pavement thickness increases of 24 and 16 percent (2 inches and 5 inches), respectively. Increasing the reliability from 90 percent to 98 percent results in additional surface and total pavement thickness increases of 16 and 10 percent (1.4 inches and 3.3 inches), respectively. Thus, design reliability can be "purchased" fairly inexpensively up to a point (90-95 percent), but becomes fairly expensive (in terms of initial costs) above that level.

Figure 3-2.26 describes the effect of selecting various values for the overall standard deviation of input values. The result is a linear shift in both surface and overall pavement design thicknesses when design reliability is held constant because this factor enters the design equation as a constant multiplier of the standard normal deviate, Z_R.

6.0 LIMITATIONS OF THE AASHTO DESIGN PROCEDURE

Major limitations of the AASHTO flexible pavement design procedure are summarized as follows:

1. Limited materials and subgrade. The Road Test used a specific set of pavement materials and one roadbed soil. The extrapolation of the performance of these specific materials to general applications is dangerous because the materials and soils available locally will probably not be identical to those used at the Road Test and will perform differently. The AASHTO design procedure addresses this deficiency through the use of several adjusting factors and inputs (M_R, F_R, drainage, etc.). However, many of these inputs are also based on empirical relationships and must be used carefully.

2. No mixed traffic. The AASHO Road Test accumulated traffic on each test section by operating vehicles with identical axle loads and axle configurations. In-service pavements are exposed to many different axle configurations and loads. The process of converting mixed traffic into equivalent 18-kip ESAL applications is based on another empirical relationship that has never been field-verified.

3. Short road test performance period. The number of years and heavy axle load applications upon which the design procedure is based represents only a fraction of the design age and load applications that many of today's pavements must endure. Current design periods range from 20 to 40 years or more and many pavements will be exposed to 100 million 18-kip ESAL or more. Even if the AASHTO equations can be extrapolated to design for so many load applications, the environmental deterioration that

occurs over time is not directly included in the design equations. The iterative procedure described earlier must be used to adjust the design performance period of the pavement section to account for reductions due to environmental effects. It is impossible to develop an initial design that provides a predetermined performance period and provides for the effects of environmental deterioration.

4. Load equivalency factors. The load equivalency factors used to determine cumulative 18-kip ESAL pertain specifically to the Road Test materials, pavement composition, climate and subgrade soils. The accuracy of extrapolating them to other regions, materials and distresses, etc., is not known, but is questionable. These factors are also based on terminal serviceability indices of 2.0 or 2.5. The use of higher terminal serviceabilities is often considered for pavements that carry high traffic volumes. Thus, the available equivalency factors may not adequately address such cases.

5. Variability. A serious limitation of the AASHTO design procedure is that it is based upon very short pavement sections where construction and material quality were highly controlled. Typical highway projects are normally several miles in length, and contain much greater construction and material variability, and hence show more variability in performance along the project in the form of localized failures. Projects designed using average inputs could be expected to exhibit significant localized failures before the average project serviceability index drops to P_t unless a level of reliability somewhat greater than that desired is selected for design.

6. Lack of guidance on some design inputs. Structural coefficients and drainage modifying factors are very significant on influencing flexible pavement layer thicknesses, and there is very little guidance provided for their selection. The design reliability also has an extremely large effect on pavement thickness and very little guidance is provided in selecting this factor.

Successful use of the AASHTO Guide requires a lot of experience and knowledge of the assumptions and underlying basis for design. It is strongly recommended that the resulting designs be checked using other procedures and mechanistic analyses.

7.0 EXAMPLE DESIGN PROBLEM

Using the AASHTO flexible pavement design procedure, design a flexible pavement for a rural primary highway given the following data:

Pavement Location: .... Rural
Pavement Functional Classification:........ Primary
Traffic:
Expected Initial Traffic (Two-Way, ESAL) ... 2.67×10^6
Expected Directional Distribution, DD 0.50
Expected Truck Distribution (Design Lane), TD 0.70
Expected Annual Truck Traffic Growth ..... 4.0 percent

Construction Materials Properties:
Asphalt Concrete Modulus of Elasticity .... 300,000 psi
Granular Base Resilient Modulus .... 25,000 psi
Granular Subbase Resilient Modulus ...... 12,000 psi
Roadbed Soil Seasonal Resilient Modulus (from tests):
Winter (mid-December to late February) .... 30,000 psi
Spring (early March to late April) ...... 1,000 psi
Summer, Fall (early May to mid-December) ... 5,000 psi
Typical Serviceability Loss due to Environment... see Figure 3-2.2

Solution:

The following design inputs might be selected based on consideration of pavement functional importance, stage construction considerations, knowledge of local construction quality, and engineering experience:

Performance Period .... 10 years
Analysis Period (to include one rehabilitation) .. 20 years
Design Reliability over Analysis Period, R .... 90 percent
Overall Standard Deviation (including traffic), S_o 0.45
Initial Pavement Serviceability Index 4.5
Terminal Pavement Serviceability Index 2.5

Since stage construction is being planned (original construction and one rehabilitation) and the overall design reliability has been selected as 90 percent, each stage must have a reliability $R_{stage} = (R_{overall})^{1/2}$. In this case, the reliability of each stage must be approximately 95 percent.

The expected traffic for the 10-year performance period is estimated using the given information and Module 2-6 as follows:

W_{18} = Traffic Growth Factor * Initial Expected Traffic * DD * TD
$= 12.01 * 2.67*10^6$ ESAL $* 0.50 * 0.70$
$= 11.22*10^6$ ESAL

The seasonal roadbed resilient modulus data is entered into the chart in Figure 3-2.3 and the effective roadbed resilient modulus can be estimated as 2100 psi.

The design serviceability loss due to traffic can be determined by subtracting the estimated serviceability loss due to environmental effects (obtained from Figure 3-2.2) from the overall design serviceability loss. In this case, the overall design serviceability loss is 4.5 - 2.5 = 2.0 and the estimated serviceability loss due to environmental effects over 10 years is 0.64. Thus, the design serviceability loss due to traffic is 2.0 - 0.64 = 1.36.

Entering Figure 3-2.11 with R = 95 percent, $S_o = 0.45$, $W_{18} = 11$ million, PSI = 1.36 and appropriate modulus values, the following structural numbers are obtained:

SN_3 (protection of the roadbed soil) = 8.6
SN_2 (protection of the subbase) = 5.0
SN_1 (protection of the base) = 3.8

Layer coefficients must be determined for each of the paving materials being used. If no local road tests have been conducted to experimentally determine such values for local materials and conditions, Figures 3-2.6, 3-2.7 and 3-2.8 can be used to estimate the layer coefficients from their modulus values as follows:

$$a_1 = a_{AC} = 0.36$$
$$a_2 = a_{base} = 0.12$$
$$a_3 = a_{subbase} = 0.12$$

Based on a knowledge of the granular materials properties (gradation, permeability, etc.), the construction site location, and the probable location of the materials in the pavement structure, it might be determined that the base has good drainage characteristics and will be saturated less than 5 percent of the time. The subbase might be determined to have poor drainage characteristics and might be expected to be saturated as much as 25 percent of the time. Using this information and Table 3-2.3, drainage modifying factors for these layers can be estimated as follows:

Layer	m_i
Base	1.12
Subbase	0.85

A cost analysis has been performed (as described in section 4.2.3) and it has been determined that the granular base material provides the most structural contribution per dollar and the asphalt concrete provides the least. This information is combined with the minimum layer thicknesses determined according to layered analysis and minimum constructable thicknesses to produce the following design thicknesses:

$$\text{Minimum Required Surface Thickness} = D_1 = 3.8/0.36 = 10.6 \text{ in}$$

Use D^*_1 = 11.0 in AC surface
$SN^*_1 = 11.0 * 0.36 = 3.96 > 3.8$ -- ok

Since the base course has been determined to be the more cost-effective material than the subbase, the use of subbase material should be eliminated unless other factors require it's use. The remaining structure is then composed of base material.

$$\text{Minimum Required Base Thickness} = D_2 = (8.6 - 3.96)/(0.12*1.12) = 34.5 \text{ in}$$

Use D^*_2 = 35.0 in granular base
$SN^*_2 = 35.0 * 0.1344 = 4.70$
$SN^*_1 + SN^*_2 = 3.96 + 4.70 = 8.66 > 8.6$ -- ok

This structure may be unacceptably expensive because of the excessive excavation costs required to place a 46-inch thick pavement. Consideration should be given to replacing some or all of the granular base with asphalt concrete to reduce excavation costs and achieve a lower initial cost pavement. A full-depth asphalt concrete pavement would need to be 8.6/0.36 = 23.88 or 24 inches thick (using the layer coefficient of the surface course for the entire pavement thickness).

8.0 SUMMARY

The AASHO Road Test was a carefully designed and constructed experiment that provided data concerning the effects of varying thicknesses of surfacing, base course and subbase. Other design factors were also studied. These data were used to develop empirical performance prediction models, which formed the basis for a pavement design procedure.

Over the years, several modifications and improvements have been made to the AASHTO procedures, with the most significant changes occurring in 1986. The current design procedure incorporates the concepts of design reliability and variability of design inputs, the use of resilient modulus testing for materials characterizations and determination of layer coefficients, direct consideration of the effects of drainage on materials performance, and a rational approach to adjusting designs to account for environmental considerations.

The current AASHTO flexible pavement design procedure can be accomplished manually, through the use of a nomograph, or using the referenced computer program. Design inputs include performance and analysis periods, cumulative expected traffic in ESAL, required design reliability, loss of serviceability due to environmental effects, performance criteria (initial and terminal serviceability), and resilient or elastic modulus and drainage characteristics of the paving materials and roadbed soil. Trial thicknesses are adjusted to minimize initial construction costs and to provide layer stability and constructability. The design process is described in detail in this module.

Most of the major limitations of the AASHTO flexible pavement design procedure are concerned with the general use of a design procedure that was developed from very specific conditions (e.g., one climate, one set of materials, no mixed traffic, etc.) over a short period of time. The successful use of the AASHTO design procedure requires experience and a knowledge of the underlying assumptions. AASHTO designs should be verified using other design procedures and mechanistic analyses.

9.0 REFERENCES

1. ____, "AASHO Interim Guide for the Design of Rigid Pavement Structures," American Association of State Highway Officials, Committee on Design, April 1962.

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3. ____, "AASHTO Interim Guide for Design of Pavement Structures; Chapter III Revised," American Association of State, Highway, and Transportation Officials, Washington D.C., 1981.

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5. ____, "The AASHO Road Test, Report 5 - Pavement Research," Highway Research Board, Special Report 61E, 1962.

6. Rada, Gonzalo and M. W. Witczak, "A Comprehensive Evaluation of Laboratory Resilient Moduli Results for Granular Materials," a paper prepared for presentation at the Annual Meeting of the Transportation Research Board, 1981.

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8. Lister, N. W., "Deflection Criteria for Flexible Pavements and Design of Overlays," Proceedings, Third International Conference on Structural Design of Asphalt Pavements, Ann Arbor, 1972.

9. Van Til, C. J., B. F. McCullough, B. A. Vallerga and R. G. Hicks, "Evaluation of AASHO Interim Guides for Design of Pavement Structures," NCHRP Report 128, 1972.

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MODULE 3-3

OTHER PROCEDURES FOR FLEXIBLE PAVEMENT THICKNESS DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

The purpose of this module is to show that there are alternate flexible pavement design procedures that can be used independently or to provide checks of the AASHTO procedure results. Particular emphasis is placed on the Asphalt Institute design method, but other methods are discussed as well. Each method is described along with its assumptions, strengths and weaknesses.

Upon completion of this module, the participants will be able to accomplish the following:

1. Describe the development of the Asphalt Institute method of flexible pavement design.
2. Design a flexible pavement for a variety of traffic, climate, support and materials inputs using the Asphalt Institute method.
3. Perform sensitivity analyses on the Asphalt Institute method to determine the effects of input variables on pavement design.
4. Describe the development of other flexible pavement thickness design procedures and identify the basic assumptions of each.
5. Compare the assumptions, strengths, weaknesses and limitations of alternate design procedures with those of the AASHTO flexible pavement thickness design procedure.
6. Compare designs produced by the AASHTO procedure with those produced using alternate procedures and explain the sources of their differences, if any.

2.0 INTRODUCTION

In light of rising construction and rehabilitation costs, it is imperative that a pavement design be responsive to the design inputs without being an over-design. Since most design procedures have <u>some</u> shortcoming, this objective, can be achieved by utilizing several different design procedures to verify the proposed pavement design.

3.0 ASPHALT INSTITUTE METHOD FOR FLEXIBLE PAVEMENT DESIGN

The Asphalt Institute procedure can be used to design an asphalt pavement composed of various combinations of asphalt surface and base, emulsified asphalt surface and base, and untreated aggregate base and subbase.

The original Asphalt Institute design methodology was an empirical approach based upon data from the AASHO Road Test, the WASHO Road Test, and other various state and local test sections. This procedure was completely revised in 1981, and the current Asphalt Institute procedure as presented in

MS-1, (1) uses multi-layer linear elastic theory for the determination of the required pavement thickness. Full friction is assumed to exist between the layers. Each elastic layer is characterized by a modulus of elasticity (E) and a Poisson's ratio ( ). Procedures for determining modulus values of various pavement materials are discussed in Block 2.

A computer program was used in the development of the design procedure to examine two critical stress-strain conditions. The first is the maximum vertical compressive strain induced at the top of the roadbed soil from an applied wheel load; the second is the maximum horizontal tensile strain induced at the bottom of the asphalt concrete layer from the applied wheel load (see Figure 3-3.1). For these stress-strain conditions, two key assumptions are made relative to design considerations (1):

1. If the vertical compressive strain at the top of the roadbed soil is excessive, rutting or permanent deformation will occur in both the roadbed soil and at the surface of the asphalt concrete layers.

2. If the horizontal tensile strain at the underside ofthe lowest asphalt bound layer is excessive, fatigue (alligator) cracking of the asphalt layers will develop under repeated traffic loading.

The Asphalt Institute flexible pavement design procedure strives to design a pavement structure that will be thick enough to prevent these excessive horizontal tensile and vertical compressive strains from occurring over a predetermined design period. Thickness design charts were developed using the computer program DAMA (3) which modelled these two stress-strain conditions. The curves in the design charts represent the more critical of the two stress-strain conditions, i.e., for a given set of inputs, the largest strain (either vertical compressive or horizontal tensile) governs the thickness requirements (1).

3.1 Design Considerations

The major design considerations required for the structural design of flexible pavements using The Asphalt Institute procedure include the selection of design input values for traffic, roadbed soil strength, material properties, and environmental conditions.

3.1.1 Traffic

The traffic analysis procedure used by the Asphalt Institute is based on the load equivalency factors developed at the AASHO Road Test. It is assumed that the loads applied to the pavement structure by mixed traffic can be expressed in terms of 18-kip equivalent single-axle load (ESAL) applications. The Asphalt Institute procedure requires that the ESAL factors are selected assuming a terminal serviceability index of 2.5 and a structural number of 5 for both single and tandem axles. Reference 1 contains more information on this topic, including recommended load equivalency factors.

The traffic input value required for design is the total number of ESAL applications which the pavement structure will sustain during the design period. Because the number of equivalent single-axle loads can be easily calculated for any number of years, the roadway can be designed for any

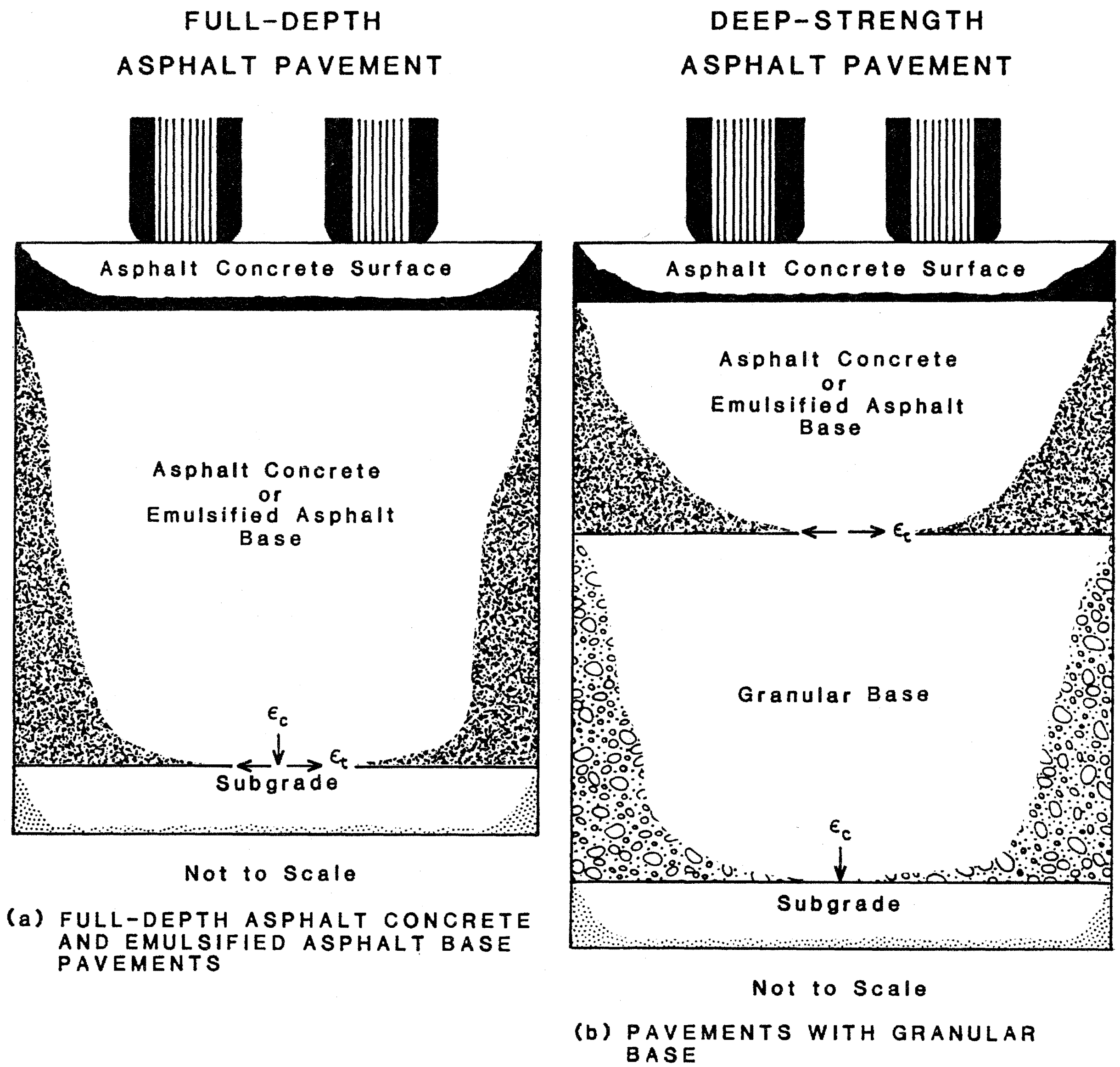

Figure 3-3.1 Assumed Critical Stain Locations for the Asphalt Institute Flexible Pavement Thickness Design Procedure (1).

desired performance period. However, since many assumptions must be made to estimate the number of ESAL applications during the design period, the estimate can be significantly different from the actual number of ESAL applications. Therefore, although not part of the Asphalt Institute design method, it may be prudent to try several different ESAL inputs to determine the effect of traffic on thickness design.

If traffic data or projections are unavailable, the Asphalt Institute method provides guidelines for estimating input ESAL values from the classification of the highway to be built and from the probable ranges of ESAL factors for the various truck volumes (1). The designer must recognize that inadequate designs may result from the use of generic traffic estimates. Such estimates are generally acceptable only when a high risk of premature failure is acceptable.

3.1.2 Roadbed Soil Strength

The second major pavement input variable is the strength of the roadbed soil. The roadbed soil is characterized by the resilient modulus (M_R), which was described in detail in Block 2. The resilient modulus used in this design procedure is the "normal" resilient modulus that is not representative of times when the roadbed soil is frozen or when it is undergoing periods of thaw.

The best method to determine a representative roadbed soil resilient modulus is to perform substantial testing on the roadbed soil. This should include all roadbed soil material that is expected to be within 2 ft of the planned subgrade elevation. If significant roadbed soil variation is present, random sampling should be done to determine the controlling (weakest) soil type, or the limits and boundaries of each roadbed soil type. The latter approach allows the project to be subdivided for separate designs if the various soil type areas are large enough.

A minimum of six to eight test values are usually used to determine the design Roadbed Soil Resilient Modulus (M_R). The design Roadbed Soil Resilient Modulus should be selected as a function of traffic, using lower values when higher traffic levels exist to ensure a more conservative design. Suggested percentile design values (select the design modulus > X% of all test values) for various traffic levels are given in the following table (1):

TRAFFIC LEVEL (CUMULATIVE ESALs)	PERCENTILE DESIGN VALUE
$<10^4$	60.0
$10^4 - 10^6$	75.0
$> 10^6$	87.5

As an example, suppose that the results of roadbed soil resilient modulus testing yielded those numbers shown in Table 3-3.1 (listed in decreasing order). Then the resilient modulus design input for the various traffic levels could be approximated as:

Table 3-3.1. Example Determination of Design Roadbed Soil Resilient Modulus.

TEST VALUE M_R, psi	NUMBER GREATER THAN OR EQUAL TO	% GREATER THAN OR EQUAL TO
16200	1	14
15700	2	29
14900	3	43
14100	4	57
12600	5	71
11500	6	86
10900	7	100

1. Traffic < 10,000 ESALs: Use M_R of 14,000 psi (design M_R > 60% of the test results).

2. Traffic > 10,000 ESALs but < 1 million ESALs: Use M_R of 12,200 psi (design M_R > 75% of the test results).

3. Traffic > 1 million ESALs: Use M_R of 11,300 psi (design M_R > 87.5% of the test results).

It may be more accurate to use a graphical procedure which plots the "% Equal To or Greater Than" versus "M_R." Then the design M_R value can be read directly off the plot for the given percentile. This is demonstrated in Reference 1.

Because resilient modulus testing equipment is not always available, correlations have been established with other widely used tests. The following equations have been suggested to convert the California Bearing Ratio (CBR) and the Resistance (R) value to resilient modulus values (1).

$$M_R = 1500*CBR$$

where:

M_R = resilient modulus, psi
CBR = California Bearing Ratio (ASTM D 1883 or AASHTO 193)

$$M_R = 1155 + 555*(R)$$

where:

M_R = Resilient modulus, psi
R = Resistance value (ASTM D 2844 or AASHTO T-190)

It should be noted that the above correlations are approximate and should be applied carefully. In particular, the R-value correlation has been developed from an extremely limited data bank. The above relationships are considered applicable for most fine-grained soils or for materials with M_R < 30,000 psi (1). Graphical solutions for the relationships between M_R and other strength measures are presented in Module 3-2. Environmental influences on soil strength must also be incorporated.

3.1.3 Material Properties

The properties of the pavement component materials are characterized by a modulus of elasticity and a Poisson's ratio. Determination of the modulus of elasticity is discussed in Block 2. The Poisson's ratios are assigned internally and are based on typical values derived from various research projects.

Other assumed characteristics of the specific materials used in the Asphalt Institute flexible pavement design are discussed below.

Asphalt Concrete

A high quality asphalt concrete was used in producing the charts for this design procedure. Thus, the design assumes that similar quality materials will be used. The asphalt concrete should meet the criteria outlined in Reference 7 for aggregate gradation, mix design, and density requirements (1).

Emulsified Asphalt Mixes

In the Asphalt Institute design method, it is permissible to use emulsified asphalt mixtures for base course layers. Three different types of emulsion mixes are allowed, depending primarily on the type of aggregate used in the mixture. The three mixes are (1):

1. Type I, emulsified asphalt mixes with processed dense-graded aggregate.
2. Type II, emulsified asphalt mixes made with semi-processed crusher-run, pit-run, or bank-run aggregate.
3. Type III, emulsified asphalt mixes made with sands or silty sands.

The aggregate and emulsions utilized must meet the requirements given in Reference 8. Typical material properties were used in development of the thickness design curves for this particular layer type.

Untreated Granular Materials

Untreated granular materials must comply with ASTM specification D 2940, except that the following requirements should apply where appropriate (1):

TEST	TEST REQUIREMENTS	
*CBR, minimum	20	80
or		
*R-Value, minimum	55	78
Liquid Limit, maximum	25	25
Plasticity Index, maximum or	6	NP
Sand Equivalency, minimum	25	35
% Passing #200 Sieve, maximum	12	7

*The roadbed soil resilient modulus relations for CBR and R-Value do not apply to untreated aggregate base and subbases.

3.1.4 Environmental Conditions

It is assumed in the Asphalt Institute method that environmental conditions can be incorporated through the effects of monthly temperature changes throughout the year on the asphalt modulus and through consideration of the effects of temperature on the roadbed soil resilient modulus and modulus of the granular materials. The effects of moisture and drainage are not considered directly.

In consideration of the asphalt concrete layers, three sets of environmental conditions were selected to represent the range of conditions to which the design manual should apply (1):

TEMPERATURE CONDITION	FROST EFFECTS		ASPHALT GRADES
Cold, mean annual Temperature < $45^{O}F$	Yes	AC-5 AR-2000 120/150 pen	AC-10 AR-4000 85/100 pen
Warm, mean annual temperature between $45^{O}F$ and $75^{O}F$	Possible	AC-10 AR-4000 85/100 pen	AC-20 AR-8000 60/70 pen
Hot, mean annual temperature > $75^{O}F$	No	AC-20 AR-8000 60/70 pen	AC-40 AR-16000 40/50 pen

Mean annual air temperatures were used to characterize the environmental conditions applicable to each region, and the characteristics of the materials were selected accordingly. In cold regions, the asphalt must be less stiff to minimize the potential for thermal cracking; in hot regions, the asphalt must be stiff to increase resistance to rutting and permanent deformation.

The roadbed soil resilient modulus is affected by seasonal variations in moisture and temperature. The resilient modulus is expected to increase during the cold winter months when the roadbed soil is frozen and to decrease in the spring when it is undergoing periods of thaw and high moisture content (1). This is illustrated in Figure 3-3.2. Similar changes occur in the elastic modulus of untreated granular materials during periods of freeze and thaw. The actual magnitude of the change in resilient modulus depends on such factors as temperature, freezing index value, groundwater conditions, roadbed soil type, drainage, and pavement cross section.

The design curves account for the seasonal influences on the roadbed soil and granular materials by using reduced moduli values for the thaw period and increased moduli values for the freezing period. Thus, it is essential that the roadbed soil modulus obtained from testing represent a "normal" value and not any extreme values. This can be accomplished by performing roadbed soil testing at times other than when the roadbed soil is frozen or undergoing spring thaw.

The Asphalt Institute method (MS-1) provides design charts only for a mean annual air temperature of $60^{O}F$ because it is assumed in that manual that if asphalt cements are selected based on the temperature guidelines previously discussed (for mean annual temperatures of $45^{O}F$, $60^{O}F$, and $75^{O}F$), the resulting concrete modulus will remain approximately constant with changes in mean annual air temperature. If this assumption is invalid, the use of MS-1 for climates with mean annual air temperatures significantly different from 60^{O} F may result in over- or underdesigns. Reference 2, which documents the development of and provides the basis for the MS-1 design method, contains a set of graphs for mean annual air temperatures of $45^{O}F$, $60^{O}F$, and $75^{O}F$ so that variation in asphalt modulus with temperature is incorporated to a certain extent. If the pavement is to be designed for extreme climates, it is recommended that these design charts be utilized.

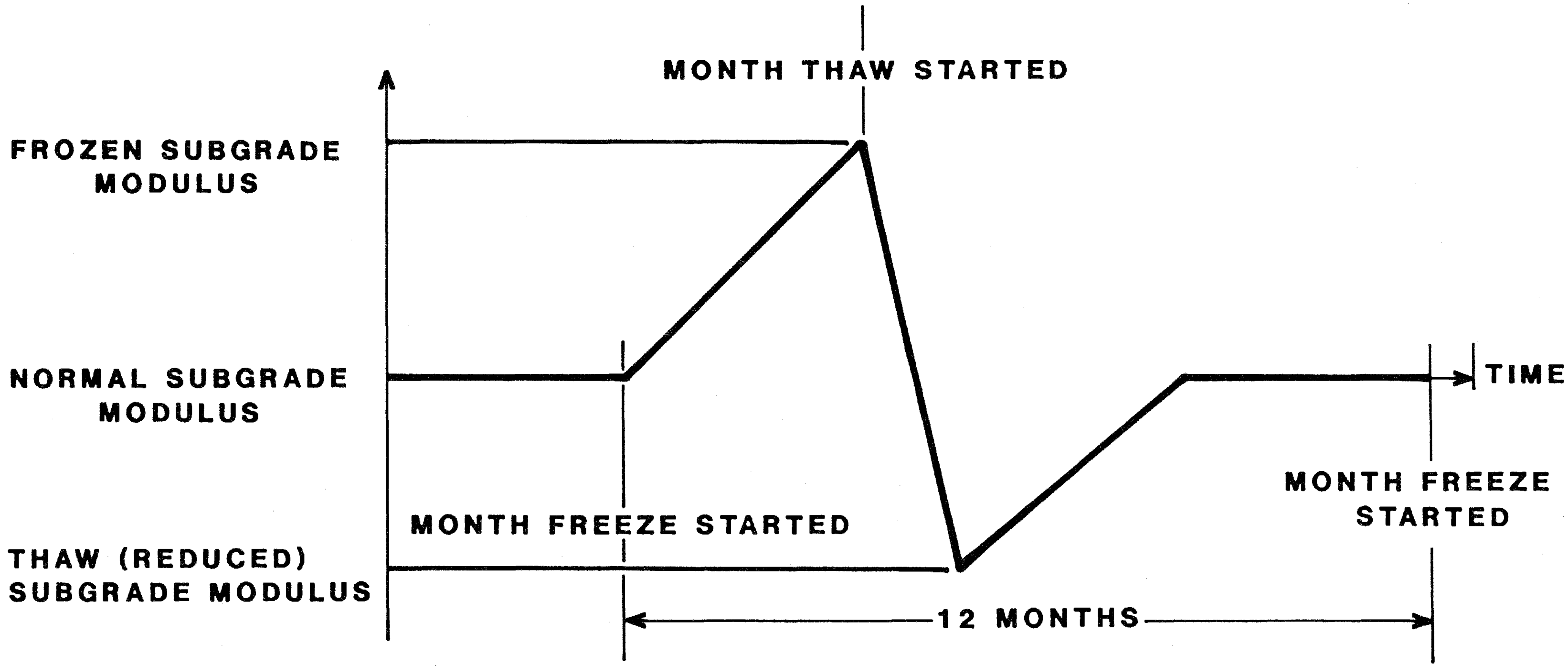

Figure 3-3.2 Conceptual Drawing of Seasonal Variation in Roadbed Soil Resilient Modulus Due to Changes in Moisture and Temperature.

3.2 Structural Design Procedure

The design criteria for the Asphalt Institute method is to restrict the amount of horizontal strain at the bottom of the asphalt-treated layer (which is a source of fatigue or alligator cracking) and to restrict the amount of vertical strain at the top of the roadbed soil (which is a source of rutting and permanent deformation). The limiting criteria for both fatigue cracking and permanent deformation are based on empirical data. Alligator cracking is limited to 20 - 25% of the pavement surface and rutting is limited to 0.5 in over the design period of the pavement. The pavement thickness obtained from these design charts satisfy the most critical of these two requirements (1).

The Asphalt Institute design procedure consists of the following steps:

1. Roadbed Soil Testing. This is necessary to determine the Design Roadbed Soil Resilient Modulus, M_R, for use in the design charts, as described earlier.

2. Material Determination. This is required to select the surface and base types to be used in the design. There are several different pavement structures which may be used for the design of a flexible pavement. These include full-depth asphalt concrete pavements, pavements with an asphalt surface and an emulsified asphalt base, and pavements with an asphalt surface and granular base.

   Since asphalt mix properties vary with temperature, it is recommended that different asphalt cement grades be used where different climate conditions prevail. A table giving the recommended asphalt grades for specific temperature conditions was provided earlier.

3. Traffic Information. It is necessary to determine the traffic to be carried by the pavement (in ESAL applications over the design period). Design life is not input directly. Different pavement design lives can be considered by using different design traffic volume inputs (e. g., 5 million ESAL for 10 years, 10 million for 20 years, etc.).

4. Thickness Design. Thickness design charts such as that shown in Figures 3-3.3, 3-3.4, and 3-3.5 for Full-Depth Asphalt Concrete, Emulsified Asphalt Mix Type II, and 6 inch Aggregate Base, respectively, are used to determine the required asphalt concrete surfacing thickness. Inputs of design traffic, roadbed soil resilient modulus, and a preselected material determination (e.g., full-depth asphalt, 4 in aggregate base, 6 in aggregate base, etc.) are required. Reference 1 contains additional charts not in this manual.

   As an example, assume a 6 inch aggregate base has been selected for use in the pavement system. If a traffic level of 2 million ESALs is anticipated over the design life and the resilient modulus has been determined to be 8000 psi, then an asphalt concrete thickness of 9 inches is required over the 6 inch aggregate base. This is shown in Figure 3-3.5.

Full-Depth Asphalt Concrete

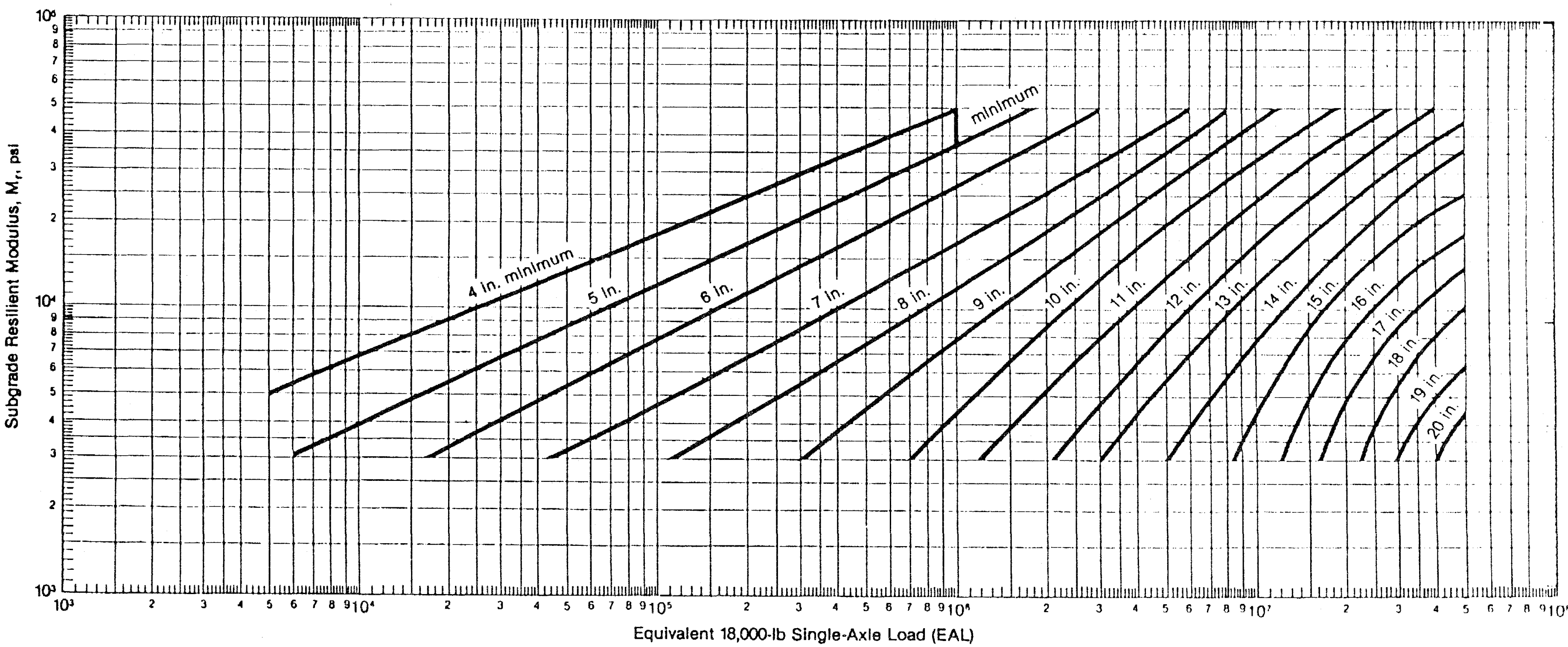

Figure 3-3.3 Example Asphalt Institute Design Chart for Full-Depth Asphalt (1).

Emulsified Asphalt Mix Type II

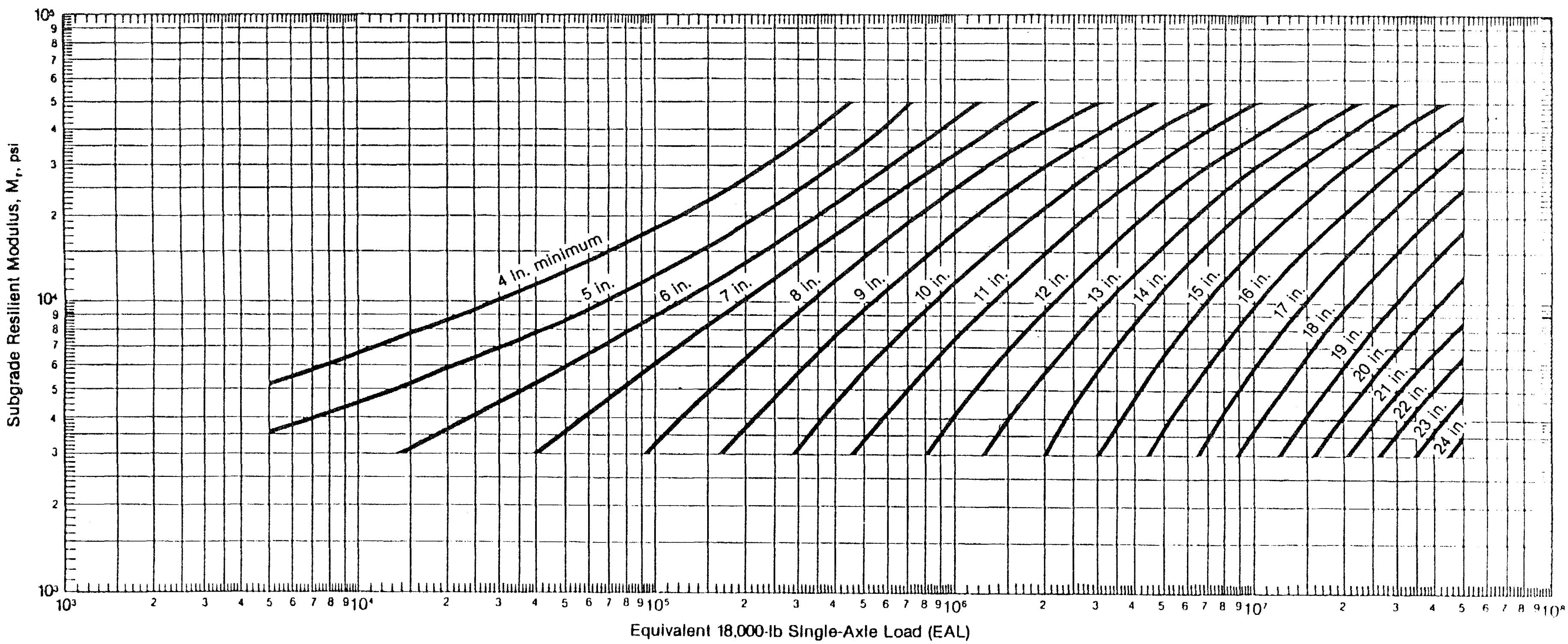

Figure 3-3.4 Example Asphalt Institute Design Chart for Emulsified Asphalt Mix Type II (1).

Untreated Aggregate Base 6.0 in. Thickness

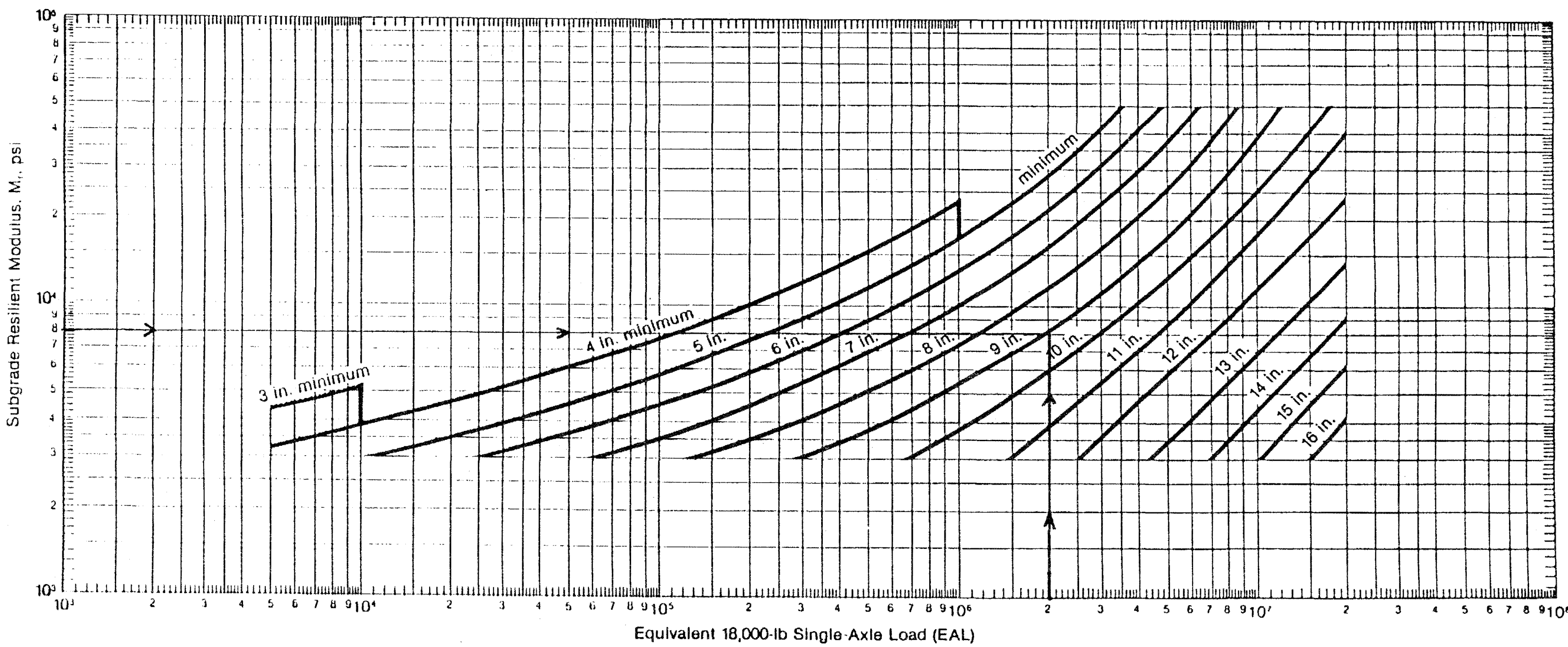

Figure 3-3.5 Example Asphalt Institute Design Chart for 6 in Untreated Aggregate Base (1).

If the pavement structure is to consist of full-depth asphalt concrete, the pavement thickness design charts can be entered directly with the resilient modulus value for the roadbed soil and the design traffic level. For full-depth asphalt concrete pavement structures, a minimum thickness of 4 inches is suggested by this design method (1).

The pavement structure can also be designed using an emulsified asphalt base course in place of a portion of the asphalt concrete layer. As discussed previously, three different types of emulsified mixes can be designed depending on the quality of the material used in the mixture. The design charts must be entered with the resilient modulus and design traffic level for a specific emulsified asphalt base type to produce a total pavement thickness. A minimum thickness of asphalt concrete is needed over the emulsion stabilized base course. The minimum asphalt thickness depends on the level of traffic anticipated for the pavement structure. For both Type II and Type III emulsified asphalt bases, the minimum asphalt concrete thickness varies from 2 inches for traffic levels of less than 10,000 ESALs to 5 inches for traffic levels greater than 1 million ESALs (1). The thickness of the emulsified base is then determined by subtracting the minimum asphalt thickness from the total thickness obtained from the charts.

Granular base courses can also be used as part of the pavement structure. For this material, a given thickness of untreated aggregate base is preselected (based on drainage, frost, or other requirements). The appropriate design chart is then entered (with a roadbed soil modulus and design traffic level) to yield the total thickness of asphalt concrete needed over the preselected granular base course thickness. The design manual provides for six different untreated base thicknesses--4, 6, 8, 10, 12, and 18 inches. The minimum thicknesses of asphalt concrete required range from 3 inches for traffic levels of less than 10,000 ESALs to 5 inches for traffic levels over 1 million ESALs (1).

3.3 Design Selection

Since there are numerous different layer configurations available when using the Asphalt Institute method (i. e., full-depth asphalt concrete, emulsified asphalt bases, granular bases), there may be some difficulty in determining the pavement system which best addresses a given design situation. Therefore, the following items should be considered when selecting the type of pavement system:

1. Full-depth asphalt concrete pavements have the advantages of better resistance to pavement stresses, less total required thickness than pavements with untreated aggregate base courses (meaning reduced excavation costs in some cases), and relative insensitivity to frost or moisture. However, materials for aggregate bases are an abundant resource and are generally inexpensive and readily available. Aggregate bases can perform well when constructed properly.

2. It is recommended that several designs be determined using different materials and then an economic analysis (as discussed in Block 6) be performed to determine the preferred alternative. However, there are other factors which should also be considered in selecting the preferred alternative, such as material

availability, geometric design problems, utility locations, agency policies, etc. The consideration of these other factors is also discussed in Block 6.

3. Stage construction (the construction of successive layers of asphalt concrete according to design requirements and on a predetermined time schedule) should also be considered in the cost analysis. This approach is beneficial when funds are insufficient for constructing a pavement with a long design life. Stage construction is also desireable when there is a great amount of uncertainty in estimating traffic. The pavement can be designed for an initial traffic level and the next stage of construction can be designed using traffic projections based on the in-service traffic data. Finally, stage construction can allow weak spots which develop in the first stage to be repaired in the next stage.

3.4 Sensitivity Analysis

The following table of design inputs was used to study the sensitivity of the Asphalt Institute flexible pavement thickness design procedure (graphical solutions) to roadbed soil resilient modulus, design traffic, mean annual air temperature and base thickness.

Design Input	Low Value	Nominal Value	High Value(s)
Roadbed Soil Resilient Modulus (psi)	3000	5000	10000
Design Traffic (ESAL*10^6)	7	14	28 and 42
Mean Annual Air Temp (^oF)	45	60	75
Base Course Thickness (in)	0	6	12

The following table summarizes the combinations of design inputs that were entered into the Asphalt Institute design charts and the pavement thickness designs that resulted. All inputs were varied one at a time, holding all other inputs constant at their nominal values. These results are presented graphically in Figures 3-3.6 through 3-3.9.

Input Description	AC Thickness (in)	Total Pavement Thickness (in)
All Nominal Values	14.6	20.6
Low Roadbed Resilient Modulus	15.8	21.8
High Roadbed Resilient Modulus	13.0	19.0
Low Traffic	13.0	19.0
High Traffic	16.5 *(extrapolated)*	22.5
Very High Traffic	18.0 *(extrapolated)*	24.0
Low MAAT	13.2	19.2
High MAAT	16.0	22.0
12" Granular Base	13.7	25.7
Full-Depth AC (No Base)	15.7	5.7

Figure 3-3.6 illustrates the effect of roadbed soil design resilient modulus on the resulting thickness design. As M_R increases, the overall pavement thickness required to protect the subgrade from permanent

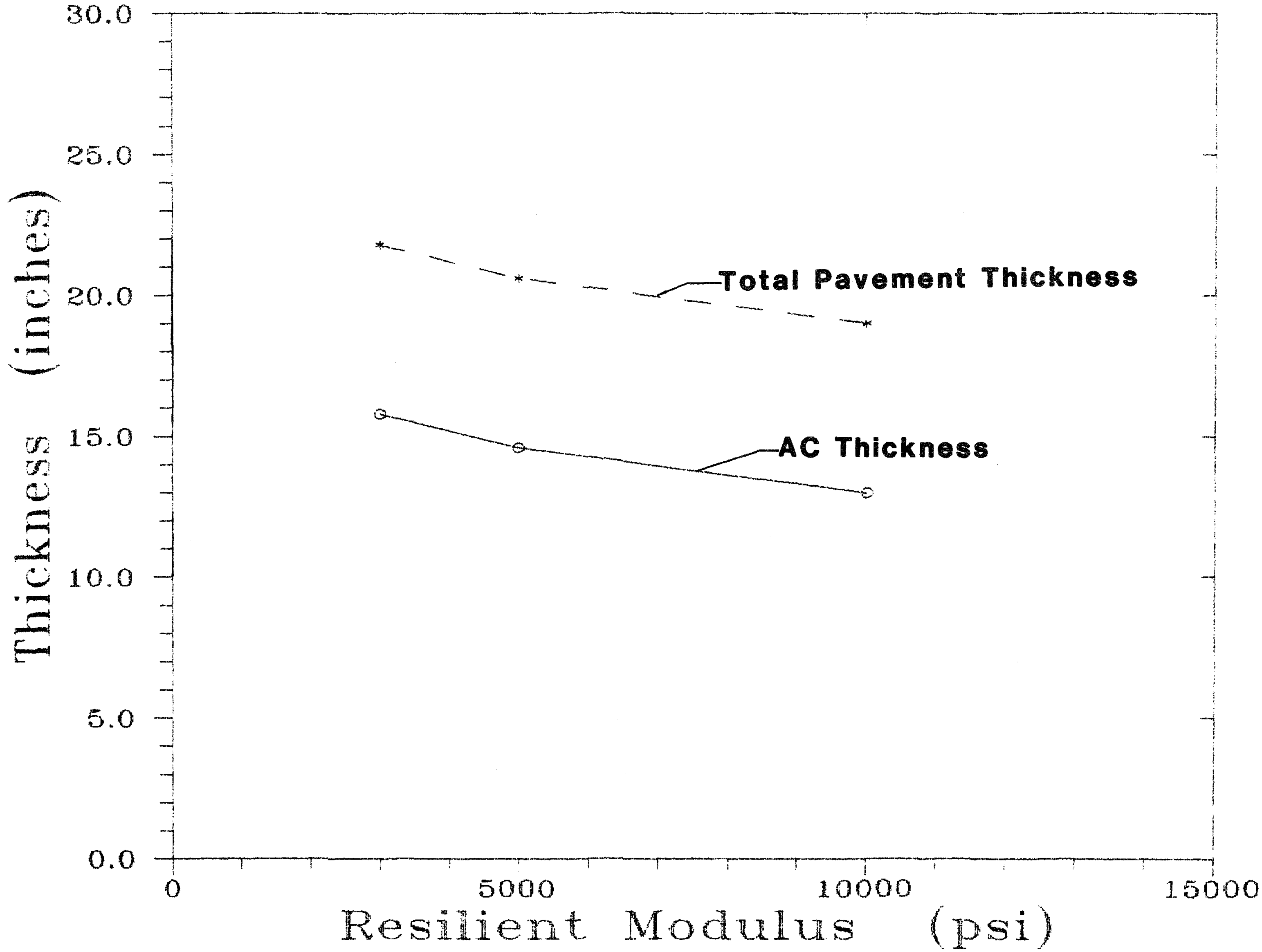

Figure 3-3.6 Sensitivity of the Asphalt Institute Flexible Pavement Thickness Design Procedure to Variations in Roadbed Soil Design Resilient Modulus.

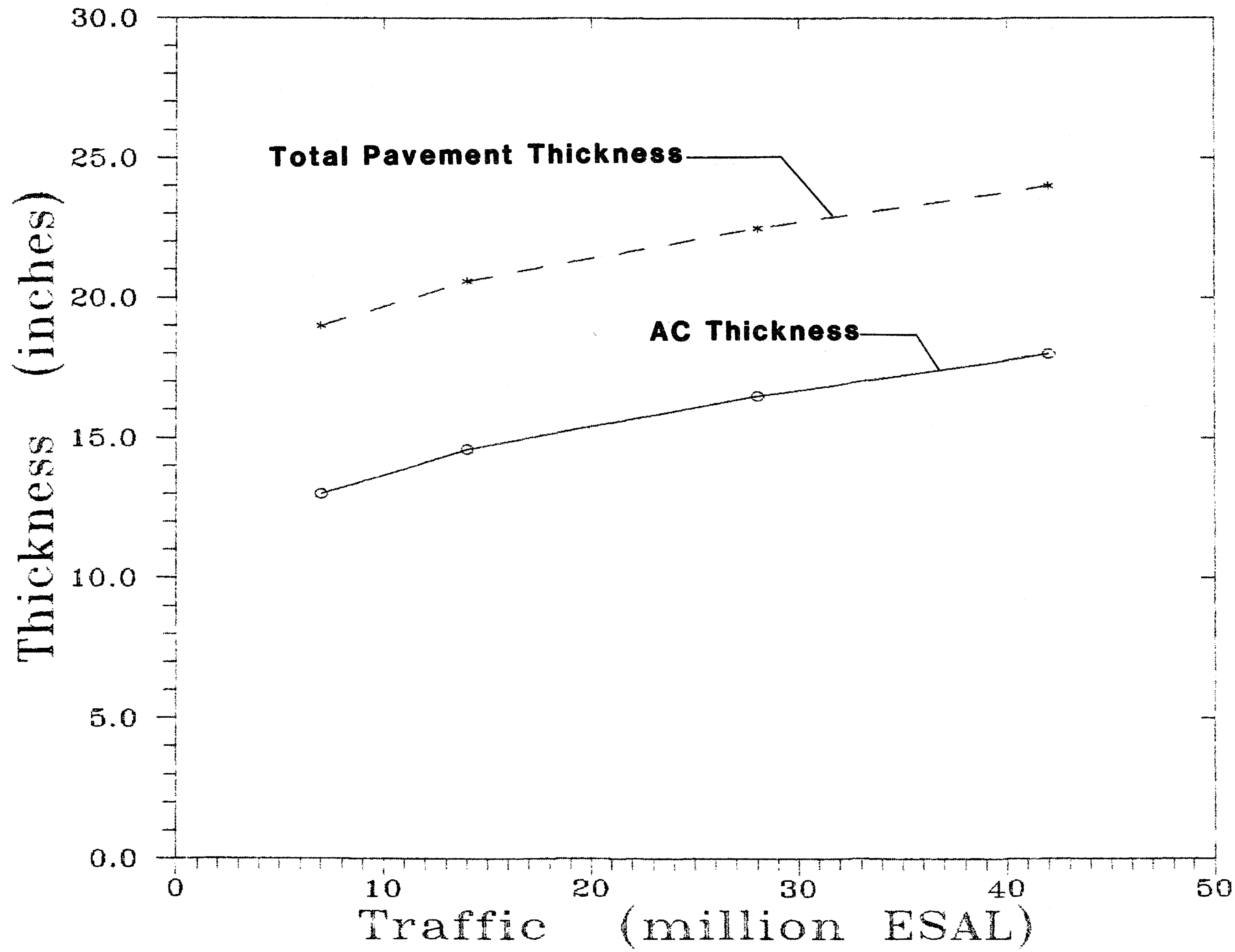

Figure 3-3.7 Sensitivity of the Asphalt Institute Flexible Pavement Thickness Design Procedure to Variations in Design Traffic.

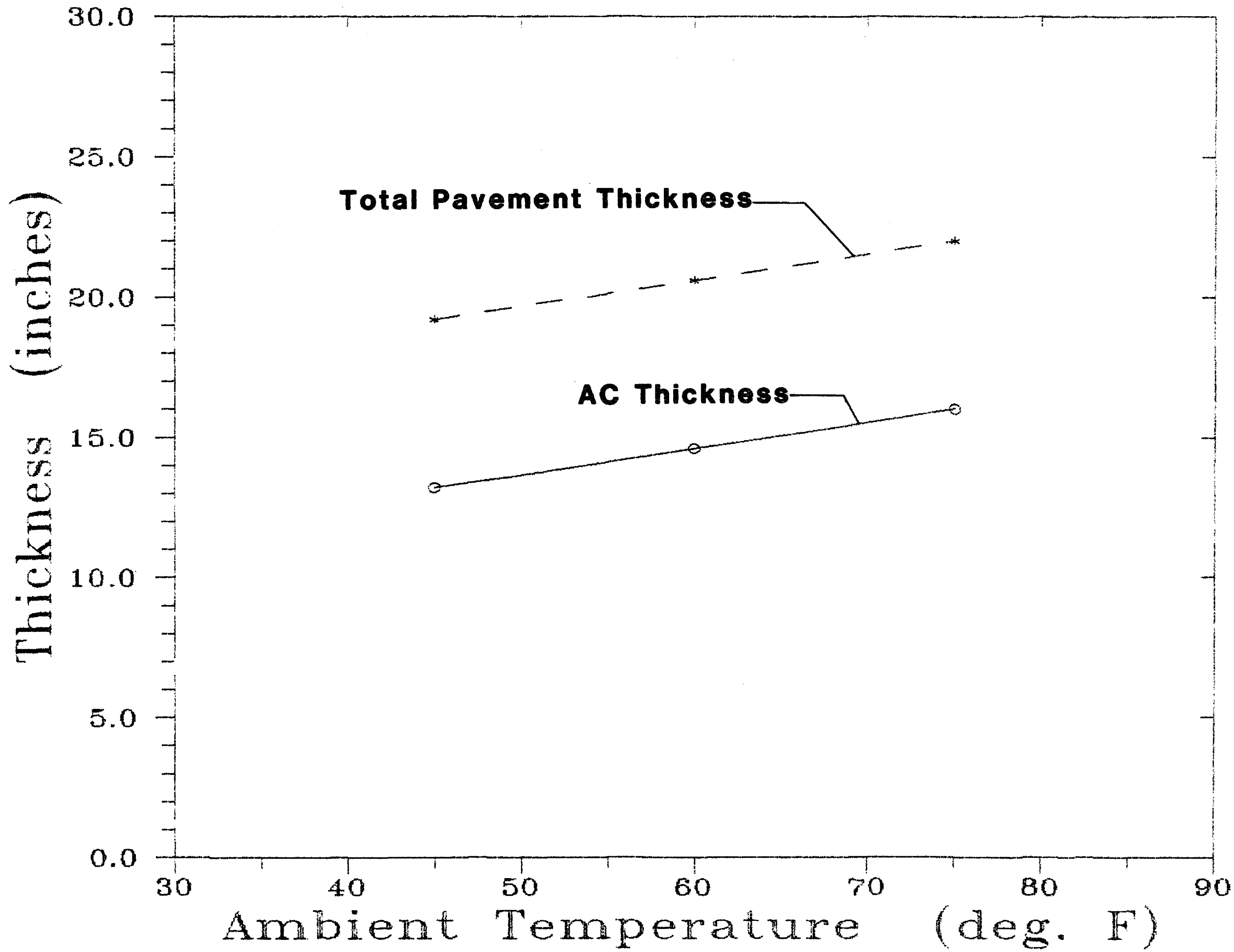

Figure 3-3.8 Sensitivity of the Asphalt Institute Flexible Pavement Thickness Design Procedure to Variations in Mean Annual Air Temperature.

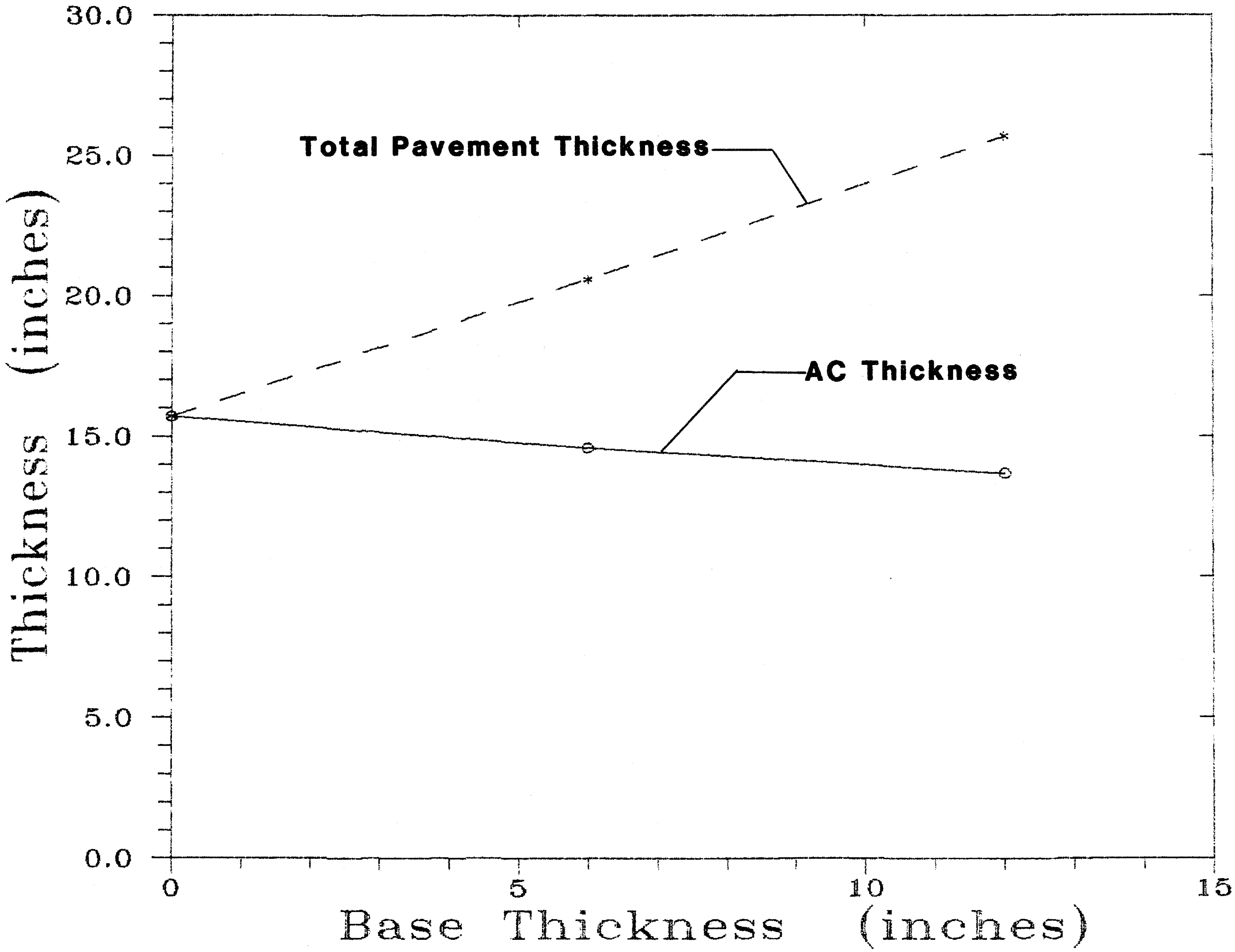

Figure 3-3.9 Sensitivity of the Asphalt Institute Flexible Pavement Thickness Design Procedure to Variations in Granular Base Thickness.

deformation decreases approximately linearly. As the resilient modulus increases from 3000 psi to 10000 psi (CBR from approximately 2 to 7), the required asphalt concrete thickness decreases from about 16 inches to 13. Thus, an increase in roadbed soil resilient modulus of more than 300 percent results in a total pavement thickness reduction of less than 13 percent in this case. Overall, the Asphalt Institute design procedure does not appear to be very sensitive to small changes in roadbed soil resilient modulus.

Figure 3-3.7 illustrates the effect of varying design traffic on pavement thickness. The largest increases in pavement thickness accompany traffic increases from relatively small volumes (<10 million ESAL). As the design traffic volume increases, further increases result in smaller increases in pavement thickness. Note that the thickness designs for traffic volumes exceeding 20 million ESAL are extrapolated from the Asphalt Institute design charts when a granular base is to be used. Design charts for full-depth asphalt concrete and emulsified asphalt mixes provide for traffic volumes up to 50 million ESAL.

Figure 3-3.8 shows the effect of assumed mean annual air temperature on pavement thickness design. The effect appears to be a fairly linear translation of the standard temperature (60^{O}F) design that adds or subtracts a little less than an inch and a half of asphalt concrete for a 15^{O}F increase or decrease in mean annual air temperature.

Figure 3-3.9 illustrates the assumed equivalency factor between asphalt concrete and granular base material. Each increase in base thickness of six inches results in a corresponding decrease in asphalt concrete surface thickness of only one inch and a net increase in overall pavement thickness of five inches. This six-to-one equivalency compares with the four-to-one equivalency found in performance studies at the AASHO Road Test.

3.5 Limitations of the Asphalt Institute Method

The major limitation of the Asphalt Institute procedure is that it does not consider environmental effects directly in the procedure. While there is an attempt to account for environmental effects in the roadbed soil resilient modulus and in the asphalt grade to be used, it does not accurately account for major climatic considerations such as seasonal variation in moisture.

Another problem lies in the limited environmental applicability of the design charts provided in MS-1. It contains only design charts for mean annual air temperatures of 60^{O}F, which accurately represents only a portion of the United States. This limitation can be overcome by using the design charts for mean annual air temperatures of 45 ^{O}F and 75 ^{O}F presented in Reference 2.

Finally, while the Asphalt Institute procedure has a firm basis in mechanistic analysis, it relies heavily on many empirical inputs. These include the computation of traffic equivalencies, the empirical assignment of limiting stress/strain criteria, and the use of empirical correlations between material strength parameters and resilient modulus by agencies without the proper testing equipment.

3.6 Asphalt Institute Computer Method (3)

The Asphalt Institute method for flexible pavement design (MS-1) was developed through the use of the computer program DAMA. DAMA was developed at the University of Maryland and is an elastic-layered pavement analysis program used to ascertain the repetitions to failure in the deformation and fatigue cracking distress modes. It was based on the Chevron N-layer program and allows for a more elaborate analysis of the input variables involved in the design process, particularly environmental effects on materials (2).

As an example, DAMA allows average traffic volumes to be entered monthly so that seasonal variations in traffic weight and density can be incorporated, (e.g., fertilizer trucks in spring and grain trucks in the fall in farming areas). Also, the computerized design method can be used to incorporate the environmental effects on the properties of each layer in the pavement structure by varying the layer moduli seasonally with provisions for monthly variation. The effect of temperature on asphaltic concrete of asphalt emulsions is done internally in the program; the user must input only mean monthly air temperatures. In addition, it allows for Poisson's ratio values to be input by the user.

There are several restrictions of the DAMA program, including the following items (2):

1. The program is limited to analyzing a five layer pavement system (4 pavement layers and a roadbed soil).

2. A maximum of one granular layer may be used, and it must be located directly above the roadbed soil.

3. An asphalt layer is required at the top of the pavement system.

4. The program can analyze dual wheel loads only; however, this can be overcome by increasing the spacing between wheels.

4.0 OTHER MECHANISTIC-EMPIRICAL DESIGN PROCEDURES

As discussed in Module 3-1, mechanistic-empirical design procedures are based on an analytical/theoretical study of pavement responses (stresses, strains, and deflections) through pavement modelling techniques. These theoretical pavement responses are empirically related to the performance of the pavement through laboratory studies and field testing to produce design procedures that are termed mechanistic-empirical approaches.

Several agencies have developed mechanistic-empirical models for general application to a variety of design considerations, including the Kentucky Department of Transportation (4) and Shell International (5). A summary of the advantages and disadvantages of mechanistic-empirical methods is given in Module 3-1.

5.0 NATIONAL STONE ASSOCIATION METHOD

The National Stone Association method is an empirical procedure based on the U. S. Army Corps of Engineers' pavement design procedure. The procedure uses a modified California Bearing Ratio test to determine the strength of roadbed soil. The NSA method incorporates crushed stone base courses as part of the pavement system. The basis of this method is to provide adequate thickness and quality of material to prevent repetitive shear deformation within any layer. Additionally, the effects of frost-action are minimized to acceptable levels (6).

This empirical design method uses only two basic input criteria to determine the total required thickness of the pavement structure. The first is the strength of the roadbed soil as determined by its California Bearing Ratio (CBR). The second is the amount of traffic (in 18-kip equivalent single-axle load applications) estimated to travel the roadway over a twenty-year design life.

The total pavement thickness is obtained from a design table using inputs of CBR and traffic (see Table 3-3.2). The thickness value obtained is then divided into asphalt concrete and granular base course layers, with a minimum thickness of asphalt concrete required for each particular level of traffic (in ESALs). The design is checked to ensure that it is adequate for frost susceptible roadbed soils.

As an example, if the roadbed soil has a CBR of 8 and if a traffic level of 350 ESAL/day (127,750 ESAL/year) is anticipated (i. e., DI-5), a total design thickness of 14 inches is obtained from Table 3-3.2. Thus, an asphalt concrete thickness of 3.5 inches will be used (from Table 3-3.2) over 10.5 inches of crushed stone.

The advantages of the National Stone Association are listed below:

1. The method is straightforward and simple to use.

2. The input requirements are minimal and usually easily obtainable.

3. The method has been revised as necessary through long-term monitoring of performance of in-service pavements.

There are also several disadvantages associated with the National Stone Association method of flexible pavement design. Among these are:

1. The strength properties of each layer above the roadbed soil are not considered (e. g., presence of a "very strong" layer would not decrease the requirements of the other layers).

2. The time and temperature dependence of the asphalt concrete layer is ignored.

3. A stabilized base course is not an option using this method.

4. Uncertainty and variability in performance may result from application of a "generalized" design procedure to a site-specific condition. This may result in over- or underdesigns.

5. The minimum thickness of asphalt as required by the method is not always sufficient to withstand the design traffic.

Table 3-3.2 National Stone Association Basic Design Thickness Table for Temperate Climate (6).

SUBGRADE SOIL Class	CBR	DESIGN THICKNESSES (INCHES) FOR INDICATED TRAFFIC INTENSITY CATEGORIES DI-1	DI-2	DI-3	DI-4	DI-5	DI-6
Excellent	15	5.0	6.0	7.0	8.0	9.0	10.0
Good	10-14	7.0	8.0	9.0	10.0	11.0	12.0
Fair	7-9	9.0	11.0	12.0	14.0	15.0	17.0
Poor*	3-6	13.5	16.5	18.5	20.5	23.0	26.0
Any class, minimum asphalt surfacing thickness (inches)		1.0**	2.0	2.5	3.0	3.5	4.0

*Poor soils should be upgraded or capped with subbase material to improve support to fair or better class.

**Use surface treatments, or increase to 1.5 inches including a prime coat on the compacted stone base if hot mixed asphalt is preferred as the surface.

TRAFFIC CATEGORIES:

DI-1: $\leq$ 5 ESAL/day ($\leq$ 1825 ESAL/year)

DI-2: 6 - 20 ESAL/day (2190 - 7300 ESAL/year)

DI-3: 21 - 75 ESAL/day (7665 - 27,375 ESAL/year)

DI-4: 76 - 250 ESAL/day (27,740 - 91,250 ESAL/year)

DI-5: 251 - 900 ESAL/day (91,615 - 328,500 ESAL/year)

DI-6: 901 - 3000 ESAL/day (328,865 - 1,095,000 ESAL/year)

In summary, the National Stone method is an empirical procedure based on a large amount of data. It is a simple procedure and only requires two inputs, roadbed soil CBR and projected design traffic. However, its simplicity is also one of its drawbacks, as several important variables are ignored.

6.0 EXAMPLE PROBLEM

Using the Asphalt Institute and National Stone Assocation flexible pavement design procedures, design a flexible pavement for a rural primary highway given the following data:

Pavement Location: .... Rural
Pavement Functional Classification:........ Primary
Traffic:
Expected Initial Traffic (Two-Way, ESAL) ... 1.33×10^6
Expected Directional Distribution, DD 0.50
Expected Truck Distribution (Design Lane), TD 0.70
Expected Annual Truck Traffic Growth ..... 4.0 percent
Construction Materials Properties:
Asphalt Concrete Modulus of Elasticity .... 300,000 psi
Granular Base Resilient Modulus .... 25,000 psi
Normal Roadbed Soil Seasonal Resilient Modulus Test Results (psi):

3500	3350	3700	3200	2900
3050	3050	3200	3550	3100

Normal Roadbed Soil CBR Test Results:

2.3	2.2	2.5	2.1	1.9
2.0	2.0	2.1	2.4	2.1

Mean Annual Air Temperature .... 60 ^{o}F

Solution:

The following design inputs might be selected based on consideration of pavement functional importance, stage construction considerations, knowledge of local construction quality, and engineering experience:

Performance Period .... 20 years
Analysis Period .... 20 years
Percentile Design Value .... 90 percent

The expected traffic for the 20-year performance period is estimated using the given information and procedures in Module 2-5.

$$\begin{aligned} EAL &= \text{Traffic Growth Factor} * \text{Initial Expected Traffic} * DD * TD \\ &= 29.78 * 1.33*10^6 \text{ ESAL} * 0.50 * 0.70 \\ &= 13.86*10^6 \text{ ESAL} \end{aligned}$$

This volume of traffic results in the daily application of nearly 2000 18-kip ESAL over the twenty-year design period.

The design roadbed soil resilient modulus can be determined as described in section 3.1.2 by selecting a value that is greater than only 10 percent (since we are using a 90 percentile design value) of the given test values. For the given data, the 90 percentile value can be estimated between 2900 and 3050 and a value of 3000 is chosen. Similarly, a 90 percentile CBR value of 2 is estimated. It is assumed that the other paving materials meet the requirements of the Asphalt Institute and the National Stone Association.

The given, assumed and computed data can be used to enter the Asphalt Institute design charts and produce the following possible design alternatives:

Alternative	AC Thickness (in)	Granular Base Thickness (in)	Emulsified Asphalt Mix Thickness (in)
1	16.5	0	----
2	5.0	0	11.5 (Type I)
3	5.0	0	14.5 (Type II)
4	5.0	0	20.5 (Type III)
5	16.0	4	----
6	16.0	6	----
7	15.5	8	----
8	15.5	10	----
9	15.0	12	----
10	14.5	18	----

The selection of the preferred design alternative must be based on several factors, including availability of materials, cost of required excavation for deep sections, frost protection and drainage requirements, etc. If the levels of performance designed for in the Asphalt Institute procedure (0.5 inches of rutting, 20-25% alligator cracking) are unacceptable, it may be desirable to increase the design traffic by some factor to produce a design that will provide better performance.

The design values can also be used to produce a design using the National Stone Association procedure, as described in section 5.0. The following design results:

Total Pavement Thickness = 26.0 inches
Minimum AC Surface Thickness = 4.0 inches

The National Stone Assocation recommends that the roadbed soil be improved or capped with a subbase to produce a pavement foundation with a CBR of 7 or more.

Extreme care must be used with any design procedure and the resulting designs should be considered with engineering experience and judgement, rather than used blindly. For example, the use of a 4-inch AC surface on a pavement as heavily-trafficked as the design example would certainly result in failure of the surface long before 20 years had passed.

7.0 SUMMARY

It is essential to consider more than one procedure when designing a pavement to ensure that a reasonable design is obtained. This module provides background information on other design procedures available for the design of flexible pavements.

The basis for mechanistic-empirical design procedures were discussed, including their advantages and disadvantages. Their main advantages lay in the accurate way that they model the behavior of a particular pavement section, predict specific distresses, and can incorporate complex solutions, but this can be a problem since they are so complex that they require a computer and good pavement knowledge.

The Asphalt Institute design method, developed through the use of the DAMA computer program, is based on limiting critical strains to allowable values. The strains considered are the horizontal tensile strain on the underside of the lowest asphalt bound layer (either asphalt concrete or emulsified asphalt treated), and the vertical compressive strain at the surface of the roadbed soil. Layer properties (modulus and Poisson's ratio) have been preselected based on various laboratory tests and research projects and represent typical values for these materials; however, the user selects the roadbed soil resilient modulus. Design charts require traffic and roadbed soil property inputs to produce a design thickness that is adequate for the more critical strain condition. The procedure is applicable to warm climates but is of questionable validity when applied to either hot or cold climates.

The computerized version of the Asphalt Institute design method (program DAMA) provides means for including climatic and environmental effects in greater detail than most other rational or semi-empirical design methods and addresses many of the weaknesses of the manual MS-1 procedure.

The National Stone Association method is an empirical procedure based on the Corps of Engineers design methodology. It requires only a CBR value for the subgrade and estimated traffic levels. While it is a simple procedure to use and is based on a large amount of data, the procedure does not consider the strength properties of the material, and it may not be applicable to all regions. The minimum thicknesses of surfacing it suggests may be inappropriate for high traffic volume applications.

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MODULE 3-4

FLEXIBLE PAVEMENT SHOULDER DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module deals with the design of shoulders for flexible pavements. Topics addressed include material selection, traffic and environmental effects, maintenance and safety considerations and thickness design concepts.

Upon completion of this module, the participant will be able to:

1. Discuss the purposes and benefits of paved shoulders.

2. Identify factors and inputs that influence shoulder design, including mainline pavement type, planned uses of the shoulder, subgrade and paving materials, environmental effects, safety and maintenance considerations.

3. Design a paved shoulder for a flexible pavement for a variety of traffic, climate, materials and other input conditions.

4. Identify common types of shoulder distress and discuss their causes and prevention.

2.0 INTRODUCTION

A highway shoulder is "the portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles, for emergency use and for lateral support of base and surface courses" (1). In addition, shoulders provide recovery space for errant vehicles, accident avoidance areas, lateral clearance to signs and guardrails, improved sight distance in cuts, and space for maintenance operations (2).

The desirable features of a shoulder include (3):

1. Clear delineation between travel lanes and the shoulder to minimize enroachment.

2. Adequate cross-slope for good drainage.

3. Sufficient width for emergency use, drainage control, and guardrail installation.

4. A flush transition at the through-lane edge.

5. Inherent structural stability.

6. A pavement-shoulder joint that remains sealed.

7. Efficient and economical maintenance.

8. Low total construction and maintenance costs.

Other added functions might include (5):

1. The accommodation of encroaching traffic.

2. Provision of extra space for construction and maintenance activities.

3. Use as an extra lane during peak hours to relieve traffic congestion.

4. Use as a bicycle path or travel lane for slow-moving vehicles and equipment.

Various types of shoulder, ranging from grass to continuously reinforced concrete, have been constructed in the U. S.; however, grass and most gravel shoulders do not meet the desirable features discussed above. Therefore, only flexible shoulder designs over granular or stabilized bases will be discussed in this module.

Recent surveys indicate that most states do not have a shoulder design method with the result that shoulder performance has generally been unsatisfactory (22). A major conclusion of one NCHRP study (4) was that most shoulders are considerably under-designed. Over the last decade poor shoulder performance has resulted in a number of shoulder studies. The results of these studies are summarized in NCHRP reports 63 and 202 and show that shoulders that have structurally adequate designs for the expected traffic generally perform satisfactorily, particularly if the pavement-shoulder joint is properly sealed, and/or if adequate drainage is provided.

3.0 FACTORS AFFECTING SHOULDER DESIGN

The following broad categories influence shoulder design:

1. Mainline pavement (and shoulder) type.

2. Traffic.

3. Environment.

4. Safety.

5. Planned maintenance strategy.

6. Thickness design concepts.

These are discussed in more detail in the subsections below.

3.1 Mainline Pavement and Shoulder Type

Selection of the shoulder type is the first step in shoulder design. It is a very important step because selection of an inappropriate shoulder material can lead to performance problems for the mainline pavement as well as the shoulder.

Hicks and Barksdale (4) have reported that one of the major problem areas with shoulders is related to the pavement-shoulder joint. This joint is particularly troublesome when the pavement and shoulder are constructed using dissimilar materials (e.g., asphalt-concrete shoulders adjacent to PCC pavements). Dissimilar materials generally have different thermal properties and expand or contract at different rates, which introduces additional stresses into joint sealants that are already being stressed by differential vertical deflections across the joint. This problem is further compounded by the fact that few joint sealants are suited for bonding to dissimilar materials.

FHWA Technical Advisory T5040.18 for pavement shoulders (5) recommends the use of shoulders constructed of materials similar to those used in the mainline pavement. It is believed that this policy facilitates construction, improves overall pavement performance, and reduces long-term maintenance costs. Longitudinal joints between flexible shoulders and flexible mainline pavements generally do not experience serious deterioration problems if the foundation material under the shoulder is designed for traffic encroachments and is properly compacted during construction. Even if some separation takes place, the sealing of these joints is much simpler than in the case of flexible-rigid combinations.

As discussed previously, grass, granular and other low-type shoulder materials have been used for shoulder construction, but are not generally desirable for high-type pavement construction.

3.2 Traffic

Traffic has generally not been considered in shoulder design, primarily on the assumption that shoulders see very little traffic. There is some evidence (4, 6, 7), however, to indicate that when there is a paved shoulder and no lateral obstructions, trucks using the outer lane tend to encroach on the shoulder by as much as 12 inches and sometimes more. A recent study by Emery (8) has shown that at least 2.4 percent of the truck traffic in the outer lane encroaches on the shoulder. Barksdale et al. (4) have recommended that 2.5 percent of the mainline truck traffic should be used in shoulder structural design. These studies were conducted on PCC pavements with either unpaved or different types of paved shoulders (other than PCC). Thus, different percentages of encroaching traffic could be expected when the shoulders are constructed of the same material as the mainline pavement. In addition, the location of the projects surveyed could influence study results. These factors indicate that a survey of local traffic conditions is necessary to obtain appropriate numbers for a specific design case.

Some states use the shoulder as an additional traffic lane during maintenance operations or during peak traffic periods in urban areas. If such a use is anticipated, this additional traffic must also be considered in the design.

In addition to planned traffic and vehicle encroachment, the shoulder is also used for parking by disabled vehicles. It is reasonable to assume that parked vehicles tend to move close to the outer edge of the shoulder, which indicates the need for a uniformly strong structural section.

The percentage of parked and encroaching traffic can be estimated from the results of national surveys or from the procedure recommended by Sawan et al. in "Structural Analysis and Design of PCC Shoulder" (9). Since the above estimates are relatively imprecise, it may also be adequate to simply estimate the projected total traffic for the highway (in terms of 18-kip equivalent single-axle loads) using the procedures outlined in Module 2-6 and assign a percentage of this traffic (e.g., 3 - 5 percent) to the shoulder for shoulder design purposes.

The procedure recommended by Sawan et al. for computing ESALs due to encroachments and parked traffic is described below.

3.2.1 Encroaching Traffic

Encroaching traffic is that part of the mainline traffic that occasionally encroaches on the shoulder and then returns to the mainline pavement. Encroaching traffic normally travels in the vicinity of the mainline/shoulder joint and encroaches 12 inches (305 mm) or less onto the shoulder.

The percentage of trucks that will encroach on the shoulder should be estimated from field surveys of projects with similar designs and traffic levels. Trucks should be selected at random and followed by observers over a selected distance (L_s) of several miles. Records are made of the time the truck travels on the shoulder to determine the longitudinal distance for each encroachment (using the average truck speed which would be approximately the same as the observer's vehicle speed). This survey can be used to determine the average number of encroachments per truck in the surveyed pavement sections (N_e) and the average length (on the shoulder) of each encroachment (L_e).

The average daily truck volume in the traffic lane nearest the shoulder ($ADTT_{outer}$) is determined by multiplying the one-directional average daily traffic (ADT) by the percent of truck traffic (%T) and the lane distribution factor for that lane (LDF):

$$ADTT_{outer} = ADT \times \%T \times LDF$$

The average daily number of truck encroachments in the surveyed section (ADTE) can be estimated by multiplying the average number of encroachments per truck (N_e) by the average daily truck traffic volume in the traffic lane nearest the shoulder ($ADTT_{outer}$):

$$ADTE = N_e \times ADTT_{outer}$$

The average daily total encroachment distance in the surveyed section (ADL_e) can be estimated by multiplying the average daily number of truck encroachments (ADTE) by the average length of each encroachment (L_e):

$$ADL_e = ADTE \times L_e$$

The average daily number of encroachments (ADE) at any given point in the surveyed section is obtained by dividing the average daily total encroachment distance (ADL_e) by the length of the surveyed section (L_s).

$$ADE = ADL_e / L_s$$

Finally, the proportion of trucks in the outer lane that encroach on the shoulder (PET) can be estimated as the ratio of the average daily number of encroachments at a given point (ADE) to the average daily truck traffic in one direction in the outer lane ($ADTT_{outer}$):

$$PET = ADE / ADTT_{outer}$$

Actual calculations show that the PET typically varies over a range of approximately 0.01 to 0.08 (1 - 8 percent) of the adjacent lane truck volume. The percentage of parking truck traffic should be added to this number because any truck has to encroach in order to park on the shoulder.

3.2.2 Parked Traffic

Parked traffic is that percentage of mainline truck traffic that parks on the shoulder for emergencies or other reasons. This input can also be estimated for the design section using traffic counts obtained from sections with similar designs and traffic. Parking traffic typically varies greatly along a given project, depending on geometric and interchange conditions. Most trucks park near interchange ramps. Thus, it may be desirable to identify separate design sections for areas where parking is likely and for areas where minimal parking is expected.

The computation required to determine the number of expected load applications that will occur along the outer shoulder edge in the selected design section is described below.

A length of pavement (L_s) that is representative of the design section must be selected. The average daily truck volume in the traffic lane nearest the shoulder ($ADTT_{outer}$) is determined as before:

$$ADTT_{outer} = ADT \times \%T \times LDF$$

The average daily number of trucks that actually park in the surveyed section (ADTP) must be determined by a visual count over one or more typical 24-hour periods. Since most parking occurs during the very early morning hours this period must be included. The average distance the trucks drive on the shoulder during a typical stop (L_p) is also determined by actual observations.

The percentage of trucks that park on the shoulder can now be estimated. The design section is first divided conceptually into "subsections" of length L_p. It is assumed that the probability that a parking truck will park in any given "subsection" is equal to P, where:

$$P = 1/(L_s/L_p)$$

Thus, the <u>percentage</u> of <u>total truck traffic</u> in one direction that parks on <u>any random "subsection"</u> (PPT) <u>is computed</u> as:

$$PPT = ADTP \times P \times 100 / ADTT_{outer}$$

Surveys and calculations from in-service pavements show that PPT may range from percentages of 0.0005 to 0.005.

3.2.3 Regular Traffic

If it is anticipated that the shoulder would be used by regular traffic at any stage of its design life, then this extra amount of traffic should be counted as a part of the shoulder design traffic and the amount of traffic at both edges of the PCC shoulder should be increased accordingly. The ultimate case is to design the shoulder as an extra lane by considering its traffic to be similar to the mainline outer lane truck traffic.

The above information must still be converted to 18-kip equivalent single-axle loads using the methods discussed in Module 2-6.

3.3 Environmental Effects

Although traffic loads are primarily responsible for shoulder fatigue, shoulder performance is also significantly affected by the magnitude and distribution of rainfall, volume change of non-stabilized materials (particularly long-term creep and consolidation or settlement), maximum and minimum temperatures, number of freeze-thaw cycles, and depth of frost penetration. Environmental effects on shoulder performance are generally more severe than for mainline pavement performance because shoulders are often built with lower quality materials and lower construction quality (e.g., lower required compaction effort). Since shoulder sections are often constructed thinner than the mainline pavement, the effects of frost penetration and freeze-thaw cycles are likely to be more significant, especially for frost-susceptible soils.

The following paragraphs address the consideration of environmental effects in pavement shoulder design.

3.3.1 Moisture and Drainage Effects

NCHRP studies (2, 4) have concluded that moisture infiltration and improper drainage are the major causes of premature shoulder failure. Surface infiltration, high groundwater, and capillary rise may result in excessive deflection, reduced load-bearing capacity and frost action. These problems may show up as alligator or fatigue cracking, potholes, stripping, ravelling and weathering, frost heave, and general pavement disintegration. Moisture problems are best addressed by including adequate drainage (through the use of permeable foundation materials and/or subdrains) or using materials that are less susceptible to the presence of moisture. These concepts are addressed briefly below and in Module 3-1, and in detail in Block 2.

Foundation soil type is an important factor in drainage design. Granular or highly permeable soils increase the effectiveness of pavement drainage systems and reduce the possibility of pavement damage due to loss of support.

Soil type takes on added importance in climates where frost may penetrate the subgrade under the shoulder pavement, particularly if the soils are fine-grained and are susceptible to frost damage. Specific design approaches to frost protection are discussed in Module 3-1. The amount of frost protection required for flexible shoulder design is often less than that required for the mainline pavement (unless the shoulder is intended to carry a large volume of traffic) because the consequences of failure are not as severe.

Dempsey et al. (10) have concluded (based on both theoretical analysis and field tests) that climate and foundation soil type also influence the location of subdrainage pipes. Their study indicates that for climatic regions with frost depths less than the thickness of the surface layer, the minimum distance from the top of an impermeable subgrade to the bottom of the subdrainage pipe should be about 1 to 2 inches (25 to 51 mm) with the pipe installed below the subgrade. At locations with extreme winter climates (either very warm or very cold) the drainage pipe should be moved closer to the surface than current specifications prescribe.

AASHTO (21) recommends avoiding the use of aggregate base courses having a significant percentage of minus 200 mesh sieve materials to prevent frost heaving, pumping, clogging of the drainage system, and base instability.

A system for the evaluation of the potential for moisture-accelerated distress and related pavement performance has been developed by Carpenter et al. (11) and designated MAD (Moisture Accelerated Distress Identification System). The MAD Index is based on the climate of the area and properties of the pavement foundation materials and can be used for both new designs and investigation of rehabilitation needs.

3.3.2 Temperature Effects

Design considerations for the effects of temperature variation on asphalt concrete shoulder performance are essentially the same as for the mainline asphalt concrete pavement, although higher levels of risk may be acceptable for the shoulder area. Temperature-related distresses are primarily limited to thermal cracking at low temperatures and pavement distortion (rutting, shoving, corrugation) at high temperatures. Both phenomena can be minimized by the proper selection of asphalt cement (hardness, viscosity) and mix design. Recommended grades of asphalt cement for various climates and layer thicknesses can be determined using the procedure developed by Basma and George (12).

The adverse effects of temperature changes on the performance of joints between dissimilar materials was discussed previously with the recommendation that shoulders should be constructed of the same material as the mainline pavement.

3.4 Safety Considerations

Various studies have shown that shoulder improvement (provision of wider or paved shoulders) is generally the single most cost-effective action with respect to safety considerations, resulting in significantly lower accident rates (13,14,15,16,17). Safety considerations, therefore, should be carefully weighed in determining the shoulder width, alignment, and continuity.

Shoulders provided adjacent to high-type facilities should be approximately 12 feet (3.7 m) wide. This allows 1 to 2 feet (0.3 to 0.6 m) of clearance from the edge of the traveled lane to parked commercial vehicles and 3 to 4 feet (0.9 to 1.2 m) for passenger cars. Somewhat narrower shoulders may be acceptable for lower volume facilities. The shoulder should also be continuous and without variations in width or elevation with respect to the mainline pavement.

It is recommended in NCHRP Synthesis Report No. 63, "Design and Use of Highway Shoulders" (2), that the shoulder cross-slope be steeper than that of the traffic lane in order to drain surface water more rapidly, but that the cross-slope should not be so steep as to be a hazard. FHWA guidelines (5) recommend shoulder cross-slopes on the order of 2 to 8 percent, provided that the algebraic difference in cross-slope between the shoulder and mainline pavement does not exceed 7 percent.

There is also some evidence (18) that delineation of the shoulder from the mainline pavement by striping, color change, or the use of rumble strips is effective in reducing wander and traffic encroachment on the shoulder.

3.5 Maintenance

The degree of maintenance that will be required depends largely on the adequacy of the structural design, type of drainage, and the similarity of the materials used in the mainline pavement and shoulder. Most existing shoulders are structurally underdesigned and exhibit severe distresses such as horizontal and vertical joint separation, fatigue cracking, rutting, potholes, and ravelling. Maintenance strategies for these shoulders varies with the severity of the distresses, is generally expensive and includes large amounts of patching and crack sealing, reconstruction, recycling, or resurfacing.

The amount of maintenance required for properly designed shoulders is often limited to maintaining the shoulder-pavement longitudinal joint and providing periodic surface seals (chip, fog, slurry, etc.). Clearly, the provision of a properly designed shoulder section provides a higher level of serviceability and is more cost-effective over the long run.

It should be noted that stage design and construction of the mainline pavement will require similar shoulder design and construction, including required overlays to maintain vertical continuity with the mainline pavement even though no shoulder deterioration is evident. In any case, a definite program of shoulder maintenance should be planned and followed to assure cost-effective and serviceable shoulders.

3.6 Thickness Design Concepts

Flexible shoulder thickness design must consider two critical areas: 1) the shoulder edge adjacent to the mainline pavement, which must withstand the effects of encroaching traffic, and 2) the outer edge, which must be designed primarily for parked vehicle loads unless the shoulder is to be used as a traffic lane during periods of congestion or construction. Possible design approaches are identical to those for flexible mainline pavement design which were discussed in previous modules in this block (empirical and mechanistic-empirical), although different specific design procedures are available.

The minimum recommended total shoulder thickness is 6 inches (152 mm) because various shoulder performance studies have shown this to be the minimum thickness for satisfactory performance. This minimum applies to both flexible and rigid shoulders. The inner edge thickness design should also be similar to that of the mainline pavement in order to avoid problems associated with varying support and material volume changes along the longitudinal pavement/shoulder joint.

Various agencies have suggested that the shoulders should be designed integrally with the mainline pavement to allow for maximum safety and performance, flexibility in shoulder use during periods of construction or congestion, and the potential for future additions of new traffic lanes. On low volume roads, similar advantages could result from the construction of a 28-30 ft mainline section with granular shoulders. The mainline pavement could then be striped to provide standard traffic lane widths and a sufficiently wide paved/granular shoulder to permit safe emergency use and reduce premature deterioration due to encroachment.

4.0 DESIGN PROCEDURE

4.1 General Approach

Figure 3-4.1 presents one approach to flexible pavement shoulder design which is discussed in more detail below.

The first step is the determination of future shoulder use. If it is likely that the most significant accumulation of loadings will result from the use of the shoulder as a traffic lane at some point in the future, the shoulder should be designed using the same procedure used for the mainline pavement, and an identical thickness design should be considered. Otherwise, the steps outlined below should be followed.

The foundation soil type should be analyzed to determine its susceptibility to frost damage. If the average annual depth of frost penetration is greater than six inches (152 mm) (the minimum thickness of bound material above the subgrade, as suggested by the referenced performance studies), then the total thickness of subbase and/or base material above the subgrade must consider frost protection using some appropriate design approach, such as the Corps of Engineers' procedure (19).

Next the environmental conditions must be evaluated using the MAD system (11). If the MAD Index indicates that drainage is needed, a drainage layer is assumed and its adequacy checked using the drainage evaluation criteria in the MAD system. If the assumed drainage is inadequate, a different design must be selected. The thickness of the drainage layer should then be compared with the minimum thickness required for frost protection. In the event that the drainage blanket is thinner than required for frost protection, a subbase is added (or its thickness increased) and the adequacy reevaluated. The MAD system can also be used to determine the need for underdrains and their size and spacing, if required.

The most difficult step in shoulder thickness design is the determination of material properties (e.g., layer moduli) for use in the design procedures. The determination of materials properties is described in detail in Block 2. The effect of drainage on base/subbase/subgrade properties should also be investigated using models such as those developed in EAROMAR (6) or the CMS system developed by Liu and Lytton (20). The EAROMAR models require less detailed climatic input data and may be preferred by many highway engineers.

The next step is to conduct the traffic analysis, including the number of projected 18-kip equivalent single-axle load applications and the location of their application (i.e., inner edge due to encroaching traffic or outer

EXAMPLE APPROACH TO FLEXIBLE PAVEMENT SHOULDER DESIGN
(NEW CONSTRUCTION)

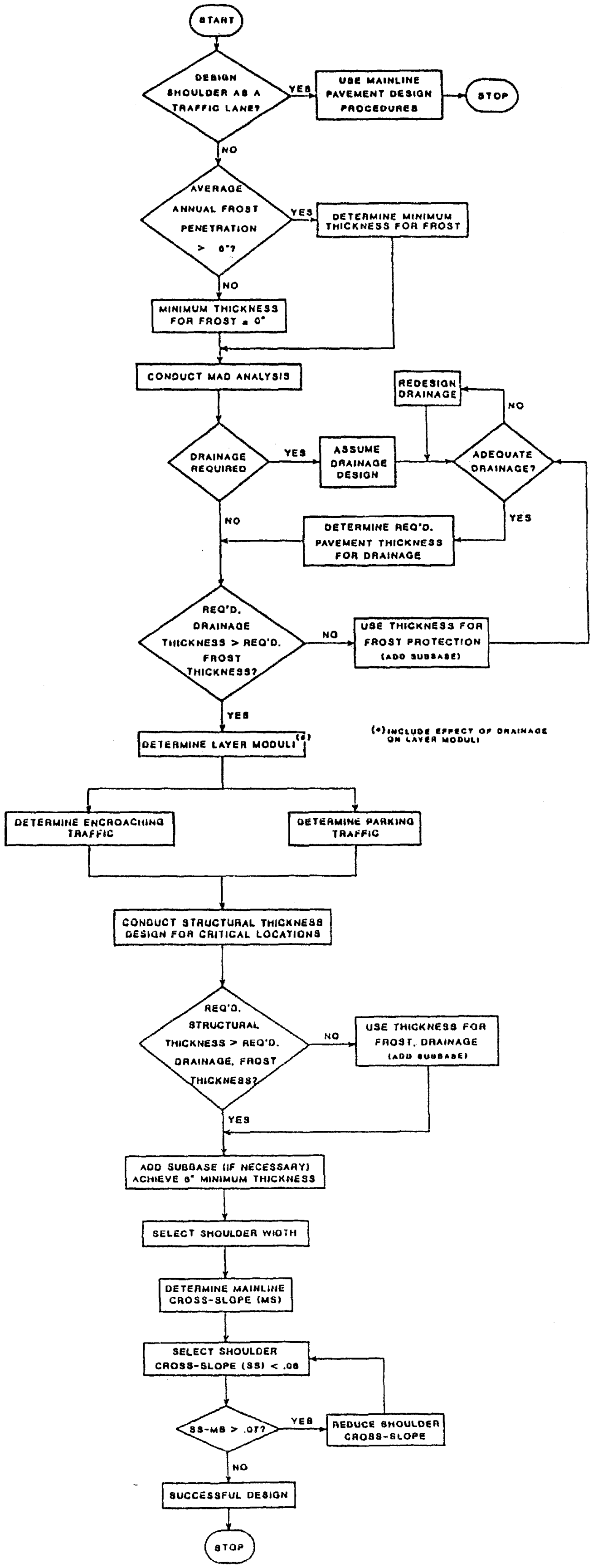

Figure 3-4.1 Flow Chart for Shoulder Design

edge due to parking traffic). The procedure described by Sawan et al. (9) in "Structural Analysis and Design of PCC Shoulders" is recommended, although other methods for estimating traffic may be appropriate.

With loading, materials properties and climate inputs in hand, the designer can conduct structural analyses using an appropriate empirical or mechanistic design procedure. The inner and outer shoulder edges must be designed separately to consider the different load types and volumes accommodated by each location (encroachments and parking) which may produce different types of failure (fatigue cracking and rutting). Design corrections to reduce the possibility of these distresses are similar to those described in Module 3-1 for mainline pavements. The more conservative design is generally selected for the entire shoulder pavement structure. The structural thickness design should be checked against the thickness designs required for frost protection and drainage and modified as necessary to meet these criteria.

The final step in the flexible pavement shoulder design is to select an appropriate shoulder width and check compliance with geometric restraints such as cross-slope restrictions. The design engineer has the option of sloping the subgrade as well as the shoulder. Current FHWA criteria suggest that the slope difference between the mainline pavement and the shoulder should not exceed 7%, with an 8% maximum slope for the shoulder.

5.0 DESIGN EXAMPLES

Design examples will be presented during the training course to illustrate the design of a flexible pavement shoulder using mechanistic and empirical design approaches.

6.0 SUMMARY

A highway shoulder is defined as that portion of the roadway contiguous to the travelled way which accommodates stopped vehicles, allows emergency use, and provides lateral support to the base and surface courses. Desirable shoulder design features are noted to include a well-defined width sufficient for designated use, flush transition from the driving lane, well-sealed joints, stable structure with good drainage, and efficient and economical construction and maintenance. These features are not all generally found in turf and granular shoulders.

Several broad categories of factors influence shoulder design, including mainline and shoulder pavement type, traffic volume and composition (including encroaching, parking and future use traffic), environment (moisture and temperature effects on materials), safety, planned maintenance and stage construction strategies, and thickness design approaches. Each of these factors should be analyzed and considered together to establish an "optimum" design.

The steps in the design procedure discussed in this module include determination of future shoulder use, thickness design for frost protection, thickness design for drainage analysis, determination of layer properties after drainage, traffic analysis, structural thickness design, and geometric design with consideration of safety factors.

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BLOCK 4

Rigid Pavement Design

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BLOCK 4.0

RIGID PAVEMENT DESIGN

Modules

4-1 BASIC PRINCIPLES OF RIGID PAVEMENT DESIGN
4-2 AASHTO METHOD FOR RIGID PAVEMENT DESIGN
4-3 OTHER DESIGN METHODS
4-4 RIGID PAVEMENT SHOULDER DESIGN

This block presents both the empirical and mechanistic design concepts and procedures and demonstrates how the inputs and other factors are considered. Steps for developing new design procedures are also described.

This block of instruction shows how each design procedure was developed, discusses their strengths and limitations, and demonstrates the sensitivity of the resulting designs to variation in the input variables. Familiarity with the AASHTO and PCA design procedures will be accomplished through the use of micro-computers for "hands-on" of example problems.

Participants will also be introduced to rigid pavement shoulder design considerations and concepts to complete the rigid pavement structural design process.

Upon completion of this block, the participants will be able to complete the instructional objectives listed for each module.

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MODULE 4-1

BASIC PRINCIPLES OF RIGID PAVEMENT DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module presents the basic principles of rigid pavement design, including stress relationships, empirical design methods (e.g., AASHTO Design Guide), and the mechanistic-empirical design approaches.

Upon completion of this module, the participants will be able to accomplish the following:

1. List the components of a rigid pavement system that need to be designed and discuss how these components interact to affect overall pavement performance.

2. Describe the stresses that occur in rigid pavements and the critical locations of those stresses.

3. List the major assumptions, capabilities and limitations of Westergaard theory.

4. List the major distress types that must be considered in a comprehensive rigid pavement design procedure.

5. Describe the general approach to empirical rigid pavement design as used in the AASHTO Design Guide.

6. Describe the general approach to the mechanistic-empirical rigid pavement design.

7. List the major factors that must be considered in design to control the major distress types for rigid pavements.

8. Identify the advantages and disadvantages of both the empirical and mechanistic-empirical approach to rigid pavement design.

2.0 OVERALL DESIGN OF RIGID PAVEMENTS

Pavement "design" may be defined as follows:

The determination of structural, material and drainage characteristics and dimensions of the pavement/subgrade structure (including all components of the pavement) through direct analytical consideration of the traffic and climatic loads that the pavement/subgrade structure is expected to be subjected to over a selected design period.

The key concept here is that true "design" includes direct analytical consideration of the loadings (both traffic and climate) for all components of the rigid pavement including the following:

1. Slab dimensions (thickness, width, length).
2. Slab strength.
3. Transverse and longitudinal joints.
4. Base/subbase materials (durability, erodability, permeability) and thickness.
5. Subdrainage of the pavement.
6. Shoulders.
7. Reinforcement (if used).
8. Subgrade treatment (if any).

2.1 Past Rigid Pavement Design Practice

In the past, rigid pavement "design" has normally consisted of either a fatigue analysis or serviceability (roughness) analysis to determine slab thickness required for the expected traffic level. Often this thickness was inadequate because the "design" of other features, such as subdrainage, was largely ignored, or was designed independently of the other components using historical standards. Very little actual design or analysis has been conducted for such features as joints, shoulder, erosion of base or reinforcement. For example, joint design has not directly included the consideration of level of traffic over the design period and its effect on joint faulting or spalling.

Little consideration has been given to the design of the concrete pavement structure as an <u>overall</u> <u>structural</u> <u>system</u>. All of the components of a rigid pavement must not only be designed to be structurally adequate by themselves, but also to function together successfully as a structural system (e.g., the traffic lanes/shoulder/pavement-layers/subgrade structure). The failure to adequately consider each of these components and to consider their interaction as a structural system in the past has led to premature failures of many pavements.

For example, design slab thickness has been determined independently of joint spacing for JPCP. It is well known that joint spacing has a great effect on transverse cracking of JPCP due to thermal and moisture gradients in the slab. Joint spacing also has a large effect on the faulting of the transverse joints where dowels are not used. In conjunction with this, joint spacing in JPCP has been selected independently of the stiffness of the supporting subbase. When a very stiff, lean concrete base is used, the thermal curling stresses increase dramatically, and if joint spacing is not shortened, serious transverse cracking will occur.

Another example of a design theory which does not consider all the important factors is the reinforcement design of JRCP based on the "subgrade drag theory." This theory does not consider the number of heavy traffic loadings which cause repeated deflection at transverse cracks. Such deflection results in deterioration of the crack if adequate reinforcement is not provided.

2.2 Approaches to Rigid Pavement Thickness Design

There are two different approaches to the thickness "design" of rigid pavements: the empirical (Figure 4-1.1) and the mechanistic (Figure 4-1.2, sometimes called mechanistic-empirical because it includes both mechanistic and empirical aspects) approach. Rigid pavement design procedures developed over many years have mostly utilized mechanistic concepts (e.g., calculation of critical stress in the slab due to wheel loads along with fatigue damage concepts) to arrive at structural designs. However, one major exception to this is the AASHTO Design procedure which was developed completely from field data.

3.0 STRESSES IN RIGID PAVEMENTS

A rigid pavement is a complex layered structure with discontinuities (joints) that is subjected to several "loadings" from traffic and climate. Stresses in rigid pavements, which result from these "loadings", are caused by traffic, thermal gradients, moisture gradients, drying shrinkage, thermal heating and cooling of the slab, and foundation movements. The following sections describe structural analysis concepts applied to rigid pavement systems.

3.1 Westergaard Theory for Calculation of Traffic-Related Stresses

The determination of stresses and deflections in slab-on-grade pavements with joints and/or cracks has been studied for many years. In the early 1920's, Westergaard developed closed-form analytical equations for stresses and deflections for jointed rigid pavements (1). In his original work, three cases of loading were investigated: (a) center slab, (b) edge, (c) corner (See Figure 4-1.3). The tire imprint is approximated by a circular loading.

Assumptions and limitations of Westergaard theory, the equations, graphical and computer solutions of the equations, and conclusions drawn from Westergaard's studies will be discussed in the following sections.

3.1.1 Assumptions and Limitations of Theory

Westergaard regarded the slab-on-grade problem to follow the classical "medium thick plate" theory. The assumptions are presented below:

1. All forces on the surface of the plate are perpendicular to the surface, i.e., no shear or frictional forces.

2. The slab is of uniform cross-section, i.e., the slab has constant thickness.

3. In-plane forces do not exist, i.e., no membrane forces.

4. The x-y plane (neutral axis) is located mid-depth within the slab, i.e., stresses and strains are zero at mid-depth.

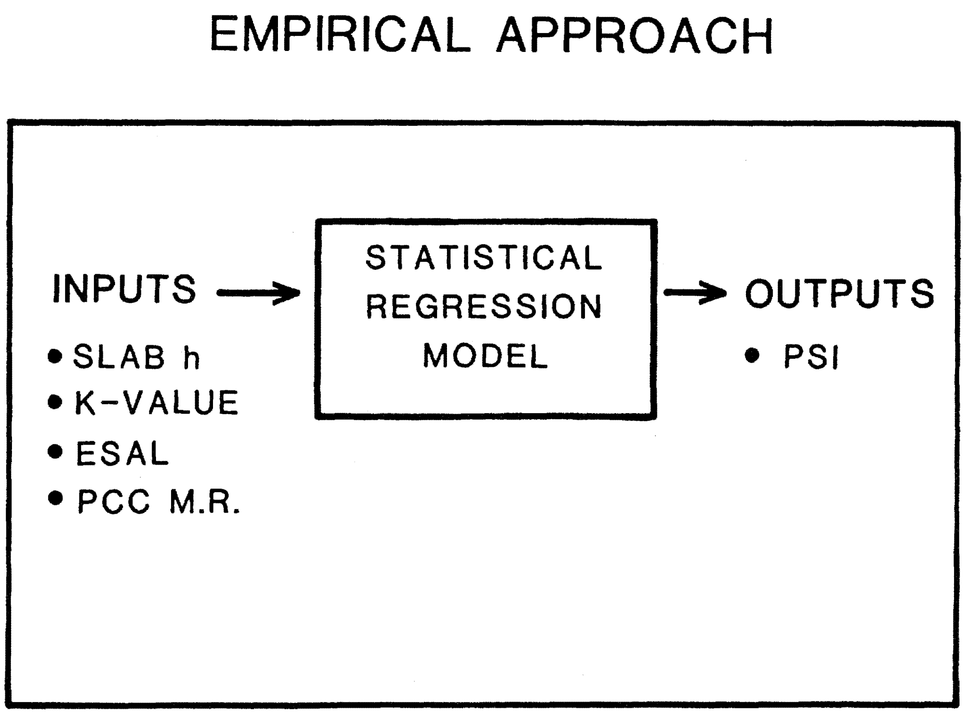

Figure 4-1.1. Illustration of the Empirical Approach to Rigid Pavement Design.

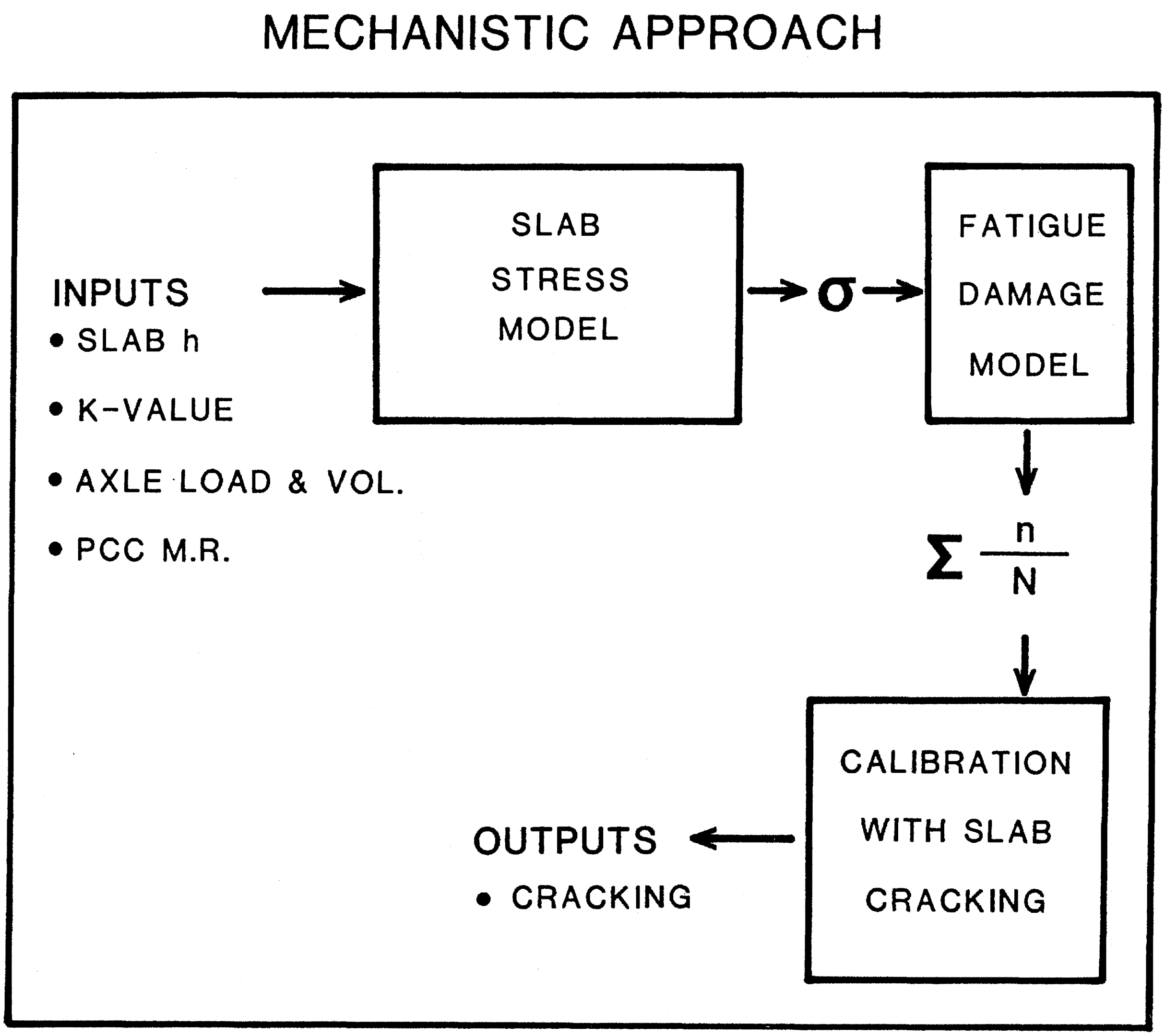

Figure 4-1.2. Illustration of the Mechanistic Approach to Pavement Design.

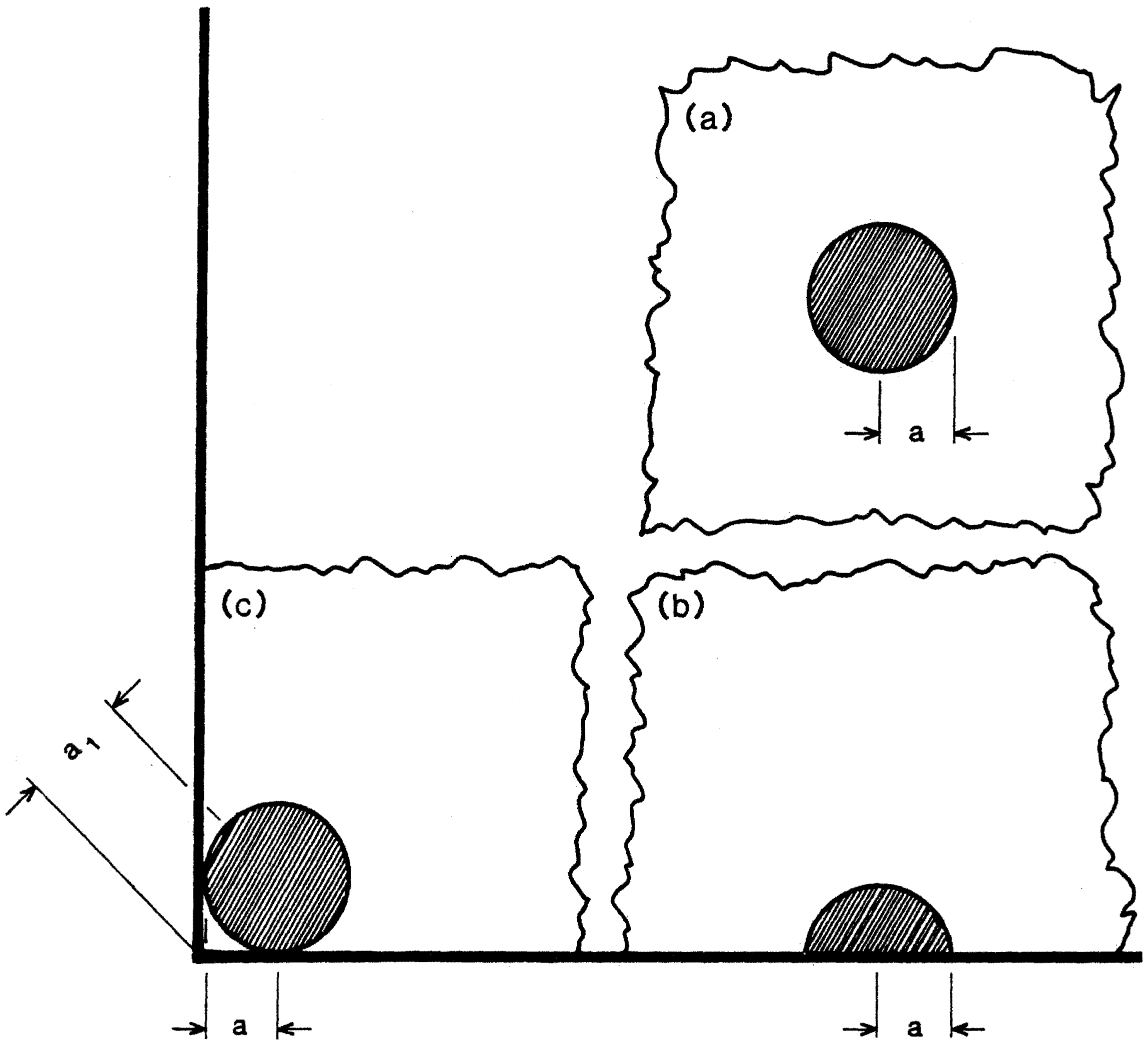

Figure 4-1.3. Loading Conditions for Westergaard Equations.

5. Deformation within the elements normal to the slab surfaces can be ignored. This is called the plane strain assumption; i.e. $_{z} =$ 0.

6. Shear deformations are small and can be ignored. However, shear forces cannot be ignored (2).

7. The slab dimensions are infinite. However, empirical guidelines have been developed for the least slab dimension, L, required to achieve the infinite slab condition (2).

8. The slab is placed on a Winkler foundation in which the subgrade is represented as discrete springs beneath the slab.

The limitations and problems with the use of the theory are presented below:

1. Only corner, edge, and mid-slab stresses and deformations can be calculated.

2. Shear and frictional forces on the slab surface may not be negligible.

3. The Winkler foundation only extends to the edge of the slab. In reality, more support is provided by the surrounding subbase and subgrade.

4. The theory assumes that the slab is fully supported; i.e. no voids or discontinuities exist beneath the slab.

5. The theory does not allow for multiple wheel loads.

6. Load transfer between joints or cracks is not considered in the stress or deflection calculations.

7. Westergaard's original equation for the edge stress is incorrect. The long ignored equation was given in a paper published in 1948 (2). Several equations ascribed to Westergaard in the literature are erroneous, usually through a series of typographical errors and/or misapplications (3).

3.1.2 Solution of the Westergaard Equations

The correct versions (3) of the closed-form Westergaard equations for maximum deflection and maximum bending stress for the three loading positions have been incorporated into the WESTY (12) computer program. The correct versions of the Westergaard equations can be found in Reference 3.

Since the original work done by Westergaard, many investigators have improved on the methods used for stress computation. Influence charts were developed by Pickett and Ray which graphically represent the Westergaard equations for single or multiple wheel loads. With the advent of high-speed computers, programs have been developed to solve the Westergaard equations and the Pickett and Ray influence charts quickly and simply.

Figure 4-1.4 shows several program screens from the WESTY program. The program uses the six input variables to calculate the bending stress and deflection at the interior, edge, and corner loading positions for a fully supported slab. Use of the program will be demonstrated in the class.

Generally speaking, the Westergaard solutions agreed fairly well with other researchers' findings for the interior loading condition. However, it failed to give even a close estimate of the response in the case of edge and corner loadings (3).

3.1.3 General Findings of Westergaard Theory

The following general conclusions are reached regarding the nature of traffic load-associated stresses in rigid pavements. The following conclusions can be verified by use of the WESTY program or the Pickett and Ray influence charts (for more information on the influence charts see Reference 4):

1. Increased slab thickness greatly reduces stress.

2. Load distribution on dual wheels at the same tire pressure significantly reduces the stresses.

3. Decreasing load radius results in higher tire pressures and increased pavement stresses.

4. Weaker subgrade support results in higher slab stresses.

5. Stress increases in slabs with higher modulus of elasticity.

6. The free edge stress is greater than the corner stress for a fully supported slab. The interior stress for a fully supported slab is much less than either the corner or free edge stress.

3.2 Finite Element Methods for the Calculation of Traffic Related Stresses

Finite element theory has provided a very great expansion in the ability to analyze a rigid pavement system. The slab is divided into many small "elements" as shown in Figure 4-1.5. The stress state of each element is calculated, assuming adjacent elements are dependent on each other.

Several finite element computer programs have been developed to analyze rigid pavement systems (i.e., ILLISLAB, JSLAB, WESLIQID, RISC...). Each of the programs were developed employing similar concepts, however, each has different capabilities. For example, JSLAB models thermal curling on a linearly supported slab (subgrade). ILLISLAB has the capability of modelling several subgrade "types" (i.e., Winkler, elastic solid, resilient, and Vlasov).

Although they are powerful tools, finite element programs should be utilized with extreme caution. They require development of a finite element "mesh" for each loading condition and slab configuration. If the mesh is inadequate, erroneous results may be produced. A knowledge of finite element concepts is recommended before using finite element programs.

Applied load on wheel (lbs)	9000
Radius of circular load (in)	6
Poisson's Ratio	.15
Slab Thickness (in)	8
Elastic Modulus of Concrete (PSI)	6000000
Modulus of Subgrade Reaction (PCI)	150
Slab Center Bending Stress (psi) =	186.9
Slab Center Deflection (in) =	.0056
Slab Edge Bending Stress (psi) =	361.9
Slab Edge Deflection (in) =	.0169
Slab Corner Bending Stress (psi) =	245.6
Slab Corner Deflection (in) =	.0406

Applied load on wheel (lbs)	9000
Radius of circular load (in)	6
Poisson's Ratio	.15
Slab Thickness (in)	8
Elastic Modulus of Concrete (PSI)	4200000
Modulus of Subgrade Reaction (PCI)	300
Slab Center Bending Stress (psi) =	166.9
Slab Center Deflection (in) =	.0047
Slab Edge Bending Stress (psi) =	314.1
Slab Edge Deflection (in) =	.0136
Slab Corner Bending Stress (psi) =	215.6
Slab Corner Deflection (in) =	.032

Applied load on wheel (lbs)	9000
Radius of circular load (in)	6
Poisson's Ratio	.15
Slab Thickness (in)	12
Elastic Modulus of Concrete (PSI)	4200000
Modulus of Subgrade Reaction (PCI)	150
Slab Center Bending Stress (psi) =	90.4
Slab Center Deflection (in) =	.0037
Slab Edge Bending Stress (psi) =	178.5
Slab Edge Deflection (in) =	.0113
Slab Corner Bending Stress (psi) =	118.7
Slab Corner Deflection (in) =	.0276

Applied load on wheel (lbs)	12000
Radius of circular load (in)	6
Poisson's Ratio	.15
Slab Thickness (in)	8
Elastic Modulus of Concrete (PSI)	4200000
Modulus of Subgrade Reaction (PCI)	150
Slab Center Bending Stress (psi) =	240.1
Slab Center Deflection (in) =	.0089
Slab Edge Bending Stress (psi) =	460.7
Slab Edge Deflection (in) =	.0266
Slab Corner Bending Stress (psi) =	314.6
Slab Corner Deflection (in) =	.0633

Applied load on wheel (lbs)	9000
Radius of circular load (in)	6
Poisson's Ratio	.15
Slab Thickness (in)	8
Elastic Modulus of Concrete (PSI)	4200000
Modulus of Subgrade Reaction (PCI)	150
Slab Center Bending Stress (psi) =	180.1
Slab Center Deflection (in) =	.0067
Slab Edge Bending Stress (psi) =	345.5
Slab Edge Deflection (in) =	.0199
Slab Corner Bending Stress (psi) =	236
Slab Corner Deflection (in) =	.0475

Figure 4-1.4. Program Screen from WESTY (12).

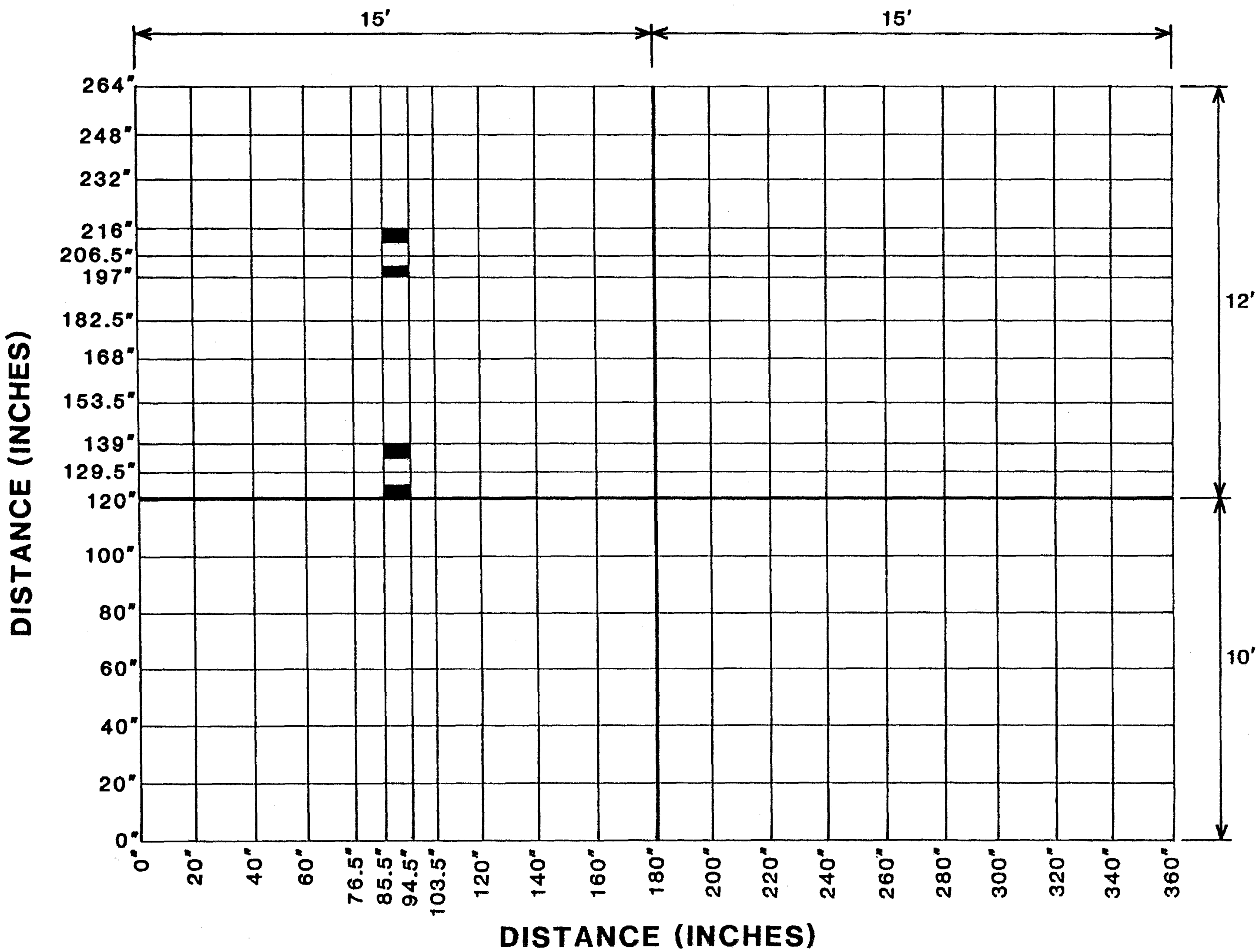

Figure 4-1.5. Example Finite Element Mesh for Edge Loading Position.

3.3 Critical Stress Location

The maximum stress in a concrete slab occurs directly beneath the load for interior and edge conditions. However, the resulting stress may or may not be the highest critical stress because the critical stress is a function of load location.

According to Westergaard's theory (assuming full support) and verified through the use of finite element methods, the highest critical stress occurs at the free edge. For example, a load is placed at the free edge of a slab. If the load is moved away from the free edge, the resultant stress will always be lower than if the load were positioned at the free edge. For fully supported slabs, the location of maximum or critical stress is at the free edge. The critical stress is a tensile located at the bottom of the slab. The free edge stress can be reduced by providing support for the slab edge through use of tied concrete shoulders or by widening the traffic load causing a heavily interior stress.

For slabs only partially supported at the corner, the location of critical stress is at the corner of the slab. It is a tensile stress located at the top of the concrete slab some distance from the corner.

The interior loading position results in substantially lower stresses for both fully and partially supported slabs.

3.4 Non-Traffic Load Stresses in Rigid Pavements

3.4.1 Drying Shrinkage Stresses

PCC contains considerably more water at placement than is needed to hydrate the cement. Within a few hours, the exposed PCC slab loses considerable water which results in shrinkage. Water content, type and gradation of aggregate, chemical admixtures, wind, moisture and temperature conditions have all been found to affect drying shrinkage. The shrinkage of the PCC slab is resisted by friction at the slab/subbase interface and thus tensile stresses develop in the slab. Also, the temperature of the PCC drops during the first night due to a reduction in hydration rate and lower nighttime temperature, resulting in additional tensile stress. Transverse cracks occur generally within the first day after placement ranging from 40 to 250 ft. in slabs that are placed without any joints. However, when joints are placed in the PCC, the tensile stresses due to drying shrinkage becomes smaller as joint spacing decreases. For relatively short joint spacings (i.e. $\leq$ 20 ft.) and under normal curling conditions tensile stresses caused by drying shrinkage are mostly relieved through joint opening, and therefore have only minor effect, if any, on transverse cracking (5).

CRCP, on the other hand, develops large stresses from drying shrinkage (and temperature shrinkage) that results in extensive cracking until an equilibrium is reached.

3.4.2 Concrete Temperature Shrinkage

Rigid slabs subjected to daily and seasonal uniform temperature changes expand and contract causing tensile stresses in the concrete. An unrestrained friction-free slab subjected to a uniform temperature change undergoes expansion and contraction without any stresses being developed in the concrete. However, under normal field conditions, friction forces exist between the concrete and support layer. These forces depend on the slab weight, the coefficient of friction, and the shearing resistance developed in the support layer.

The tensile stresses developed in the concrete for equilibrium conditions must be equal to the frictional forces developed from the center of the slab to its free end. The maximum value of these stresses can be roughly estimated using the "subgrade drag theory" where the slab is pulled over the foundation and the stress required is computed. Frictional resistance tests have been conducted by several researchers (5) which show that the coefficient of frictional resistance is not a constant, but increases with increasing displacement of slab, until a maximum is reached where the slab slides freely. Sliding friction values ranging from less than 1.0 to over 2.0 have been obtained depending on foundation conditions. Stresses computed using conventional subgrade drag theory are given in Table 4-1.1 for a range of conditions using equation 4-1.1 (5).

$$S = (Lf\gamma_c/288) \tag{4-1.1}$$

Where:

S = tensile stress in PCC, psi
f = coefficient of frictional resistance
L = slab length, ft.
γ_c = unit weight of PCC, pcf

EXAMPLE:

Calculate the tensile stress for a 12-foot slab given γ_c = 150 pci and f = 1.35.

ANSWER:

$S = 8.4$ psi

The tensile stress increases linearly with joint spacing which contributes to the increase in cracking with increased joint spacing. These stresses are very small for short joint spacings (i.e. <20 ft.), but are of significant magnitude for longer joint spacings. They are believed to be negligible for relatively short slabs, and only of minor significance for slabs up to 30 feet in length.

3.4.3 Slab Thermal Gradients

A difference in temperature between the top and bottom of slabs (thermal gradient, G) has been shown both experimentally and analytically to cause significant stress in slabs. Under a positive nighttime thermal gradient, i.e., when the top of the slab is cooler than the bottom, the pavement corners tend to curl upward and are resisted by the weight of the slab and joints in keeping the slab in its original position. Under a negative daytime thermal gradient, the pavement corners tend to curl downward, theoretically leaving the center of the slab unsupported. A weightless and unrestrained slab would curl upward or downward without any stresses being developed.

Westergaard has developed equations for curling stresses due to differential temperatures and for slabs of infinite width and length as well as finite width and length dimensions. The Westergaard equations were simplified as follows:

Edge Stress

$$\sigma_e = (1/2)\, C E \alpha_T \Delta_T \qquad (4\text{-}1.2)$$

Interior Stress

$$i = (1/2)(E \alpha_T \Delta_T) * (C1 + C2)/(1 + \mu^2) \qquad (4\text{-}1.3)$$

Where:

C, C1, C2 =	Coefficients for edge stress, interior stress for the direction used in the analysis, and the stress coefficient for perpendicular direction, respectively. C1 and C2 depend on the length and width of the slab and radius of relative stiffness (see Figure 4-1.6 (4)).
Δ_T =	Differential temperature between top and bottom of slab.
E =	Modulus of elasticity.
α_T =	Coefficient of temperature expansion.
μ =	Poisson's ratio.

EXAMPLE:

Calculate the edge curling stress given the following inputs:
L_x = 40 ft., L_y = 12 ft., k = 100 pci, Δ_T = 3°F/in.,
E = 4 x 10^6 psi, h = 10 in.

ANSWER:

σ_e = 315 psi

The differential temperature for a pavement slab ranges from 0.5 to 2 degrees F/inch of thickness; however, the temperature gradient is dependent on the season of the year. Tests have shown that maximum positive temperature differentials occur during the day in the spring and summer months. During the spring the subgrade is cool, and the slab which is exposed to the sun warms faster than the subgrade. During the summer months the slab cools during the nighttime, and its top becomes very warm during the daytime. For example, maximum temperature differentials for slabs 6 and 9 inches thick approach 2 1/2 to 3 degrees per inch of slab.

There is evidence from finite element results that equations 4-1.2 and 4-1.3 overestimate the curling stress considerably, especially for stiffer subgrade support (5). Equations derived from finite element theory are given in Reference 5. Table 4-1.1 shows some comparative curling stresses for each method.

Curling stresses are of such a significant magnitude that they are believed to definitely contribute to slab cracking. Figure 4-1.7 shows some results from two test roads. The shapes of the curves are similar to the slope of Figure 4-1.6.

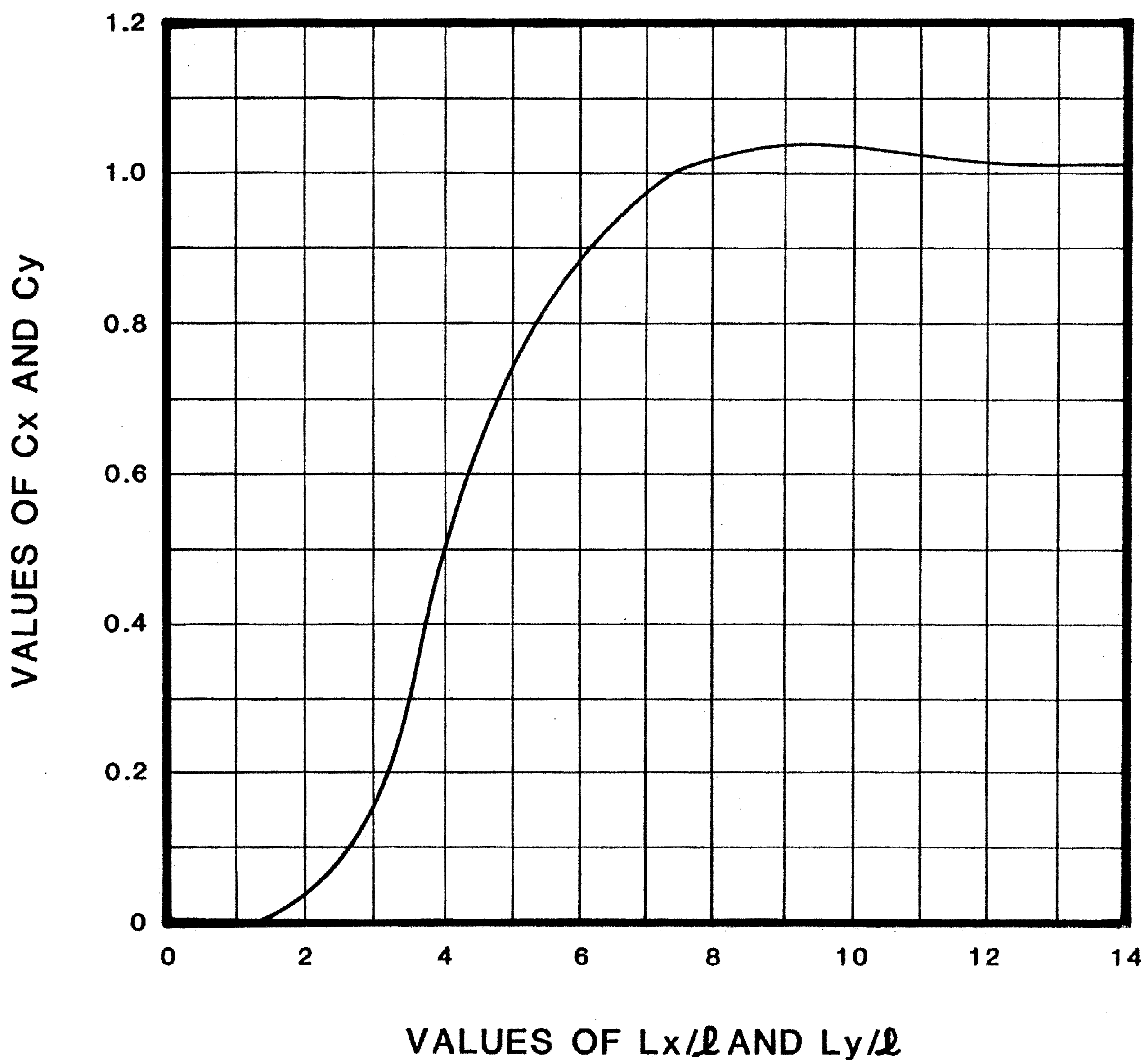

Figure 4-1.6. Curling Stress Coefficients (4).

$$\ell = [Eh^3/12k(1-\mu^2)]^{0.25}$$

Table 4-1.1. Computed Tensile Stress in PCC Slabs Due to Temperature Reduction and Subbase Frictional Resistance (5).*

Joint Spacing (ft)	Coefficient of Frictional Resistance 1.0	1.5	2.0
10	5 psi**	8	10
15	8	12	16
20	10	16	21
30	16	23	31
50	26	39	52
100	52	78	104

*PCC unit wt. = 150 pcf

**Computed from Equation 4-1.1

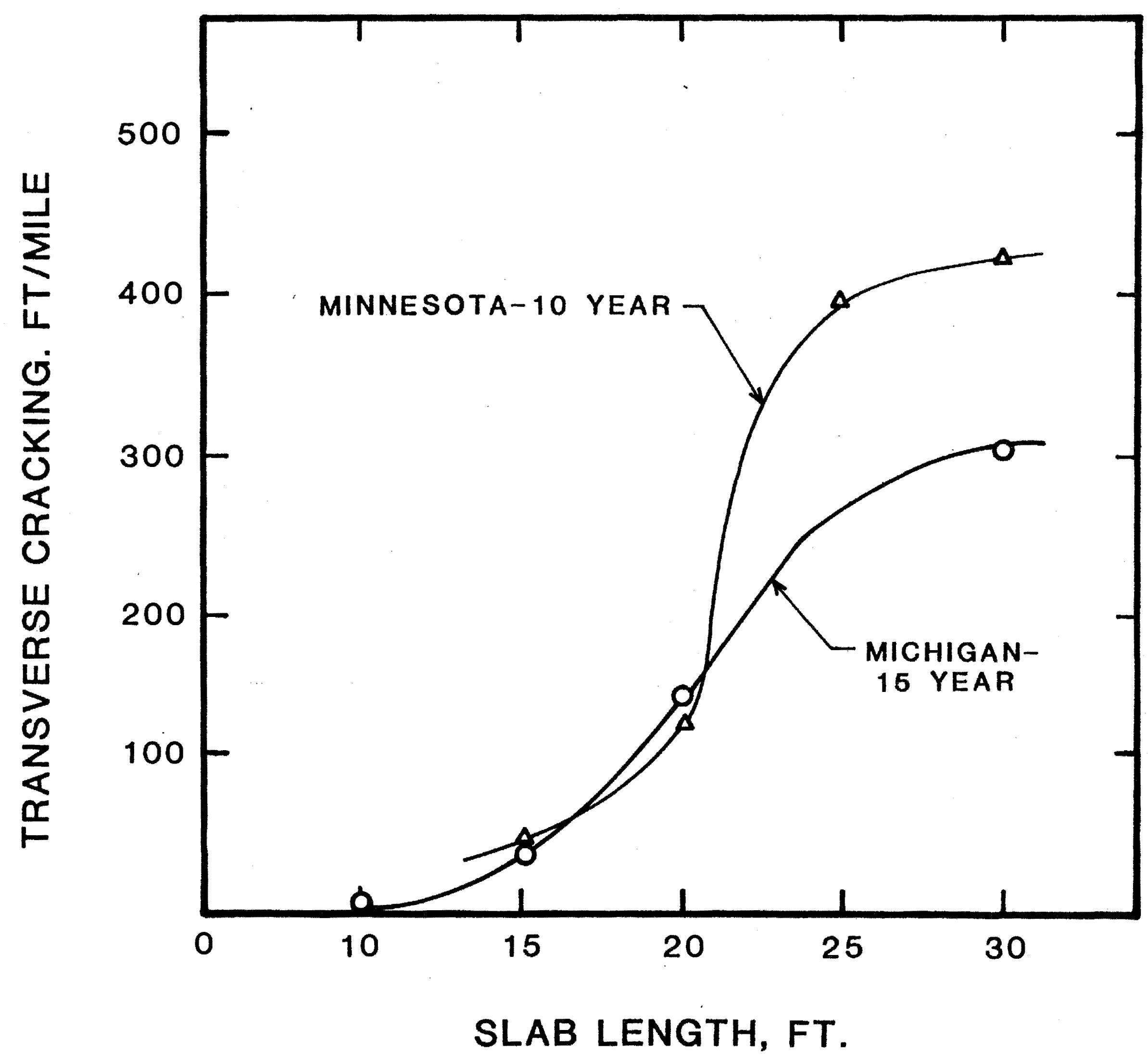

Figure 4-1.7. Effect of Slab Length on Transverse Cracking of Slabs for Two Road Tests.

3.4.4 Moisture Stresses

Research has shown that slab warping is caused by moisture differences between the top and bottom of the slab. The stresses caused by moisture gradients through the slab are referred to as warping stresses. The weight of the slab, resistance from the subgrade, and any resistance at the slab edges apply restraint to warping of the slab, and thus stresses occur at the top and bottom of the slab.

Several studies were conducted on the effect of moisture gradients by the Bureau of Public Roads (BPR) in the 1930's. The results showed that the warping of a pavement slab from moisture gradients is mostly a seasonal change occurring over a considerable time interval. For example, during the winter months in Phoenix, Arizona, when the relative humidity is very low, the PCC slabs are noticeably warped upward due to the severe drying of their surface. The warp is less severe during other seasons.

The BPR tests also showed that as the seasonal warping occurs, the slab settles somewhat into the subgrade, thus reducing the restraint due to slab weight. Any creep of the concrete would also tend to reduce the stress from moisture warping (5).

Because of the many difficulties involved (i.e. inability to measure moisture contents in slabs, settlement, creep, etc.) attempts have not yet been successful to compute or measure strains due to moisture gradients, and their relative magnitudes are unknown. However, the following conclusions appear justified based upon the available information:

1. The top of the slab is usually dryer than the bottom through most of the year, causing some compressive stresses at the bottom of the slab.

2. These stresses are greater during the warm weather portion of the year because of a dryer slab surface.

3. Moisture warping stresses at the slab bottom are generally of opposite sign of the critical stresses, and hence tend to reduce the combined stress occurring at the slab edge or interior.

4. There is not enough information presently available to consider moisture gradient warping stress in design; however, as joint spacing increases, the warping stress will also increase similar to thermal curling stress.

3.5 Combined Stresses

The combined stress state in a rigid pavement is determined by superimposing the environmentally related stresses, such as curling and warping stresses, on the load associated stresses. The critical condition results when curling and loading stresses are additive, i.e.:

1. The slab corners are curled upward, and the load is placed at the corner.

2. The slab is curled downward and the load is placed at an edge or interior location.

Thermally induced stresses are dependent on the dimensions of the slab. To minimize the effect of these stresses, the use of short slabs is recommended. The combination of load and curling stresses cannot be accomplished with the Westergaard equations. They are not directly additive. Figure 4-1.8 shows the dramatic effect that slab length has on edge stress due to curling and load. The stress increases rapidly until a slab length of 25 feet and then levels off. The combination of load and thermal curling can produce very high stress states in the slab supported on bases of moderate to stiff support.

The ZMAN (13) computer program incorporates regression equations developed under the FHWA research contract entitled "Design of Zero-Maintenance for Plain Jointed Concrete Pavement" (5). The equations are based on a series of finite element computer program executions. A finite element program was executed to establish a large database. The regression equations were derived from this database. Reference 5 details the equation development as well as providing the actual regression equations. The program can be used to calculate stress for single or tandum axles.

Figure 4-1.9 shows the program screen for the ZMAN computer program. The program uses seven input variables to calculate the combination stress (loading stress + curling stress) for the edge loading position.

3.6 Stresses in Dowel Bars

Dowel bar stresses have a great effect on the resulting faulting of the joint (6). This is a greatly overlooked aspect of rigid pavement design. It it important for the design engineer to understand the causes and relative magnitudes of bearing stresses.

When loads are applied at a joint, a portion of that load is transferred through the dowel bar to the next slab. The dowel bar immediately under the load assumes the major portion of the load with the other dowels assuming progressively lesser amounts. The action of a group of dowels was analyzed by Frieberg (4). Frieberg concluded that the maximum negative moment occurs at an effective length of 1.8*l from the load. However, further research concludes that an effective length of 1.0*l form the load is more accurate (15). Thus, if a series of dowel bars are designed, the dowel bar immediately under the applied load carries full capacity, decreasing at a distance of 1.8*l (l is the radius of relative stiffness of the pavement system which is defined in Figure 4-1.10) from this dowel (4) as shown in Figure 4-1.10.

A graph of concrete bearing stress versus dowel diameter for various dowel spacings in Figure 4-1.11. This figure shows the dowel diameter has an enormous effect on bearing stress. The dowel spacing has a smaller effect on bearing stress with larger diameter dowel bars.

Allowable bearing stress depends upon the amount of faulting allowed. The following equation was developed from the extended AASHO Road test sections for dowelled and non-dowelled pavements. Figure 4-1.12 is a graphical representation of these equations.

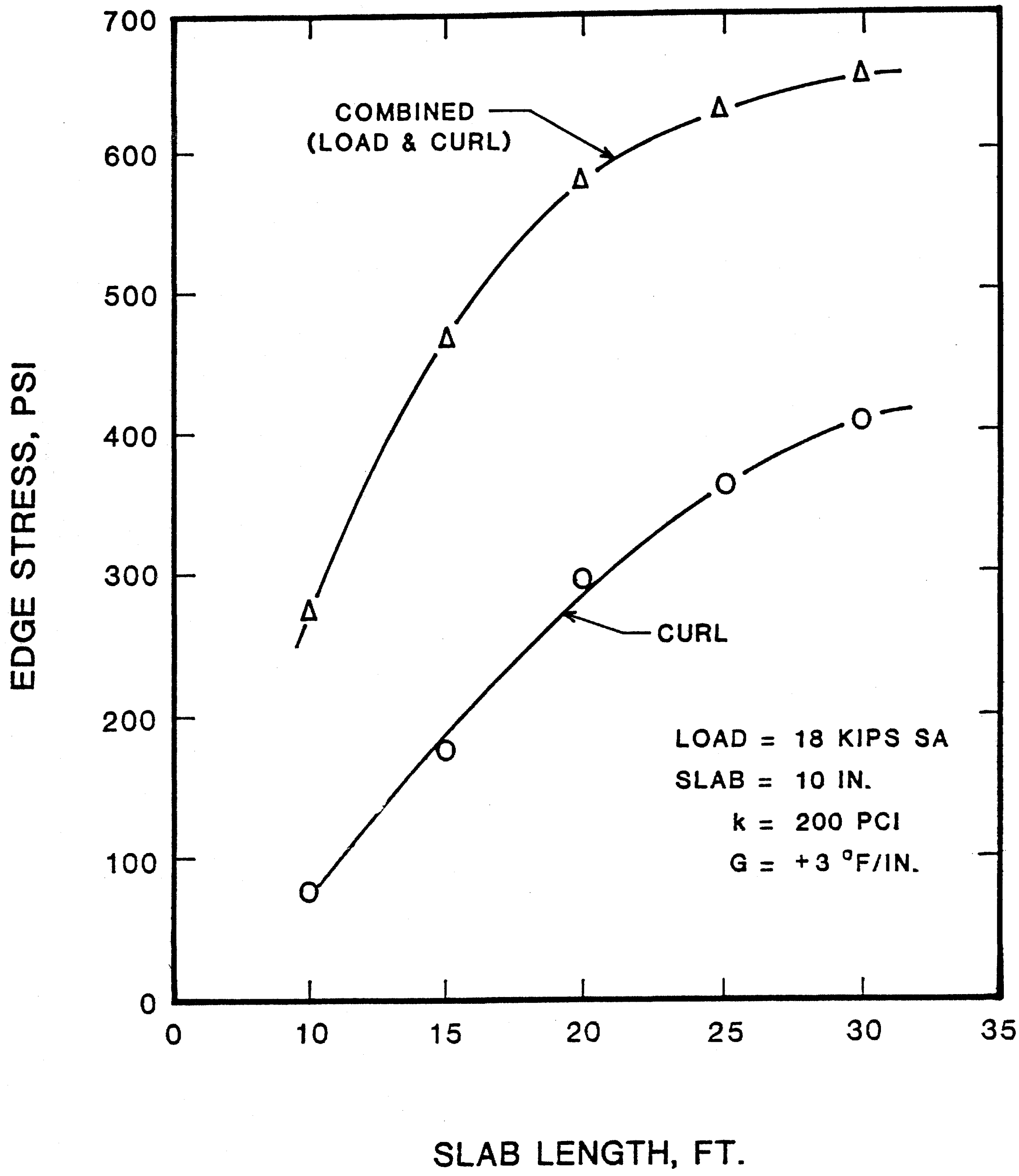

Figure 4-1.8. Edge Stress Due to Load and Thermal Gradient for Varying Joint Spacing (5).

```
Z M A N

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Written by Michael T. Darter


Press Any Key to continue
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Zero-Maintenance Edge Curling Equations

  Axle (S)ingle or (T)andem  S
              Axle Load      18000
        Slab Thickness (in)  10
     Thermal Gradient (f/in) -1.5
     Subgrade Modulus (pci)  200
           Slab Length (ft)  12
           Erodability (in)  36

 Calculated Load Stress (psi): 250.8
 Calculated Curl Stress (psi): -69.4
   Combined Edge Stress (psi): 189.8
```

```
Zero-Maintenance Edge Curling Equations

  Axle (S)ingle or (T)andem  s
              Axle Load      18000
        Slab Thickness (in)  10
     Thermal Gradient (f/in) -1.5
     Subgrade Modulus (pci)  200
           Slab Length (ft)  12
           Erodability (in)  0

 Calculated Load Stress (psi): 222.7
 Calculated Curl Stress (psi): -67.3
   Combined Edge Stress (psi): 156.8
```

```
Zero-Maintenance Edge Curling Equations

  Axle (S)ingle or (T)andem  s
              Axle Load      18000
        Slab Thickness (in)  10
     Thermal Gradient (f/in) 3
     Subgrade Modulus (pci)  200
           Slab Length (ft)  12
           Erodability (in)  36

 Calculated Load Stress (psi): 250.8
 Calculated Curl Stress (psi): 138.7
   Combined Edge Stress (psi): 429.1
```

```
Zero-Maintenance Edge Curling Equations

  Axle (S)ingle or (T)andem  s
              Axle Load      18000
        Slab Thickness (in)  10
     Thermal Gradient (f/in) -1.5
     Subgrade Modulus (pci)  400
           Slab Length (ft)  12
           Erodability (in)  36

 Calculated Load Stress (psi): 229.8
 Calculated Curl Stress (psi): -66.5
   Combined Edge Stress (psi): 170.5
```

```
Zero-Maintenance Edge Curling Equations

  Axle (S)ingle or (T)andem  t
              Axle Load      36000
        Slab Thickness (in)  10
     Thermal Gradient (f/in) -1.5
     Subgrade Modulus (pci)  200
           Slab Length (ft)  12
           Erodability (in)  36

 Calculated Load Stress (psi): 215.7
 Calculated Curl Stress (psi): -69.4
   Combined Edge Stress (psi): 154.7
```

```
Zero-Maintenance Edge Curling Equations

  Axle (S)ingle or (T)andem  s
              Axle Load      18000
        Slab Thickness (in)  10
     Thermal Gradient (f/in) -1.5
     Subgrade Modulus (pci)  200
           Slab Length (ft)  15
           Erodability (in)  36

 Calculated Load Stress (psi): 250.8
 Calculated Curl Stress (psi): -104.5
   Combined Edge Stress (psi): 159.3
```

Figure 4-1.9. ZMAN Program Screens.

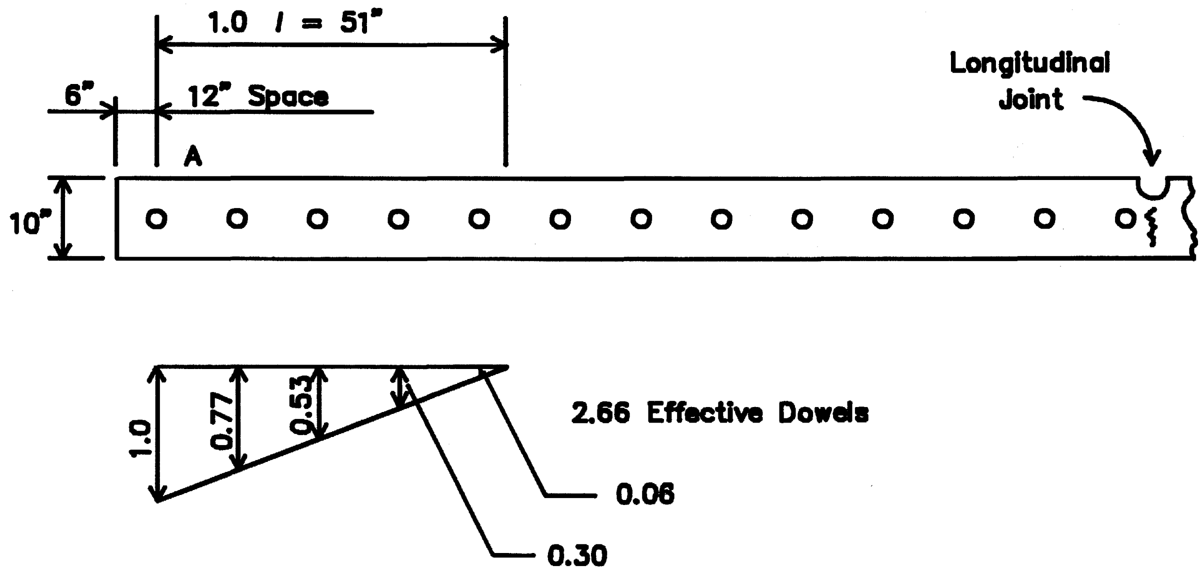

Figure 4-1.10. Loads on Dowel Group; Pavement = 10 inches, k = 50 psi, 3/4-inch Round Dowel Spaced 12 inches c-c. Effective Dowels Due to Load at A.

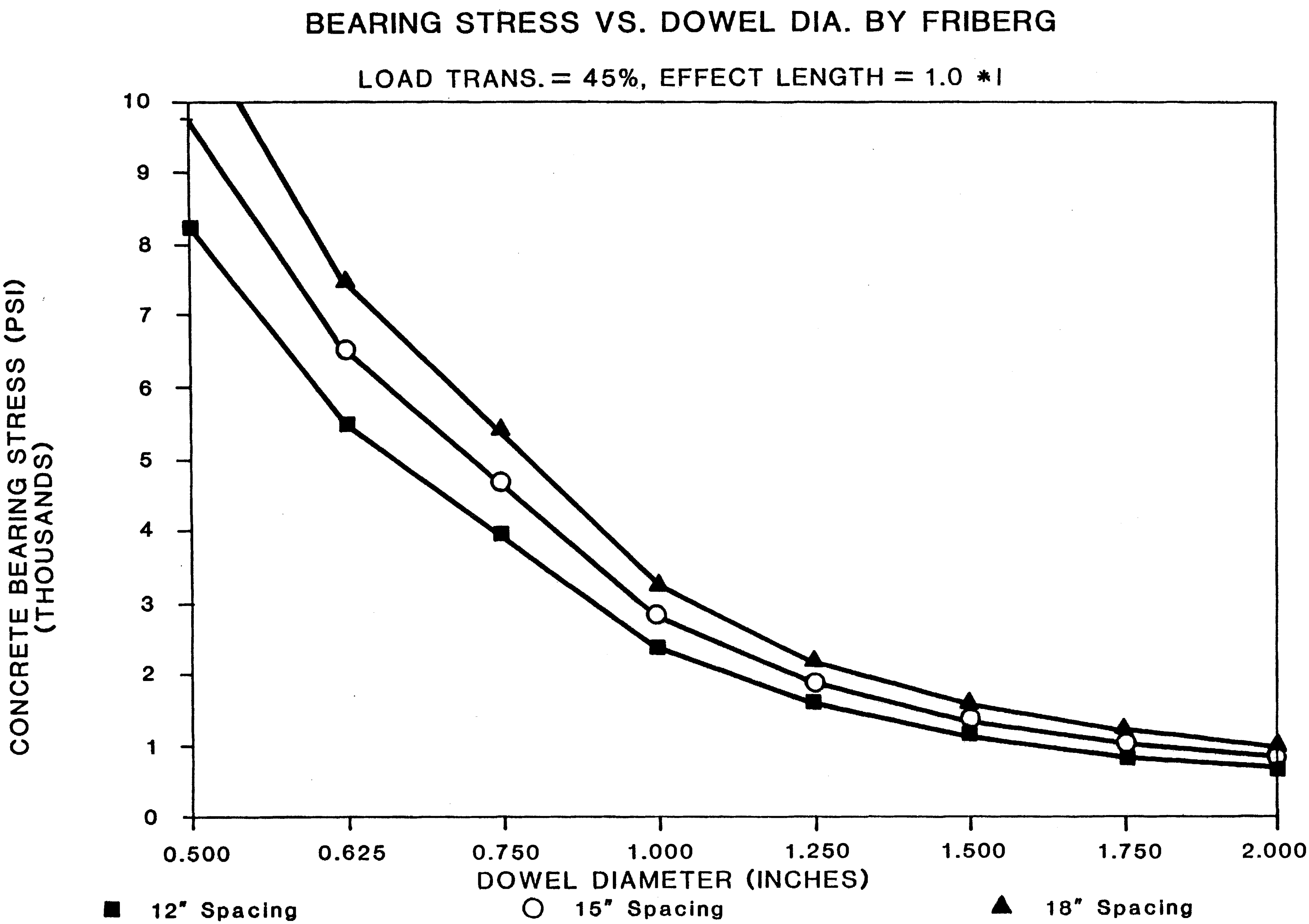

Figure 4-1.11. Effect of Dowel Diameter and Spacing on Bearing Stress.

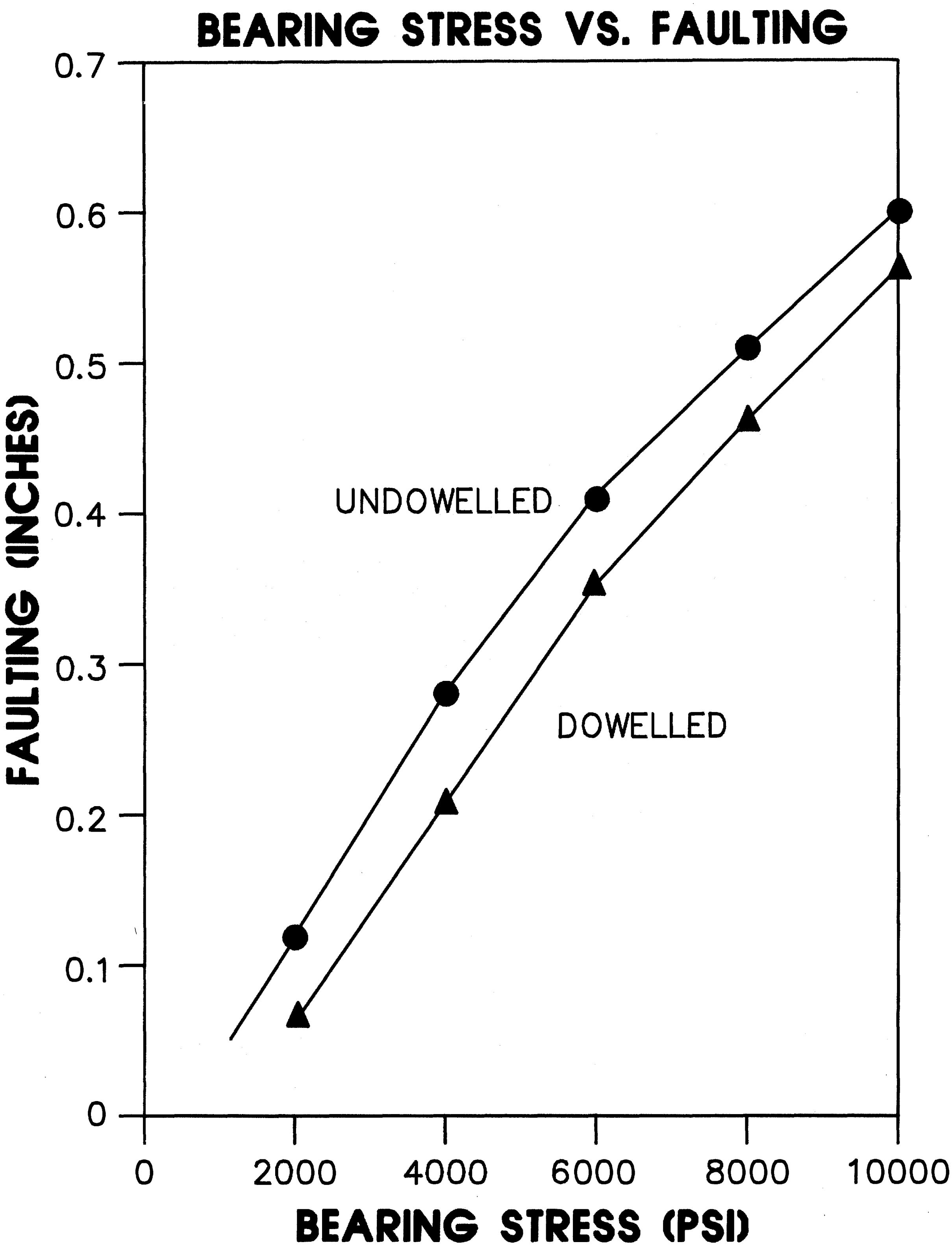

Figure 4-1.12. Allowable Faulting Versus Bearing Stress for Dowelled and Undowelled Pavements.

UNDOWELLED

$$FAULT = ESAL^{0.2692} [-2.995 + 0.00779\ BSTRESS^{0.4527} + 2.766\ JSPACE^{0.00676}]$$

DOWELLED

$$FAULT = ESAL^{0.5398} [2.128 + 0.00296\ BSTRESS^{0.4584} + 0.000493\ JSPACE^{0.9993} - 2.066\ KVALUE^{0.0136}]$$

Where:

FAULT = Average faulting (inches)
ESAL = Commulative 18-kip equivalent single axle loads over design life (millions)
BSTRESS = Maximum concrete bearing stress (psi)
JSPACE = Transverse joint spacing (ft)
KVALUE = Effective modulus of subgrade reaction (psi/in)

3.7 Sensitivity Analysis

A finite element program was used to perform the sensitivity analysis (5). The traffic loading includes an 18-kip single axle and a 36-kip tandem axle located at the edge of the slab at midpoint between transverse joints. The critical stress for this load position is at the bottom of the slab edge, parallel to the edge beneath the wheel load. This stress is used in all of the subsequent analyses and is referred to as edge stress. The stress is caused by traffic load (referred to as load stress), thermal gradient through the slab (referred to as curl stress), or a combination of load and thermal gradient (referred to as load and curl stress). Stresses were computed for a complete factorial of six factors (5), including:

1. Slab thickness (H): 8, 10, and 14 ins.

2. Modulus of foundation support (k): 50, 200, and 500 pci

3. Thermal gradient (G): -1.5 (nighttime where bottom warmer than top of slab), 0, +3.0 (daytime), °F/in.

4. Slab length (L): 15, 20, 25, and 30 ft. (width was constant at 12 ft.)

5. Erodability of support (ES): 0, 12, 36, and 60 ins. (longitudinal strip of width ES along outer slab edge).

Some of these results are illustrated considering a selected "standard pavement" (see Figure 4-1.13) and then varying each factor over a typical range and plotting the change in edge stress that results. The standard pavement is selected as follows:

1. Slab Thickness (H) = 10 ins.

2. Modulus of Foundation Support (k) = 200 pci.

3. Slab Length (L) = 15 ft.

STANDARD PAVEMENT SECTION:

SLAB THICKNESS (H) = 10 INS.

MODULUS OF FOUNDATION SUPPORT (k) = 200 PCI

SLAB LENGTH (L) = 15 FT.

SLAB WIDTH = 12 FT.

THERMAL GRADIENT THROUGH SLAB (G) = 0° F/IN.

(-1.5 INDICATES NIGHTTIME WHERE BOTTOM IS WARMER THAN TOP OF SLAB; + 3.0 INDICATES DAYTIME WHERE TOP IS WARMER THAN BOTTOM OF SLAB)

PCC MODULUS OF ELASTICITY = 5×10^6 PSI

PCC THERMAL COEFFICIENT OF EXPANSION = $5 \times 10^{-6}/^\circ F$

ERODABILITY OF SUPPORT ALONG EDGE OF SLAB (ES) = 0 IN.

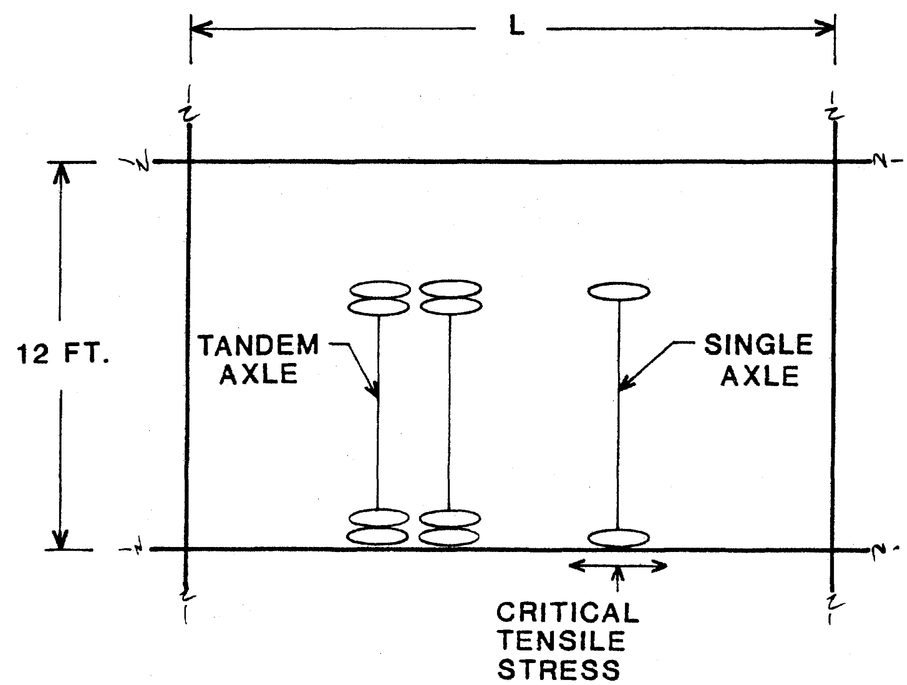

Figure 4-1.13. Standard Pavement for Sensitivity Analysis.

4. Slab Width = 12 ft.

5. Thernal Gradient through Slab (G) = 0^{o}F/in.

6. PCC Modulus of Elasticity = 5 X 10^{6} psi.

7. PCC Thermal Coefficient of Expansion = 5 x $10^{-6}/^{o}$F.

8. Erodability of Support (ES) = 0 in.

Data are plotted showing the change in edge stress due to changes in H, k, E, G, and L, and their interactions. Results for edge stress caused by the combination of traffic load and thermal gradients are given in Figures 4-1.13 through 4-1.18. Results showing the effect of an edge stress at the slab bottom for only thermal gradients are shown in Figures 4-1.19 through 4-1.22. A comparison of stresses caused by 18-kip single axle and 36-kip tandem axles is shown in Figure 4-1.23. These graphs should be carefully studied because they have many important design implications. A brief summary of the most significant results as illustrated in Figures 4-1.14 through 4-1.23 is as follows:

1. As joint spacing (L) increases from 15 to 30 ft., the edge stress caused by thermal gradients change greatly. Stress increases for daytime gradients (i.e. top of slab warmer than bottom) and decreases for nighttime gradients (i.e. bottom warmer than top) as joint spacing increases (Figure 4-1.8 shows combined load and curl edge stress, Figure 4-1.19 shows curl stress only). Joint spacing does not have significant effect when only traffic load is applied.

2. As slab thickness increases, edge stress caused by either traffic load, thermal gradient, or load and gradient combined, decreases significantly for slabs of about 20 ft. or less (Figure 4-1.8, 4-1.20 (b)). The combined stress generally increases as slab thickness increases for slabs with length greater than about 20 ft. Slab thickness also interacts with other parameters such as the k-value and erodability. The effect of change in stress for thick slabs for changes in k-value or erodability of support is less than it is for thin slabs (Figures 4-1.15 (a) and 4-1.15 (b)).

3. As erodability of support increases, combined load and curl edge stress increase with one exception (Figures 4-1.16 and 4-1.21). When a daytime gradient exists, there is a slight decrease in combined edge stress because of reduced restraint of the slab.

4. As the subgrade modulus of support increases, combined load and curl stresses decrease with one important exception (Figure 4-1.17). With a daytime thermal gradient (+3.0 ^{o}F/in.), the stress generally increases as the k-value increases (Figure 4-1.17 (c)). This has implications in the use of very stiff bases for slabs.

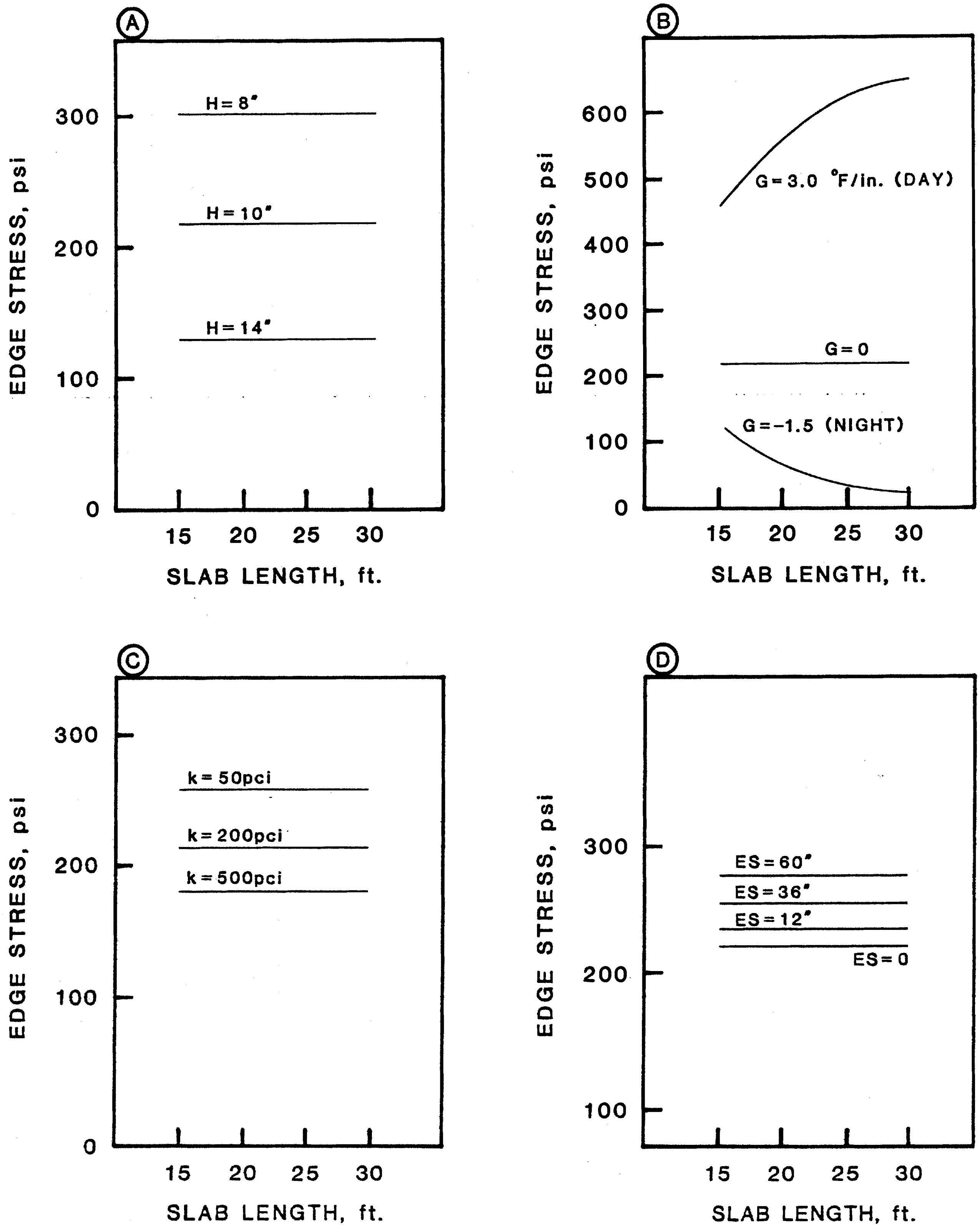

Figure 4-1.14. Effect of Slab Length on Total Edge Stress (Load & Curl) for a Single Axle Load (5).

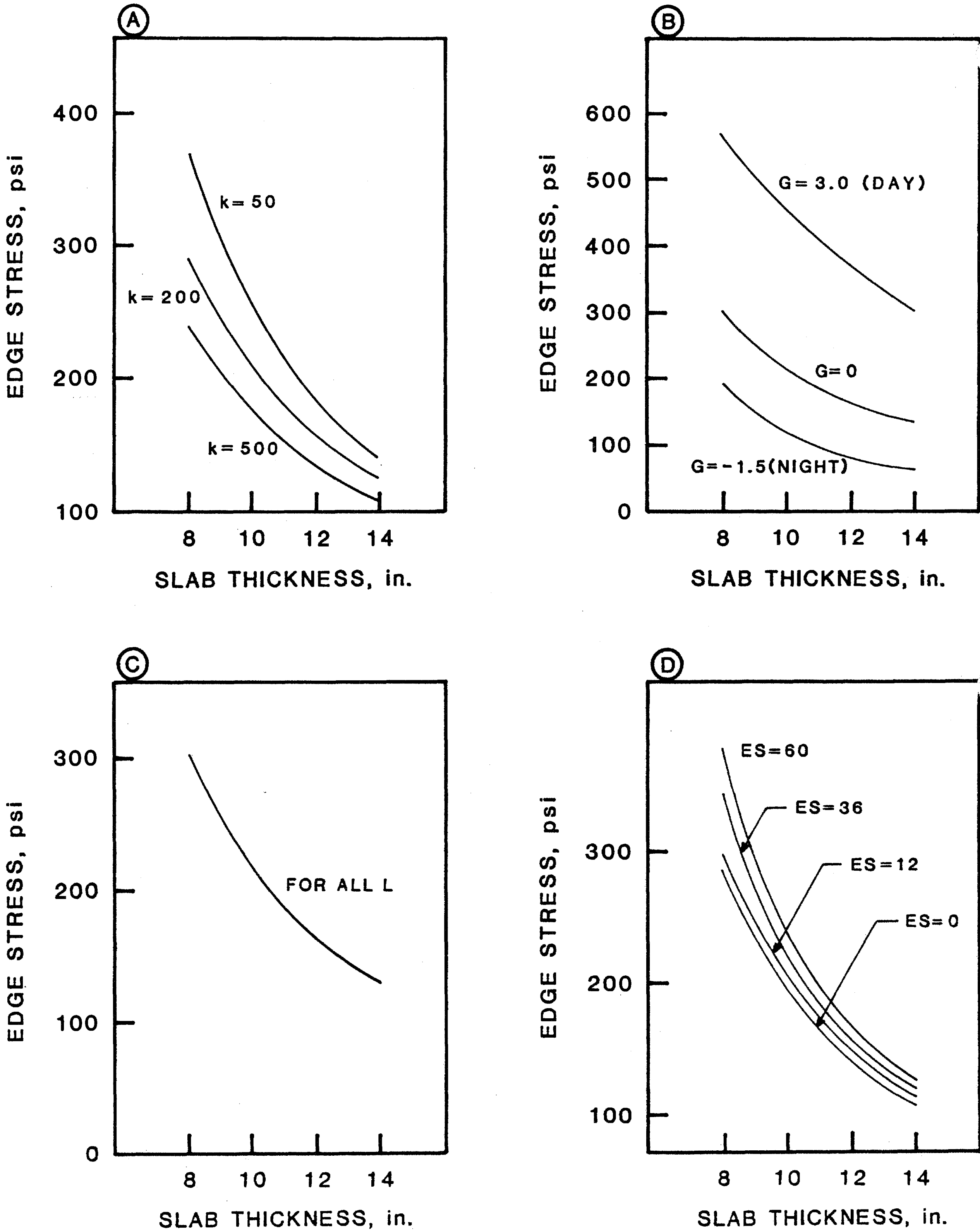

Figure 4-1.15. Effect of Slab Thickness on Total Edge Stress for a Single Axle Load (5).

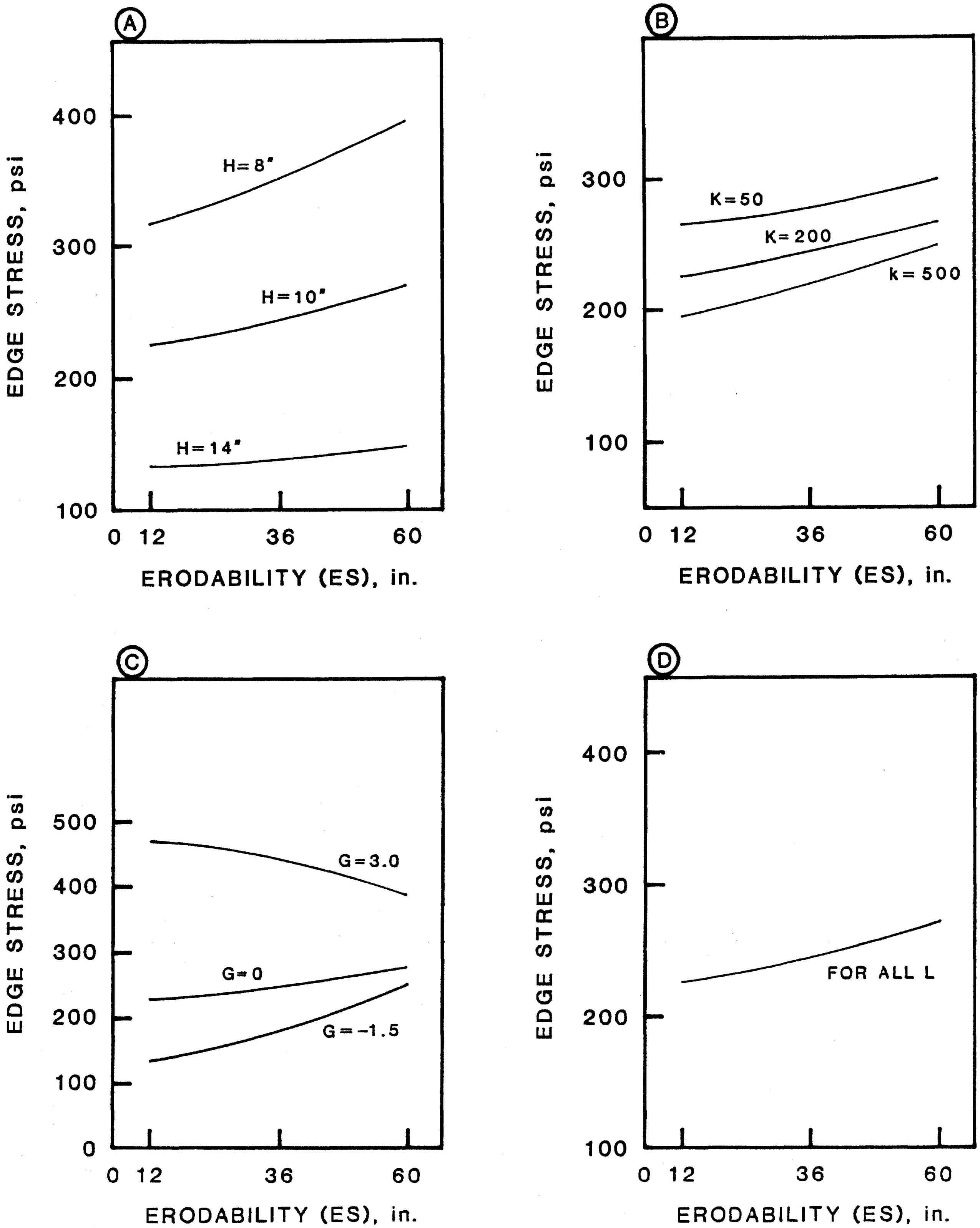

Figure 4-1.16. Effect of Subbase Erosion on Total Edge Stress for a Single Axle Load (5).

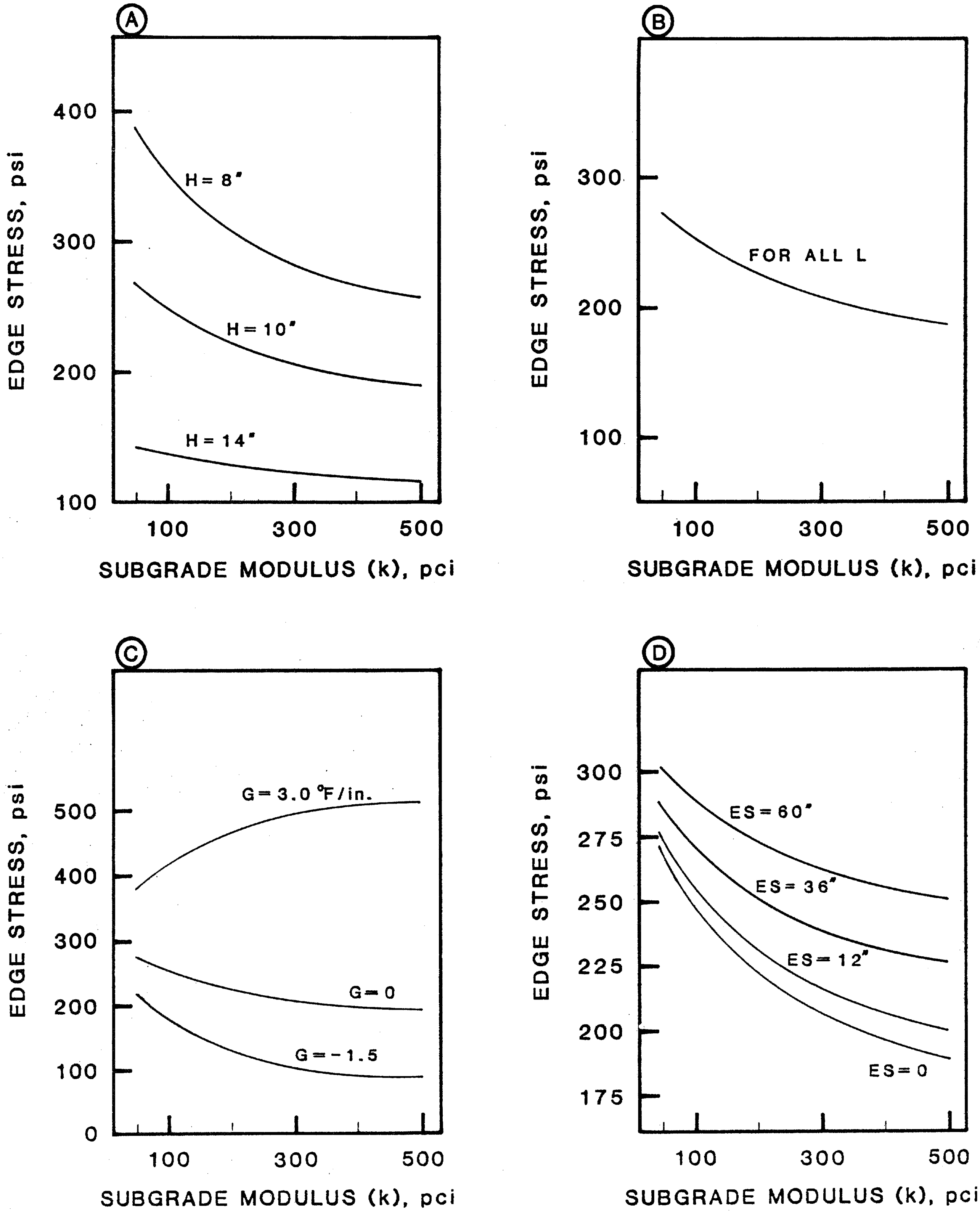

Figure 4-1.17. Effect of Subgrade Modulus on Total Edge Stress for a Single Axle Load (5).

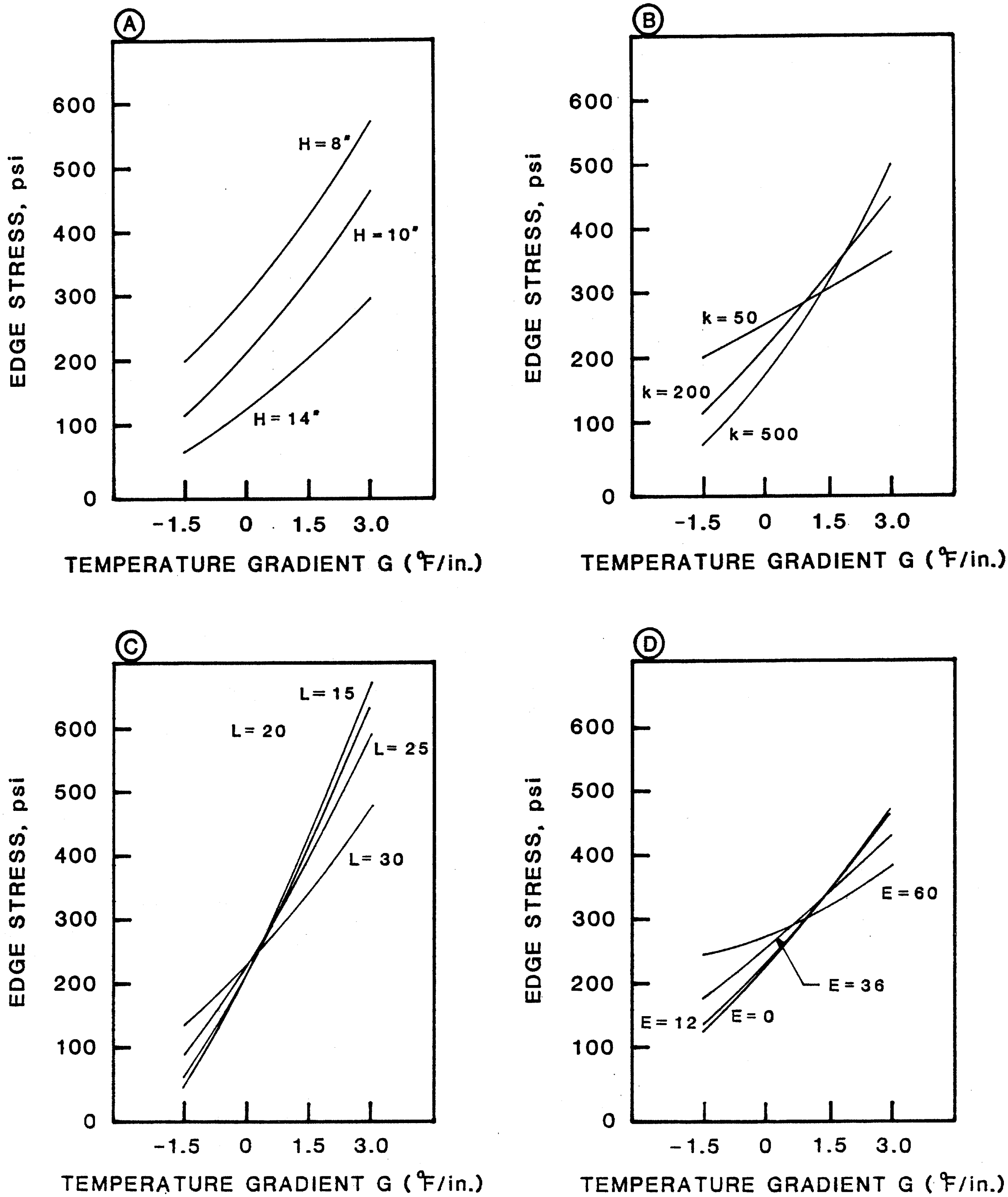

Figure 4-1.18. Effect of Thermal Gradient on Total Edge Stress for a Single Axle Load (5).

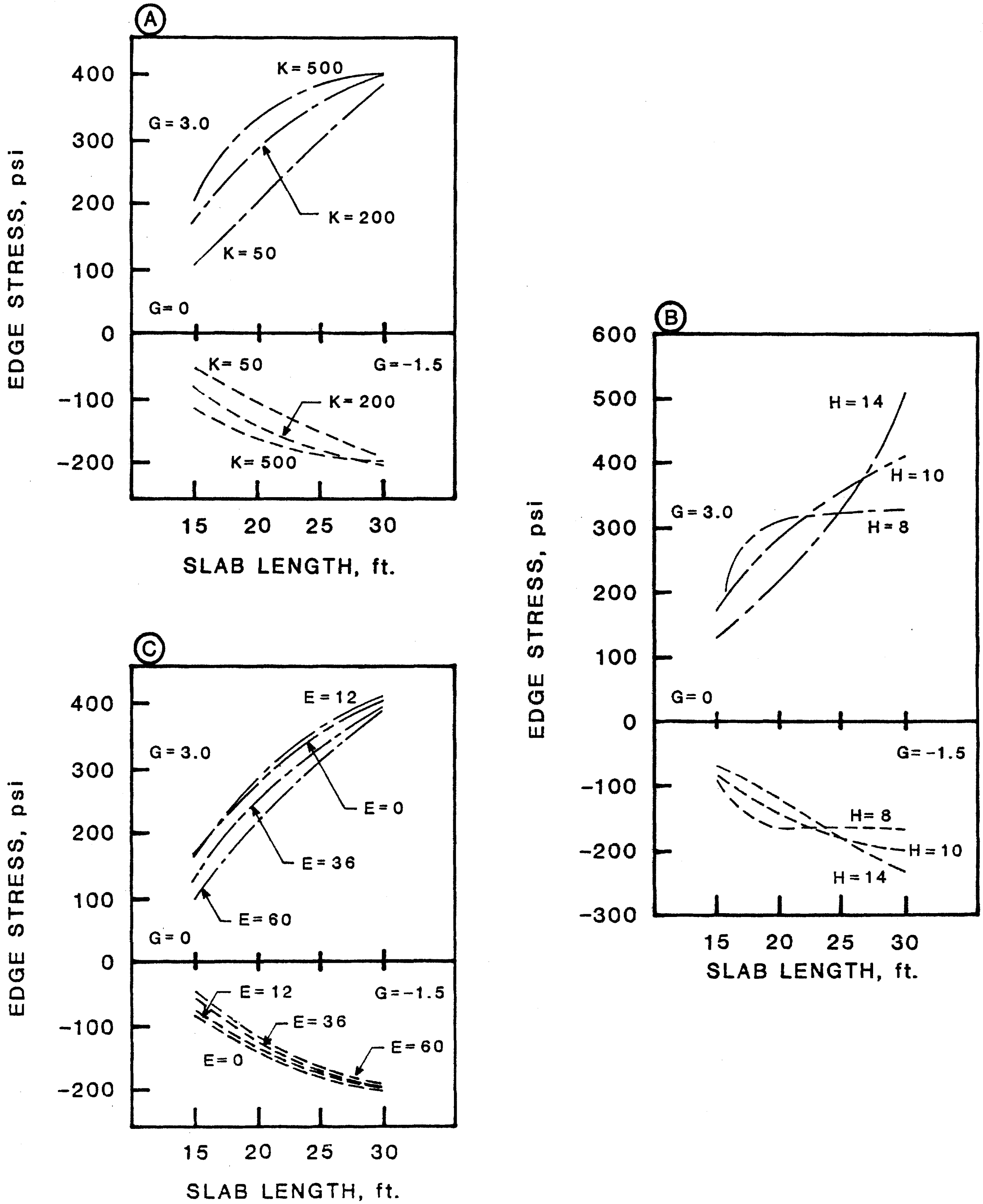

Figure 4-1.19. Effect of Slab Length on Thermal Curl Edge Stress for a Single Axle Load (5).

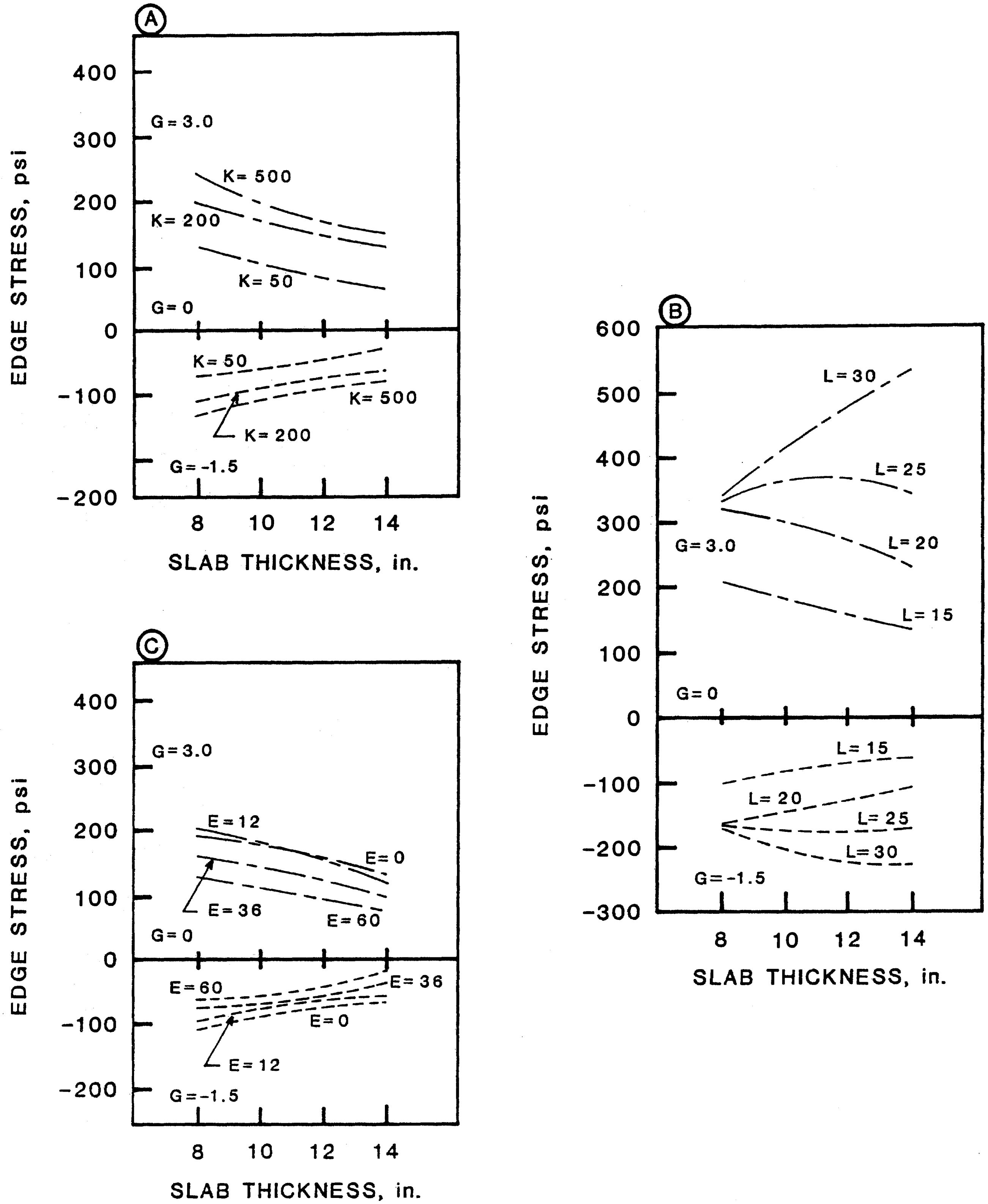

Figure 4-1.20. Effect of Slab Thickness on Thermal Curl Edge Stress for Single Axle Load (5).

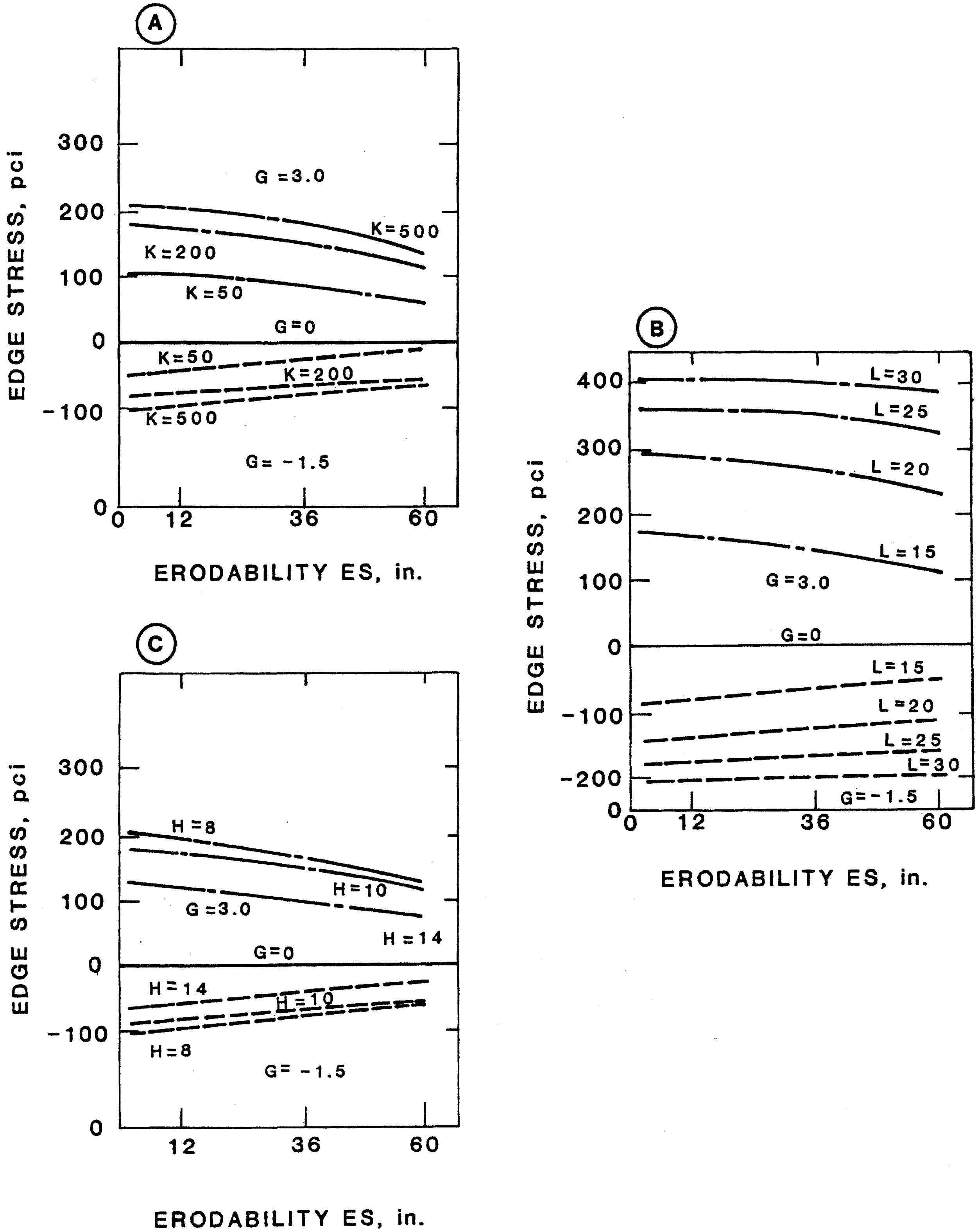

Figure 4-1.21. Effect of Subbase Erosion on Thermal Curl Edge Stress for Single Axle Load (5).

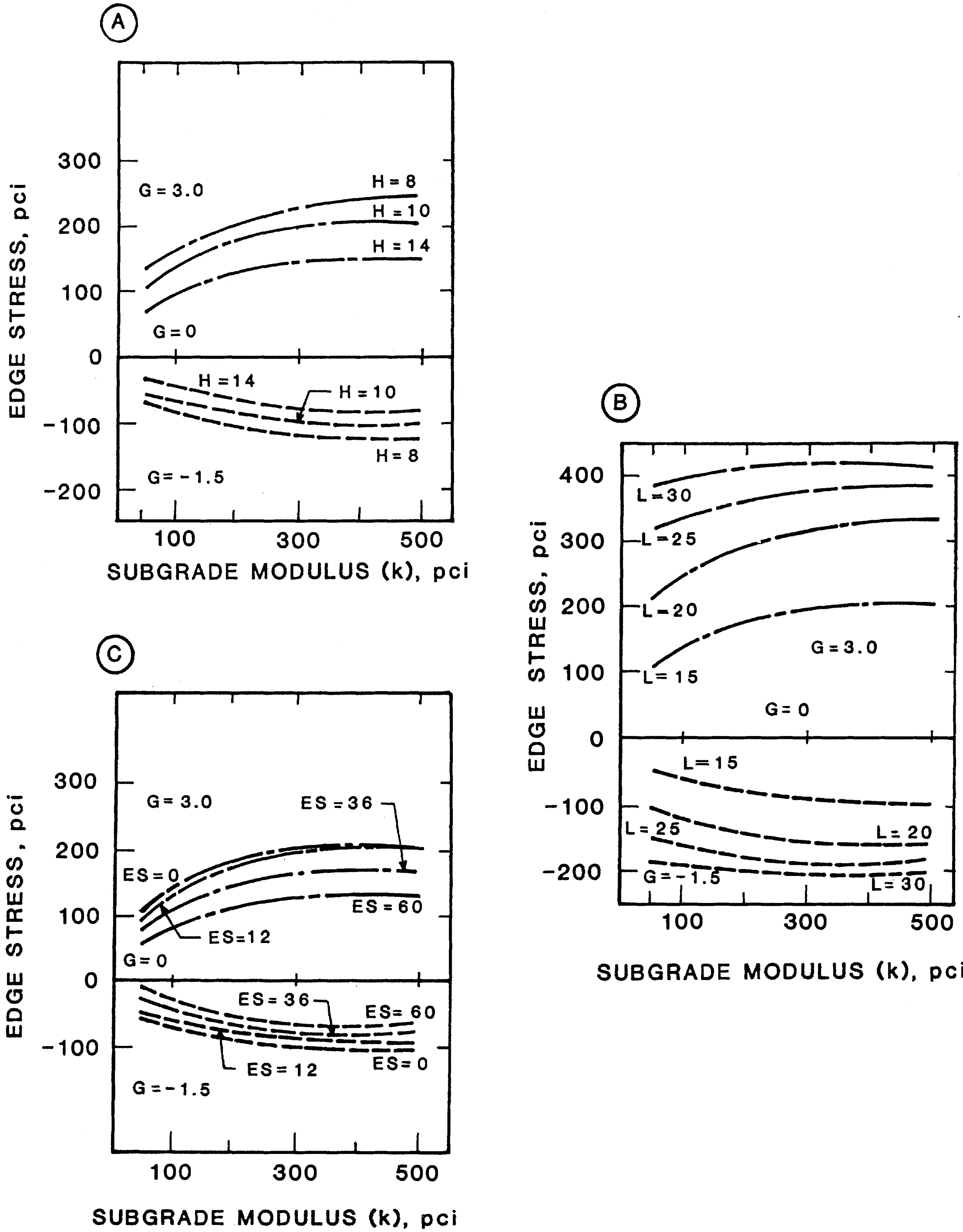

Figure 4-1.22. Effect of Subgrade Modulus on Thermal Curl Edge Stress for a Single Axle (5).

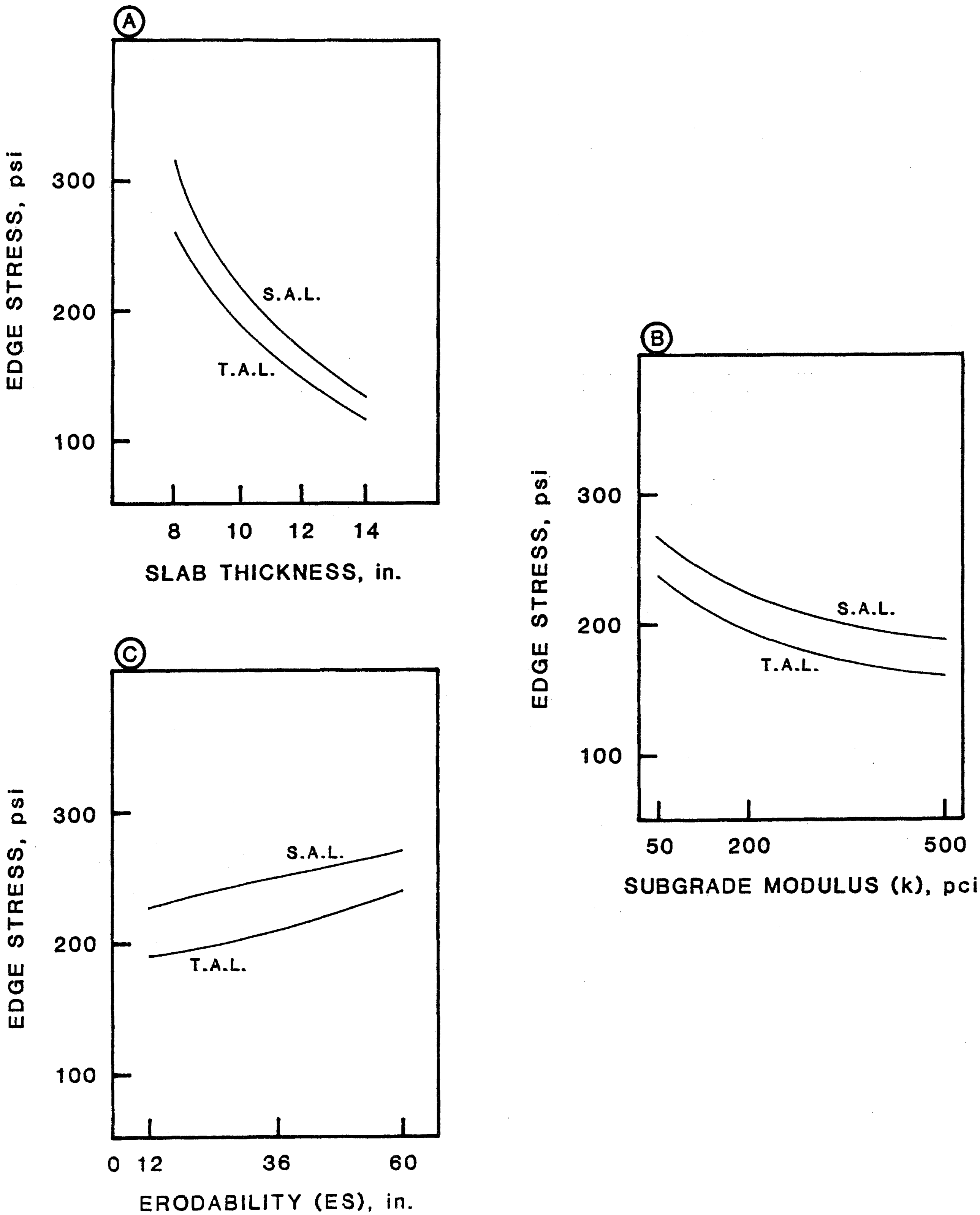

Figure 4-1.23. Comparison of Single and Tandem Axles on Edge Stress (5)

5. As the thermal gradient through the slab increases from a negative (nighttime) to positive (daytime) edge stress caused from both the combined load and curl and curl only increases very significantly (Figures 4-1.18 through 4-1.22). When the gradient is positive (daytime) the edge stress at the slab bottom caused by only thermal gradient is tensile, but if the gradient is negative (nighttime) the edge stress is compressive.

6. Edge stress resulting from an 18-kip single axle load is approximately 15 percent greater than edge stress from a 36-kip tandem axle (Figure 4-1.23).

4.0 EMPIRICAL THICKNESS DESIGN CONCEPTS FOR RIGID PAVEMENTS

The empirical approach to rigid pavement thickness design is based on experience or measurements of field performance alone, normally without consideration of structural theory.

4.1 AASHTO Empirical Design Procedure For Slab Thickness

The only major rigid pavement design procedure which utilizes a totally empirical approach is the AASHTO Design Guide (7). The structural slab thickness design equation is based upon field performance data from the AASHO Road Test conducted from 1958 to 1960 in northern Illinois. The details of this procedure are described in Module 4-2. The basic concept of the empirical development of the slab thickness determination is as follows:

1. Over 200 test sections of JPCP and JRCP were specially constructed having a range of slab and subbase thicknesses over a single subgrade soil and in a single climate. The sections were subjected to single or tandem axle loads (1.1 million) over a two year period.

2. The pavements were monitored for roughness and distress over the two years. Roughness and distress were used to compute the present serviceability index (PSI) of each section. Thus, the present serviceability index of each pavement test section was known over time and traffic loadings.

3. The performance data were used to develop an empirical model (equation) using regression techniques. Serviceability loss was related to slab and subbase thickness and traffic axle type and number of loadings. The basic empirical equation is shown below:

$$\log_{10}(W) = \log_{10}(r) + G/B \qquad (4\text{-}1.5)$$

where:

W = axle load applications, for load magnitude L1 and axle type L2, to a serviceability index of P2

$\log_{10} r = 5.85 + 7.35 \log(D + 1) + 4.62 \log(L1 + L2) + 3.28 \log(L2)$

B = $1.0 + 3.63(L1 + L2)^{5.2}/((D + 1)^{8.46} * L2^{3.52})$

G = $\log((P1 - P2)/(P1 - 1.5))$

D = PCC slab thickness, inches

L1 = load on a single or a tandem axle, kips
L2 = axle code, 1 for single axles and 2 for tandem axles
P1 = initial serviceability index
P2 = terminal serviceability index

This model is limited because it is applicable only to the northern Illinois climate and the specific subgrade and materials used for the pavement/subgrade structure. It also is based only on two years of aging which would not normally cause such problems as concrete durability deterioration, incompressible buildup on joints, and corrosion of reinforcement and dowels.

4. The basic performance equation was modified by the inclusion of a corner stress equation that incorporates slab thickness, k-value, load transfer coefficient and Poisson's ratio in an attempt to make the model more useful (1960). This equation was further extended (1985) to include empirical factors for loss of support, drainage and reliability. The current design equation (Equation 4-2.6) is shown below:

$$\log_{10} W_{18} = Z_R * S_o + 7.35 * \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5 - 1.5}\right]}{1 + \frac{1.624 * 10^7}{(D+1)^{8.46}}} + (4.22 - 0.32 p_t) * \log_{10}\left[\frac{S_c' * C_d \left[D^{0.75} - 1.132\right]}{215.63 * J \left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$

Where:

W_{18} = $W_{18}/F_R = W_{18}/10^{(-Z_R S_o)}$ = Predicted number of 18-kip single axle load applications
F_R = Reliability design factor
Z_R = Standard normal deviate corresponding to selected level of reliability
S_o = Overall standard deviation for rigid pavement
D = Thickness of pavement slab, inches
P_i = Initial serviceability index
P_t = Terminal serviceability index
S'_c = Modulus of rupture for PCC on specific project
J = Load transfer coefficient used to adjust for load transfer characteristics of specific design
C_d = Drainage coefficient
E_c = Modulus of Elasticity for PCC, psi
k = Modulus of subgrade reaction, pci

5. This empirical design equation (without the drainage, loss of support and reliability factors) was utilized to develop a design nomograph that has been used for over 20 years for design of jointed rigid pavements.

6. The design of other components are based largely on the AASHO Road Test pavement design and experience from past performance of other jointed pavements. There does exist a semi-mechanistic design approach for determining reinforcement for JRCP (using the subgrade drag theory).

4.2 Empirical Models to Predict Deterioration of Concrete Pavements

Several predictive models have been developed for rigid pavement serviceability and distresses. Each of these models were developed using a specific pavement database and multiple regression techniques. Each of the models have significant deficiencies because they are based upon a limited database and, thus, must be used within the limits of the data from which they were derived.

4.2.1 Jointed Concrete Pavements

Table 4-1.2 shows empirical equations developed by several different researchers for distresses developing in jointed concrete pavements. Information on the development of the predictive equations as well as additional equations for other distress types may be found in the referenced report for each equation.

The PREDICT program (14) incorporates the empirical equations developed under NCHRP Project 1-19. Under the project, concrete pavements from six states were surveyed, a comprehensive database was developed, and predictive equations were developed for JPCP and JRCP for the following distress types:

1. Slab cracking.
2. Transverse joint faulting.
3. Joint deterioration.
4. Pumping.
5. PSR.

The input screens for the PREDICT program are shown in Figure 4-1.24. The important variables for JPCP are shown. The program outputs can be displayed in graphical form or tabular form. Figures 4-1.25 and 4-1.26 show the outputs for the JPCP case. Use of the program will be demonstrated in the class.

The findings of the NCHRP 1-19 project with respect to development of distress are summarized briefly below. These can be verified through use of the PREDICT program.

Slab Cracking. Slab cracking is a fatigue damage phenomenon which takes a considerable time to develop. Although shrinkage cracks usually occur within a few years after placement in JRCP, the deterioration of cracks requires time to develop. Small cracks begin at the bottom of the JRCP or JPCP slab. Once these "mirco-cracks" develop, cracking accelerates rapidly. The rate of crack development will eventually slow as the slab becomes cracked in several pieces. Most pavements are rehabilitated well before that point is reached.

Transverse Joint Deterioration. Joint deterioration is a more common problem in JRCP than in JPCP due to the longer joint spacings. The distress progresses similarly in both types of pavements. It is believed that joint deterioration is caused by many cycles of opening

Table 4-1.2 Sample Empirical Design Equations

SLAB CRACKING FOR JRCP
Equation Developed from AASHO Road Test (8)

Single axle

$$\log W = 4.95 + 0.5*\log(C) - 2.30*\log(L1) + 3.57*\log(D)$$

Tandem axle

$$\log W = 6.37 + 0.5*\log(C) - 3.13*\log(L1) + 3.96*\log(D)$$

Where:

W = cummulated number of axle loads of magnitude L1
C = cracking index, ft./1000 sq.ft. (essentially working, spalled cracks)
D = PCC slab thickness, ins.

JOINT DETERIORATION FOR JPCP
Developed Under NCHRP Project 1-19 (6)

$$DETJT = AGE^{1.695}*(0.9754*DCRACK) + AGE^{2.841}*(0.01247*UNITUBE) + AGE^{3.038}*(0.001346*INCOMP)$$

Where:

DETJT = Number or deteriorated joints/mile (medium and high severity only)
AGE = Time since construction, years (represents annual cycle of joint opening and closing)
UNITUBE = 0, If no Unitube joint inserts exist
1, If Unitube joint inserts exist
INCOMP = 0, If no incompressibles are present in the joint
1, If incompressibles are present in the joint

TRANSVERSE JOINT FAULTING FOR JPCP
Developed by Packard (9)

$$F_{ave.} = 1.29 + 48.95*((T*A^2)^{0.465}/D^{3.9})*S^{0.610}*(J - 13.5)^{b6})$$

Where:

$F_{ave.}$ = Average joint faulting, 32^{nds} of an inch
T = Number of tractor-semi-trailer and combination trucks in one-direction (average number per day during A years of service)
A = Age of the pavement, years
D = Pavement thickness, inches
S = Type of subgrade with respect to drainage (1 for good, 2 for poor)
J = Joint spacing, feet
b6 = 0.241 for granular subbase
0.037 for stabilized subbase

Screen #1

```
P R E D I C T

Version 1.1

Developed by Michael I. Darter
Copyright (C) 1986
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Written by Michael T. Darter



Press Any Key to continue
```

Screen #2

```
Prediction of Jointed
Concrete Pavement Deterioration


Your Selection:


(P)redict Performance of Specified Pavement.

(C)alibrate Models and Predict Performance
   of Specified Pavement.

(I)nformation about Program.

(Q)uit and return to DOS.
```

Screen #3

```
Type of Pavement to Analyze


(P)lain or (R)einforced
```

Screen #4

```
                               Slab and Shoulder Design Inputs

                                  Slab Thickness, Ins.   10___
                          Transverse Joint Spacing, Ft.  12___
Diameter Of Dowel Bars, Ins. (0 if no dowels)            0____
                   Modulus Of Rupture (3rd Pt. 28-Day)   600__
                              Unitube Inserts Used (Y/N) N
                        Tied Concrete Shoulders (Y/N)    N


                               Foundation and Drainage Inputs

              Type Of Base Course (G)ranular, (S)tabilized  G
          Subgrade Soil Type (F)ine grained, (C)oarse grained  F
   Modulus Of Subgrade Reaction(k-value top of base, psi/in.)  200
```

Screen #5

```
                    Age and Traffic Loading Inputs
Press Ctrl D when done
        Reference Number      Age (Years)      ESAL (Millions)
               1                 5__                3__
               2                 10_                8__
               3                 15_                12_
               4                 20_                20_
               5                 __
```

Screen #6

```
                     Potential Pavement Condition Inputs


     'D' Cracking Aggregate In Slab (Y/N)   N
Incompressibles In Transverse Joints (Y/N)  Y



                          Climatic Inputs

                  Mean Annual Precipitation, Cms   85__
     Corps Of Engineers Freezing Index, Degree Days  1000
                Mean Annual Temperature, Degrees C  11_
Temp. Range (Differance Between Avg. Max. Temp.
in July and Avg. Min. Temp. in January), Degrees C  28
```

Figure 4-1.24. Input Screens from PREDICT Program (14).

Data: 08-13-1987 --

J O I N T E D P L A I N C O N C R E T E P A V E M E N T

Slab Thickness, Ins.	10
Transverse Joint Spacing, Ft.	12
Diameter Of Dowel Bars, Ins.	None
Modulus Of Rupture (28-Day), psi	600
Unitube Inserts Used (Y/N)	No
Tied Concrete Shoulders (Y/N)	No
Type of Base Course	Granular
Subgrade Soil Type	Fine Grained
Modulus of Subgrade Reaction, pci	200
'D' Cracking Aggregate in Slab	No
Incompressibles in Transverse Joints	Yes
Mean Annual Precipitation, Cms	85
Corps of Engineers Freezing Index, Degree Days	1000
Mean Annual Temperature, Degrees C	11
Temperature Range, Degrees C	28

Age (Years)	Single Lane ESAL (Millions)
5	3
10	8
15	12
20	20

P R E D I C T E D R E S U L T S

AGE	ESAL	PUMP	FAULT	CRACK	JTDET.	PSR
0.00	0.00	0.0	0.00	0	0	4.5
5.00	3.00	2.0	0.14	71	0	3.7
10.00	8.00	3.0	0.16	285	1	3.5
15.00	12.00	3.0	0.17	639	5	3.2
20.00	20.00	3.0	0.19	1984	12	2.7
Years	Millions	0 = Low 3 = High	Ins.	Ft./Mile	No./Mile	0 to 5

Figure 4-1.25. Tabular Output from PREDICT Program (14).

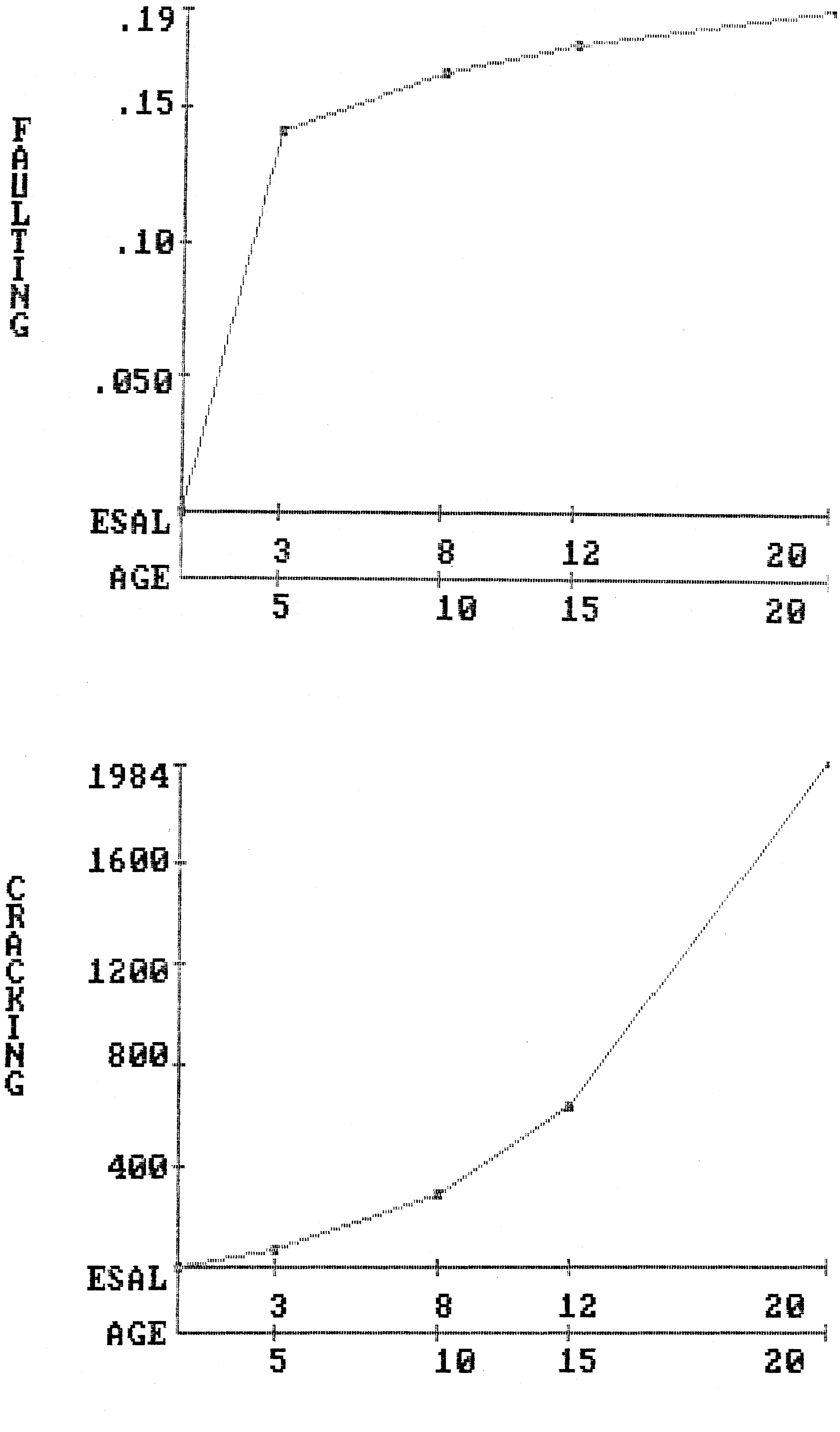


Figure 4-1.26. Graphical Output from PREDICT Program (14).

and closing of the joint which is infiltrated with incompressibles. Freezing and thawing, "D" cracking, and deicing salt (which corrode dowel bars) also contribute greatly to the phenomenon. Aging is required for progression of the distress. Joint deterioration starts out slowly then progresses rapidly as more and more of the working joints begin to lock-up.

Transverse Joint Faulting. Faulting begins rapidly and then levels off. The specific reasons for the rapid, early occurrence of faulting are not known, but are most likely related to the almost immediate tendency of a pavement to pump soon after opening to traffic. The presence of dowel bars reduced the amount of faulting. However, an initial looseness in the dowels from the grease coating may contribute to early faulting.

Pumping. Pumping begins early in the life of the pavement and continues almost constantly throughout the life of the pavement. Several factors influence pumping, including, slab thickness, annual precipitation, drainability of soil, and the presence of subdrains.

Present Serviceability Index/Rating. The present serviceability rating drops off rapidly during the early life of the pavement. This could be due to the rapid increase in faulting which causes roughness. Overall pavement serviceability is affected by many of the factors that influence the distresses presented above because it is closely related to the major distress types.

4.2.2 Empirical Model for Punchouts in Continuously Reinforced Pavements

The punchout is the critical structural failure mode of CRCP. One predictive empirical model for punchouts has been developed using field data from CRCP in Illinois (10). The dependent variable, failures/mile, include punchouts, permanent repairs and high severity transverse crack steel ruptures.

$$\begin{aligned} FAIL = {} & 0.0001673*ESAL^{1.9838}*THICK^{(-4.2772)}*ASTEEL^{(-5.0)} \\ & + 0.4127*ESAL^{1.9553}*(0.01584*BAM + 1.9080*CAM - 0.02005*BAR) \end{aligned} \tag{4-1.7}$$

Where:

FAIL = Total number of punchouts plus steel ruptures plus number of patches per lane mile
ESAL = Accumulated 18-kip equivalent single-axle loads in the outer lane, millions
THICK = Slab thickness, inches
ASTEEL = Area of reinforcement, square inches per inch width of slab
BAM & CAM = Both zero (0), if subbase material is granular
1 & 0, if subbase material is BAM
0 & 1, if subbase material is CAM
BAR = 0, if deformed welded steel fabric is used
1, if deformed rebars are used

A sensitivity of some main factors and the functional form of the typical development of punchouts for a CRCP is shown in Figure 4-1.27. This is a fatigue phenomenon and requires many repeated loadings before it rapidly increases.

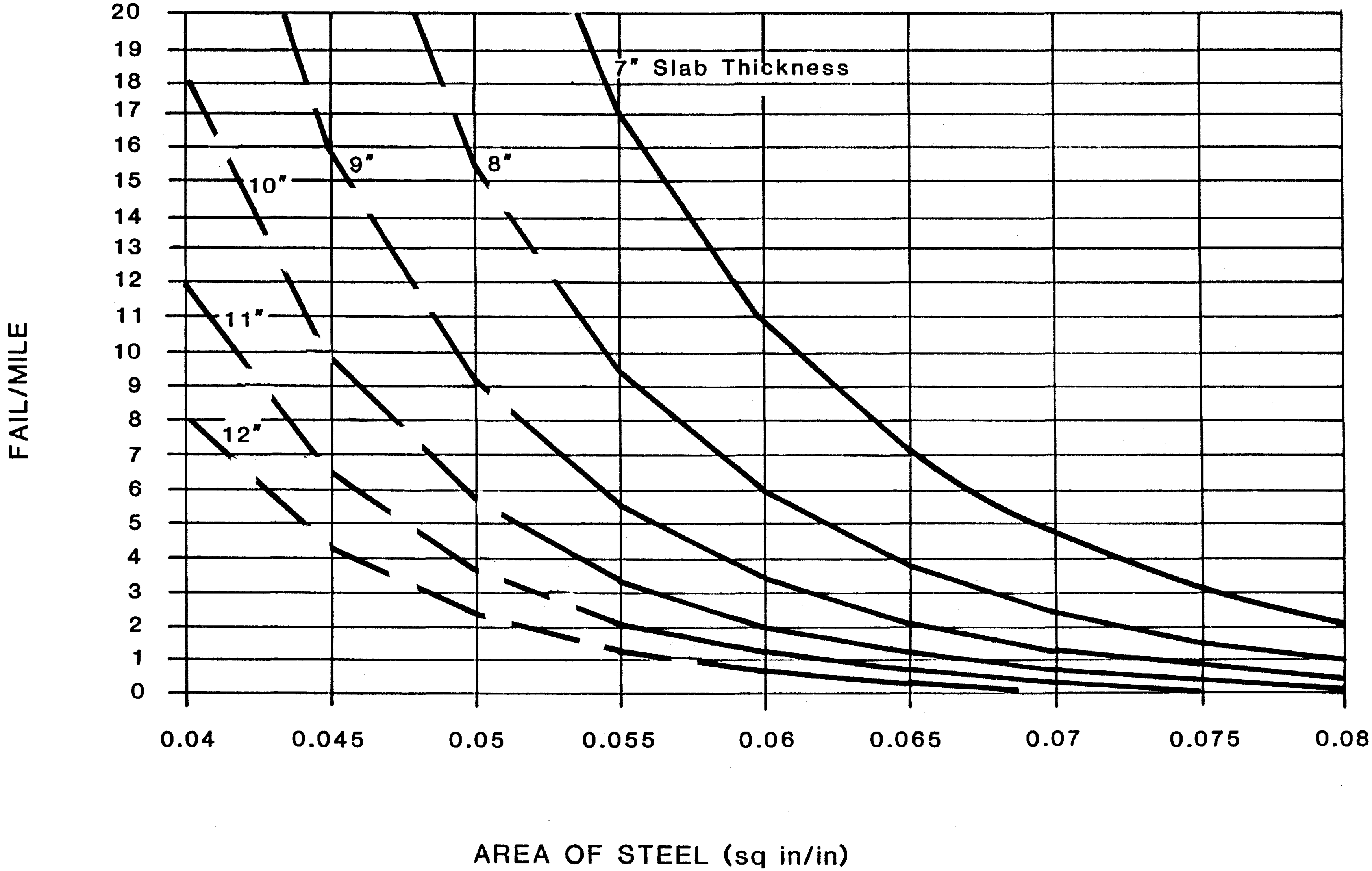


Figure 4-1.27. CRCP Failures Per Mile vs. Area of Steel (BAM Subbase, 15 million ESAL and Deformed Re-Bars) (10).

4.2.3 Limitations of Empirical Models

Empirical Models have several serious limitations as stated below:

1. The model reflects the database completely.

2. Databases may be inadequate for future design needs.

3. Models cannot be applied to design situations which were not included in the database. For example, widened lanes, permeable basis, and tied shoulders must be included in the developmental database in order to examine the effects of these features.

5.0 MECHANISTIC THICKNESS DESIGN CONCEPTS FOR RIGID PAVEMENTS

The mechanistic approach to rigid pavement thickness design uses the concepts presented in Section 3.0 to structurally model the pavement. Several relationships have been developed to relate the stresses calculated using theoretical models to actual pavement performance.

5.1 Stress Computation

Mechanistic design requires the calculation of critical stresses in the PCC slab caused by all significant factors. Stresses in a rigid pavement are caused by traffic, thermal gradients, moisture gradients, drying shrinkage, thermal heating and cooling of the slab, and foundation movements. The calculation of these stresses was discussed in Section 3.0.

5.2 Fatigue Damage Computation

Many laboratory and field tests have shown that concrete beams and slabs experience fatigue cracking failure when subjected to high repetitive flexural stresses. Every stress application produces some amount of damage which accumulates until a fracture occurs. This section summarizes some basic concepts in fatigue damage to concrete pavement slabs.

5.2.1 Concrete Fatigue Curves

Concrete fatigue curves for PCC beams have been derived in many studies such as the data presented in Module 2-3. The number of repeated loads that concrete can sustain in flexure before fracture depends upon the ratio of applied flexural stresses to the ultimate static flexural strength or modulus of rupture.

5.2.2 Miner's Fatigue Damage Accumulation

Because there are usually many different load magnitudes applied to a highway pavement, some means of combining the damage caused by the different loads must be available to assist in the design of the pavement. The fatigue damage concept published by Miner (11) has found widespread use in pavement design and evaluation. The Miner fatigue damage accumulation hypothesis is given as follows:

$$\text{DAMAGE} = \sum n_i/N_i \qquad (4\text{-}1.8)$$

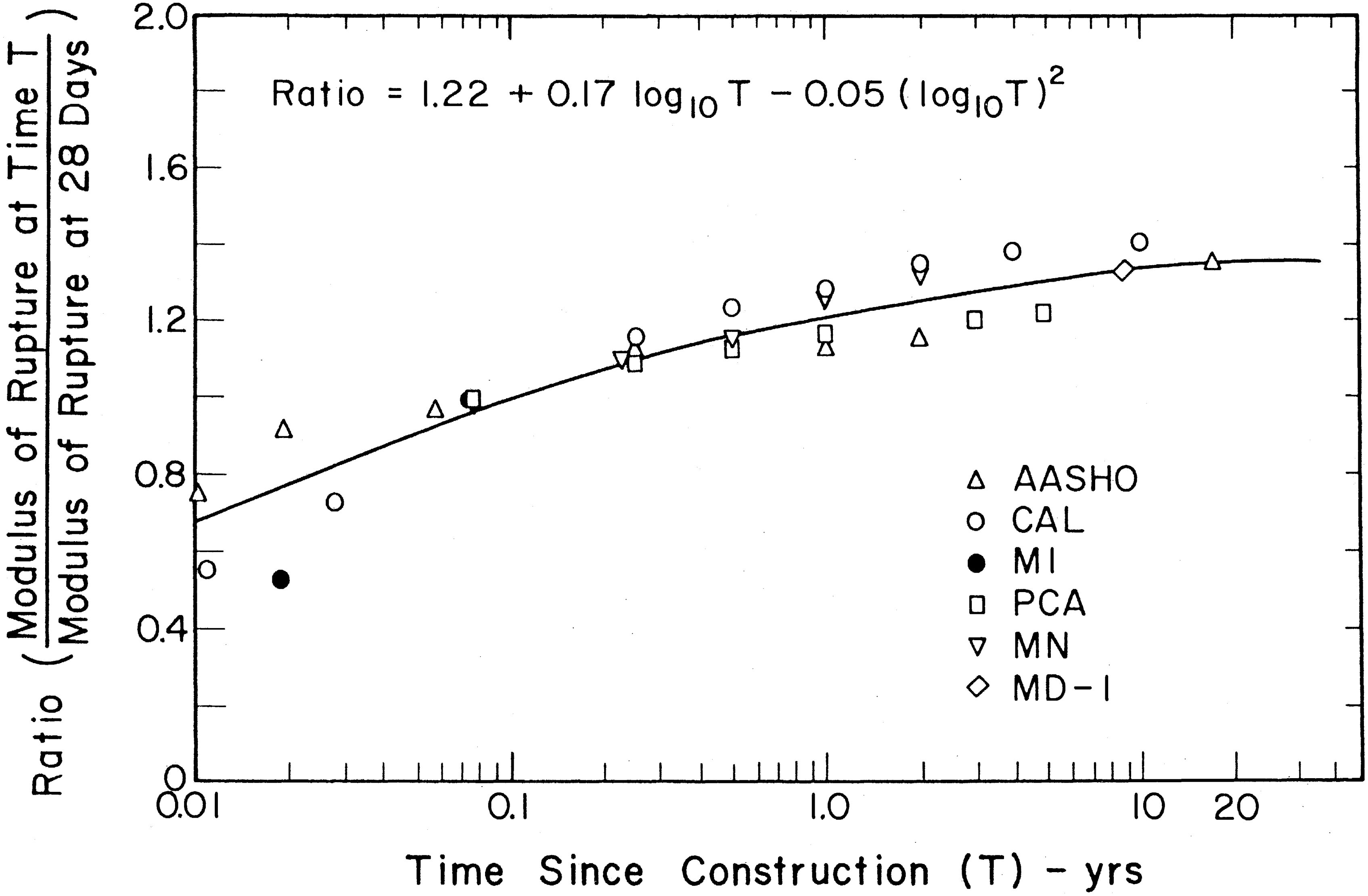


Figure 4-1.28. Change in PCC Modulus of Rupture With Time.

where:

n_i = no. of actual load applications applied of i^{th} magnitude

N_i = no. of allowable load applications applied of i^{th} magnitude until cracking occurs

5.2.3 Concrete Strength Gain

The ratio of stress to flexural strength of PCC is a major factor in determining the number of allowable load applications to cracking. Flexural strength increases over time after placement of the pavement slab as illustrated in Figure 4-1.28. The strength after a few years can be over 30 percent greater than it is after 28 days. This change in strength can have a large effect on the fatigue damage in the slab and should be considered in design.

5.2.4 Fatigue Damage Number

What does it mean when a computed fatigue damage is equal to 0.31, 1.00 or 3.65? Figure 4-1.29 illustrates this concept. A value of 1.0 theoretically means 50 percent of the slabs are created (for this particular example).

5.2.5 Critical Fatigue Location In Slab

The location of the critical point at which cracking initiates in the slab is vital to the development of a fatigue analysis with the objective of preventing slab fatigue cracking. Results from road tests and in-service pavements that do not have tied concrete shoulders show that cracking initiates at the longitudinal edge of the slab, where the critical stresses occur.

If the pavement has widened lanes or tied PCC shoulders, the critical fatigue damage point occurs either at the transverse joint or somewhere at the interior of the slab depending upon conditions.

5.2.6 Computation of Total Fatigue Damage

A comprehensive fatigue damage analysis would include the following considerations:

1. The critical fatigue damage point in the slab must be located and all stresses computed at that point.

2. Critical stresses caused by traffic loads and at least thermal curling should be considered. Other stresses may be neglected as they are generally much smaller in magnitude. Correct procedures must be used to combine traffic loading and thermal curling stresses (finite element models).

3. The proportion of traffic occurring near the critical fatigue point must be determined.

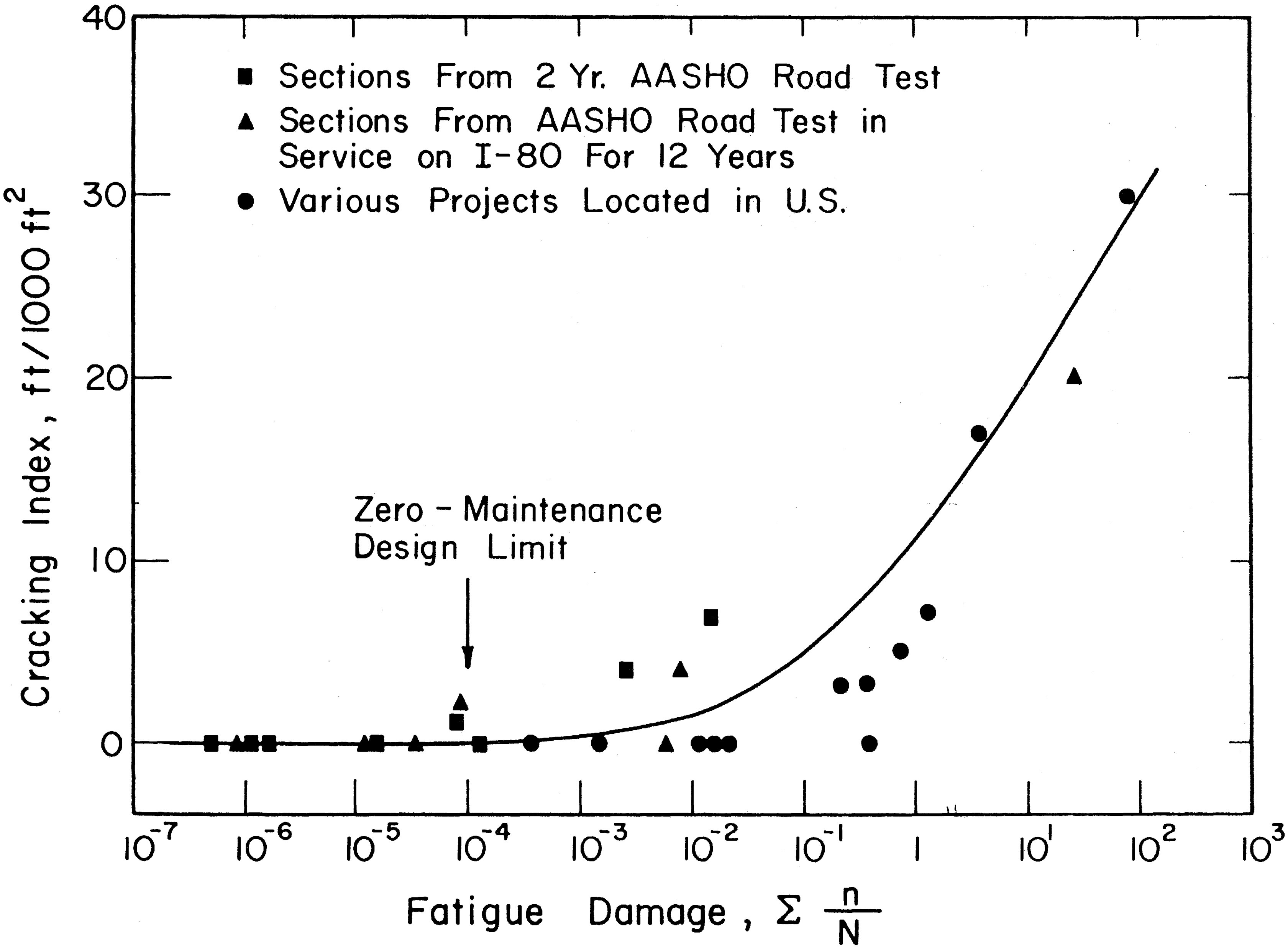


Figure 4-1.29. Effect of Fatigue "Damage" on Cracking of PCC Plain Jointed Concrete Pavements (5).

4. Concrete strength increases over time which makes the fatigue analysis time dependent.

5. The Miner hypothesis to compute total accumulated fatigue damage should be used.

6. Field calibration of the computed fatigue damage with actual slab cracking should be performed.

5.2.7 Field Calibration of Fatigue Damage With Slab Cracking

Even though considerable effort has been made to compute the actual fatigue damage in the pavement slab, there are simplifying assumptions made in the computations (such as the use of beam fatigue models). Therefore, it is necessary to calibrate the computed fatigue damage with field measured cracking.

An example of this result is shown in Figure 4-1.29 where computed Miner's damage is correlated with measured linear feet of cracking of several actual field JPCP sections. If the theory was exactly correct, there would be exactly 50 percent slabs cracked when the computed fatigue damage equaled 1.0. If 50 percent of the slabs were cracked a cracking index would be about 25 ft. per 1000 ft. square. Thus, this fatigue computation procedure provides damage calculation estimates that are not far off from the theoretical value.

6.0 BASIC CONCEPTS FOR JOINT DESIGN

The design of transverse and longitudinal joints is critical to the successful performance of the rigid jointed concrete pavement. Inadequate joint design has led to much serious failure in jointed rigid pavements.

6.1 Critical Factors In Joint Design

The following components of joint system must be designed:

1. Load transfer of transverse joints.

2. Spacing of transverse joints.

3. Spacing (or lane width) between longitudinal joints.

4. Joint sealant reservoir.

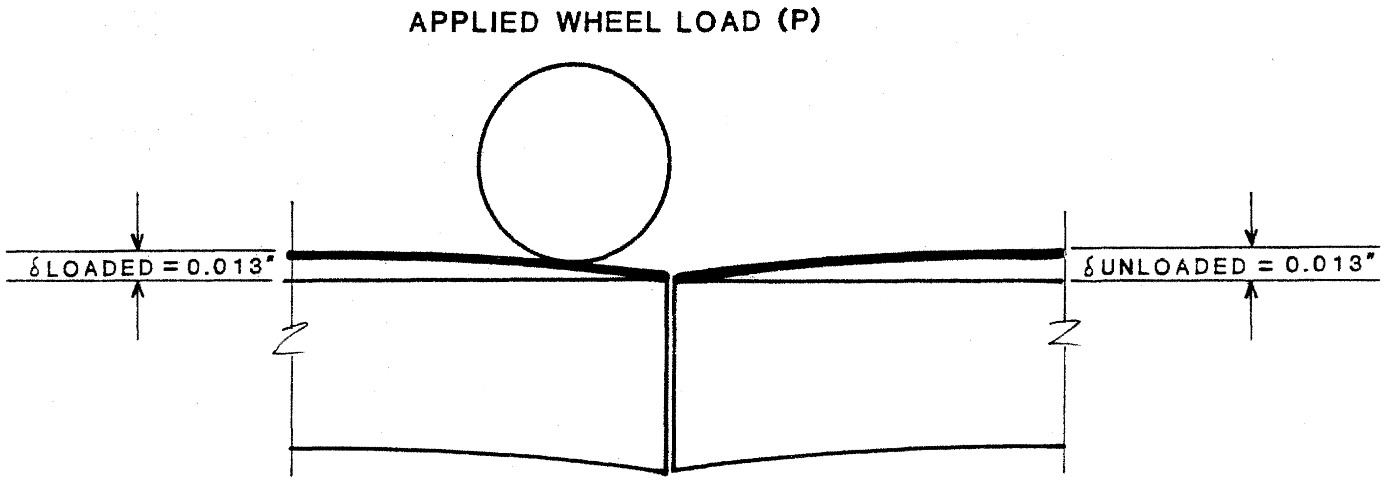
5. Ties between longitudinal joints.

Each of these factors will be briefly discussed in this module. More details on the design of each component is provided under each design procedure.

6.2 Load Transfer of Transverse Joints

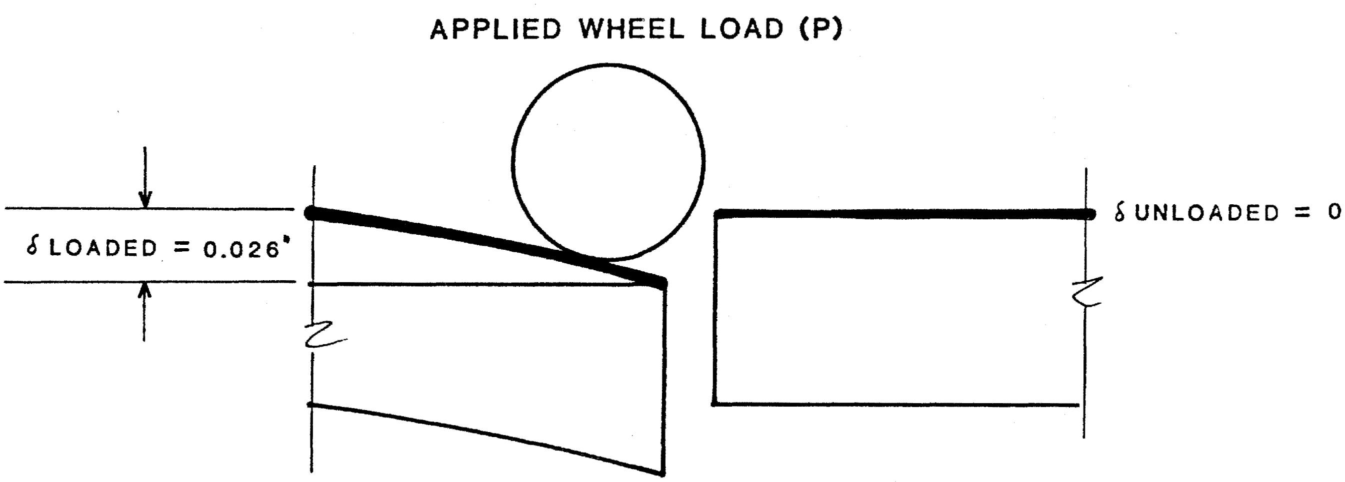
Deflection load transfer is defined as:

$$LT_{\delta} = (\delta_{unloaded}/\delta_{loaded}) \times 100\% \qquad (4\text{-}1.9)$$



100% LOAD TRANSFER

$$\delta_{LOADED} = \delta_{UNLOADED} = 1/2\ \delta_{TOT} = 0.013"$$



0% LOAD TRANSFER

$$\delta_{TOT} = \delta_{LOADED} = 0.026" \qquad \delta_{UNLOADED} = 0$$

WHERE:

δ_{LOADED} = DEFLECTION OF LOADED SLAB

$\delta_{UNLOADED}$ = DEFLECTION OF UNLOADED SLAB

δ_{TOT} = TOTAL DEFLECTION

Figure 4-1.30. Effect of Load Transfer.

Also;

$$\delta_{tot} = \delta_{unloaded} + \delta_{loaded} \qquad (4\text{-}1.10)$$

where:

LT_δ = Deflection load transfer
$\delta_{unloaded}$ = Deflection of the adjacent unloaded slab
δ_{loaded} = Deflection of the loaded slab
δ_{tot} = Total system deflection

If the load transfer is perfect or 100%, the unloaded slab deflects the same amount as the loaded slab. If the load transfer is 0%, the unloaded slab does not deflect at all. This concept is illustrated in Figure 4-1.30.

Stress transfer can be calculated using the ratio of the stress of the loaded slab to the stress of the unloaded slab. The equation is given below:

$$LT_\delta = (\delta_{unloaded}/\delta_{loaded}) \times 100\% \qquad (4\text{-}1.11)$$

$$\sigma_{tot} = \sigma_{unloaded} + \sigma_{loaded} \qquad (4\text{-}1.12)$$

where:

LT_σ = stress load transfer
$\sigma_{unloaded}$ = stress in loaded slab
σ_{loaded} = stress in adjacent unloaded slab
σ_{tot} = total free edge stress

It is important to note that LT_δ does not equal LT_σ. Figure 4-1.31 shows the approximate relationship between stress-calculated load transfer and deflection-calculated load transfer as determined by a finite element computer program.

For example, assume that it is desired to compute the stresses at a joint where dowels will be used to transfer load. It is assumed that a deflection load transfer of 80% can be achieved with the dowels.

$$LT_\delta = 0.80$$

The free edge stress from the slab is computed to be 400 psi for the given conditions.

From Figure 4-1.30 determine stress load transfer:

$$LT_\sigma = 41\% = (\sigma_{unloaded}/\sigma_{loaded}) \times 100\%$$

$$0.41\sigma_{loaded} = \sigma_{unloaded}$$

Using equation 4-1.12, and substituting from above, calculate the stress in each slab:

$$400 = 0.41\sigma_{loaded} + \sigma_{loaded}$$

σ_{loaded} = 284 psi, $\sigma_{unloaded}$ = 116 psi

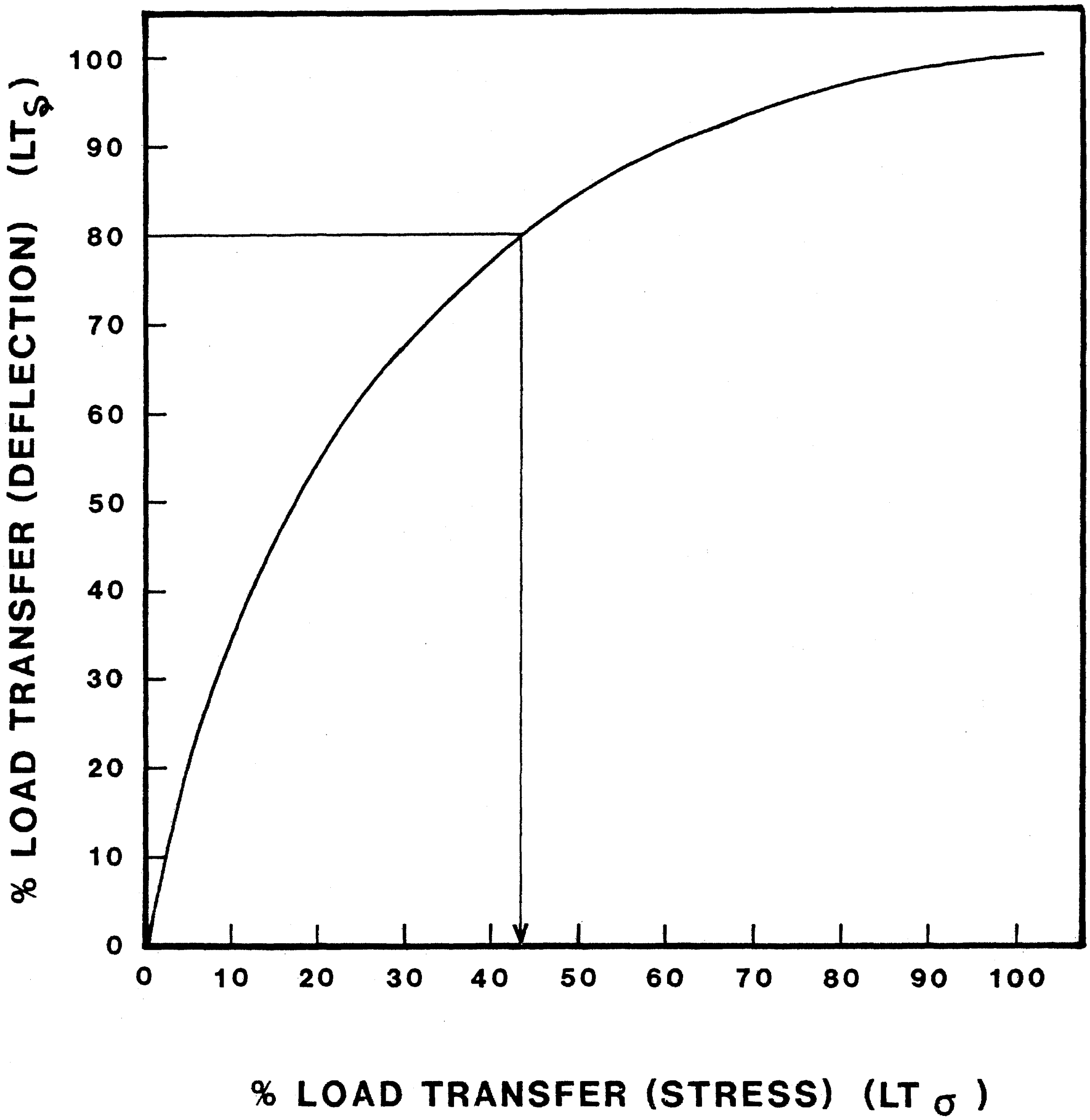


Figure 4-1.31. Relationship Between Load Transfer Efficiency Based on Deflection and that Based on Stress.

Dowel bars, keyed joints, and aggregate interlock are methods of achieving load transfer. Aggregate interlock provides vertical shear resistance between two adjacent slabs. If the slabs are in close contact, aggregate interlock can provide good load transfer. However, since slabs contract from drying shrinkage and cooler temperatures, the faces are not always in contact. This causes the loss of load transfer.

6.2.1 Need For Load Transfer

The key reasons for maintaining adequate load transfer is to prevent faulting and to keep corner deflections and stresses low to reduce pumping (and hence faulting) and corner breaks from loss of support.

6.2.2 Criteria For Mechanical Load Transfer Devices

Dowels or other mechanical load transfer devices are always used on JRCP because it is clearly recognized that the large joint openings cause a complete loss of aggregate interlock load transfer. This condition (without dowels) would result in rapid and serious faulting of the joint.

However, there is great difference of opinion as to the use of dowels for shorter jointed JPCP. In fact, one of the most controversial design issues is the need for and the use of dowels in JPCP.

Practically all JPCP projects located in the dryer western states have not used dowel bars. Many of these have performed for many years without serious faulting, but some have developed serious faulting. Also, a number of projects in many other states in the East have not used dowels in the transverse joints. These pavements have typically developed greater amounts of faulting than in the dryer western states, due to greater precipitation.

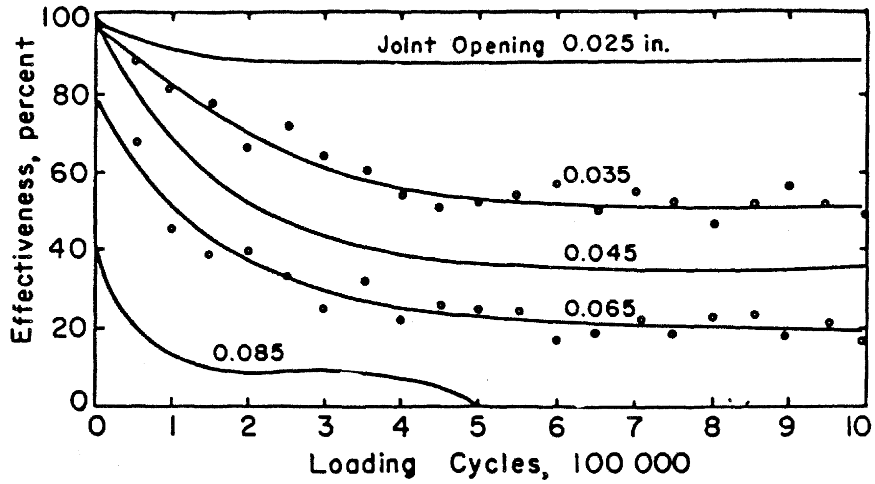
Figure 4-1.32 shows some of the factors that affect joint load transfer such as joint opening and k-value of the base. Other factors include types of aggregate and thickness of slab. An interesting graph is shown in Figure 4-1.33 for joint faulting with and without dowels.

Some empirical models to estimate joint faulting are discussed in Section 4.0. These models show that dowels reduce transverse joint faulting (6,9). However, dowels are costly and there may be other design features that could be utilized (such as permeable bases, shorter joint spacing, thicker slabs) to control faulting more economically under certain climatic and traffic conditions. This is definitely an area where additional research is needed.

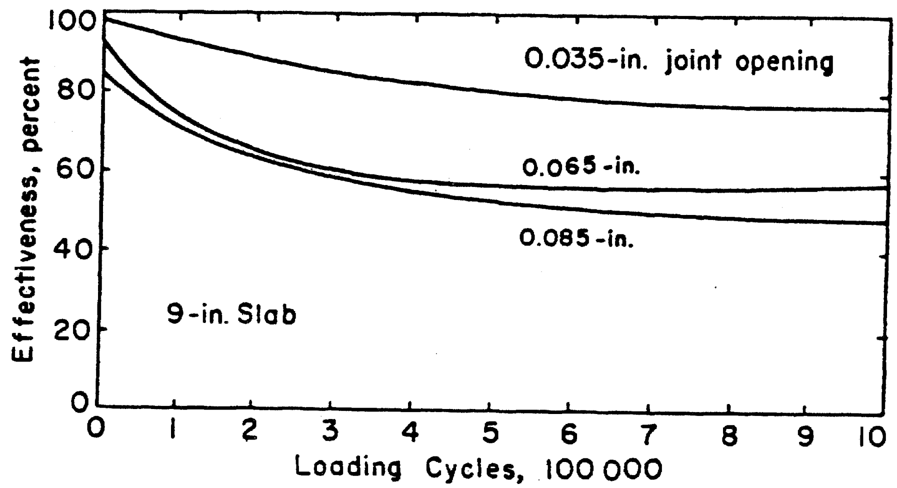
6.3 Relationship Between Dowel/Concrete Bearing Stress and Faulting

Truck loadings have increased in volume and weight over the past 20 or so years. Slab thicknesses have also increased some. Dowel bar load transfer design has not changed, however, and a substantial amount of faulting is occurring on pavements with dowels.

Analysis of field data has shown a strong relationship between dowel bearing stress and transverse joint faulting. Figure 4-1.34 shows some data from the "extended" AASHO Road Test on I-80. Although slab thickness is varying along with dowel bar diameter, the dowel bar diameter actually has by



(a) Influence of joint opening on effectiveness
(9 in. slab, 6 in. gravel subbase, k = 145 psi)



(b) Influence of joint opening on effectiveness
(9 in. slab., cement stabilized subbase,
k = 542 pci)

Figure 4-1.32. Joint load effectiveness for gravel and cement treated subbase for various joint openings.

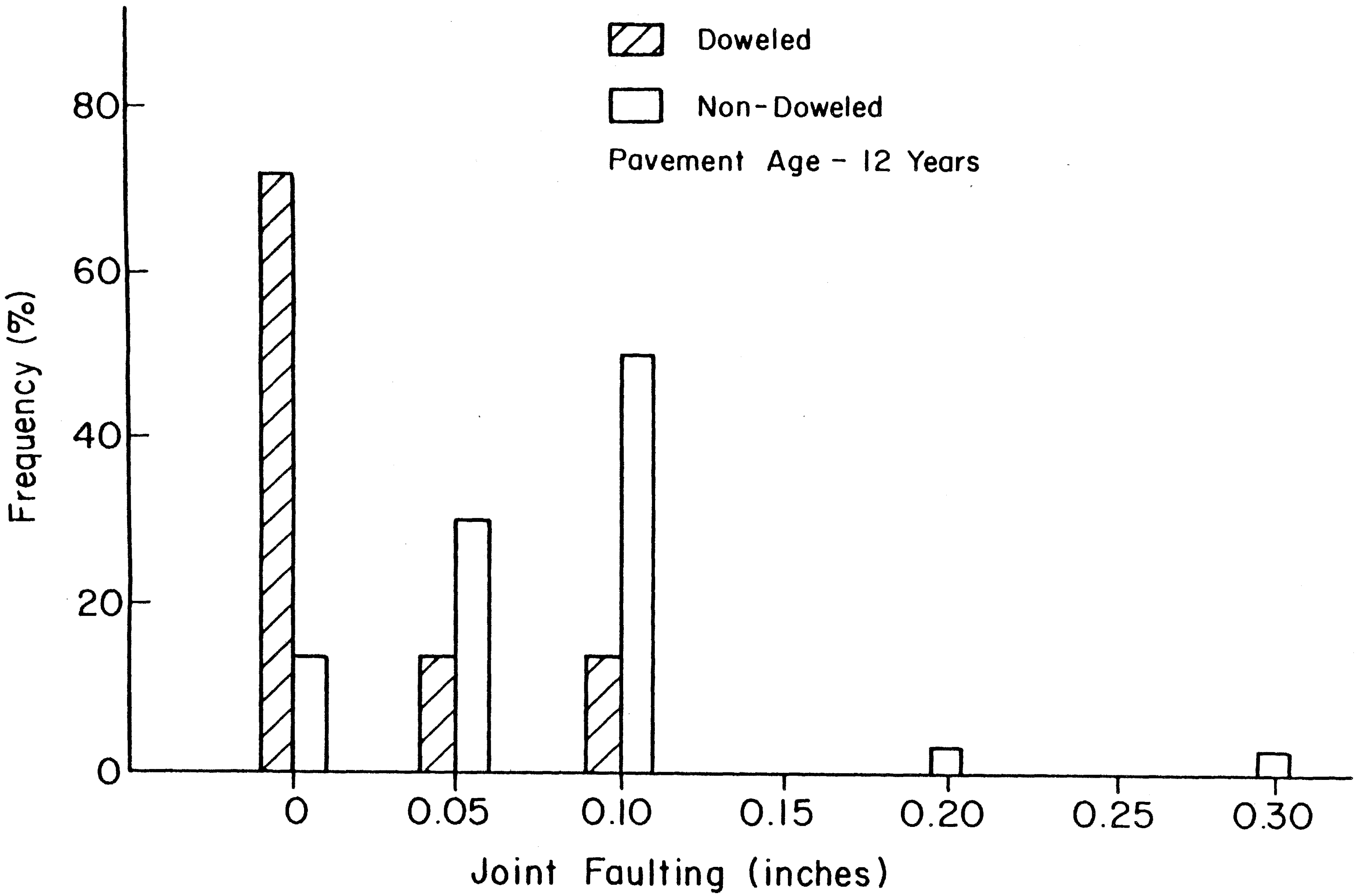


Figure 4-1.33. Comparison of Joint Faulting Between Doweled and Non-doweled Joints - Florida Test Sites 3, 6, and 7 - River Gravel and Crushed Stone Aggregate in PCC.

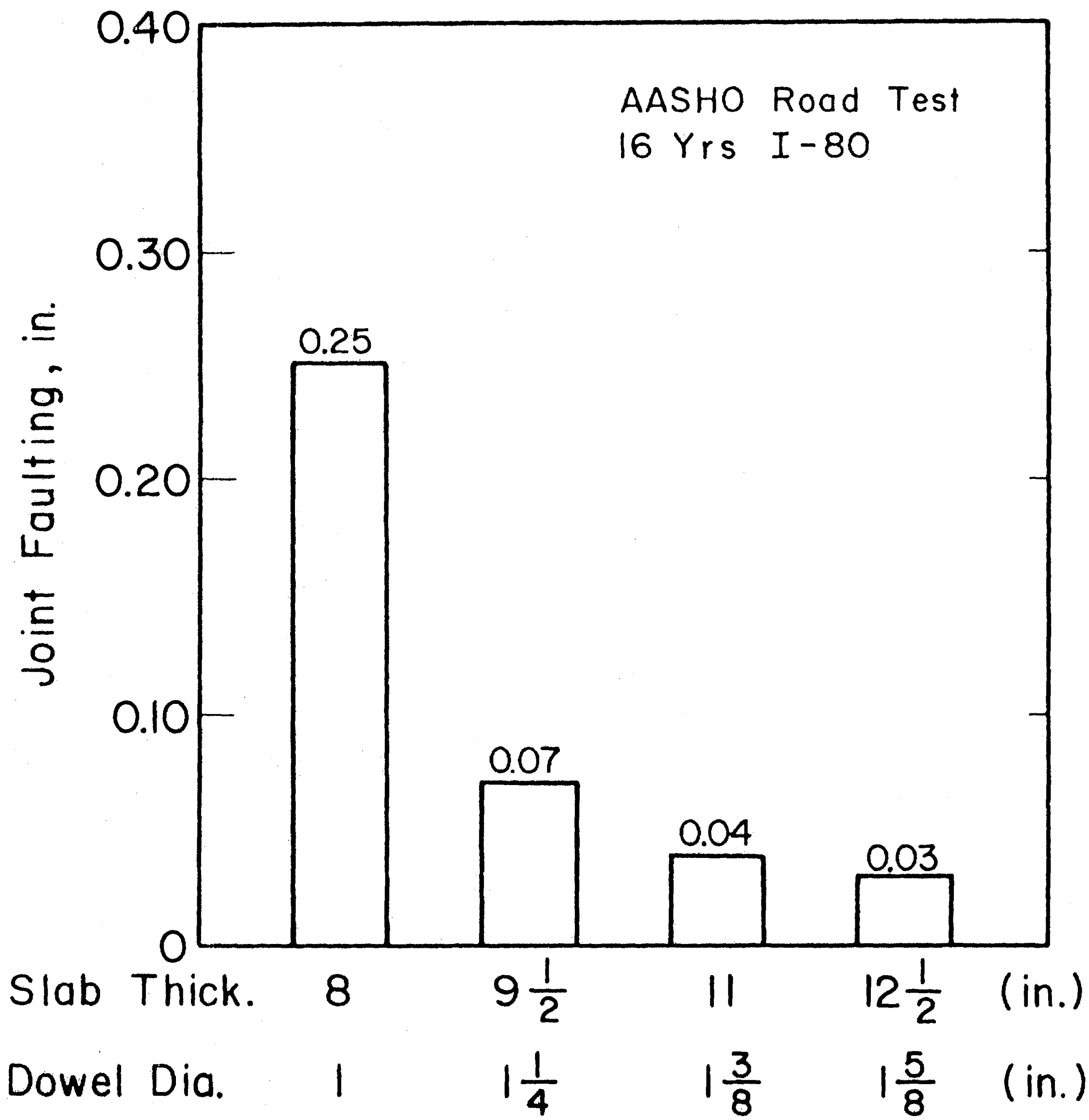


Figure 4-1.34. Joint Faulting on Plain Jointed Concrete AASHO Road Test Sections Left In-Service on I-80.

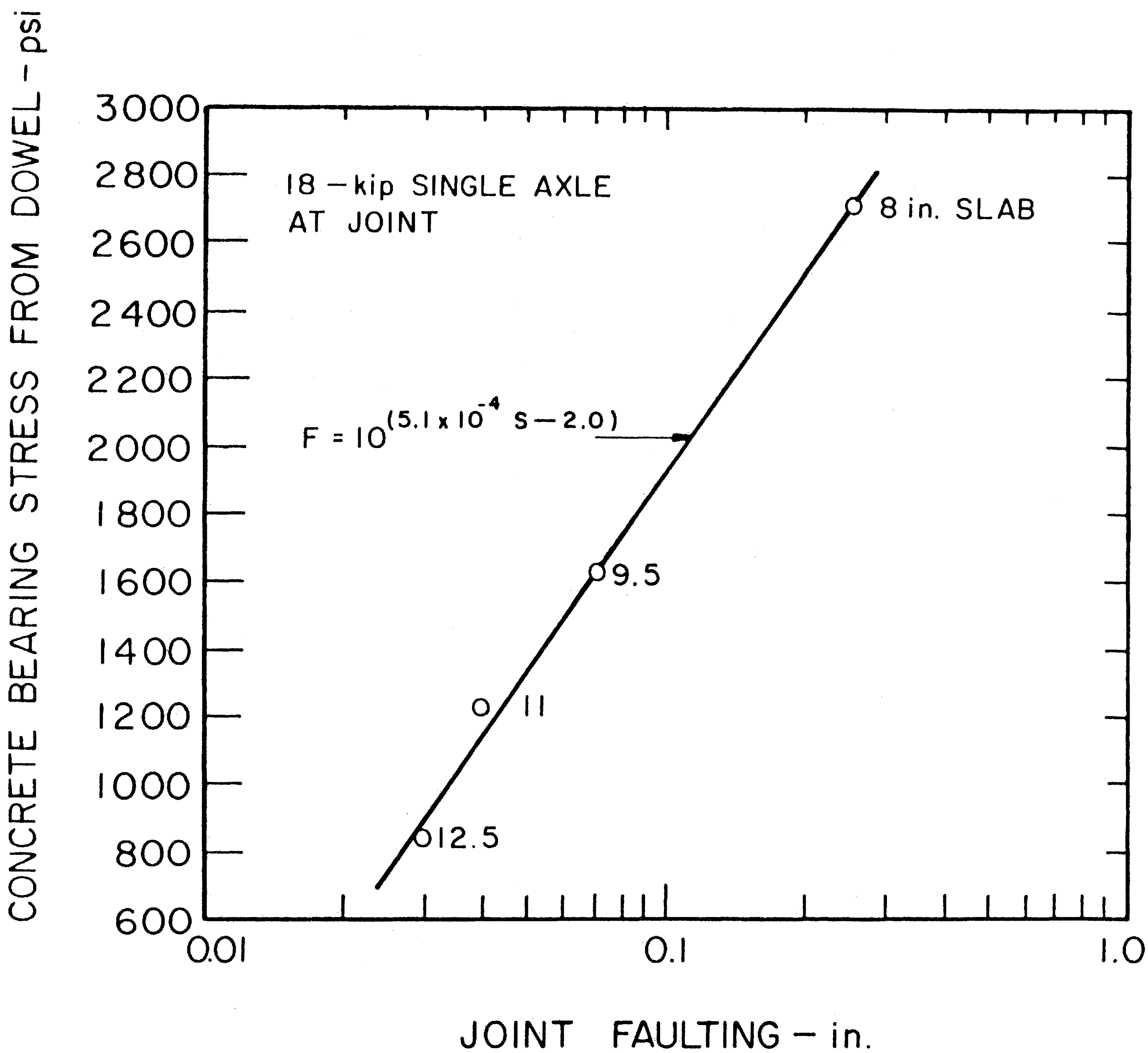


Figure 4-1.35. Concrete Bearing Stress Under Dowel Bars Versus Mean Joint Faulting for JCP-1 to 25 After Being Subjected to 13-19 Million 18-kip ESAL Over 16 Years.

far the strongest effect on bearing stress between the concrete slab and dowel as shown in Figure 4-1.35. An equation which was developed from this study was presented in Section 3.6. It is strongly recommended that some model be utilized to check the conventional design of doweled joints to estimate the amount of faulting that may occur. Heavy truck routes may require substantially heavier dowel bars than presently used to control faulting.

6.4 Joint Design When Doweled Load Transfer Devices Are Not Used

The following design recommendations are for situations where dowels are not considered feasible to minimize joint faulting:

1. Permeable base beneath slab to reduce pumping.
2. Shorten transverse joint spacing (e.g., less than 13 ft.).
3. Thicker slabs.
4. Tied concrete shoulders.

6.5 Transverse Joint Spacing

The spacing of transverse joints is one of the most critical design factors for either JPCP or JRCP. The following summarizes the concepts and factors involved.

6.5.1 Transverse Joint Spacing for JPCP

The major considerations involved are joint faulting and transverse slab cracking. The longer the joint spacing, the greater the thermal curling stresses as explained in Section 3.4.3, and the greater the cracking. For example, pavement with the typical random joint spacing of 12, 13, 18 and 19 ft. rarely have cracking in the 12 or 13 ft. slabs whereas the 18 and 19 ft. slabs often have transverse cracking. This is especially true for slabs placed on stiff bases, such as lean concrete. There have been major pavement failures due to the use of too long joints for JPCP.

A general rule of thumb to prevent cracking is that joint spacing should not exceed 1.5 to 1.75 times the slab thickness. Thus, an 8 inch slab would have a maximum joint spacing of 12 ft. and a 12 inch slab would have a maximum joint spacing of 18 ft.

Another advantage of using short joint spacing for JPCP is a reduction of joint opening as illustrated in Figure 4-1.36 to keep average joint opening to 0.03 inches. This reduction will lead to reduced faulting of the joints. One study in California determined that faulting could be reduced by approximately one-half through a reduction in joint spacing from 12-13-18-19 ft. to about one-half these values.

6.5.2 Transverse Joint Spacing For JRCP

Joint deterioration is by far the most serious type of failure for JRCP. Joint spacing for reinforced jointed pavements has typically ranged from 40 to 100 ft. In the past few years, design guidelines have recommended a maximum of 40 ft. spacing.

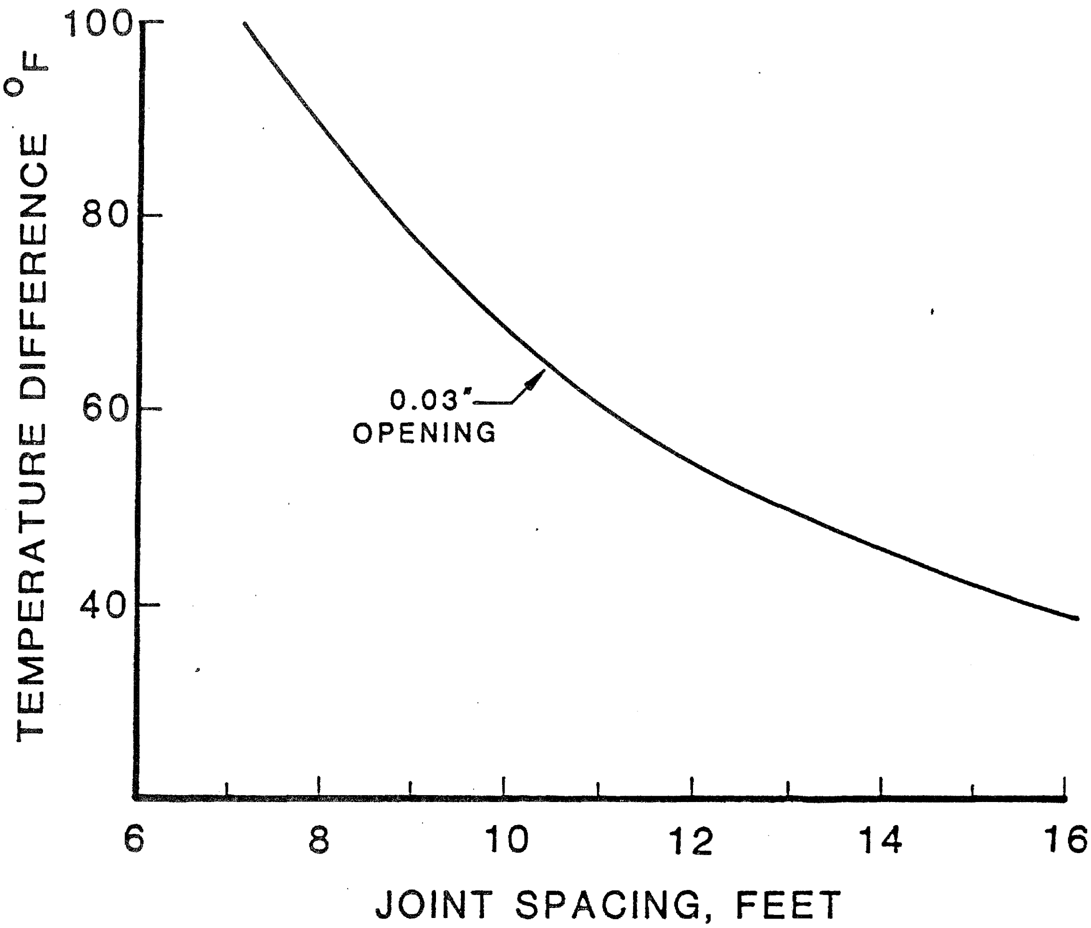


Figure 4-1.36. Temperature Difference Versus Joint Spacing to Obtain Joint Opening of 0.03 inches.

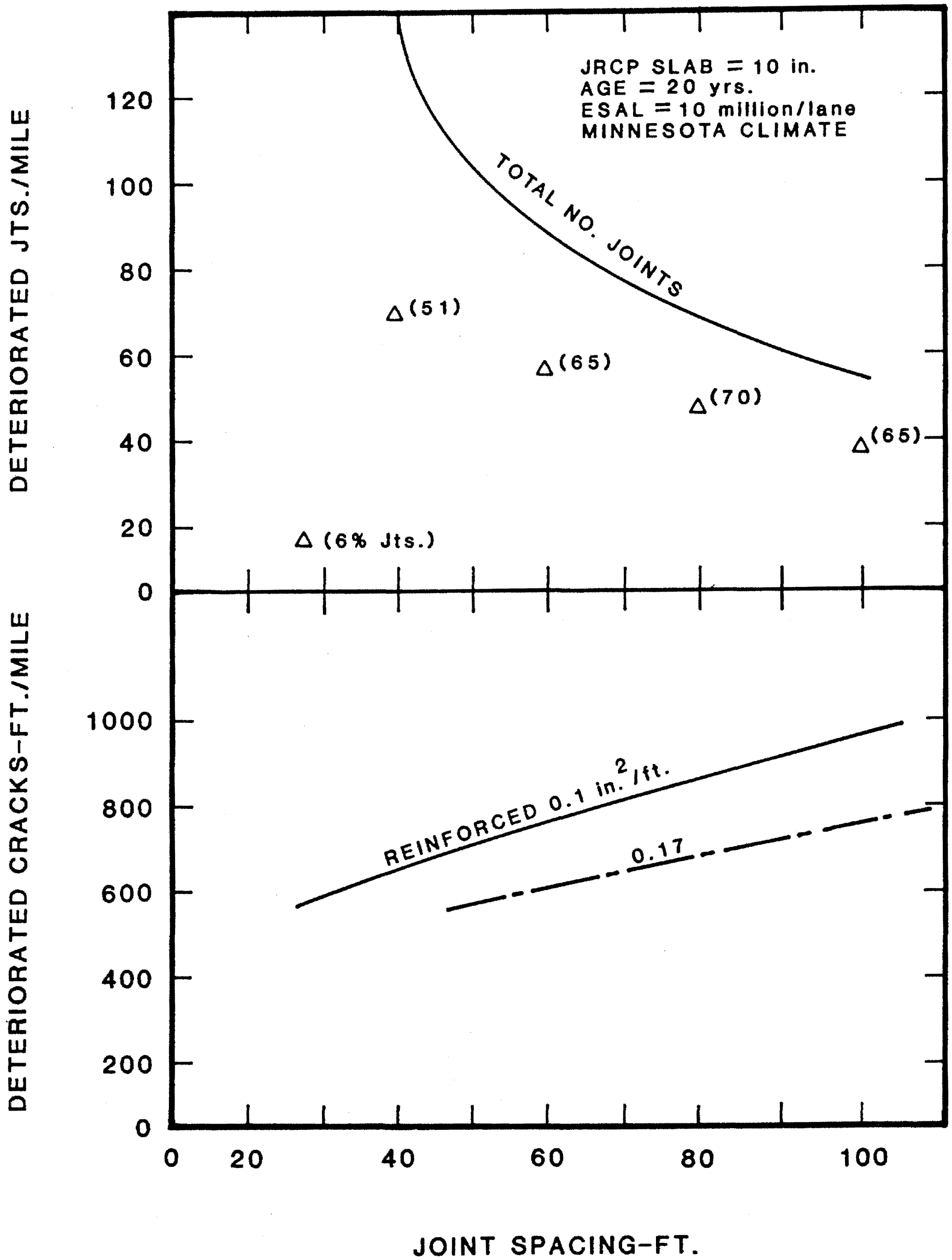


Figure 4-1.37. Results from Joint Deterioration Model from JRCP Showing Effect of Joint Spacing on Number of Deteriorated Joints and Cracks (3).

Figure 4-1.37 shows the results of an analysis of joint spacing on the number of deteriorated joints and cracks per mile. This graph shows the effect of joint spacing on deteriorated joints and cracks. As joint spacing is decreased from 100 to 40 ft. the total number of joints increases, as does the total number of joints that deteriorate per mile (38 per mile at 100 ft. and 66 per mile at 40 ft.). However, as joint spacing is decreased further, the amount of joint deterioration decreases rapidly. At 27 ft. spacing the number is 16 per mile for the example.

As joint spacing decreases from 100 ft. to 40 ft., deteriorated transverse cracks decrease as shown. This data shows that the currently recommended 40 ft. joint spacing may be the worst possible spacing because a large proportion of them fail (probably due to incompressibles). A joint spacing of less than 30 ft. may provide a much lower number of total joint failures per mile as expereinced by at least one state.

7.0 DESIGN CONSIDERATIONS FOR MINIMIZING DISTRESS

There are several key distress types that can be directly considered in rigid pavement design. Each of these are listed, major causes and design factors identified and information on their effect on rigid pavement performance provided. These recommendations are based upon field observations and theoretical considerations.

7.1 Slab Cracking -- JPCP only

There are three major factors: slab thickness, length and width. Increased slab thickness has a large effect on reducing cracking. Decreasing slab length has a large effect on reducing transverse slab cracking due to reduced thermal curling stresses. The provision of a base that does not erode but is not extremely stiff is critical to reduce thermal curling stresses. It is likely that a permeable base has great potential in reducing pumping although it is not extremely stiff; however, field data are not yet conclusive on this topic. Increasing slab width or tying on a PCC shoulder would reduce the critical edge stress and thus result in much less fatigue damage.

7.2 Crack Deterioration -- JRCP only

There are two major factors: amount of longitudinal reinforcement, slab thickness, and joint spacing. Increased reinforcement provides improved capability to hold shrinkage cracks tight so that aggregate interlock is maintained. Increased slab thickness reduces deflection and provides more depth to maintain aggregate interlock. Shorter joint spacing results in less deterioration of transverse cracks, but the exact cause is unknown. It is likely due to decreased shrinkage stresses from shorter joint spacing. Increased slab width would also reduce critical deflections at the edge and thus increase slab crack deterioration life.

7.3 Joint Deterioration -- JRCP & JPCP

There are two major factors: joint spacing and incompressibles in the joint. Joint spacing is the major factor with shorter spacings (e.g., less than 30 ft.) showing far less joint deterioration than 40 to 100 ft.

spacing. Incompressibles also result in serious spalling and blowups in jointed rigid pavements. A sealant that will keep out incompressibles is very beneficial.

7.4 Pumping -- JRCP, JPCP & CRCP

There are three major factors: an erodible base and shoulder, free moisture within the pavement section and large deflection of the slab. A permeable base, that remains unclogged, may be the only way to prevent serious pumping. Improved joint load transfer or lane to shoulder load transfer will also reduce deflection.

7.5 Faulting of Transverse Joints -- JRCP & JPCP

The major factors are: reduce pumping of fines and load transfer. The reduction of pumping would reduce the amount of fines to lift the approach slab and thus reduce faulting. Maintaining high deflection load transfer across transverse joints is a very important factor in reducing faulting since this reduces differential deflection when a wheel load rolls over the joint.

7.6 Serviceability (Roughness) -- JRCP & JPCP

All of the above recommendations will improve the roughness condition of the pavement over its service life.

7.7 Punchouts -- CRCP only

There are three critical factors: slab thickness, amount of reinforcement, and loss of support beneath the CRCP slab. The reduction of pumping through improved drainage will be critical to preventing increased deflections. Increased slab thickness reduces deflection and increased reinforcement keeps transverse shrinkage cracks tighter.

8.0 SUMMARY: ADVANTAGES AND DISADVANTAGES OF MECHANISTIC DESIGN PROCEDURES

8.1 Advantages Of Mechanistic Approach Over The Empirical Approach

1. The capability to structurally analyze practically any rigid pavement cross section and jointing condition is the most significant advantage. This allows for new and unique designs to be developed and analyzed, whereas the empirical approach can only design rigid pavement designs similar to the field sections that were used at the AASHO Road Test with any confidence.

2. Direct consideration of the critical stress in the slab and the computation of fatigue damage in the slab, which can be correlated to field cracking. This provides for the design capability to limit the amount of slab cracking.

3. Resulting designs over a wide range of conditions will be much improved over those developed using empirical procedures because of the many different factors that can be directly considered using mechanistic procedures.

8.2 Disadvantages Of Mechanistic Approach Over The Empirical Approach

1. The mechanistic approach is more complicated and requires the use of a computer. The computations required to conduct a fatigue analysis are extensive.

2. Mechanistic procedures have only been developed for slab cracking to date. Additional procedures are needed for key distress type of faulting, joint spalling and pumping.

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MODULE 4-2

AASHTO METHOD FOR RIGID PAVEMENT DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module provides comprehensive presentation of the AASHTO Guide for the design of rigid pavements. This includes jointed plain concrete, jointed reinforced concrete and continuously reinforced concrete pavements.

Upon completion of this module, the participants will be able to accomplish the following using the AASHTO Guide:

1. Describe the AASHTO Road Test for rigid pavements and the development of the original empirical performance/design model for relating performance to design and traffic factors.

2. Identify the basic assumptions used in developing the design model and describe its advantages and disadvantages.

3. Describe the adjustments made to the original design model in the current version of the AASHTO design guide.

4. List the input factors required to use the AASHTO guide and describe how to determine each factor.

5. Design rigid pavement slab thickness and base/subbase using a set of design inputs.

6. Design rigid pavement joints for JPCP and JRCP using a set of design inputs.

7. Design rigid pavement reinforcement for JRCP and CRCP using a set of design inputs.

8. Determine the sensitivity of the major design factors for the AASHTO procedure.

9. List the major strengths, weaknesses, and limitations of the procedure and how some of these may be overcome or considered indirectly through other means.

2.0 INTRODUCTION

The original "AASHO Interim Guide for Design of Pavement Structures" (2) was developed in 1962 by the AASHO Design Committee through its subcommittee on Pavement Design Practices. The Guide was evaluated and partly revised in 1972 (3) and 1981 (4, 13). From 1984 to 1985 the Subcommittee on Pavement Design and a team of consultants revised the existing guide under NCHRP Project 20-7/24 and issued the new version entitled "AASHTO Guide for Design of Pavement Structures--1986" (5). Several major modifications were made in the rigid pavement design procedures in the 1986 version.

Computer programs are available for solving the basic equations and generating multiple design strategies so that the designer may select an optimum economical solution (11, 12).

3.0 DESIGN ASSUMPTIONS AND PROCEDURES

3.1 Development of the Design Model

The 1986 AASHTO rigid pavement performance/design model is an extension of the original pavement performance model developed from the results of the two-year AASHO Road Test conducted near Ottawa, Illinois, from 1958 to 1960. The several extensions to the model makes it more applicable to different climates, designs, materials and soils that exist across the U.S.

The original empirical regression model related the present serviceability index to slab thickness and axle load magnitude, type and repetition. This model was modified and extended by using Spangler's corner stress formula (6) in 1962 and by considering drainage, loss of support, effective k-value, variations of joint load transfer using the J factor and incorporation of design reliability in 1986. A complete description of the development of the structural design model is given in the AASHO Road Test report (7), the appendix of the 1981 Interim Guide (4), NCHRP Report No. 128 (13) and the 1986 AASHTO Guide (5).

The following major design variables (and values) were included at the AASHO Road Test for rigid pavements:

1. Slab thickness - 2.5 to 12.5 inches
2. Subbase type - untreated gravel/sand with plastic fines
3. Subbase thickness - 0 to 9 inches
4. Subgrade soil - silty clay (A-6) only
5. JRCP joint spacing - 40 ft.
6. JPCP joint spacing - 15 ft.
7. Joint load transfer - all joints doweled with dowel diameter related to slab thickness
8. JRCP reinforcement - smooth welded wire fabric, varies with slab thickness
9. Truck axle types - single and tandem
10. Single axle weights - 2 to 30 kips
11. Tandem axle weights - 24 to 48 kips
12. Climate - northern Illinois climate
13. No. axle load applications - 1,114,000
14. Maximum 18-kip ESAL - 10,000,000 (heaviest loop)

The Road Test data provided empirical relationships between PCC slab thickness, load magnitude, axle type, number of load applications, and serviceability loss of the pavement for Road Test conditions (i.e., specific environment and materials). The original empirical model derived from road test data is as follows:

$$\log_{10}(W) = \log_{10}(r) + G/B$$

where:

W = axle load applications, for load magnitude L1 and axle type L2, to a serviceability index of P2

$\log_{10} r = 5.85 + 7.35*\log(D + 1) + 4.62*\log(L1 + L2) + 3.28*\log(L2)$

$B = 1.0 + 3.63*(L1 + L2)^{5.20}$

$G = \log((P1 - P2)/(P1 - 1.5))$

D = PCC slab thickness, inches

$L1$ = load on a single or a tandem axle, kips

$L2$ = axle code, 1 for single axles and 2 for tandem axles

$P1$ = initial serviceability index

$P2$ = terminal serviceability index

This empirical model (Equation 1) was modified and extended in 1962 using the Spangler corner stress equation (6) to include material properties including PCC flexural strength (F), modulus of elasticity (E), and foundation support (k). The following basic assumptions were made in this extension:

1. There will be no variation in W for different load magnitudes if the level of the ratio of tensile stress/strength of the PCC slab is kept constant and such W will be accounted for by the AASHO Road Test Equation 1.

2. Any change in the ratio tensile stress/strength resulting from changes in the values of E, k, and F (modulus of rupture) will have the same effect on W as an equivalent change in slab thickness (calculated by Spangler's equation) will have on W.

Reliability concepts were introduced into the design process in 1986 to decrease the risk of premature structural deterioration below acceptable levels of serviceability. The reliability design factor (F_R) accounts for chance variations in both the traffic prediction and the pavement performance prediction for a given W_{18}. This factor provides a predetermined level of design reliability (R%) that pavement sections will structurally carry the traffic for which they were designed. However, it does not increase the reliability against such problems as "D" cracking or misalignment of dowel bars.

A drainage coefficient (C_d) based on the quality of drainage and the percent of the time the pavement structure is exposed to moisture levels approaching saturation was added to the design equation in 1986 to provide an approximate way to consider the effect of drainage. The C_d provides a relative basis of comparison from the condition at the AASHO Road Test.

The potential effect of subgrade swelling and frost heave are considered through the potential rate of loss in serviceability ($PSI_{SW,FH}$).

A Loss of Support factor (LS) was added to the design model in 1986 to account for the potential loss of support arising from subbase erosion and/or differential vertical soil movement by diminishing the overall effective k-value based on the size of the void that may develop beneath the slab. The resulting final 1986 AASHTO structural design model is given as follows:

$$\log_{10}(W_{18}) = Z_R{*}S_o + 7.35{*}\log_{10}(D+1) - 0.06 + \frac{\log_{10}(\text{PSI}/(4.5-1.5))}{(1+(1.624{*}10^7/((D+1)^{8.46})}$$

$$+ (4.22-0.32p_t){*}\log_{10}\left(\frac{S'_c{*}C_d[D^{0.75}-1.132]}{215.63{*}J[D^{0.75}-(18.42/((E_c/k)^{0.25})}\right)$$

where:

W_{18} =	$W_{18}/F_R = W_{18}/10^{(-Z_R S_o)}$ = Predicted number of 18-kip single axle load applications
F_R =	Reliability design factor
Z_R =	Standard normal deviate corresponding to selected level of reliability
S_o =	Overall standard deviation for rigid pavement
D =	Thickness of pavement slab, inches
P_i =	Initial serviceability index
P_t =	Terminal serviceability index
S'_c =	Modulus of rupture for PCC on specific project
J =	Load transfer coefficient used to adjust for load transfer characteristics of specific design
C_d =	Drainage coefficient
E_c =	Modulus of Elasticity for PCC, psi
k =	Modulus of subgrade reaction, pci

3.2 Specific Conditions of the AASHO Road Test

The general conditions under which the basic structural design equation was developed from the field performance results are as follows:

1. Construction control. Construction was of extremely high quality; therefore, variations in concrete, aggregates, moisture, density, subgrade soil properties, etc., were much lower than can be expected in most normal highway construction.

2. Length of test pavements. The length of the test section was 120 ft for the JPCP and 240 ft for the JRCP. The slab lengths are discussed under Item 6. Thus, variability in materials and foundation support that normally occurs along a several mile project did not exist at the AASHO Road Test.

3. Subbase. An untreated densely graded sand-gravel with significant fines served as the subbase. This material pumped extensively on many sections which was a major reason for the failure of these sections. The gradation of the subbase is as follows:

Sieve Size	% Passing
1 1/2 in	100
1	98
1/2	74
No. 4	49
No. 40	23
No. 200	9
PI (-40)	3.5

4. Subgrade. The subgrade was a fine grained A-6 soil with a CBR ranging from 2 to 4. A modulus of subgrade reaction of 45 pci was measured in the spring after the initial thaw.

5. Climate. The climate in northern Illinois receives about 30 inches of annual precipitation and retains 4 inches more annual precipitation than evaporation, thus yielding a positive Thornthwaite Index of 30. The average depth of frost penetration is about 30 inches, and the number of freeze-thaw cycles is 12 per year at the subbase level in the pavement. The AASHO pavements were only subjected to two years of climatic effects.

6. Joints and Reinforcement. All joints were contraction type joints with non-coated steel dowel bars. Reinforcement with smooth wire mesh was placed in JRCP slabs with 40 ft joint spacing. No reinforcement was used in the JPCP slabs with 15 ft joint spacing. Table 4-2.1 summarizes the reinforcing and joint design used.

7. Length of Test. The test was conducted over a two-year time period, too short for effective evaluation of corrosion of mesh or dowels and deterioration of concrete.

8. Number of Load Applications. The total number of load applications applied to each loop was 1,114,000. The maximum 18-kip ESAL were nearly 10 million on the heaviest loaded loop.

3.3 Accuracy of Structural Design Model

The empirical model of Equation 1 was derived from results from the Road Test data, and relates specifically to the conditions listed above. Within these conditions, the ability of Equation 1 to predict the exact number of load applications to any given level of serviceability index for a pavement section at the road test site was as shown in Figure 4-2.1 (7). The shaded band indicates the range in load applications that includes approximately 90 percent of all the performance data.

Referring to the top curve in Figure 4-2.1, for example, if the slab thicknesses were 8 inches, the resulting number of 30-kip single axle applications to a terminal serviceability index of 2.0 ranged between 400,000 to 1,910,000 for controlled AASHO Road Test conditions. If Equation 1 is used for conditions different than those for which it was developed, its range of accuracy or associated error of prediction will be greater. This may be particularly true for different climatic conditions. The modified expression, Equation 3, allows for changes in k, E, F, subdrainage and loss of support, but the accuracy of these adjustments is unknown.

Table 4-2.1. Reinforcement Used at AASHO Road Test.

Pavement Thickness (in.)	Maximum Size of Aggregate (in.)	Depth of Sawing (in.)	Joints[1] Transverse Dowels[2] Diam. × Length (in.)	Joints[1] Longitudinal, Deformed Tie Bars[3] Size × Length (no.) (in.)	Reinforcement in Test Pavements: Fabric Style[4]	Reinforcement in Test Pavements: Fabric Weight (lb/100 sq ft)	Reinforcement in Test Pavements: Depth in Pavement (in.)
2½	1½	¾	⅜ x 12	3 x 20	66–1010	21	1¼
3½	1½	1	½ x 12	3 x 20	66–88	30	1¾
5	2½	1¼	⅝ x 12	3 x 20	612–66	32	2
6½	2½	1½	⅞ x 18	4 x 24	612–44	44	2
8	2½	1¾	1 x 18	4 x 24	612–33	51	2
9½	2½	2	1¼ x 18	5 x 30	612–22	59	2
11	2½	2¼	1⅜ x 18	5 x 30	612–11	69	2
12½	2½	2½	1⅝ x 18	5 x 30	612–00	81	2

[1] All joints formed by sawing groove approximately ⅛-in. wide.
[2] All transverse joints doweled, on 12-in. centers, and spaced at 15 ft in plain sections and at 40 ft in reinforced sections.
[3] At 30-in. centers.
[4] Code:

```
6        12   -   4        4
|        |        |        |________ Gage of transverse wires
|        |        |__________________ Gage of longitudinal wires
|        |___________________________ Longitudinal spacing of transverse wires (inches)
|____________________________________ Transverse spacing of longitudinal wires (inches)
```

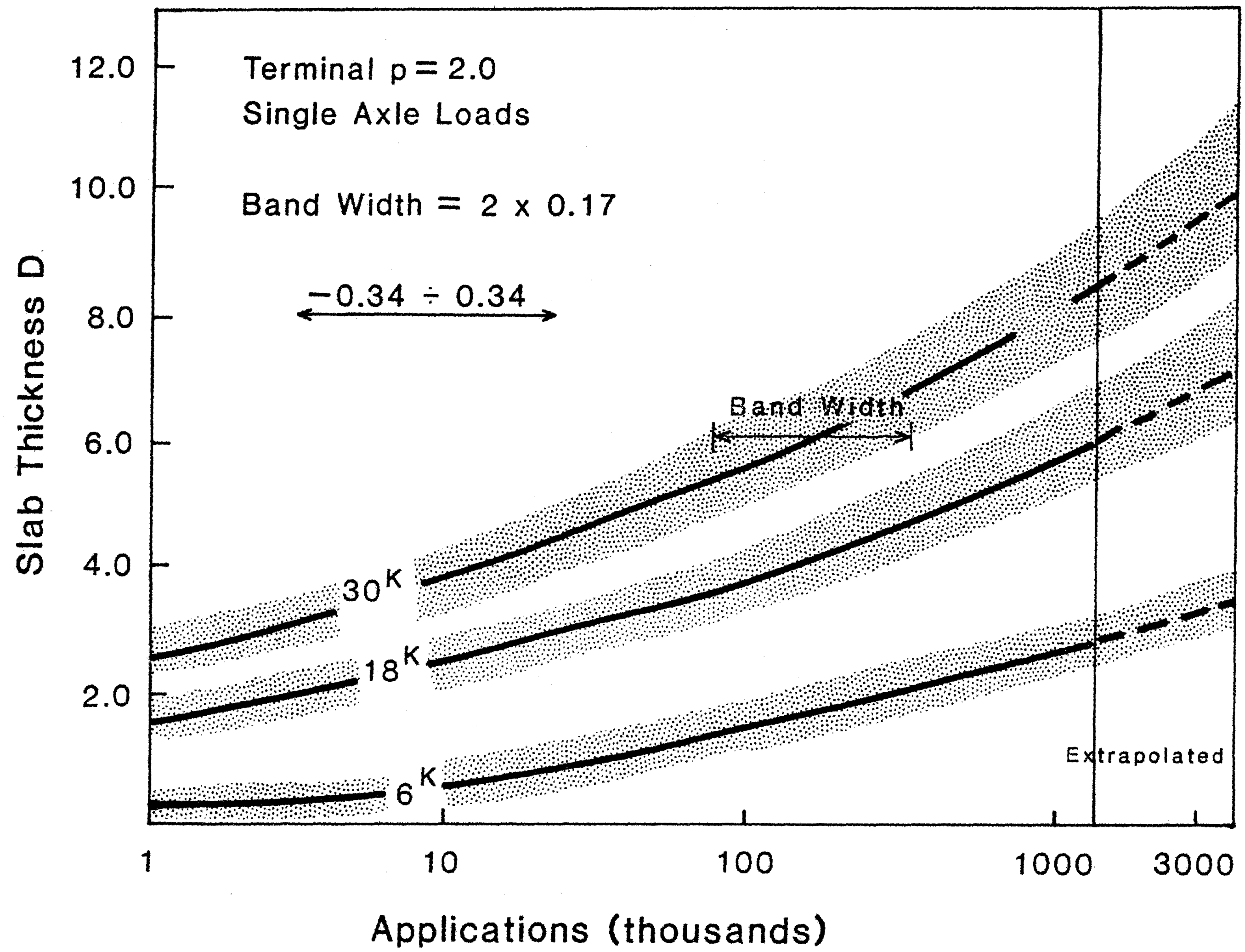


Figure 4-2.1. Illustration of Error of Predication of Basic AASHO Design Model.

An analytical evaluation of the original road test model was conducted by comparing predicted vs. actual ESALs using in-service pavement performance data from the NCHRP Project 1-19 database (8). The actual number of ESALs was compared to the predicted ESAL due to the measured loss in present serviceability index using AASHTO performance Equation (3). This was done for each section of pavement of JPCP and JRCP in the COPES database. The actual pavement thicknesses, material properties, serviceability at the time of the study, and traffic from the COPES database were input into the equation. The drainage indicator value for the equation was set at 1.0. The value of the J factor was assumed as 3.2 for joints with dowels and 4.1 with aggregate interlock (without dowels). The analysis was run at the 50 percent level of reliability.

The pavement sections in the COPES databank were divided into four broad climatic zones (1) and the results compared by zone. Following is the classification and data locations for the four climatic zones:

Climatic Zone	Annual Rainfall (cms)	Freezing Index (degree-days)	JPCP Location	JRCP Location
Wet-Nonfreeze	equal or greater than 70	less than 100	GA, LA	IL, LA
Wet-Freeze	equal or greater than 70	Equal or greater than 100	IL	IL, MN
Dry-Nonfreeze	less than 70	less than 100	CA	- -
Dry-Freeze	less than 70	equal or greater than 100	UT	MN, NB

The plots of predicted vs. actual ESALs for all JPCP and JRCP for each climatic region are given in Figure 4-2.2 and Figure 4-2.3, respectively. A summary of the results of the predicted vs. actual ESALs for JPCP and JRCP is given in Table 4-2.2. The significance and comparison of the results are discussed in the following sections.

3.3.1 JPCP

The results are highly dependent on climate. Almost all the sections in the dry-nonfreeze region performed better than the original AASHO model predicted (52 out of 53 sections, or 98 percent). The average predicted ESALs to existing present serviceability index in this region is 6.4 million (or 69.5 percent) less than the actual ESALs. Those sections in the dry-freeze and wet-nonfreeze climates did perform generally as predicted with 60 and 71 percent of sections acceptable, respectively.

The JPCP sections in the wet-freeze climatic region (same as AASHO Road Test) performed worse than the AASHO model predicted with only 9 out of 36 sections (or 25 percent) acceptable. The average predicted ESALs in wet-freeze region is 7.36 million (or 92 percent) greater than the actual ESALs.

3.3.2 JRCP

The results in Table 4-2.2 show that the JRCP sections did not perform as well as the original AASHO model predicted in any climatic zone. Only 41

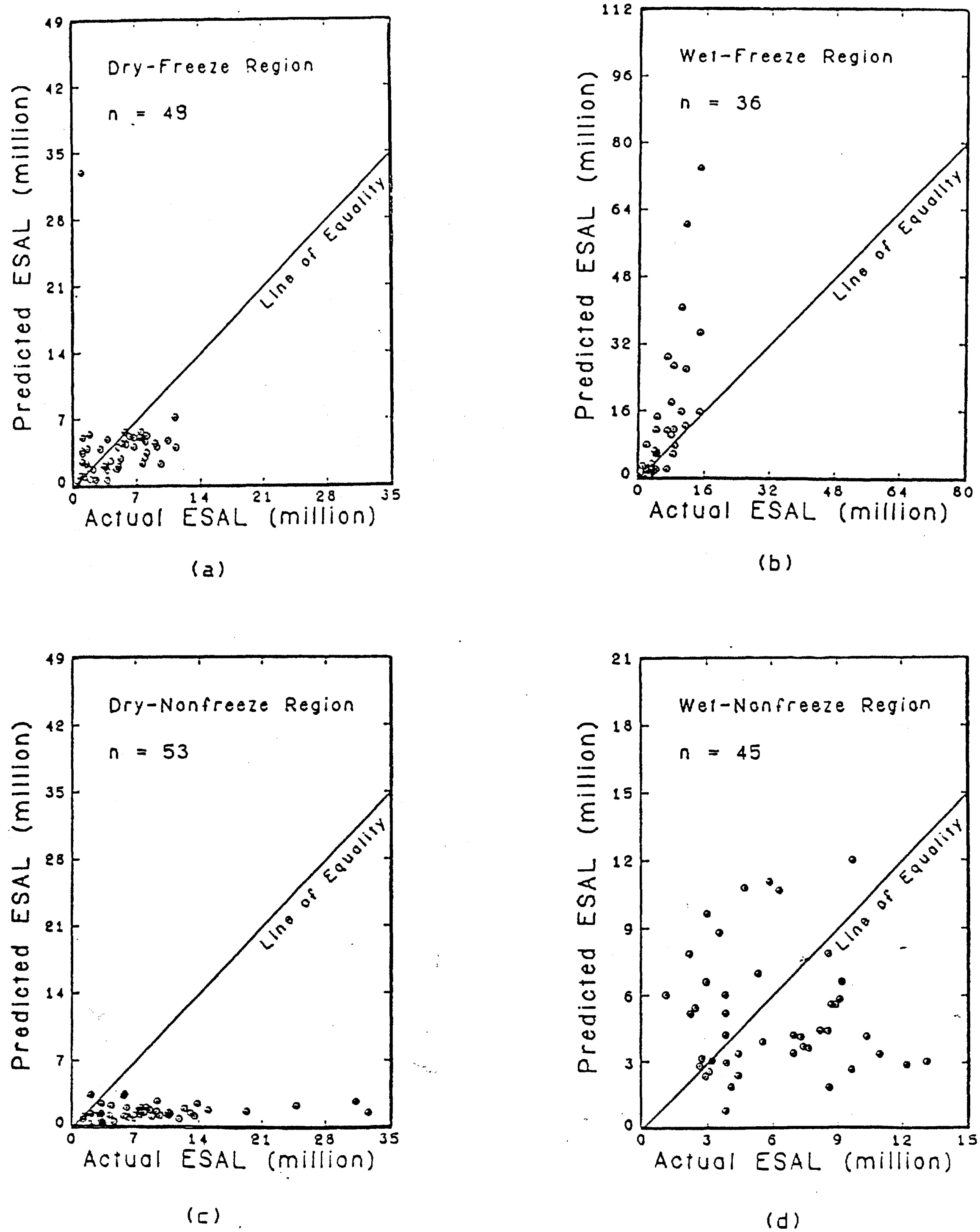


Figure 4-2.2. Predicted ESAL vs. Actual ESAL for JPCP Using Original AASHO Road Test PSI Prediction Model (15).

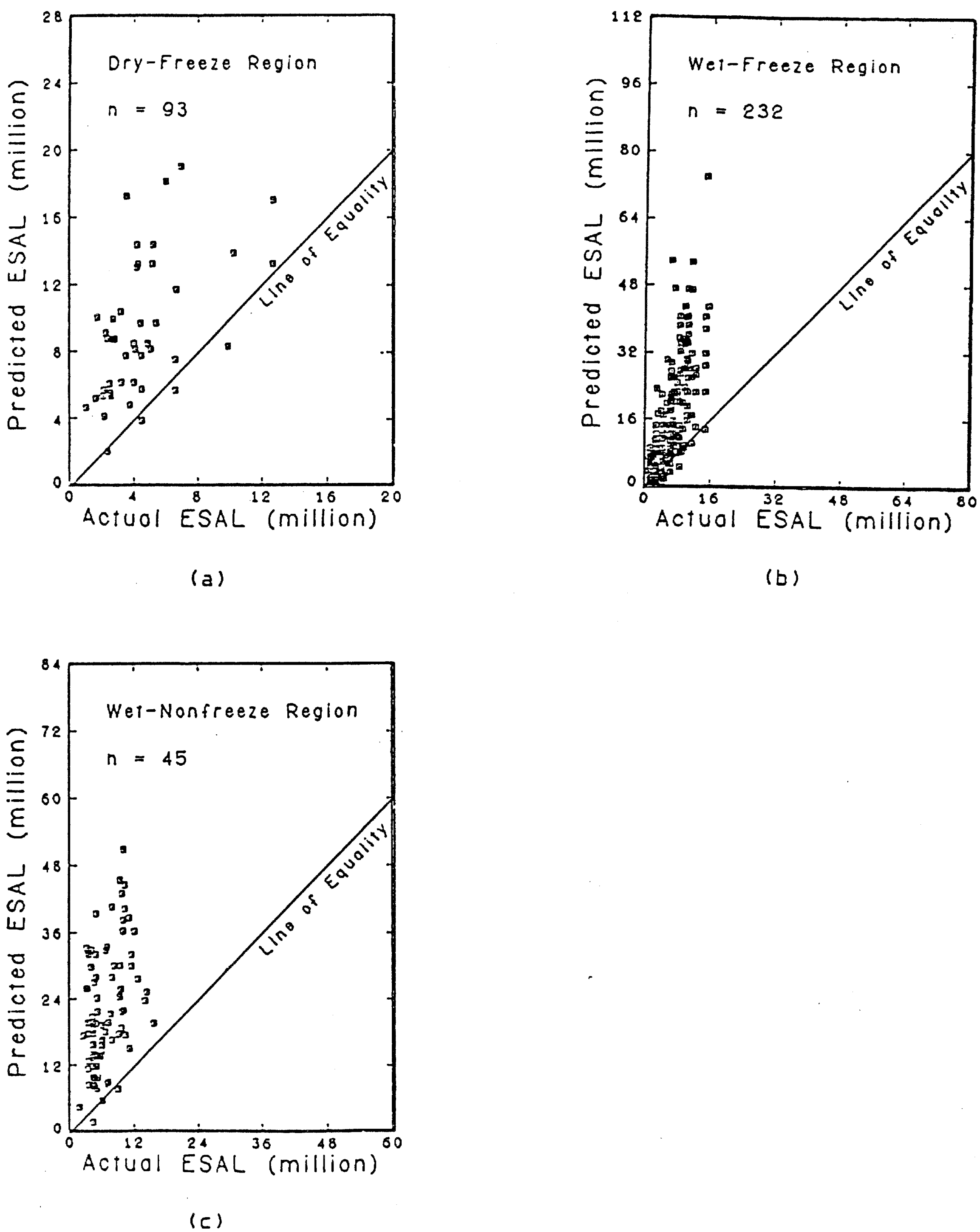


Figure 4-2.3. Predicted ESAL vs. Actual ESAL for JRCP Using Original AASHO Road Test PSI Prediction Model (15).

Table 4-2.2. Summary of Results for Original AASHO Road Test PSI Prediction Model (15).

Climatic Region (1)	# of Cases (2)	# of Section Acceptable* (3)	Percent Acceptable (4)=(3)/(2)	Mean Difference** (5)	Mean Percent Difference*** (6)
JPCP:					
Wet-Nonfreeze	45	27	60%	-0.911	17.1%
Wet-Freeze	36	9	25%	7.355	92.0%
Dry-Nonfreeze	53	52	98%	-6.405	-69.5%
Dry-Freeze	49	35	71%	-0.763	86.9%
Overall	183	123	67%	-0.836	25.4%
JRCP:					
Wet-Nonfreeze	93	3	3%	15.460	282.0%
Wet-Freeze	232	34	15%	9.886	168.8%
Dry-Nonfreeze	-****	-	-	-	-
Dry-Freeze	49	4	8%	4.584	143.0%
Overall	374	41	11%	10.577	193.6%

* "Acceptable" means the actual number of 18-kip ESALs is equal or greater than the predicted ESALs, i.e., the pavement section performed as good as or better than the AASHO model predicted, otherwise is "unacceptable".

** Difference = Predicted ESAL - Actual ESAL, in millions

*** Percent Difference = ((Predicted ESAL - Actual ESAL)/Actual ESAL) x 100%

**** No JRCP sections available in dry-nonfreeze region in COPES.

out of 374 sections (or 11 percent) performed better than predicted. The average predicted ESALs for the all JRCP sections is 10.58 million (or 193.6 percent) greater than the actual ESALs.

Climatic factors affect JRCP more severely than JPCP. Many of these JRCP sections are deteriorated from causes other than traffic loading, such as a build up of compressive stress with long joint spacing resulting in corrosion of dowels and mesh, "D" cracking or reactive aggregate, infiltration of incompressibles into joints causing blowups and joint spalling, etc.

The types of failures indicated that other factors besides the thickness of the pavement must be addressed. The designer must give attention to proper drainage, base and joint design. Increasing the thickness through the use of drainage coefficients, J factors and higher reliability factors may not provide for increased pavement. life. The road test models assume that the dowels and reinforcement will not corrode.

It should be noted that the ESAL estimated for the COPES data base were based on W-4 Tables. Actual ESAL may be up to 20% greater than these estimates. This result would not change these conclusions, however, concerning the AASHO model.

4.0 INPUT VARIABLES

The following design inputs are required to use the AASHTO Guide for Design of Pavement Structures--1986. The inputs are categorized according to general type:

4.1 General Design Variables

4.1.1 Performance Period

The period of time that an initial pavement structure will last before it needs rehabilitation and the performance time between rehabilitation operations. The designer must select minimum and maximum bounds.

4.1.2 Analysis Period

The period of time for which the analysis is to be conducted (i.e., the length of time that any design strategy must cover). The following general guidelines are recommended:

Highway Conditions	Analysis Period (years)
High volume urban	30 - 50
High volume rural	20 - 50
Low volume paved	15 - 25

4.1.3 Traffic

The design procedures are based upon cumulative expected 18-kip equivalent single axle loads (ESAL) during the analysis period (W_{18}). The computation of cumulative ESAL requires the following traffic data:

1. Average daily traffic over analysis period.

2. Average daily truck traffic over analysis period.

3. Truck distribution between lanes for multi-lane facilities.

4. Truck axle weight distribution over analysis period (including consideration of the expected increase in the equivalency per truck over time).

4.1.4 Reliability

The procedures for incorporating design reliability into the pavement design process is presented in Module 2-7. The following guidelines are recommended for design reliability:

Functional Classification	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

A standard deviation (S_O) must be selected that is representative of local conditions. The following values are recommended for general use, but should be evaluated for local usage. These data were developed from AASHTO Road Test conditions.

Design Condition	Standard Deviation
Variation in pavement performance prediction without traffic error	0.25 rigid 0.35 flexible
Total variation in pavement performance prediction and in traffic estimation	0.35 rigid 0.45 flexible

4.1.5 Roadbed Swelling

If a swelling clay exists, and the pavement design does not take steps to prevent swelling directly, its effect on serviceability loss can be considered by estimating the loss of serviceability over the analysis period.

4.1.6 Frost Heave

If a frost heave potential exists, and the pavement design does not take steps to prevent frost heave directly, then its effect on serviceability can be considered by estimating the loss over the analysis period.

It is recommended that all design address roadbed swelling and frost heave directly through material and construction techniques (see Module 4-3). Figure 4-2.4 shows a conceptual example of serviceability loss due to frost or swelling soil.

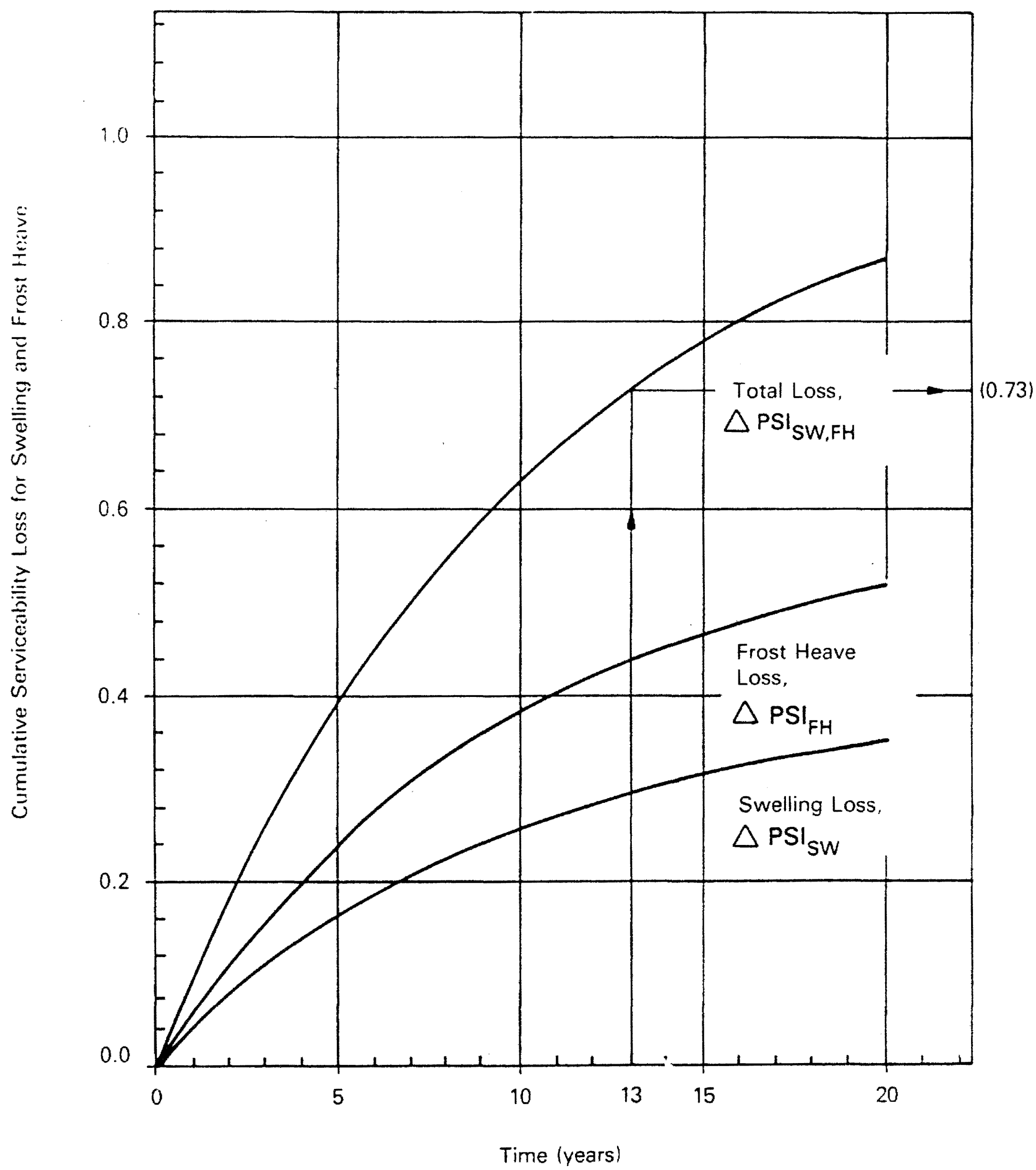


Figure 4-2.4. A <u>Conceptual</u> Example of the Environmental Serviceability Loss Versus Time Graph that may be Developed for a Specific Location (<u>5</u>).

4.2 Performance Criteria

The following values are recommended for the initial and terminal serviceability.

Pavement Type	Initial Serviceability Level
Rigid	4.5
Flexible	4.2

Highway Type	Terminal Serviceability Level
Major highways	2.5 or higher
Highways with lesser traffic	2.0

4.3 Material Properties for Structural Design

4.3.1 Effective Modulus of Subgrade Reaction

The determination of the effective k-value for design is a complex process requiring seven steps:

1. Identify the combinations (or levels) of factors that are to be considered and enter them in the heading of Table 4-2.3 (example filled out).

    a. Subbase types have different strengths or modulus values.
    b. Subbase thicknesses (inches).
    c. Loss of support, LS.
    d. Depth to rigid foundation (feet).
    e. For each combination of these factors that is to be considered for design, it is necessary to prepare a separate table and develop a corresponding effective modulus of subgrade reaction.

2. Identify the seasonal roadbed soil resilient modulus values and enter them in Column 2.

3. Assign subbase elastic (resilient) modulus (E_{SB}) values for the season. Enter these in Column 3 of Table 4-2.3 and should correspond to those for the seasons used to develop the roadbed soil resilient modulus values.

4. Estimate the composite modulus of subgrade reaction for each season, assuming a semi-infinite subgrade depth (i.e., depth to bedrock greater than 10 ft.) and enter in Column 4. This is accomplished with the aid of Figure 4-2.5.

5. Develop a k-value which includes the effect of a rigid foundation near the surface. This step should be disregarded if the depth to a rigid foundation is greater than 10 ft. Figure 4-2.6 provides the chart that may be used to estimate this modified k-value for each season. The values for each modified k-value should subsequently be recorded in Column 5 of Table 4-2.3.

Table 4-2.3. Table to Calculate Effective Modulus of Subgrade Reaction (5).

SUBBASE: TYPE GRANULAR
THICKNESS (inches) 6
LOSS OF SUPPORT. LS 1.0
DEPTH TO RIGID FOUNDATION (feet) 5
PROJECTED SLAB THICKNESS (inches) 9

(1) MONTH	(2) ROADBED MODULUS. M_R (PCI)	(3) SUBBASE MODULUS. E_{SB} (PCI)	(4) COMPOSITE K-VALUE (PCI)	(5) K-VALUE (PCI) ON RIGID FOUNDATION	(6) RELATIVE DAMAGE M_r
Jan.	20,000	50,000	1100	1350	0.35
Feb.	20,000	50,000	1100	1350	0.35
Mar.	2,500	15,000	160	230	0.86
April	4,000	15,000	230	300	0.78
May	4,000	15,000	230	300	0.78
June	7,000	20,000	410	540	0.60
July	7,000	20,000	410	540	0.60
Aug.	7,000	20,000	410	540	0.60
Sep.	7,000	20,000	410	540	0.60
Oct.	7,000	20,000	410	540	0.60
Nov.	4,000	15,000	230	300	0.78
Dec.	20,000	50,000	1100	1350	0.35

AVERAGE: $\bar{u}_r = \frac{\sum u_r}{n} = \frac{7.25}{12} = 0.60$ SUMMATION: $\sum u_r =$ 7.25

EFFECTIVE MODULUS OF SUBGRADE REACTION: K (PCI) = 540

CORRECTED FOR LOSS OF SUPPORT: K (PCI) = 170

Example:

D_{SB} = 6 inches

E_{SB} = 20,000 psi

M_R = 7,000 psi

Solution: k_∞ = 400 pci

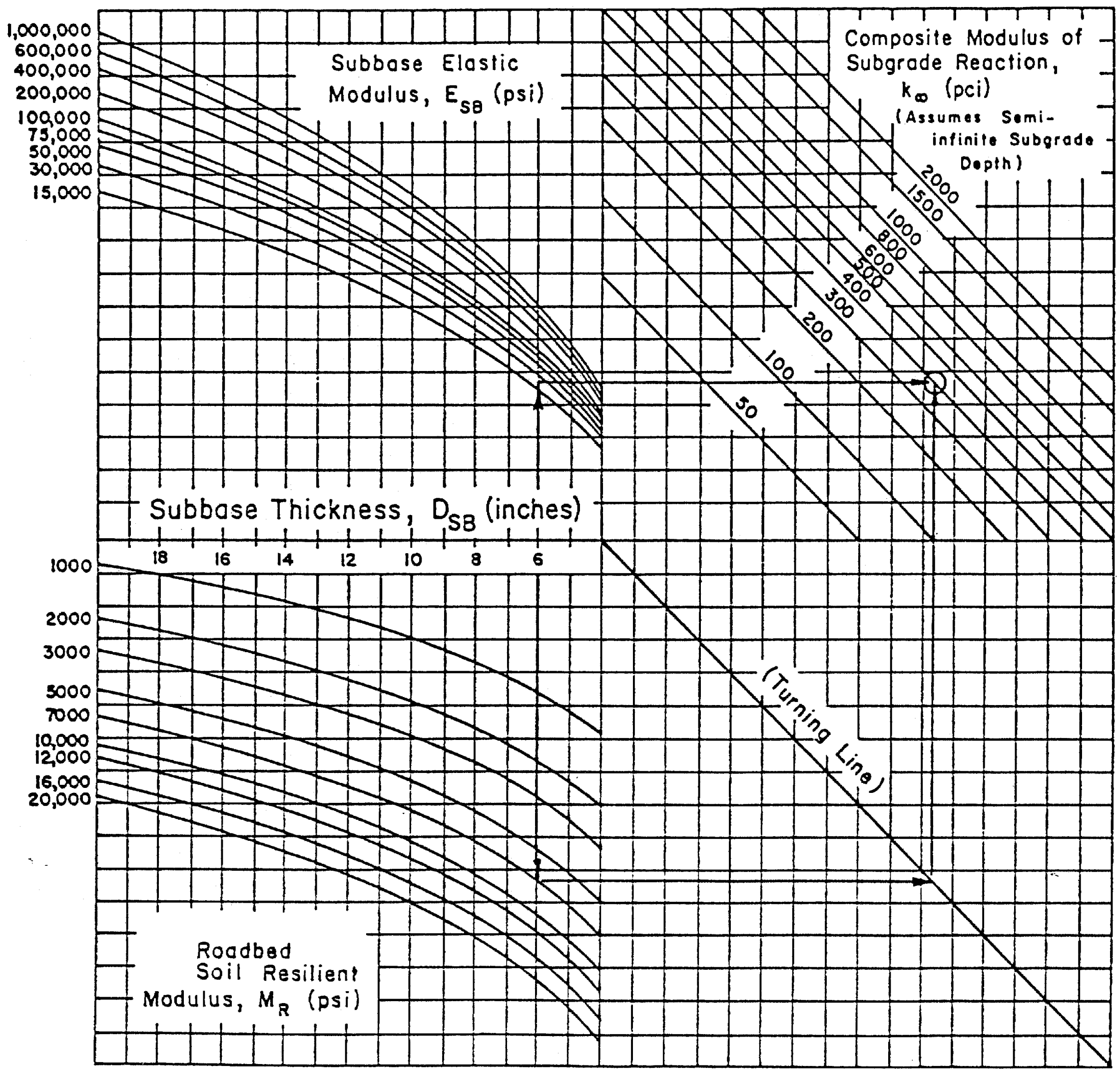


Figure 4-2.5. Chart for Estimating Composite Modulus of Subgrade Reaction, k_∞, Assuming a Semi-Infinite Subgrade Depth. (For Practical Purposes, a Semi-Infinite Depth is Considered to be Greater Than 10 Feet Below the Surface of the Subgrade.) (5).

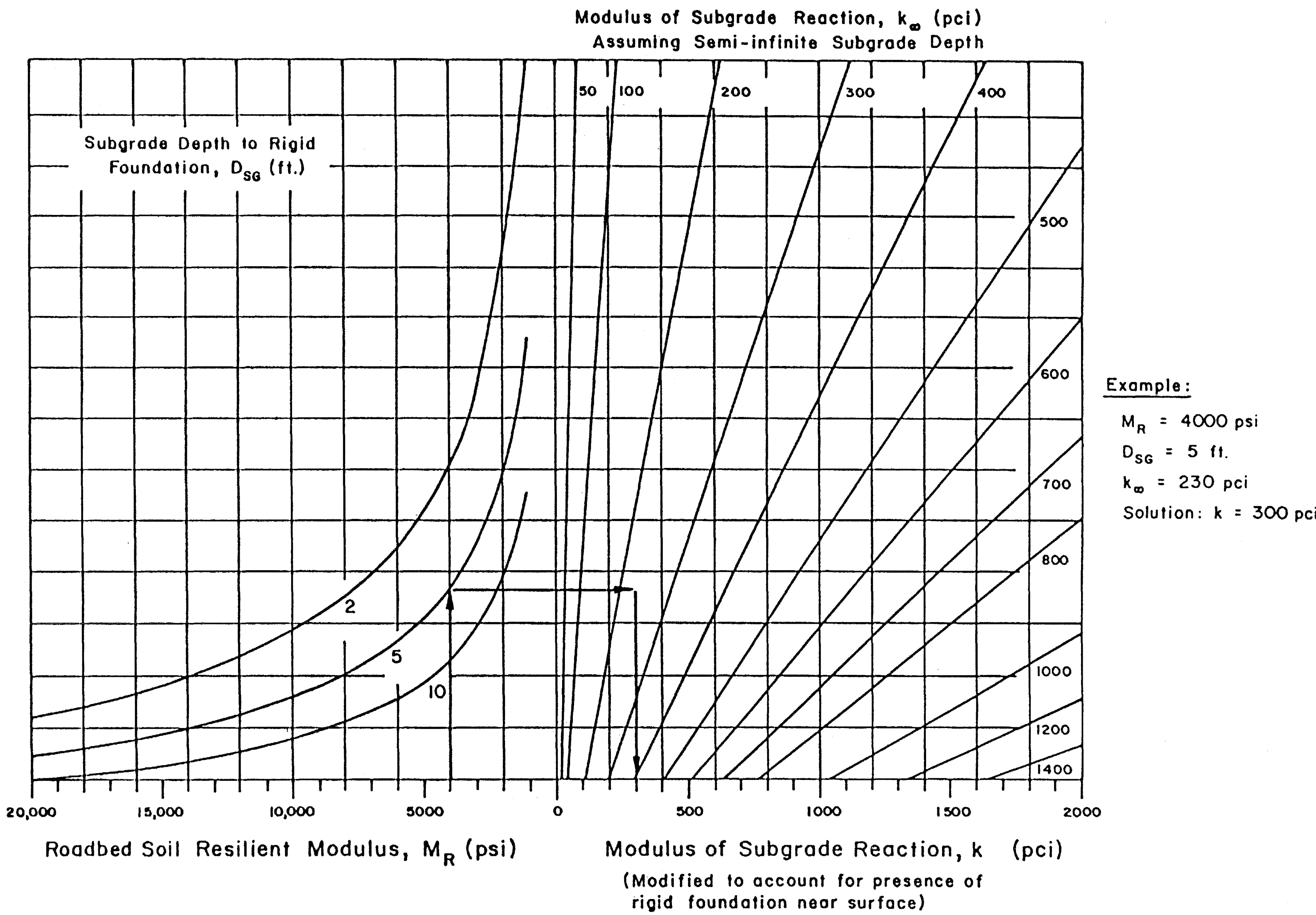


Figure 4-2.6. Chart to Modify Modulus of Subgrade Reaction to Consider Effects of Rigid Foundation Near Surface (Within 10 Feet) (5).

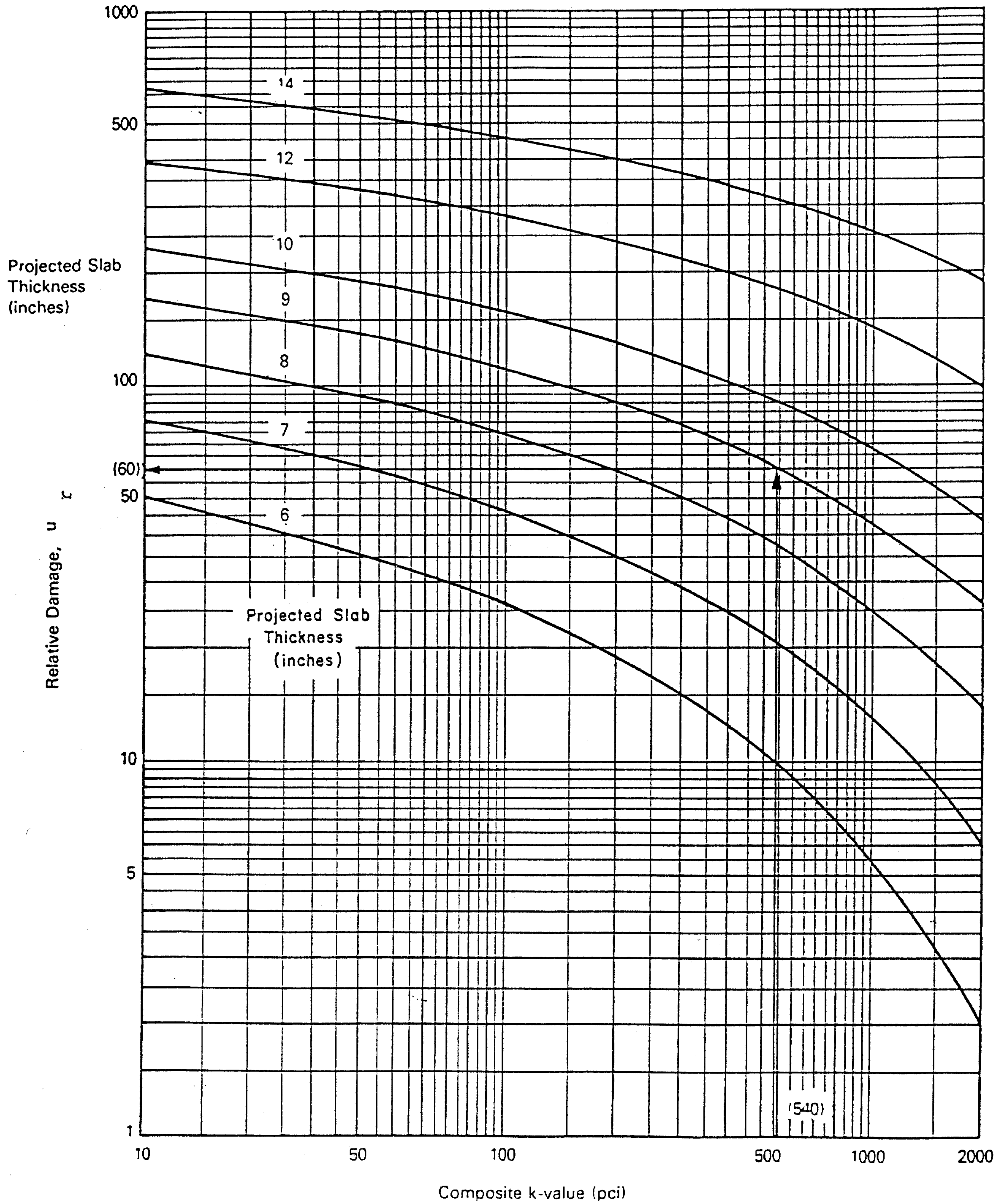


Figure 4-2.7. Chart for Estimating Relative Damage to Rigid Pavements Based on Slab Thickness and Underlying Support (5).

6. Estimate the thickness of the slab that will be required, and then use Figure 4-2.7 to determine the relative damage, u_r in each season and enter them in Column 6 of Table 4-2.3.

7. Add all the u_r values (Column 6) and divide the total by the number of seasonal increments (12 or 24) to determine the average relative damage, u_r. The effective modulus of subgrade reaction is the value corresponding to the average relative damage (and projected slab thickness) in Figure 4-2.7.

8. Adjust the effective modulus of subgrade reaction to account for the potential loss of support arising from subbase erosion. Figure 4-2.8 provides the chart for correcting the effective modulus of subgrade reaction based on the loss of support factor, LS. Space is provided in Table 4-2.3 to record this final design k-value.

4.3.2 Pavement Layer Materials Characterization

The elastic modulus of the PCC can be estimated using the following relationship:

$$E_c = 57{,}000\,(f'_c)^{0.5}$$

where:

E_c = PCC elastic modulus, psi
f'_c = PCC compressive strength as determined using AASHTO T22, T140 or ASTM C39, psi

The elastic modulus of other materials can be determined using either correlations with other material tests or appropriate AASHTO and ASTM elastic modulus standard tests.

4.3.3 PCC Modulus of Rupture

The mean value should be determined after 28 days using third-point loading.

4.4 Pavement Structural Characteristics

4.4.1 Drainage

The expected level of drainage for a rigid pavement is considered through use of a drainage coefficient, C_d. Recommended values for C_d are provided in Table 4-2.4. The quality of drainage rating is based upon the following guidelines:

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	water will not drain

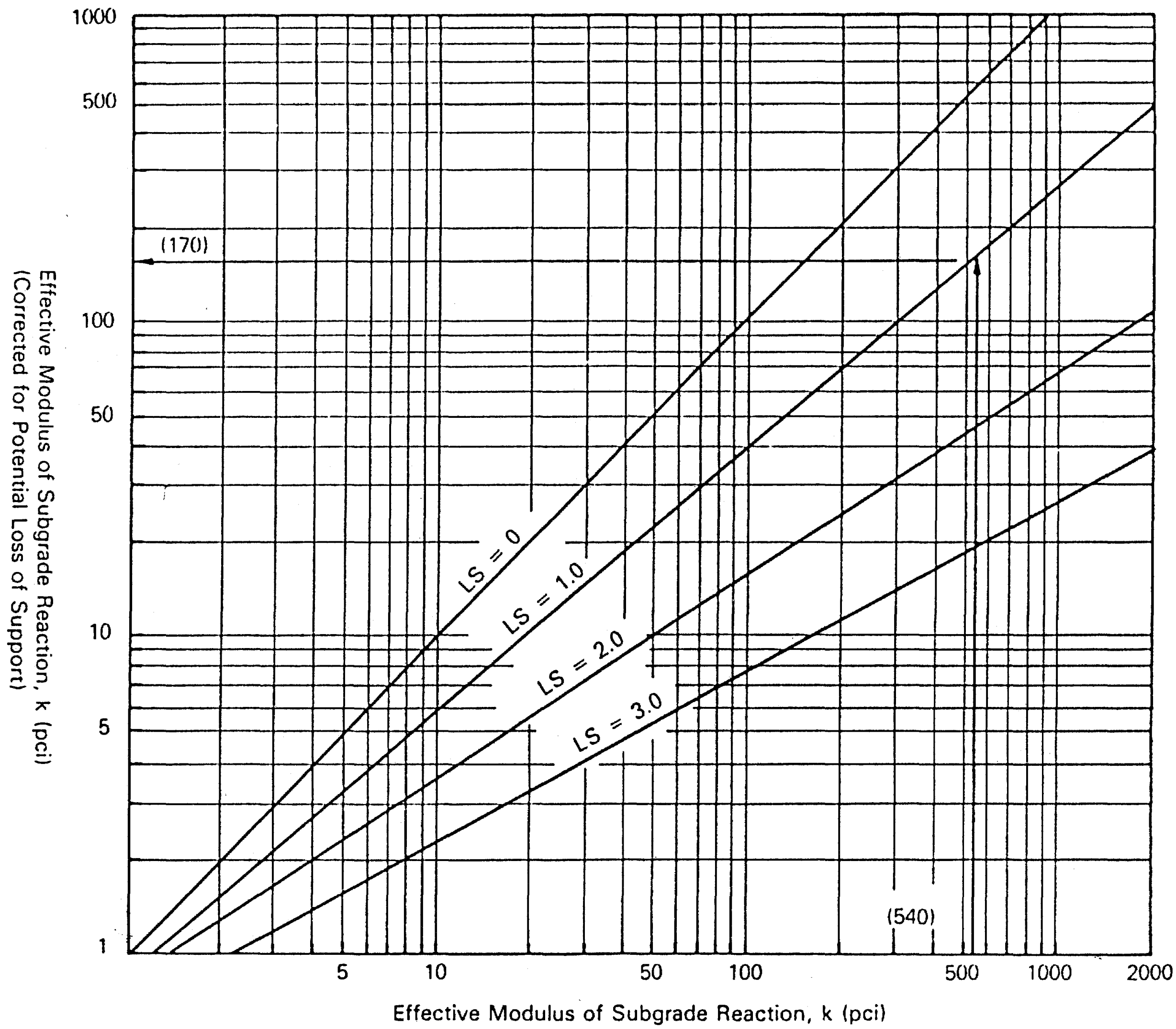


Figure 4-2.8. Correction of Effective Modulus of Subgrade Reaction for Potential Loss of Subbase Support (5).

Table 4-2.4. Recommended Values of Drainage Coefficient, C_d, for Rigid Pavement Design (5).

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.25 - 1.20	1.20 - 1.15	1.15 - 1.10	1.10
Good	1.20 - 1.15	1.15 - 1.10	1.10 - 1.00	1.00
Fair	1.15 - 1.10	1.10 - 1.00	1.00 - 0.90	0.90
Poor	1.10 - 1.00	1.00 - 0.90	0.90 - 0.80	0.80
Very Poor	1.00 - 0.90	0.90 - 0.80	0.80 - 0.70	0.70

The value of C_d for the AASHO Road Test is likely to be approximately 0.70 since the base and subgrade were saturated much of the time and neither material had significant drainability. The pumping was extremely high at the Road Test.

4.4.2 Load Transfer

The load transfer coefficient, J, is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to transfer load across joints and cracks. Table 4-2.5 provides recommendations for ranges of load transfer coefficients for different conditions. The AASHO Road Test conditions represent J = 3.2 as all joints were doweled and no tied shoulders.

4.4.3 Tied Shoulders or Widened Outside Lanes

These both reduce slab deflections and stresses. To take this into account, lower J-values may be used for the design of both jointed and continuous pavements. For CRCP with tied concrete shoulders, the range of J is between 2.3 and 2.9, with a recommended value of 2.6. For jointed concrete pavements with dowels and tied shoulders, the value of J should be between 2.5 and 3.1 based on the agency's experience. The lower J-value for tied shoulders assumes traffic is not permitted to run on the shoulder.

4.4.4 Loss of Support

The LS factor accounts for the potential loss of support arising from subbase erosion and/or differential vertical soil movements. The effective k-value is reduced to consider the loss of support. Table 4-2.6 provides some suggested ranges of LS. If different types of bases or subbase are to be considered for design, then the corresponding values of LS should be determined for each type.

4.5 Reinforcement Variables -- JRCP

4.5.1 Slab Length

The spacing of transverse contraction joints in feet. No specific guidelines are given for JRCP.

4.5.2 Working Stress

The allowable working stress, f_s, of the steel reinforcement is recommended to be equal to 75 percent of the steel yield strength.

4.5.3 Friction Factor

F represents the frictional resistance that exists between the bottom of the slab and the top of the underlying subbase or subgrade layer and is basically equivalent to a coefficient of friction. Recommended values for natural subgrade and a variety of subbase materials are presented in Table 4-2.7.

Table 4-2.5. Recommended Load Transfer Coefficient for Various Pavement Types and Design Conditions (5).

Shoulder	Asphalt		Tied P.C.C.	
Load Transfer Devices	Yes	No	Yes	No
Pavement Type				
1. Plain Jointed and Jointed Reinforced	3.2	3.8 - 4.4	2.5 - 3.1	3.6 - 4.2
2. CRCP	2.9 - 3.2	N/A	2.3 - 2.9	N/A

Table 4-2.6. Typical Ranges of Loss of Support (LS) Factors for Various Types of Materials (5).

Type of Material	Loss of Support (LS)
Cement Treated Granular Base (E = 1,000,000 to 2,000,000 psi)	0.0 to 1.0
Cement Aggregate Mixtures (E = 500,000 to 1,000,000 psi)	0.0 to 1.0
Asphalt Treated Base (E = 350,000 to 1,000,000 psi)	0.0 to 1.0
Bituminous Stabilized Mixtures (E = 40,000 to 300,000 psi)	0.0 to 1.0
Lime Stabilized (E = 20,000 to 70,000 psi)	1.0 to 3.0
Unbound Granular Materials (E = 15,000 to 45,000 psi)	1.0 to 3.0
Fine Grained or Natural Subgrade Materials (E = 3,000 to 40,000 psi)	2.0 to 3.0

Note: E in this table refers to the general symbol for elastic or resilient modulus of the material.

Table 4-2.7. Recommended Friction Factors (7).

Type of Material Beneath Slab	Friction Factor (F)
Surface treatment	2.2
Lime stabilization	1.8
Asphalt stabilization	1.8
Cement stabilization	1.8
River gravel	1.5
Crushed stone	1.5
Sandstone	1.2
Natural subgrade	0.9

4.6 Reinforcement Variables -- CRCP

4.6.1 Concrete Tensile Strength

This is defined as the indirect tensile strength covered under AASHTO T198 and ASTM C496. The strength at 28 days should be used. The indirect tensile strength is normally 86 percent of the concrete modulus of rupture.

4.6.2 Concrete Shrinkage

This is the drying shrinkage that occurs from loss of water. The value of shrinkage at 28 days is used for the design shrinkage value. Table 4-2.8 may be used as a guide in selecting a value corresponding to the indirect tensile strength.

4.6.3 Concrete Thermal Coefficient

Recommended values of PCC thermal coefficient as a function of aggregate type are presented in Table 4-2.9.

4.6.4 Bar Diameter

The design nomographs provide for deformed bars from No. 4 to 7 for longitudinal reinforcement in CRCP.

4.6.5 Steel Thermal Coefficient

A value of 5.0×10^{-6} in./in./degree F is recommended.

4.6.6 Design Temperature Drop

This is the difference between the average concrete curing temperature and a design minimum temperature. The design temperature drop computed as follows:

$$\Delta T_D = T_H + T_L$$

where:

ΔT_D = design temperature drop, degrees F

T_H = average daily high temperature during the month the pavement is constructed, degrees F

T_L = average daily low temperature during the coldest month of the year, degrees F

4.6.7 Friction Factor

The friction factor is identical to that for jointed concrete pavement.

5.0 SLAB THICKNESS DESIGN PROCEDURE

5.1 Initial Performance Period

The final rigid pavement design model was computerized and also nomographed as shown in Figure 4-2.9. Either may be used for determining slab thickness for each alternative design section. The following inputs are required to determine slab thickness using the nomograph:

Table 4-2.8. Approximate Relationship Between Shrinkage and Indirect Tensile Strength of Portland Cement Concrete (5).

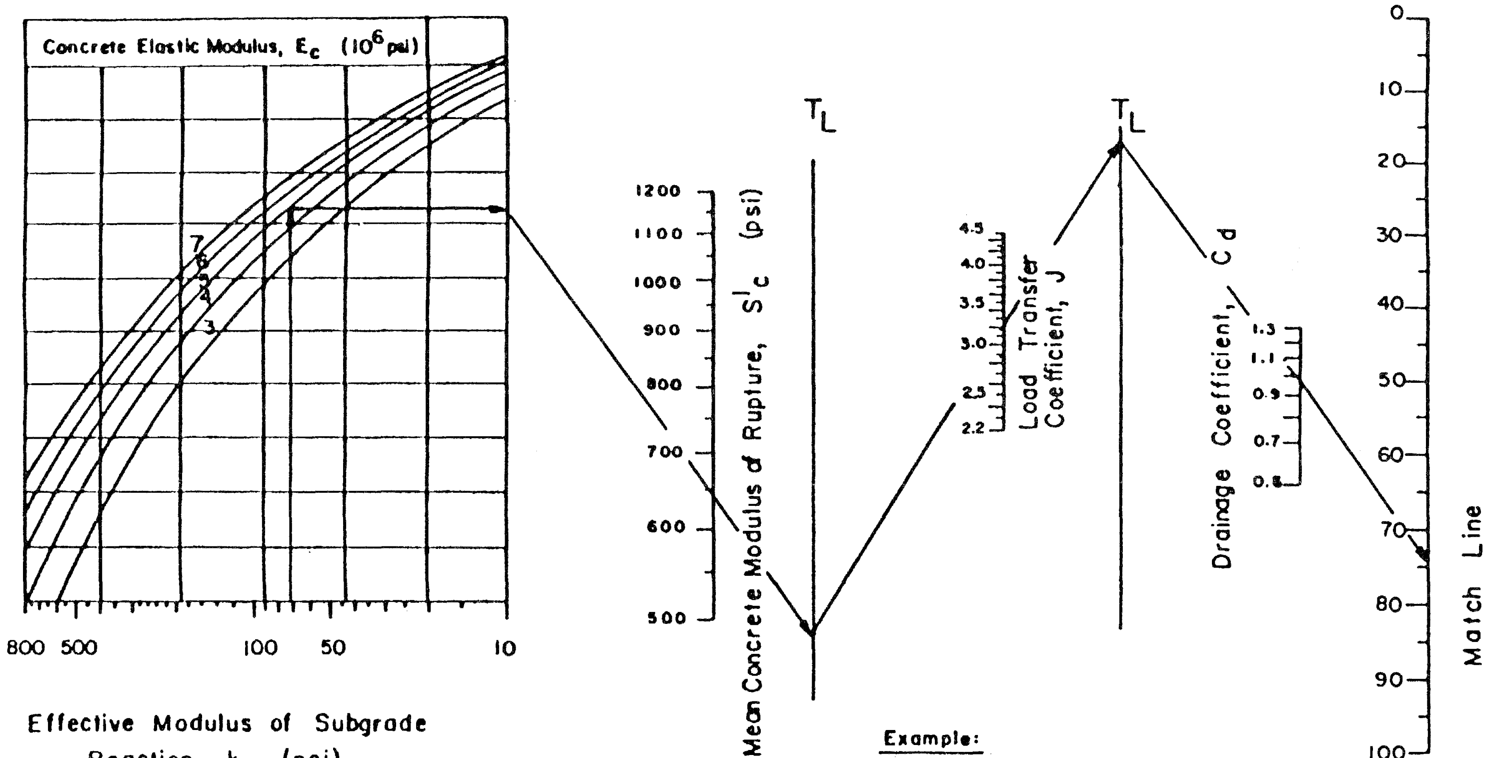
Indirect Tensile Strength (psi)	Shrinkage (in./in.)
300 (or less)	0.0008
400	0.0006
500	0.00045
600	0.0003
700 (or greater)	0.0002

Table 4-2.9. Recommended Value of the Thermal Coefficient of PCC as a Function of Aggregate Types (5).

Type of Coarse Aggregate	Concrete Thermal Coefficient(10^{-6}/°F)
Quartz	6.6
Sandstone	6.5
Gravel	6.0
Granite	5.3
Basalt	4.8
Limestone	3.8

NOMOGRAPH SOLVES:

$$\log_{10}W_{18} = Z_R{*}S_o + 7.35{*}\log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\dfrac{\Delta PSI}{4.5-1.5}\right]}{1+\dfrac{1.624{*}10^{7}}{(D+1)^{8.46}}} + (4.22-0.32p_t){*}\log_{10}\left[\frac{S'_c{*}C_d\left[D^{0.75}-1.132\right]}{215.63{*}J\left[D^{0.75}-\dfrac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$



Example:

k = 72 pci
E_c = 5×10^6 psi
S'_c = 650 psi
J = 3.2
C_d = 1.0

S_o = 0.29
R = 95 % (Z_R = -1.645)
ΔPSI = 4.2 - 2.5 = 1.7
W_{18} = 5.1×10^6 (18 kip ESAL)
Solution: D=10.0 inches (nearest half-inch, from segment 2)

Figure 4-2.9. Design Chart for Rigid Pavement Based on Using Mean Values for Each Input Variable (Segment 1) (5).

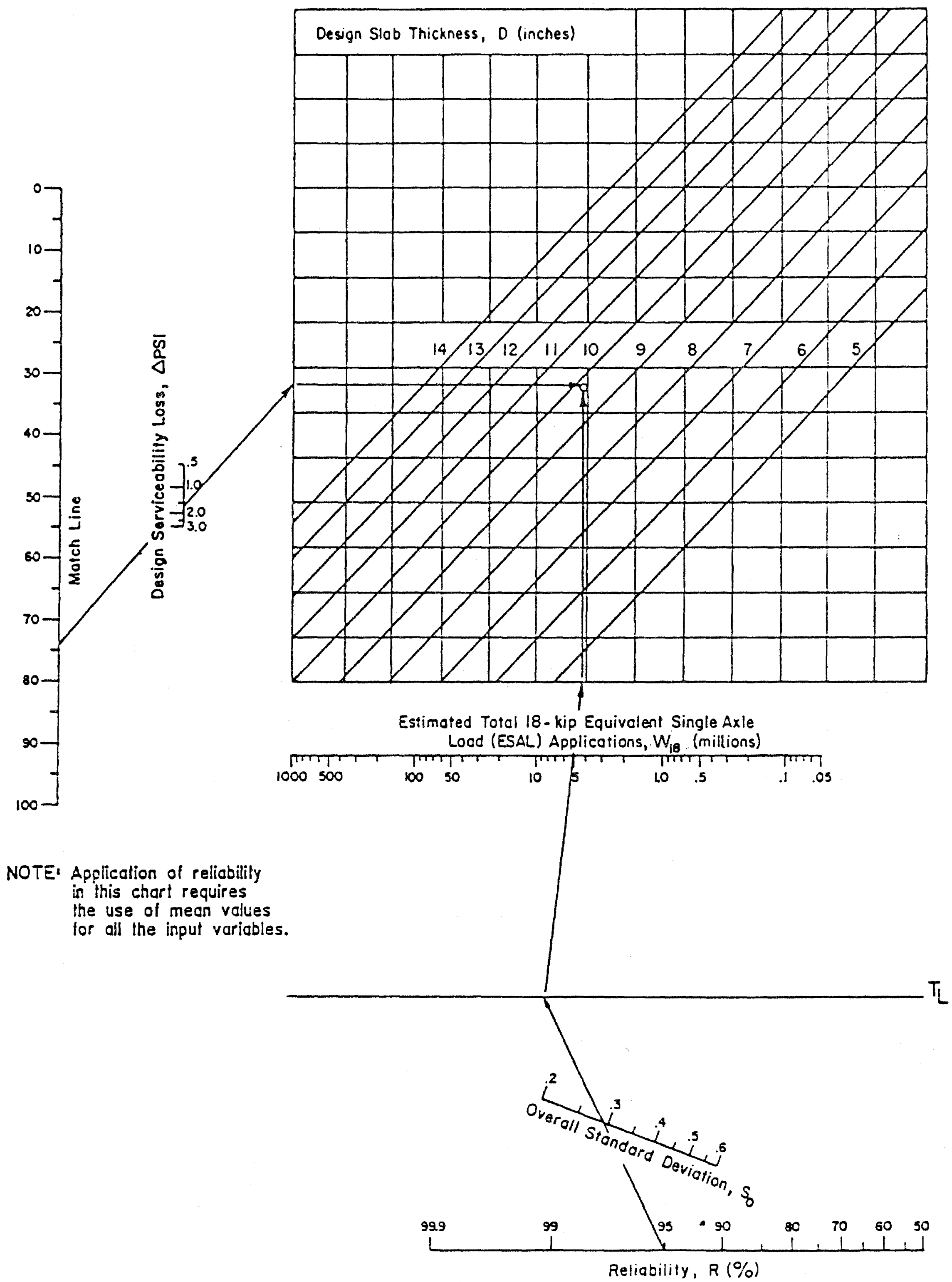


Figure 4-2.9. Design Chart for Rigid Pavements Based on Using Mean Values for Each Input Variable (Segment 2) (Cont.) (5).

1. Effective modulus of subgrade reaction, psi/inch.

2. Estimated future traffic, W_{18}, for the performance period.

3. Design reliability, R.

4. Overall standard deviation, S_o.

5. Design serviceability loss, $\Delta PSI = p_i - p_t$.

6. Concrete elastic modulus, E_c.

7. Concrete modulus of rupture, S'_c.

8. Load transfer coefficient, J.

9. Drainage coefficient, C_d.

This structural design does not consider the effects of roadbed swelling and frost heave in reducing the serviceability index. These effects are described in Section 5.3.

5.2 Stage Construction

A rigid pavement is normally designed to last over the entire analysis period through provision of a structural design to carry expected traffic. However, the designer may wish to consider stage construction of the rigid pavement by including one or more major rehabilitations over the course of the analysis period. This would be needed if the initial performance period does not last as long as the desired analysis period. This may occur due to environmental problems (such as swelling soil) or the desirability to consider thinner slabs with shorter lives for potential economic benefits. The type of rehabilitation could be restoration type work or different types of overlays.

5.3 Roadbed Swelling and Frost Heave Considerations

Swelling refers to the localized volume changes that occur in expansive roadbed soils as they absorb moisture. A drainage system can be effective in minimizing roadbed swelling if it reduces the availability of moisture for absorption.

Frost heave refers to the localized volume changes that occur in the roadbed as moisture collects, freezes into ice lenses, and produces distortions on the pavement surface. The effects of frost heave can be decreased by providing some type of drainage system, or by providing a layer of nonfrost susceptible material thick enough to prevent the frost from penetrating into the roadbed soil. This not only protects against frost heave, but also significantly reduces or eliminates the subgrade softening that may occur during the spring thaw.

The following iterative procedure can be used to directly consider the effect of swelling or frost heave on serviceability loss and the need for rehabilitation. The objective of this iterative process is to determine when the combined serviceability loss due to traffic and environment reaches the terminal level. An example worksheet is provided in Table 4-2.10 to complete the following:

Table 4-2.10. Example of Process Used to Predict the Performance of an Initial Rigid Pavement Structure Considering Swelling and/or Frost Heave (5).

Slab Thickness (inches) 9.5
Maximum Possible Performance Period (years) 20
Design Serviceability Loss, Δ **PSI** = $p_i - p_t$ = 4.2 - 2.5 = 1.7

(1) Iteration No.	(2) Trial Performance Period (years)	(3) Total Serviceability Loss due to swelling and Frost Heave $\Delta PSI_{SW,FH}$	(4) Corresponding Serviceability Loss Due to Traffic ΔPSI_{TR}	(5) Allowable Cumulative Traffic (18-kip ESAL)	(6) Corresponding Performance Period (years)
1	14.0	0.75	0.95	3.1×10^6	9.6
2	11.8	0.69	1.01	3.3×10^6	10.2
3	11.0	0.67	1.03	3.4×10^6	10.4

Column No.	Description of Procedures
2	Estimated by the designer (Step 2).
3	Using estimated value from Column 2 with Figure 2.2, the total serviceability loss due to swelling and frost heave is determined (Step 3).
4	Subtract environmental serviceability loss (Column 3) from design total serviceability loss to determine corresponding serviceability loss due to traffic.
5	Determined from Figure 3.5 keeping all inputs constant (except for use of traffic serviceability loss from Column 4) and applying the chart in reverse (Step 5).
6	Using the traffic from Column 5, estimate net performance period from Figure 2.1 (Step 6).

1. Select an appropriate slab thickness for the initial pavement considering the traffic level, assuming no swelling or frost heave. Any practical slab thickness less than this value may be appropriate for swelling or frost heave conditions, so long as it does not violate the minimum performance period.

   It is important to note that for this example, an overall reliability of 90 percent is desired. Since it is expected that one overlay will be required to reach the 20-year analysis period, the individual reliability that must be used for the design of both the initial pavement and the overlay is $0.90^{0.5}$ or 95 percent.

2. Select a trial performance period that might be expected under the swelling/frost heave conditions anticipated and enter in Column 2. This number should be less than the maximum possible performance period corresponding to the selected initial slab thickness. In general, the greater the environmental loss, the smaller the performance period will be.

3. Using a graph of cumulative environmental serviceability loss versus time, estimate the corresponding total environmental serviceability loss due to swelling and frost heave ($PSI_{SW,FH}$) that can be expected for the trial period from Step 2 and enter in Column 3.

4. Subtract this environmental serviceability loss (Step 3) from the desired total serviceability loss (4.2 - 2.5 = 1.7 in this example) to establish the corresponding traffic serviceability loss. Enter in Column 4.

$$\Delta PSI_{TR} = \Delta PSI - \Delta PSI_{SW,FH}$$

5. Use Figure 4-2.9 to estimate the allowable cumulative 18-kip ESAL traffic corresponding to the traffic serviceability loss determined in Step 4 and enter in Column 5. It is important to use the same levels of reliability, effective modulus of subgrade reaction, etc., when applying the rigid pavement design chart to estimate the allowable traffic.

6. Estimate the corresponding year at which the cumulative 18-kip ESAL traffic (determined in Step 5) will be reached and enter in Column 6. This should be accomplished with the aid of the cumulative traffic versus time plot.

7. Compare the trial performance period with that calculated in Step 6. If the difference is greater than 1 year, calculate the average of the two and use this as the trial value for the start of the next iteration (return to Step 2). If the difference is less than 1 year, convergence is reached and the average is said to be the predicted performance period of the initial pavement structure corresponding to the selected design slab thickness. In the example, convergence was reached after three iterations and the predicted performance period is about 10.5 years.

   The basis of this iterative process is exactly the same for the estimation of the performance period of any subsequent overlays.

6.0 REINFORCEMENT DESIGN PROCEDURE

The purpose of steel reinforcement in the pavement is to hold the cracks that form very tight, thus maintaining the pavement as an integral structural unit. Both deformed wire fabric and deformed reinforcement bars may be used for either CRCP or JRCP.

6.1 Jointed Reinforced Concrete Pavements

The required amount of reinforcement for JRCP is determined using the subgrade drag theory (Module 4-1). The inputs required for determining the amount of reinforcement are:

1. Slab length, L.
2. Steel working stress, f_s.
3. Friction factor, F.

The nomograph for estimating the minimum percent of steel reinforcement required in a JRCP is shown in Figure 4-2.10. This chart applies to both longitudinal and transverse reinforcement for JRCP (it also applies to transverse reinforcement for CRCP). The amount of reinforcement required is expressed as a percentage of the total PCC cross-sectional area. This minimum percentage must be converted into the size of bars or wires and their spacings.

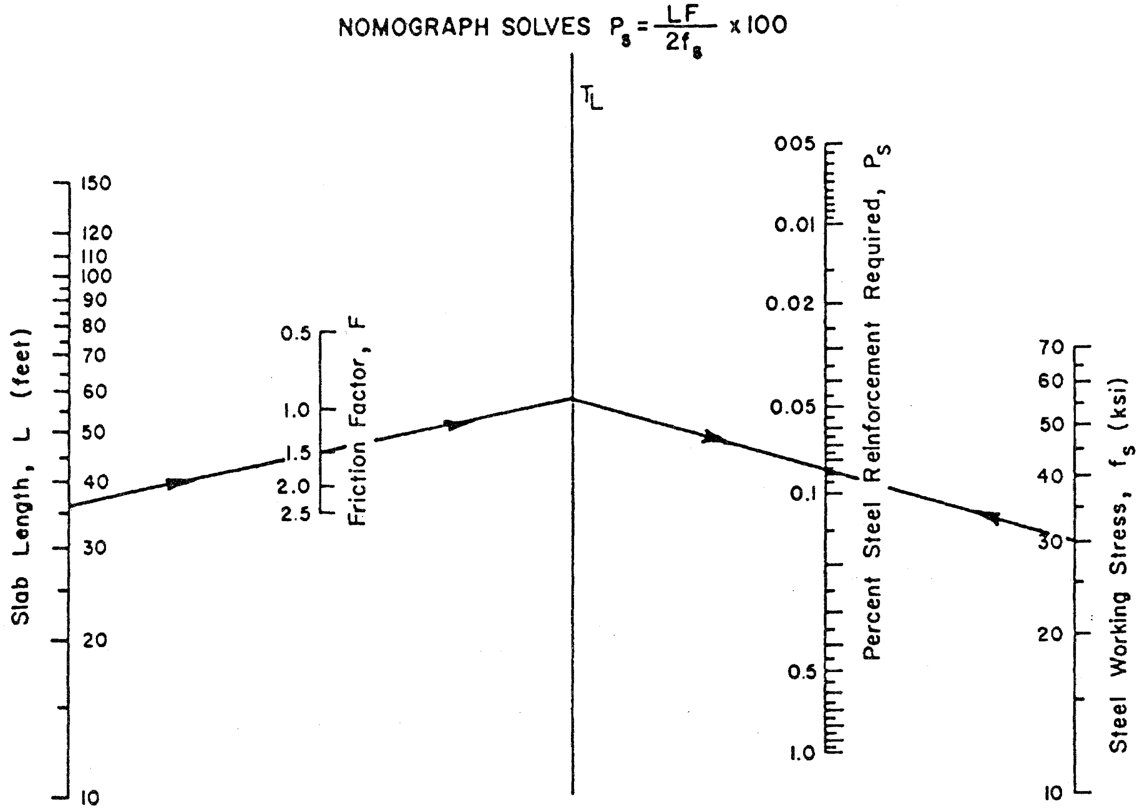
6.2 Continuously Reinforced Concrete Pavements

The amount of longitudinal reinforcement may be determined using the worksheet in Table 4-2.11. Space is provided for entering the design inputs, intermediate results and calculations. A separate worksheet is presented in Table 4-2.12 for design revisions.

The design inputs required by this procedure are as follows:

1. Concrete indirect tensile strength, f_t.
2. Concrete shrinkage at 28 days, Z.
3. Concrete thermal coefficient, A_c.
4. Reinforcing bar or wire diameter, d.
5. Steel thermal coefficient, A_s.
6. Design temperature drop, ΔT_D.

An additional input required by the procedure is the wheel load tensile stress developed during initial loading of the constructed pavement by either construction equipment or truck traffic. Figure 4-2.11 may be used to estimate this wheel load stress based on the design slab thickness, the magnitude of the wheel load, and the effective modulus of subgrade reaction.



Example:
L = 36 ft.
F = 1.5
f_s = 30,000 psi

Solution:
P_s = .085%

Figure 4-2.10. Reinforcement Design Chart for Jointed Reinforced Concrete Pavements (5).

Table 4-2.11. Worksheet for Longitudinal Reinforcement Design (5).

DESIGN INPUTS

Input Variable	Value	Input Variable	Value
Reinforcing Bar/Wire Diameter, ϕ(inches)		Thermal Coefficient Ratio, α_s/α_c (in/in)	
Concrete Shrinkage, Z (in/in)		Design Temperature Drop, DT_D(°F)	
Concrete Tensile Strength, f_t(psi)		Wheel Load Stress, σ_w(psi)	

DESIGN CRITERIA AND REQUIRED STEEL PERCENTAGE

	Crack Spacing, $\bar{x}$ (feet)	Allowable Crack Width, CW_{max} (inches)	Allowable Steel Stress, $(\sigma_s)_{max}$ (ksi)	
Value of Limiting Criteria	Max: 8.0 Min. 3.5			
Minimum Required Steel Percentage				(P_{min})*
Maximum Allowable Steel Percentage				P_{max}

*Enter the largest percentage across line

**If $P_{max} < P_{min}$, then reinforcement criteria are in conflict, design not feasible.

Table 4-2.12. Worksheet for Revised Longitudinal Reinforcement Design (5).

Parameter		Change in Value from Previous Trial				
		Trial 2	Trial 3	Trial 4	Trial 5	Trial 6
[2]Reinforcing Bar/Wire Diameter, ϕ(inches)						
Concrete Shrinkage, Z (in/in)						
[2]Concrete Tensile Strenght, f_t(psi)						
Wheel Load Stress, σ_w(psi)						
[1]Design Temperature Drop, DT_D($^{\circ}$F)						
Thermal Coefficient Ratio, α_s/α_c						
Allowable Crack Width Criterion, CW_{max}(inches)						
Allowable Steel Stress Criterion, $(\sigma_s)_{max}$(ksi)						
Required Steel % for Crack Spacing	min.					
	max.					
Minimum Required Steel % for Crack Width						
Minimum Required Steel % for Steel Stress						
Minimum % Reinforcement, P_{min}						
Maximum % Reinforcement, P_{max}						

[1]Change in this parameter will affect crack width criterion.

[2]Change in this parameter will affect steel stress criterion.

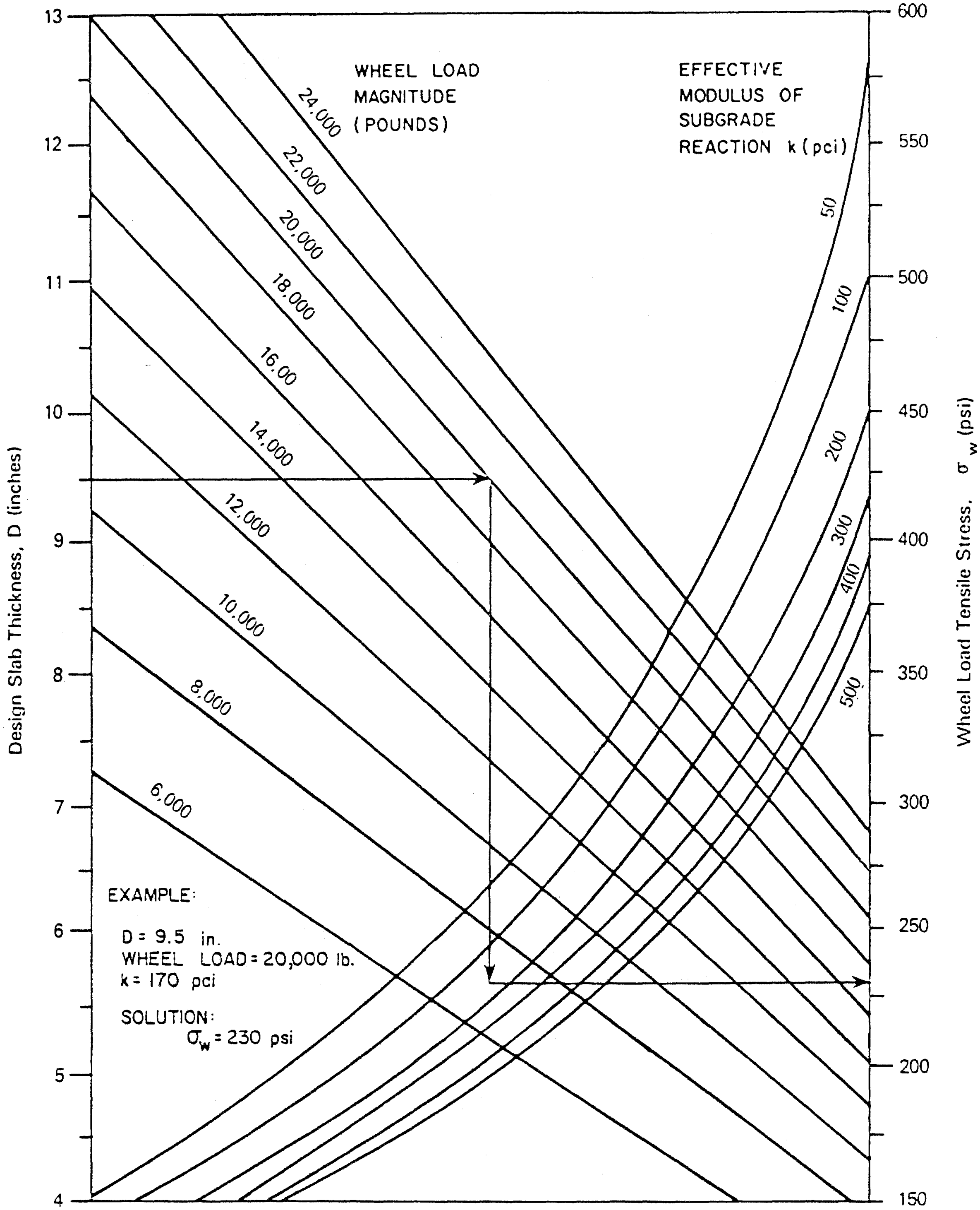


Figure 4-2.11. Chart for Estimating Wheel Load Tensile Stress (5).

Three limiting criteria should also be considered:

1. Crack spacing. To minimize the potential for punchouts, the minimum crack spacing for design is 3.5 ft.

2. Crack width. The allowable crack width should not exceed 0.04 ins. (the crack width should be reduced as much as possible through the selection of a higher steel percentage or smaller diameter reinforcing bar/wire).

3. Steel stress. A limiting stress of 75 percent of the ultimate tensile strength is recommended.

Values of allowable mean steel working stress for use in this design procedure are listed in Table 4-2.13 as a function of reinforcing bar size and concrete strength. The indirect tensile strength should be that determined in AASHTO T198 or ASTM C496. The limiting steel working stresses in Table 4-2.13 are for the Grade 60 steel (meeting ASTM A615 specifications) recommended for longitudinal reinforcement in CRC pavements determination of allowable steel stress for other types. Once the allowable steel working stress is determined, it should be entered in the space provided in Table 4-2.11.

6.3 Design Procedure for Longitudinal Reinforcement

The following procedure may be used to determine the amount of longitudinal reinforcement required:

1. Solve for the required amount of steel reinforcement to satisfy each limiting criterion using the design charts in Figure 4-2.12, Figure 4-2.13, and Figure 4-2.14. Record the resulting steel percentages in the spaces provided in the worksheet in Table 4-2.11.

2 If P_{max} is greater than or equal to P_{min}, go to Step 3. If P_{max} is less than P_{min} then:

   a. Review the design inputs and decide which input to revise.
   b. Indicate the revised design inputs in the worksheet in Table 4-2.12. Make any corresponding change in the limiting criteria as influenced by the change in design parameter and record this in Table 4-2.12. Check to see if the revised inputs affect the subbase and slab thickness design. It may be necessary to reevaluate the subbase and slab thickness design.
   c. Rework the design nomographs and enter the resulting steel percentages in Table 4-2.12.
   d. If P_{max} is greater than or equal to P_{min}, go to Step 3. If P_{max} if less than P_{min}, repeat this step using the space provided in Table 4-2.12 for additional trials.

3. Determine the range in the number of reinforcing bars or wires required:

   a. $N_{min} = 0.01273 \times P_{min} \times W_S \times D/\phi^2$, and
   b. $N_{max} = 0.01273 \times P_{max} \times W_S \times D/\phi^2$

Table 4-2.13. Allowable Steel Working Stress, ksi, (10).

Indirect Tensile Strength of Concrete at 28 days, psi	Reinforcing Bar Size* No.4	No.5	No.6
300 (or less)	65	57	54
400	67	60	55
500	67	61	56
600	67	63	58
700	67	65	59
800 (or greater)	67	67	60

*For DWF proportional adjustments may be made using the wire diameter to bar diameter.

Nomograph Solves : $\bar{X} = \dfrac{1.32\left(1+\dfrac{f_t}{1000}\right)^{6.70} \times \left(1+\dfrac{\alpha_s}{2\alpha_c}\right)^{1.15} \times \left(1+\phi\right)^{2.19}}{\left(1+\dfrac{\sigma_w}{1000}\right)^{5.20} \times \left(1+P\right)^{4.60} \times \left(1+1000Z\right)^{1.79}}$

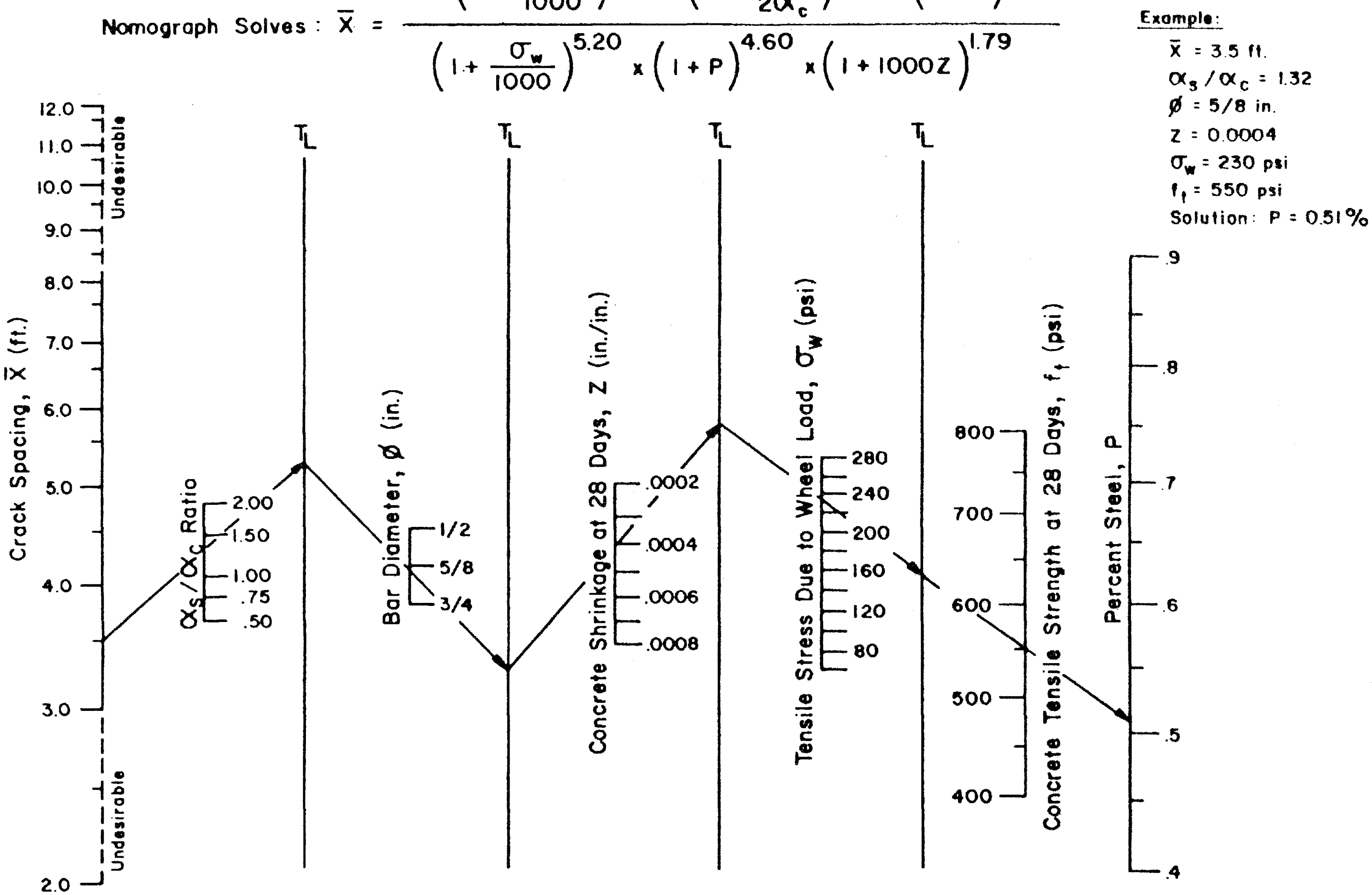


Figure 4-2.12. Percent of Longitudinal Reinforcement to Satisfy Crack Spacing Criteria (5).

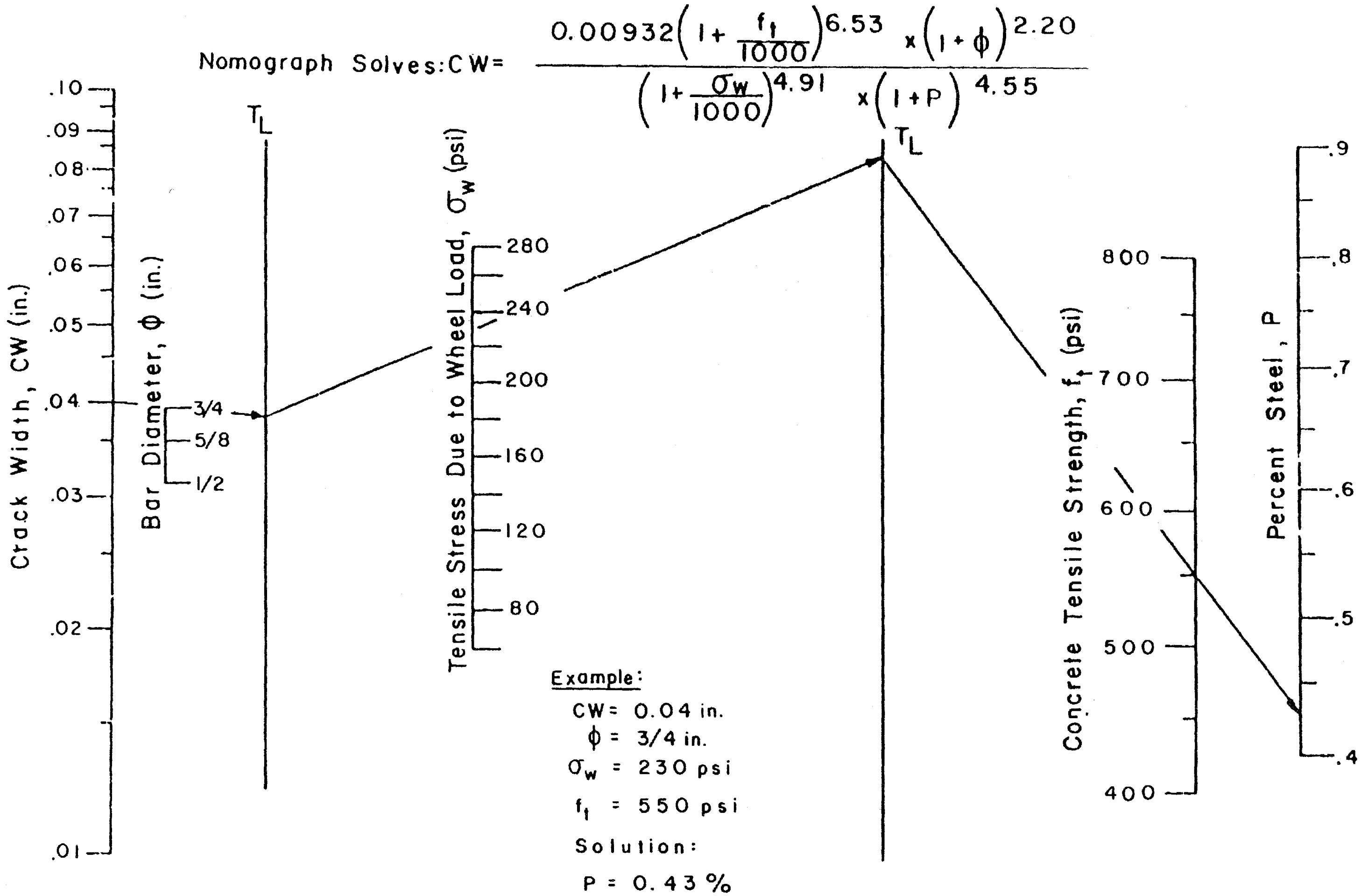


Figure 4-2.13. Minimum Percent Longitudinal Reinforcement to Satisfy Crack Width Criteria (5).

Nomograph Solves: $$\sigma_s = \frac{47{,}300\left(1+\frac{DT_D}{100}\right)^{0.425} \times \left(1+\frac{f_t}{1000}\right)^{4.09}}{\left(1+\frac{\sigma_w}{1000}\right)^{3.14} \times \left(1+1000Z\right)^{0.494} \times \left(1+P\right)^{2.74}}$$

Example:

σ_s = 57 ksi
DT_D = 55° F
Z = 0.0004
σ_w = 230 psi
f_t = 550 psi
Solution: P = 0.47%

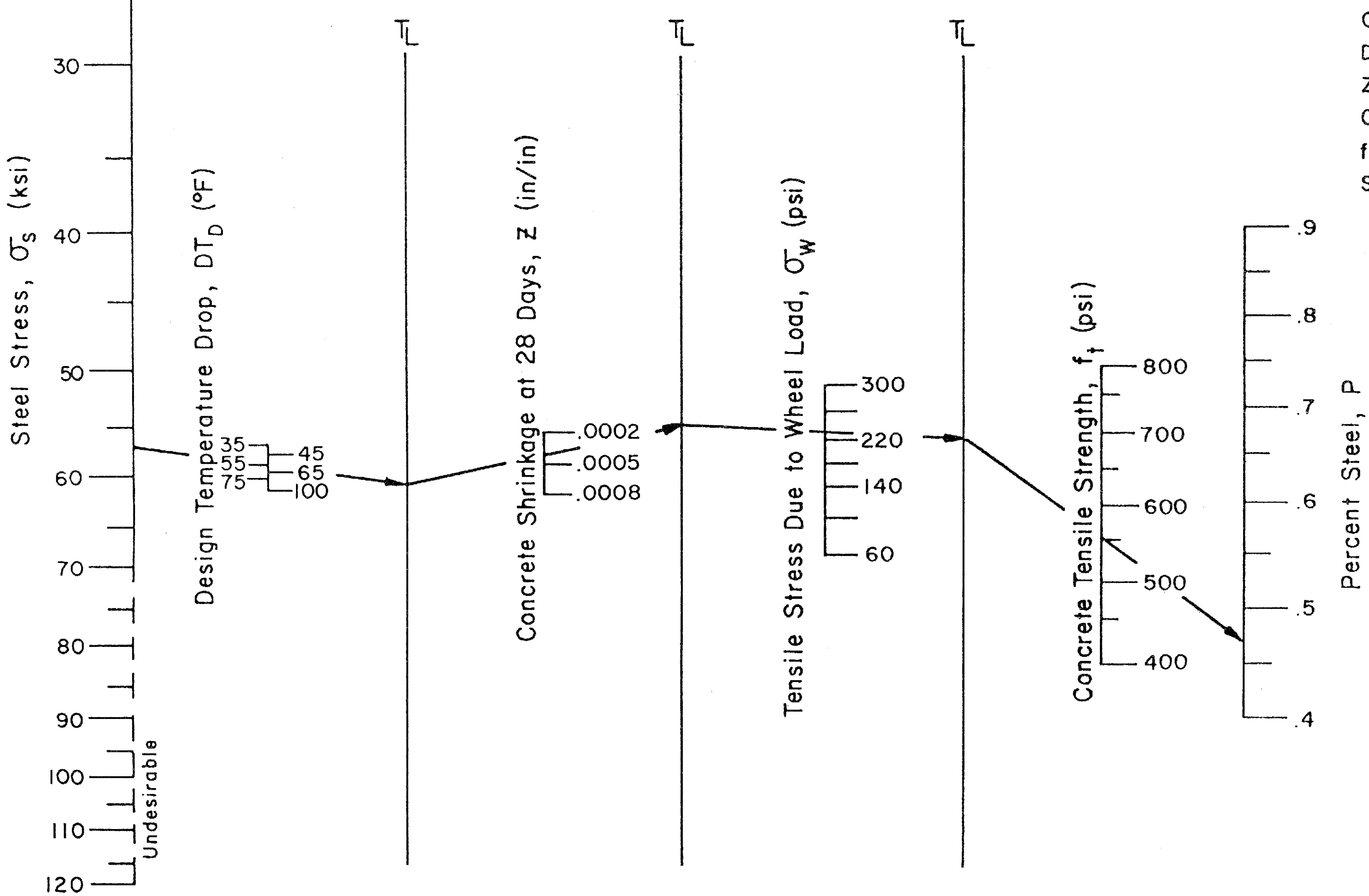


Figure 4-2.14. Minimum Percent Longitudinal Reinforcement to Satisfy Steel Stress Criteria (5).

where:

N_{min} = minimum required number of reinforcing bars or wires,
N_{max} = maximum required number of reinforcing bars or wires,
P_{min} = minimum required percent steel,
P_{max} = maximum required percent steel,
W_s = total width of pavement section (inches),
D = thickness of concrete layer (inches), and
ϕ = reinforcing bar or wire diameter (inches), which may be increased if loss of cross section is anticipated due to corrosion.

4. Determine the final steel design by selecting the total number of reinforcing bars or wires in the final design section, N_{Design}, such that N_{Design} is a whole integer number between N_{min} and N_{max}. The appropriateness of these final design alternatives may be checked by converting the whole integer number of bars or wires to percent steel and working backward through the design charts to estimate crack spacing, crack width, and steel stress.

6.4 Design Procedure for Transverse Reinforcement

Transverse steel is included in either jointed or continuous pavements for conditions where soil volume changes (due to changes in either temperature or moisture) can result in longitudinal cracking. Steel reinforcements will prevent the longitudinal cracks from opening excessively, thereby maintaining maximum load transfer and minimizing water entry.

When transverse reinforcement and/or tie bars are desired, the "slab length" should be considered as the distance between free longitudinal edges. If tie bars are placed at a longitudinal joint, then that joint is not a free edge.

For normal transverse reinforcement, Figure 4-2.10 may be used to determine the percent transverse steel. The percent transverse steel may be converted to spacing between reinforcing bars as follows:

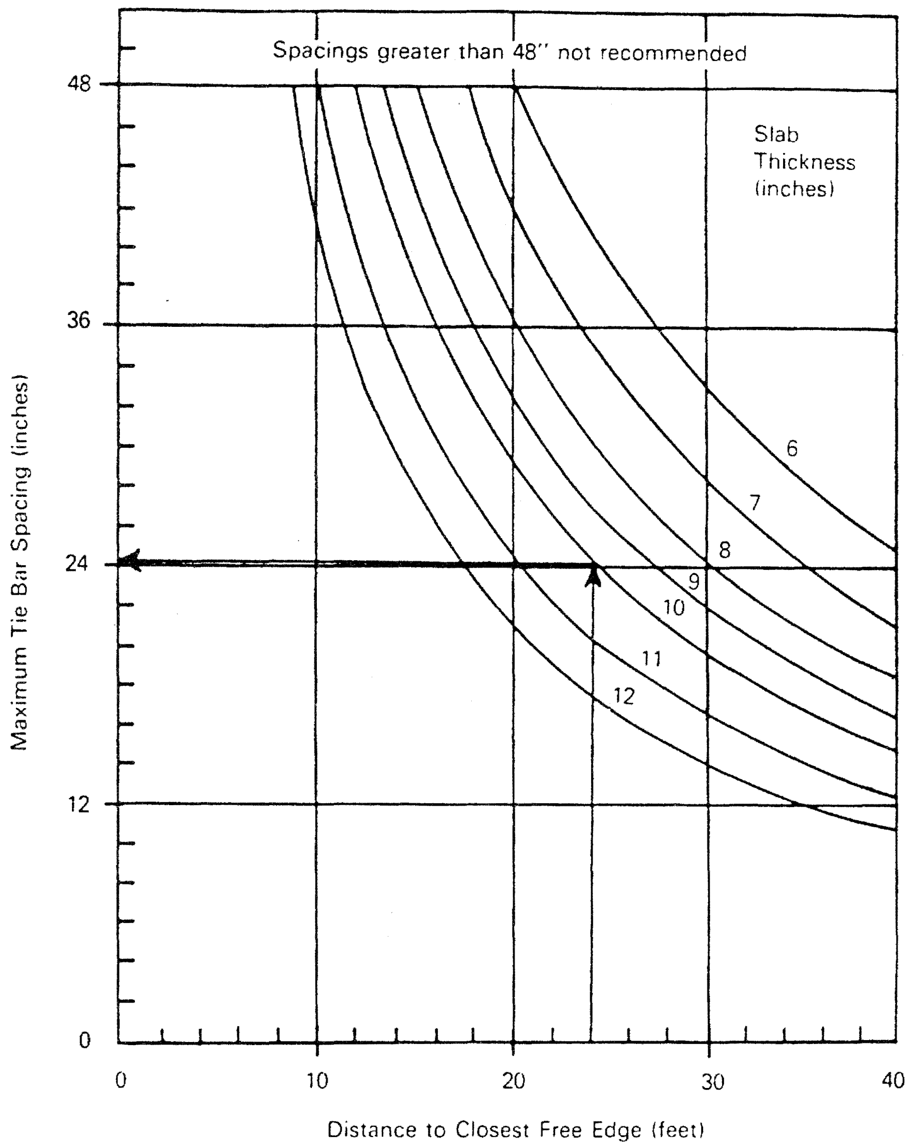
$$Y = (A_s/P_tD) \times 100$$

where:

Y = transverse steel spacing (inches),
A_s = cross-sectional area of transverse reinforcing steel (in^2),
P_t = percent transverse steel, and
D = slab thickness (inches).

Figure 4-2.15 and Figure 4-2.16 may be used to determine the tie bar spacing for 1/2-inch and 5/8-inch diameter deformed bars, respectively. The designer enters the figure on the horizontal with the distance to the closest free edge axis and proceeds vertically to the design pavement thickness. From the pavement thickness, move horizontally and read the tie bar spacing from the vertical scale. These nomographs are based on Grade 40 steel and a subgrade friction factor of 1.5.

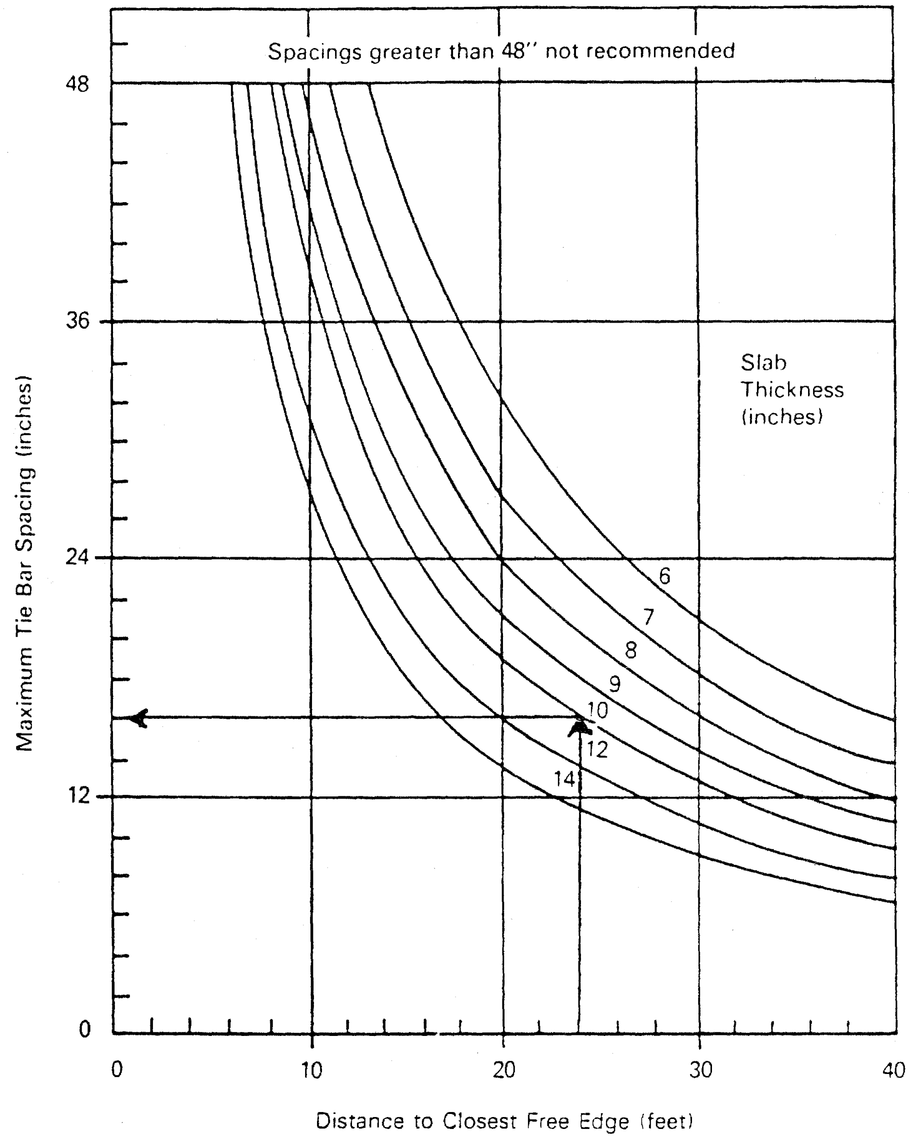
Note that since steel stress decreases from a maximum near the center of the slab (between the free edges) to zero at the free edges, the required minimum tie bar spacing increases. Thus, in order to design the tie bars efficiently, the designer should first select the layout of the longitudinal construction joints.



Example: Distance from free edge = 24 ft.
D = 10 in.

Answer: Spacing = 24 in.

Figure 4-2.15. Recommended Maximum Tie Bar Spacings for PCC Pavements Assuming 5/8 inches Diameter Tie Bars, Grade 40 Steel, and Subgrade Friction of 1.5 (5).



Example: Distance from free edge = 24 ft.
D = 10 in.

Answer: Spacing = 16 in.

Figure 4-2.16. Recommended Maximum Tie Bar Spacings for PCC Pavements Assuming 1/2-inches Diameter Tie Bars, Grade 40 Steel, and Subgrade Friction Factor of 1.5 (5).

Finally, if bending of the tie bars is to be permitted during construction, then to prevent steel failures, the use of brittle (high carbon content) steels should be avoided and an appropriate steel working stress level selected.

7.0 JOINT DESIGN

Joint design requires the determination of spacing of longitudinal and transverse joints, joint load transfer and joint forming and sealant dimensions. Recommendations provided in the AASHTO Guide are summarized below.

7.1 Joint Spacing

Contraction joint spacing for JPCP to control transverse cracking depends upon several factors as described in Module 6-1. Local conditions of materials and environment are very important and should be the most significant factor. As a rough guide, the joint spacing (in ft.) should not exceed twice the slab thickness (in ins.). No such guideline is provided for joint spacing of JRCP.

Skewing and randomization of joints minimize the effect of joint faulting, thereby improving the pavement riding quality. The obtuse angle at the outside pavement edge should be ahead of the joint in the direction of traffic (counter clockwise in direction of traffic), since that corner receives the greatest impact from the sudden impact of load. Skewed joints have the following advantages:

1. Reduced deflection and stress at joints.

2. Less impact reaction in vehicles as they cross joints.

7.2 Joint Load Transfer

No specific guidance for joint load transfer type selection for JPCP or JRCP is provided (e.g., use of dowels or aggregate interlock). However, some guidance is given through use of the load transfer coefficient, J, which is used to account for the ability of a concrete pavement structure to transfer load across a joint or crack. Dowels or other mechanical load transfer devices, aggregate interlock, reinforcement and tied concrete shoulders all have an effect on this value. Recommended values for J for all rigid pavement types are given in Table 4-2.5.

Dowel diameter should be equal to the slab thickness multiplied by 1/8 inch (e.g., 8 inch slab would utilize a 8 x 1/8 = 1 inch diameter dowel. The dowel spacing and length are normally 12 ins. and 18 ins., respectively.

7.3 Joint Forming and Reservoir Dimensions

7.3.1 Depth of Initial Joint

The depth of the transverse and longitudinal joints should be adequate to ensure that cracking occurs at the joint. The following criteria are recommended:

1. Transverse joints - 1/4 slab thickness.

2. Longitudinal joints - 1/3 slab thickness

These joints may be developed by sawing, inserts, or forming. The time of sawing is critical to preventing uncontrolled cracking, and joints should be sawed consecutively to ensure all commence working together. The time between placement and sawing is variable and depends upon temperature, curing conditions and mix proportions.

7.3.2 Joint Reservoir Dimensions

Contraction joint sealant reservoir dimensions require direct consideration of joint opening and closing movements and the type of sealant. The opening and closing of a joint depends upon slab length, temperature change, thermal coefficient of the PCC and friction between slab and subbase. The expected mean opening of a joint can be computed from the following equation:

$$\Delta L = C*L*(A*\Delta T + Z)$$

where:

ΔL = the joint opening caused by temperature changes and drying shrinkage of the PCC, ins.
A = thermal coefficient of contraction of the PCC slab, degrees F
ΔT = temperature range from PCC placement to minimum temperature, degrees F
Z = drying shrinkage coefficient of the PCC slab (neglect for resealing project), ins./ins.
ΔL = joint spacing, ins. (not ft.)
C = an adjustment factor for friction between slab and subbase, 0.80, for granular untreated subbase 0.65, for stabilized granular subbase (e.g., asphalt, cement)

The required joint design width is then computed from the following equation:

$$W = \Delta L/S$$

where:

W = design width of transverse contraction joint, ins.
S = allowable strain in the joint sealant material (e.g., different sealants have different requirements, most asphaltic based sealants allow a maximum tensile strain in the sealant of 25 percent, thus S would be 0.25; whereas silicone sealants require 50 percent)

For field molded sealants, the depth of the sealant reservoir is determined by the desired joint reservoir shape factor for the sealant (width to depth). Different sealants require different shape factors, and manufacturers recommendations should be followed for their specific type of sealant. For many sealants, the shape factor should be within the range of 0.67 to 1.5. A minimum depth of 3/8 and 1/2 inch is recommended for longitudinal and transverse joints, respectively. A backer rod may be used to achieve the proper shape factor.

For premolded sealants, the sealant reservoir is much different. The sealant should be compressed between 20 to 50 percent of its normal width at all times. The selection of a proper preformed sealant width and joint reservoir width is a trial and error procedure.

Expansion and construction joint sealants should also be designed based upon expected movement and materials.

8.0 SENSITIVITY ANALYSIS FOR SLAB THICKNESS

Some of the design inputs have a much stronger effect than others on required slab thickness. It is important to develop an understanding of the relative effect that the inputs have upon the design of the rigid pavement. This section provides some illustrations of the sensitivity of selected inputs to the AASHTO Design Guide.

A set of "standard" design inputs were selected as shown below.

Analysis period . . . . 20 years

Number of Traffic lanes . . . . . . 3 one direction

Roadbed soil resilient moduli

Season:	1	2	3	4	5	6
Modulus (psi):	30,000	30,000	6,000	6,000	12,000	12,000
Season:	7	8	9	10	11	12
Modulus (psi):	12,000	12,000	12,000	12,000	12,000	12,000

Reliability Level. . . . . 50 percent

Roadbed soil swelling . . . . Not considered
Frost heave . . . . Not considered

Performance period . . . . 20 years

Serviceability Index, Initial . . . 4.5
Final 3.0

Traffic
- Growth rate . . . . 4 percent
- Initial yearly 18-kip ESAL . . 2,700,000 both directions
- Directional dist. factor . . . 50 percent
- Lane distribution factor . . . 65 percent

Overall standard deviation . . . . . 0.35 (log of repetitions)

Subbase
- Type . . . . GRANULAR
- Thickness . . . . 6 inches
- Elastic modulus . . . . 30,000 psi

Portland cement concrete slab
Type of construction ..... JPCP
PCC elastic modulus ...... 4,500,000 psi
Average PCC modulus of rupture 725 psi

Structural characteristics
Load transfer coefficient ... 3.2
Drainage coefficient 1.00
Loss of support factor 1.00

This set of inputs were considered "standard" conditions and several key input factors were varied over a practical range, one at a time, while all other factors were held constant. The required slab thickness was determined for each combination of inputs. Graphs showing the results are shown in Figures 4-2.17 to 4-2.22. The change in required slab thickness for the corresponding change in each design input is as follows:

Design Input	Range	Change in Slab Thickness, ins.
Terminal Serviceability	2.0 - 3.0	0.8
Type of Shoulders	AC (J=3.2)/PCC (J=2.8)	0.8
Annual growth rate	1 % - 8 %	1.2
Load transfer	J = 3.2 - 4.1	1.3
Drainage coefficient, Cd	0.8 - 1.2	2.1
Reliability level	50 % - 99 %	3.2

The design reliability level and drainage coefficient have a very large effect on required slab thickness (2 to 3 inches). The other inputs also cause a change in required slab thickness of about one inch. The effects of other design inputs could also be determined in a similar manner. Some inputs, however, require the corresponding change of other inputs to keep the results realistic. For example, if base type is varied, the drainage coefficient and loss of support should also be varied to keep the analysis realistic.

The designer is encouraged to conduct a small sensitivity analysis of some of the inputs to his/her design problem that may be difficult to determine, to see how much they effect the required slab thickness. The results could be useful in the determination of the required slab thickness for the project.

9.0 EXAMPLE DESIGN PROBLEM

This section provides a detailed design example for the design of a rigid pavement using the AASHTO rigid pavement design procedure and the DNPS86 computer program (computerized version of the AASHTO procedure) (14). In the first example, frost heave is handled through the provision of a layer of non-frost susceptible material thick enough to insulate the soil from significant frost penetration, but not thick enough to prevent some thaw-weakening. No loss of serviceability due to frost heaving is expected. In the second example, a loss of serviceability is expected due to frost heaving action.

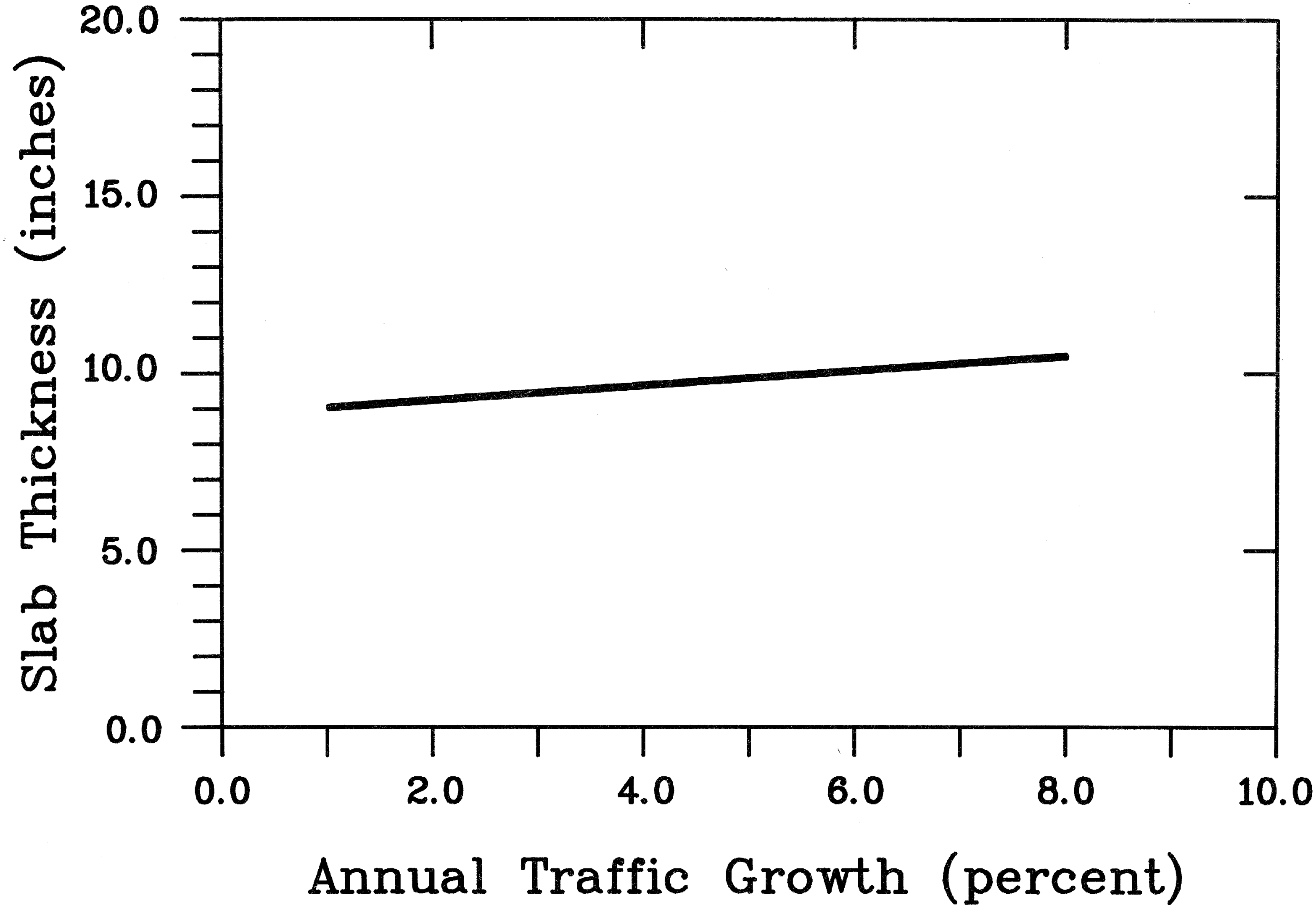


Figure 4-2.17. Slab Thickness vs. Annual Traffic Growth.

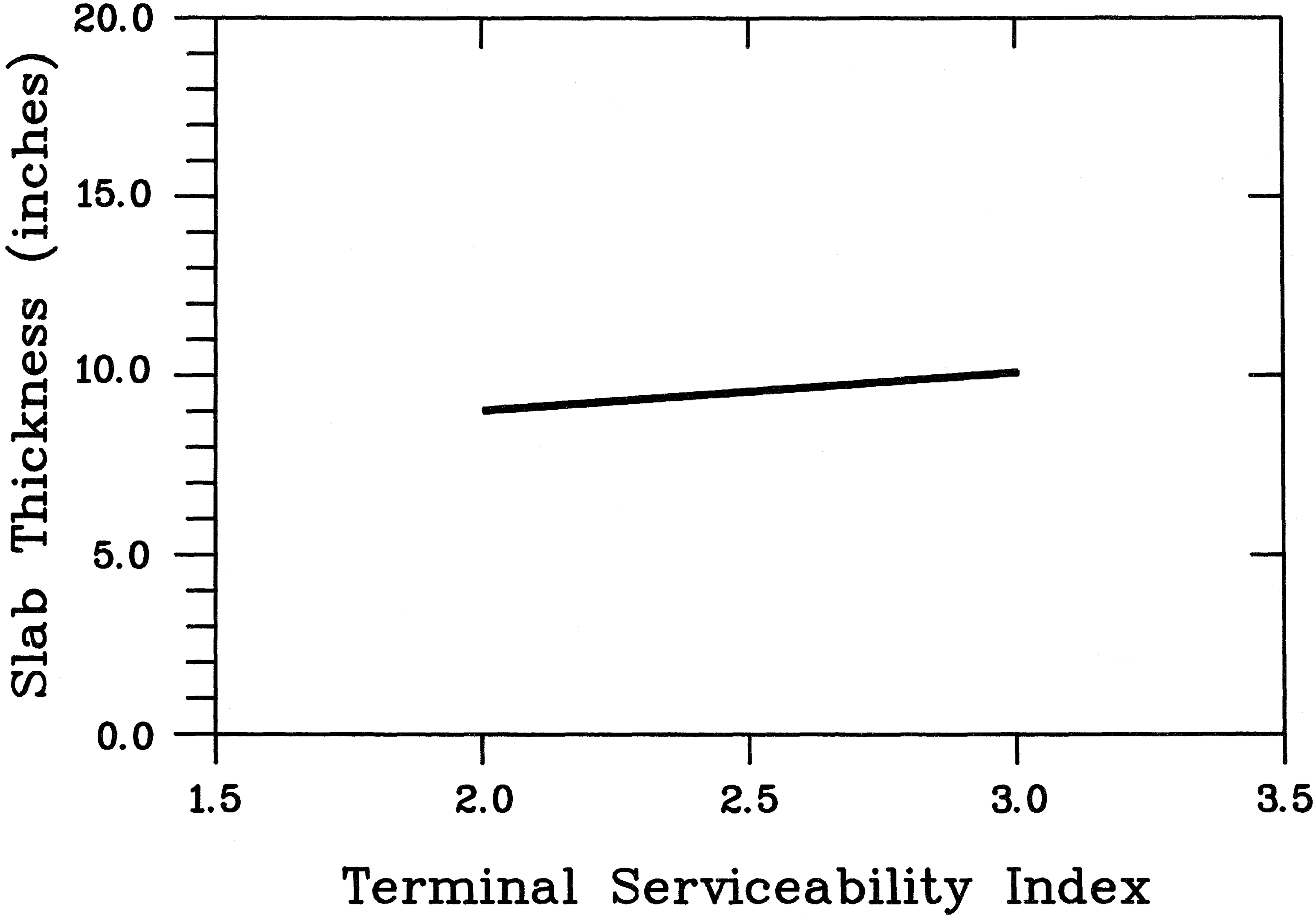


Figure 4-2.18. Slab Thickness vs. Terminal Serviceability.

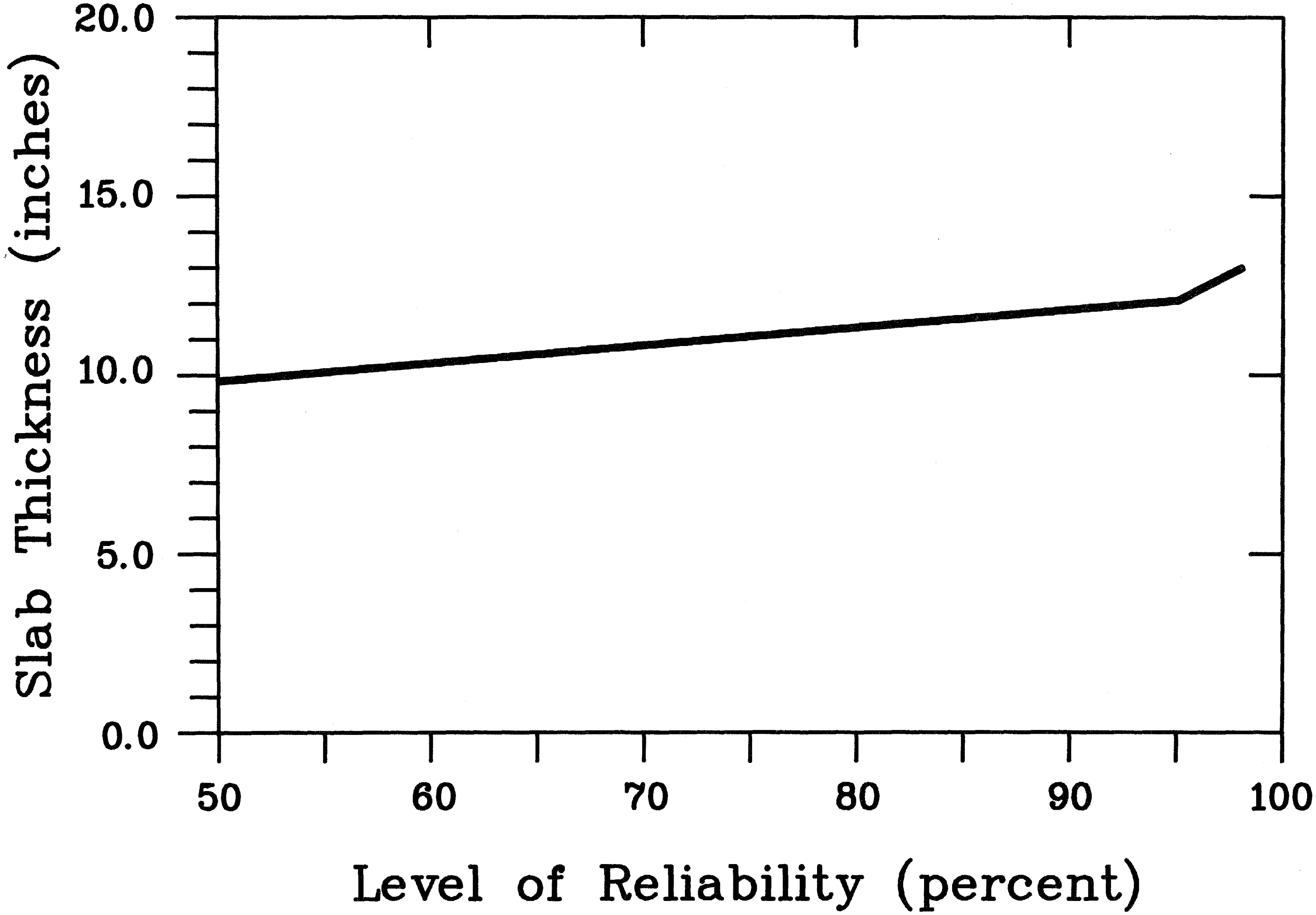


Figure 4-2.19. Slab Thickness vs. Reliability.

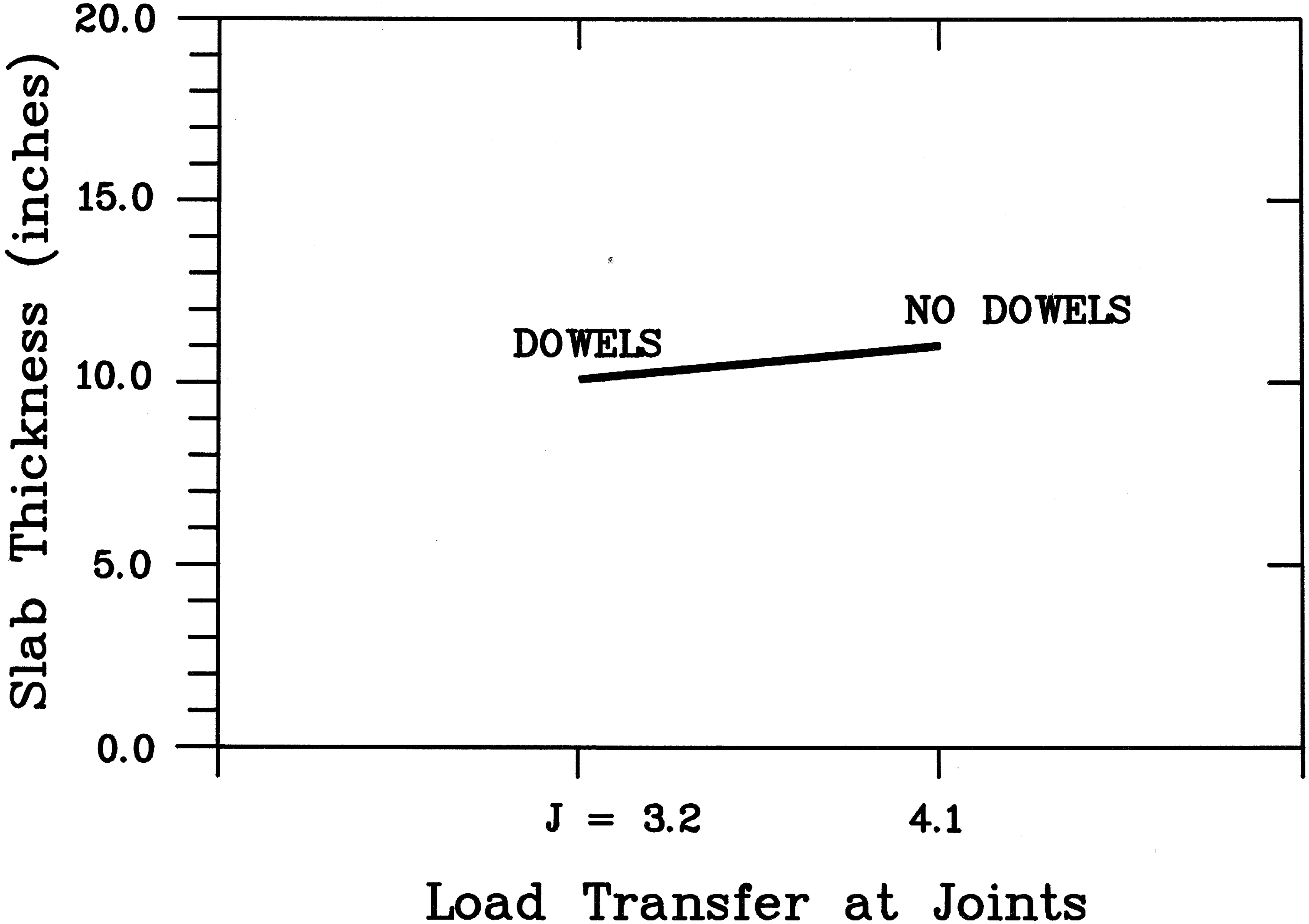


Figure 4-2.20. Slab Thickness vs. Load Transfer.

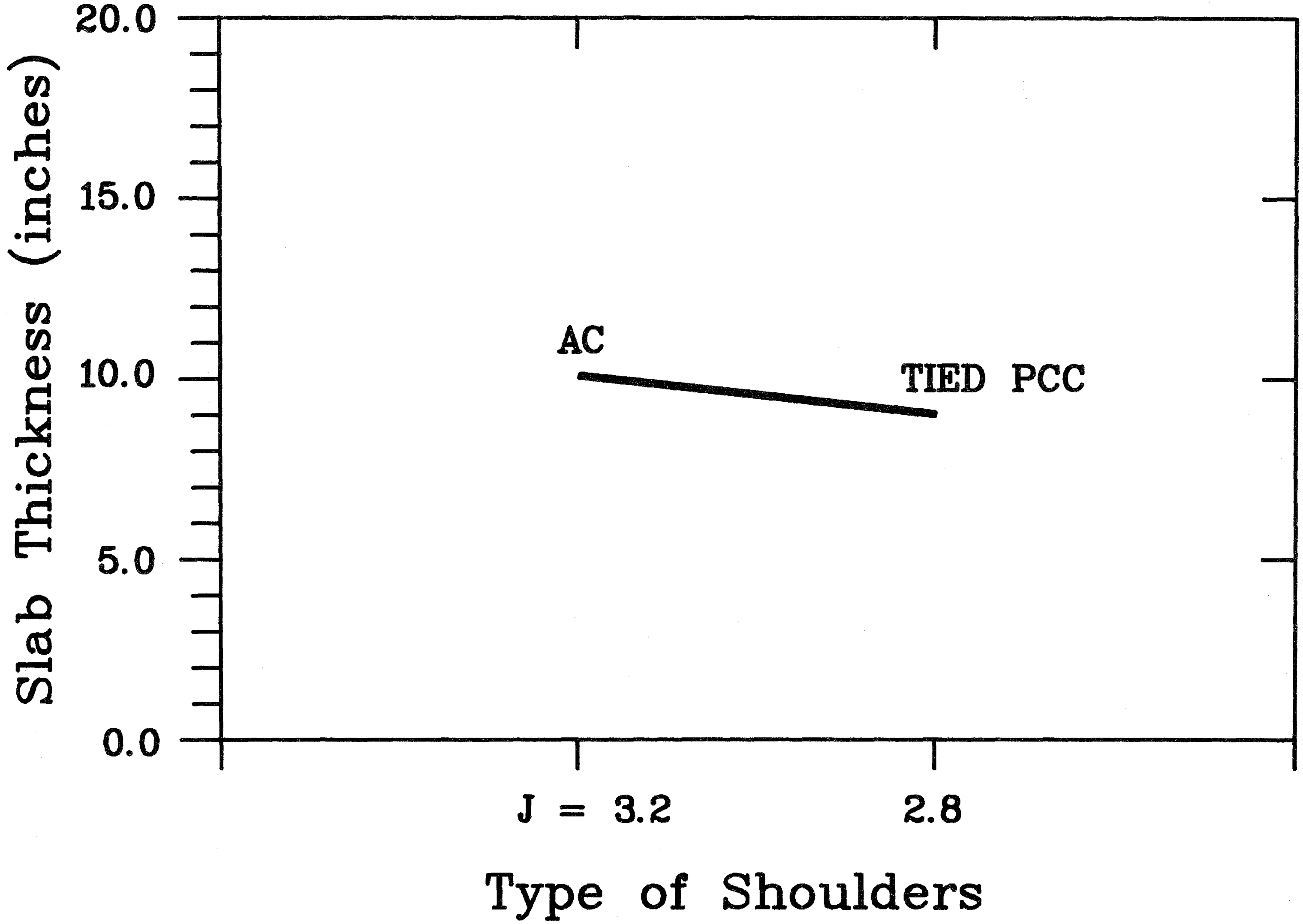


Figure 4-2.21. Slab Thickness vs. Type of Shoulders.

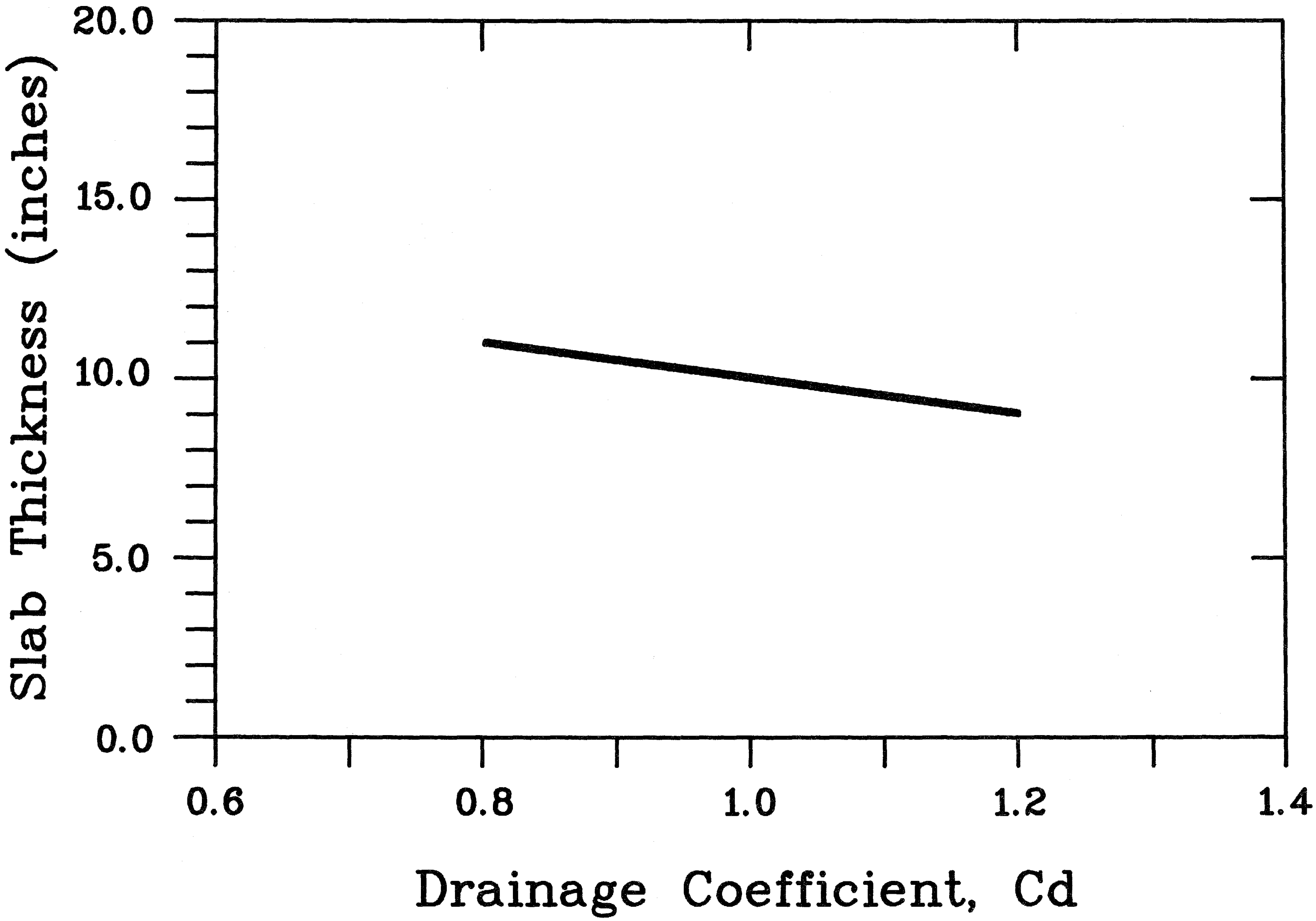


Figure 4-2.22. Slab Thickness vs. Drainage Coefficient.

9.1 Determine the Required Slab and Subbase Thickness

DESIGN INPUTS (no loss of serviceability due to frost heave)

Pavement location	Rural
Functional classification	Primary arterial
Analysis period	30 years
Number of Traffic lanes	2 one direction

Roadbed soil resilient moduli

Season:	1	2	3	4	5	6
Modulus (psi):	50,000	50,000	8,000	8,000	15,000	15,000
Season:	7	8	9	10	11	12
Modulus (psi):	15,000	15,000	15,000	15,000	15,000	15,000

Reliability Level.	90 percent
Roadbed soil swelling	Does not exist in project
Frost heave	Frost depth penetration to 36 inches average. For example 1 assume that corrective measures will be taken to prevent frost heave.
Performance period	30 years
Serviceability Index, Initial	4.5
Final	3.0

Traffic

ADT and truck volumes	Initial ADT is 15,700. Initial ADTT is 2200, or 14%. Of ADTT, 330 are single units and 1870 are multiple units.
Growth rate of truck traffic. . .	6 percent
Directional dist. factor	50 percent
Lane distribution factor	80 percent
Initial yearly 18-kip ESAL . . .	Computed using design truck factors: Single units = 0.319 Multiple units = 1.700 18-kip ESAL/truck factor over 30 year design period = (0.319*330 + 1.700*1870)365 = 1,200,000 both directions.

Total 18-kip ESAL........	= Traffic Growth Factor * Initial Yearly Traffic * Directional Distribution * Lane Distribution = 79.06*1,200,000*0.5*0.8 = 37,948,800 (for 50 percent design reliability).

Overall standard deviation ..... 0.39 (log of repetitions)

Subbase

Type .... PERMEABLE CEMENT-TREATED
Thickness........... 6 inches
Elastic modulus........ 100,000 psi

Effective modulus of subgrade reaction determined from Table 4-2.14 is 1000 pci

Portland cement concrete slab

Type of construction	JPCP
PCC elastic modulus	4,500,000 psi
Average PCC modulus of rupture .	725 psi

Structural characteristics

Load transfer coefficient....	Dowels (J=2.8, tied PCC shoulder)
Drainage coefficient	Drainage of the roadbbed soilsis poor, however, the permeable base provides for rapid drainage of the structural section of the pavement = 1.00
Loss of support factor	The permeable base is expected to minimize pumping and thus loss of support = 0.5
Corrected Modulus of Subgrade Reaction for Loss of Support..	500 pci (See Figure 4-2.8).

<u>REQUIRED SLAB THICKNESS</u> (no loss of serviceability due to frost heave)

The required slab thickness can be determined using the above inputs and Figure 4-2.9. The result is 11.0 inches. This result is checked using the DNPS86 computer program. The input screens and output of the program is shown in Tables 4-2.15 and 4-2.16.

<u>DESIGN INPUTS</u> (assuming loss of serviceability due to frost heave)

An estimate of the loss of serviceability due to frost heaving must be made if the pavement is placed directly on the roadbed soil (i.e., a filter layer placed directly on the soil, followed by the permeable cement treated base followed by the PCC slab). This is accomplished as follows.

Table 4-2.14. Table for Estimating Effective Modulus of Subgrade Reaction.

TRIAL SUBBASE: TYPE PERMEABLE CEMENT TRT.

THICKNESS (inches) 6

LOSS OF SUPPORT, LS 0.5

DEPTH TO RIGID FOUNDATION (feet) ____

PROJECTED SLAB THICKNESS (inches) 11

(1) MONTH	(2) ROADBED MODULUS M_R (psi)	(3) SUBBASE MODULUS, E_{SB} (psi)	(4) COMPOSITE k-VALUE (psi) (Fig. 3.3)	(5) k-VALUE (psi) ON RIGID FOUNDATION (Fig. 3.4)	(6) RELATIVE DAMAGE u_r (Fig. 3.5)
Jan.	50,000	100,000	1,500		80
Feb.	50,000	100,000	1,500		80
Mar.	8,000	100,000	600		120
Apr.	8,000	100,000	600		120
May	15,000	100,000	1,000		100
Jun.	15,000	100,000	1,000		100
Jul.	15,000	100,000	1,000		100
Aug.	15,000	100,000	1,000		100
Sep.	15,000	100,000	1,000		100
Oct.	15,000	100,000	1,000		100
Nov.	15,000	100,000	1,000		100
Dec.	15,000	100,000	1,000		100

Summation: $\Sigma u_r =$ 1,200

Average: $\bar{u}_r = \frac{\Sigma u_r}{n} = \underline{100}$

Effective Modulus of Subgrade Reaction, k (pci) = 1,000

Corrected for Loss of Support: k (pci) = 500

Table 4-2.15. DNPS86 Input Screens (14).

Screen #1

```
AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
         Version 1 - September 1986

   00000  00   00 00000   0000     0000    0000
   000000 00   00 000000 000000   000000 000000
   00   00 000 00 00   00 00   00   00   00 00   00
   00   00 000 00 00   00 00        00   00 00
   00   00 000000 000000 00000      0000   00000
   00   00 000000 00000    00000    0000   000000
   00   00 00 000 00          00  00   00 00   00
   00   00 00 000 00       00   00  00   00 00   00
   000000 00   00 00      000000  000000 000000
   00000  00   00 00       0000    0000    0000

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    American Association of State Highway
        and Transportation Officials

    444 N. Capitol Street, N.W., Suite 225
          Washington, D.C. 20001

        Press Any Key to Continue ...
```

Screen #2

```
             * * *  IMPORT/CREATE DATA FILE  * * *

DATA FILE TO IMPORT . . . . . . . . . . . . . in.dat
  This allows the user to import and
  edit an existing data file.  This
  may be left blank if a new file is
  to be created.

DATA FILE TO CREATE AND ANALYZE . . . . . . . in.dat
  If left blank, a default name
  (DNPSTMP.DAT) will be assumed.
```

Screen #3

```
            * * *  PROBLEM NUMBER AND DESCRIPTION  * * *

      PROBLEM NUMBER  . . . . . . . . . . . . . . . . .           1

      PROBLEM DESCRIPTION
      EXAMPLE 1

F1: HELP  F2: IMPORT/STORE  F3: ANALYZE/PRINT/EXIT  F4: DISPLAY RESULTS
```

Screen #4

```
          * * *  GENERAL DESIGN INPUT REQUIREMENTS  * * *

      ANALYSIS PERIOD (YEARS) . . . . . . . . . . . . .        30.0

      DISCOUNT RATE (PERCENT) . . . . . . . . . . . . .        4.00

      NUMBER OF TRAFFIC LANES (ONE DIRECTION) . . . . .           2

      LANE WIDTH (FEET) . . . . . . . . . . . . . . . .       12.00

      COMBINED WIDTH OF SHOULDERS (FEET, ONE DIRECTION)       16.00

F1: HELP  F2: IMPORT/STORE  F3: ANALYZE/PRINT/EXIT  F4: DISPLAY RESULTS
```

Table 4-2.15. (Cont.) DNPS86 Input Screens (14).

Screen #5

* * * ROADBED SOIL RESILIENT MODULI * * *

Season No.	Resilient Modulus (psi)	Season No.	Resilient Modulus (psi)
1	50000	13	0
2	50000	14	0
3	8000	15	0
4	8000	16	0
5	15000	17	0
6	15000	18	0
7	15000	19	0
8	15000	20	0
9	15000	21	0
10	15000	22	0
11	15000	23	0
12	15000	24	0

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

Screen #6

ROAD SURFACE
(P)aved or (A)ggregate . . . . P

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

Screen #7

* * * DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS * * *

DESIRED LEVEL OF RELIABILITY (PERCENT) 90.00

DESIGN TERMINAL SERVICEABILITY 3.00

ROADBED SOIL SWELLING AND/OR FROST HEAVE
Consider? (Y)es or (N)o . . . . N

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

Table 4-2.15. (Cont.) DNPS86 Input Screens (14).

Screen #8

```
PAVEMENT TYPE
  (F)lexible or (R)igid . . . . . . . . . . . . .              R









F1: HELP  F2: IMPORT/STORE  F3: ANALYZE/PRINT/EXIT  F4: DISPLAY RESULTS
```

Screen #9

```
           * * *  RIGID PAVEMENT DESIGN INPUTS  * * *

PERFORMANCE PERIOD FOR INITIAL PAVEMENT (YEARS) .        30.0

SERVICEABILITY INDEX AFTER INITIAL CONSTRUCTION .        4.50

TRAFFIC
  Growth Rate (percent per year)  . . . . . . . .        6.00
  (S)imple or (C)ompound Growth . . . . . . . . .           C
  Initial Yearly 18-kip ESAL (both directions)  .     1200000
  Directional Distribution Factor (percent) . . .          50
  Lane Distribution Factor (percent)  . . . . . .          80
  Calculated Total 18-kip ESAL During the
    Analysis Period (in the design lane)  . . . .    37947929

OVERALL STANDARD DEVIATION (LOG REPETITIONS)  . .       0.390



F1: HELP  F2: IMPORT/STORE  F3: ANALYZE/PRINT/EXIT  F4: DISPLAY RESULTS
```

Screen #10

```
     * * *  ADDITIONAL RIGID PAVEMENT DESIGN INPUTS  * * *
                    AND ASSOCIATED COSTS

SUBBASE
  Subbase Type  . . . . . . . . . . . . . . . . . .   PERMEABLE
  Thickness (inches)  . . . . . . . . . . . . . . .        6.00
  Elastic Modulus (psi) . . . . . . . . . . . . . .      100000
  Unit Cost ($/CY)  . . . . . . . . . . . . . . . .       30.00
  Salvage Value (percent) . . . . . . . . . . . . .          30

PORTLAND CEMENT CONCRETE SLAB
  Type of Construction  . . . . . . . . . . . . . .        JPCP
  PCC Elastic Modulus (psi) . . . . . . . . . . . .     4500000
  Average PCC Modulus of Rupture (psi)  . . . . . .         725
  Unit Cost of PCC ($/CY) . . . . . . . . . . . . .       70.00
  Salvage Value (percent) . . . . . . . . . . . . .          30

STRUCTURAL CHARACTERISTICS
  Load Transfer Coefficient . . . . . . . . . . . .        2.80
  Drainage Coefficient  . . . . . . . . . . . . . .        1.00
  Loss of Support Factor  . . . . . . . . . . . . .        0.50

F1: HELP  F2: IMPORT/STORE  F3: ANALYZE/PRINT/EXIT  F4: DISPLAY RESULTS
```

Table 4-2.15. (Cont.) DNPS86 Input Screens (14).

Screen #11

```
            * * *  ADDITIONAL RIGID PAVEMENT COSTS  * * *

   OTHER CONSTRUCTION RELATED COSTS
     Shoulders, If Not Full Strength ($/linear ft) .        30.00
     Drainage  ($/linear ft) . . . . . . . . . . . .         3.00
     Mobilization and Other Fixed Costs ($/lin ft) .         2.00

   MAINTENANCE COST
     Initial Year Costs Begin to Accrue  . . . . . .            0
     Yearly Increase ($/lane mile/year)  . . . . . .        50.00

F1: HELP  F2: IMPORT/STORE  F3: ANALYZE/PRINT/EXIT  F4: DISPLAY RESULTS
```

Screen #12

```
   * * *  PERFORM ANALYSIS, PRINT RESULTS OR EXIT  * * *

OPTIONS

   1. Perform Analysis
   2. Perform Analysis and Print Results
   3. Print Previous Results
   4. Return to Edit Session
   5. Exit

   Enter Desired Option  . . . . . . . . . . . . .        1
```

Table 4-2.16. DNPS86 Output Screens (14).

```
DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
             VERSION 1 - SEPTEMBER 1986

PROBLEM NO.       1                                       Page 2
  EXAMPLE 1


RIGID PAVEMENT DESIGN INPUTS

  Performance Period for Initial Pavement (years)           30.0

  Serviceability Index After Initial Construction           4.50

  Traffic
    Growth Rate (percent)                                   6.00
    Type of Growth                                      COMPOUND
    Initial Yearly 18-kip ESAL (both directions)        1200000.
    Directional Distribution Factor (percent)                50.
    Lane Distribution Factor (percent)                       80.

  Overall Standard Deviation (log repetitions)              .390

  Subbase
    Subbase Type                                       PERMEABLE
    Thickness (inches)                                      6.00
    Elastic Modulus (psi)                                100000.
    Unit Cost ($/CY)                                       30.00
    Salvage Value (percent)                                  30.

  Portland Cement Concrete Slab
    Type of Construction                                    JPCP
    PCC Elastic Modulus (psi)                           4500000.
    Average PCC Modulus of Rupture (psi)                    725.
    Unit Cost of PCC ($/CY)                                70.00
    Salvage Value (percent)                                  30.

  Structural Characteristics
    Load Transfer Coefficient                               2.80
    Drainage Coefficient                                    1.00
    Loss of Support Factor                                   .50

  Other Construction Related Costs
    Shoulders, If Not Full Strength ($/linear foot)        30.00
    Drainage ($/linear foot)                                3.00
    Mobilization and Other Fixed Costs ($/linear foot)      2.00

  Maintenance Cost
    Initial Year Costs Begin to Accrue                        .0
    Yearly Increase ($/lane mile/year)                     50.00
```

```
DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
             VERSION 1 - SEPTEMBER 1986

PROBLEM NO.       1                                       Page 3
  EXAMPLE 1


RIGID PAVEMENT STRUCTURAL DESIGN

  Effective Modulus of Subgrade Reaction (pci)              488.
  Subbase Type                                         PERMEABLE
  Subbase Thickness (inches)                                6.00
  Pavement Type                                             JPCP
  Required Slab Thickness (inches)                         11.05
  Performance Life (years)                                  30.0
  Allowable 18-kip ESAL Repetitions                    37947850.

LIFE-CYCLE COSTS ($/SY)

  Initial Construction                                     39.61
  Maintenance                                               1.68
  Salvage Value                                            -2.45

  First Overlay Construction                                 .00
  First Overlay Maintenance                                  .00
  First Overlay Salvage Value                                .00

  Second Overlay Construction                                .00
  Second Overlay Maintenance                                 .00
  Second Overlay Salvage Value                               .00

  Total Net Present Value                                  38.84
```

Table 4-2.16. (Cont.) DNPS86 Output Screens (14).

```
DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
             VERSION 1 - SEPTEMBER 1986

PROBLEM NO.      1                                        Page 1
  EXAMPLE 1


GENERAL DESIGN INPUT REQUIREMENTS

  Analysis Period (years)                                   30.0

  Discount Rate (percent per year)                          4.00

  Number of Traffic Lanes (one direction)                      2

  Lane Width (feet)                                         12.0

  Combined Width of Shoulders (feet, one direction)          16.

ROADBED SOIL RESILIENT MODULI

  Season:                 1       2       3       4       5       6
  Modulus (psi):     50000.  50000.   8000.   8000.  15000.  15000.

  Season:                 7       8       9      10      11      12
  Modulus (psi):     15000.  15000.  15000.  15000.  15000.  15000.

DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS

  Desired Level of Reliability (percent)                   90.00

  Design Terminal Serviceability                            3.00

  Roadbed Soil Swelling                          (Not Considered)

  Frost Heave                                    (Not Considered)
```

The roadbed soil is a gravelly clay, CL classification with approximately 50 percent finer than 0.02 mm. Figure 4-2.23 can be used to determine the average rate of heave to be 5 mm/day. Then given the depth of frost penetration of 3 ft. and the drainage quality of fair to poor the maximum serviceability loss due to frost heave is determined from Figure 4-2.24 to be 1.0.

The estimated percentage of the project that will experience frost heave is 50 percent based upon soil profile data.

Finally, an estimate of the loss of serviceability due to frost heave is determined using Figure 4-2.25. The result is approximately 0.5.

<u>REQUIRED SLAB THICKNESS</u> (loss of serviceability due to frost heave)

The total loss of serviceability allowable to traffic loads is

4.5 (Initial Value) - 3.0 (terminal value) - 0.5 (frost heave) = 1.0.

Using this loss of serviceability and the inputs given previously, the required slab thickness is 11.8 inches. Thus, theoretically the adverse effects of frost heave on loss of serviceability can be handled through a slightly thicker slab (0.8 inches). The thicker slab results in a smaller loss is serviceability (e.g., 0.5) due to traffic loadings, and thus this loss is available for frost heaving.

It is still recommended, however, that the effects of frost heave (and swelling soil) be handled through other means than increased slab thickness.

9.2 Determine the Joint Design For JPCP and JRCP

Joint design requires consideration of spacing, load transfer and sealant reservoir. The transverse joints for the jointed plain concrete pavement (JPCP) must be limited to control thermal curling stresses, especially when the slab is placed on a stabilized base layer with a high k-value. A maximum slab length of twice the slab thickness (2 * 11.0) or 22 ft. is recommended by the AASHTO Guide. However, due to the stiff support, a lesser thickness of 17 ft. (1.5 * 11) is recommended.

Load transfer at the transverse joints is dowels due to the high truck traffic level. A dowel diameter of 1/8 times the slab thickness is recommended, or 1 3/8 inches. Dowel spacing of 12 inches is recommended. The dowels must be coated to prevent corrosion of the steel.

Joint sealant reservoir dimensions depend upon the type of sealant specified. For long life, a preformed compression sealant is recommended. The width of the joint must be determined in accordance with manufacturers recommendations. Normally the sealant should be compressed between 20 and 50 percent of its nominal width. The sealant should be placed 1/8 to 1/4 inch below the surface of the pavement.

9.3 Determine Reinforcement Design for JRCP

If a jointed reinforced concrete pavement (JRCP) is to be considered, the minimum amount of reinforcement to hold transverse cracks tightly

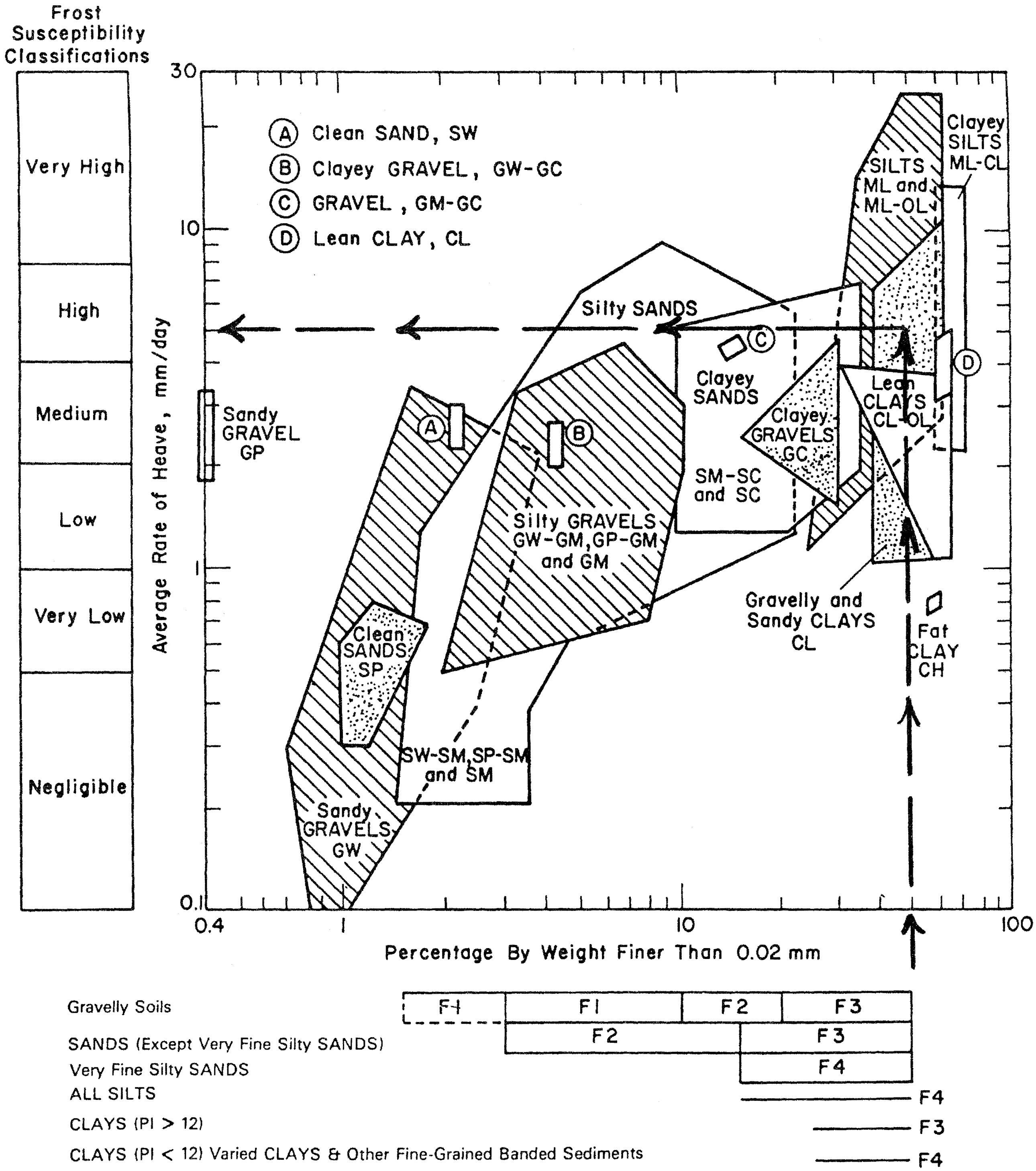

Figure 4-2.23. Chart for Estimating Frost Heave Rate for a Roadbed soil, Part II (11).

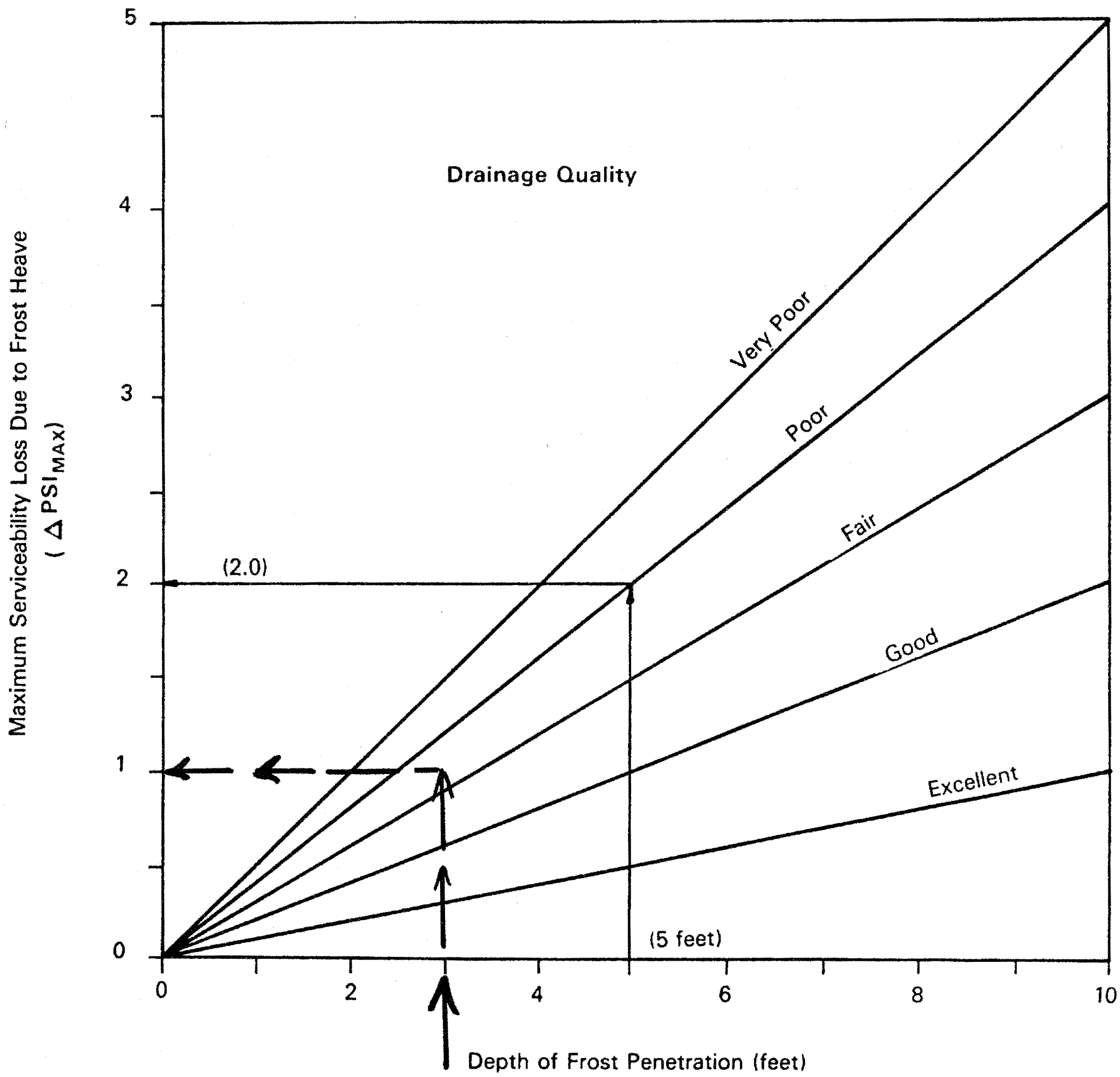

Figure 4-2.24. Graph for Estimating Maximum Serviceability Due to Frost Heave (5).

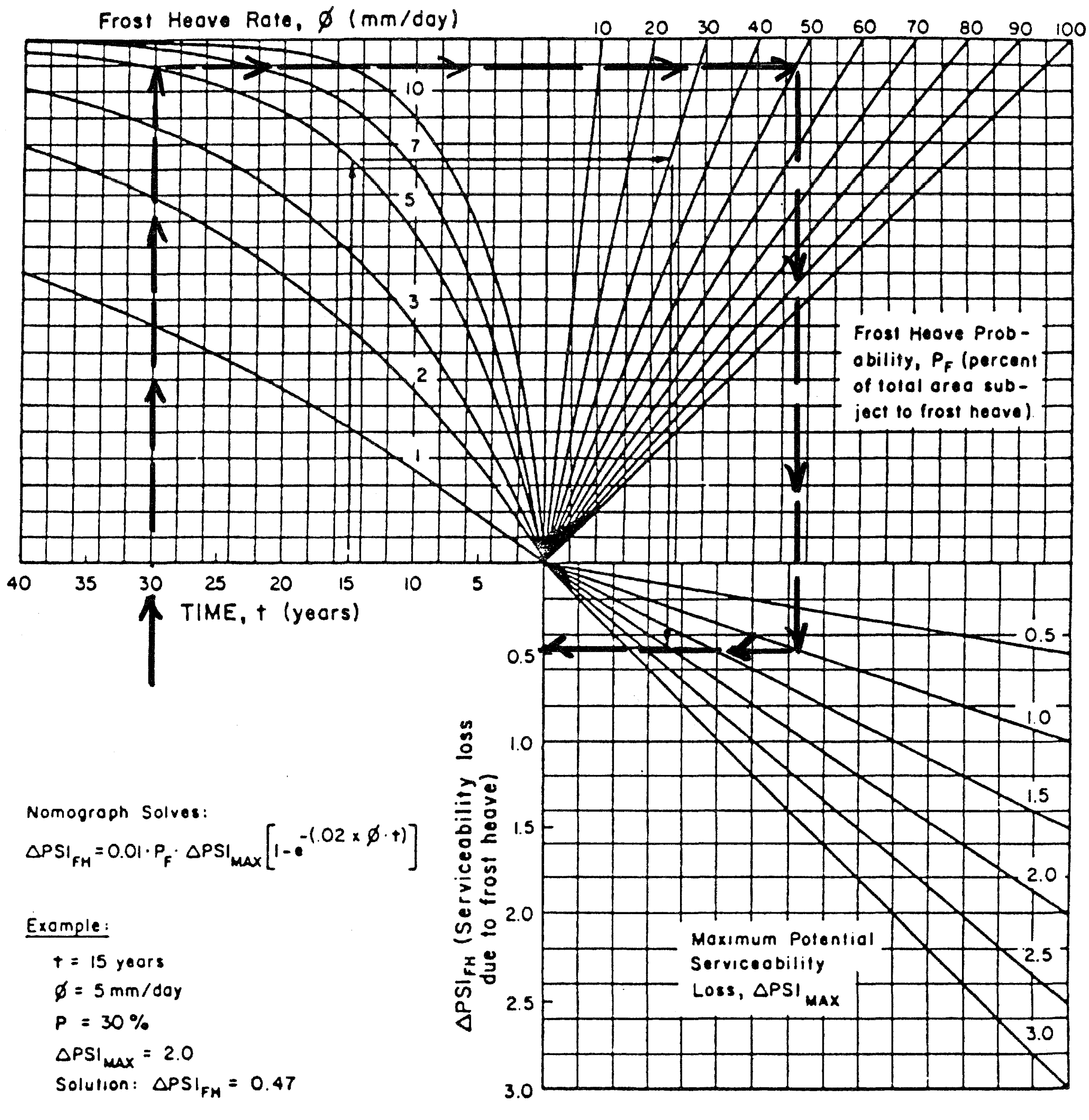

Figure 4-2.25. Chart for Estimating Serviceability Loss Due to Frost Heave (5).

together must be determined. Joint spacing can be lengthened beyond that of the JPCP because transverse cracks caused from shrinkage and thermal curling stresses will be held together by the reinforcement. However, joint spacing cannot be extended too much because of the difficulties of sealing joints and the amount of increased movement at the joints can create problems when incompressibles infiltrate.

No guidance is provided in the AASHTO Guide for joint spacing for JRCP. Joint spacings of 40 ft. have resulted in many joint failures from a build up of incompressibles. The shorter the joint spacing the better from a performance standpoint. Joint spacings of 27 ft. have worked well in some states and that is recommended for this design.

The amount of reinforcement recommended is determined as follows.

Slab length = 27 ft.

Steel working stress, f_s = 48,750 psi (0.75 f_y)
(welded smooth wire fabric)

Friction factor, F = 1.8 (stabilized base)

Entering this information in the following equation gives the minimum percentage of reinforcement required.

$$P_s = LF/2f_s = 0.05 \text{ percent}$$

9.4 Determine Shoulder Design

A tied PCC shoulder is recommended that has the same thickness as the traffic lane slab at the longitudinal joint. This thickness may be tapered somewhat to the outside edge.

9.5 Determine Subdrainage Design

A permeable layer of cement treated granular material is recommended to be placed immediately beneath the PCC slab. This layer will provide for rapid drainage of moisture that infiltrates through joints and cracks in the PCC slab. A filter layer of some type (either granular or geotextile fabric) must be provided to prevent the movement of fines from the roadbed soil into the permeable base layer.

A subdrainage pipe system is also recommended to carry the water from the permeable layer out of the pavement section.

10.0 LIMITATIONS OF THE AASHTO RIGID PAVEMENT DESIGN PROCEDURE

Major limitations of the AASHTO rigid pavement design procedure are summarized as follows:

1. Variability. A serious limitation of the AASHTO design procedure is that Equation 1, 2 and 3 are based upon very short pavement sections where construction and material quality was highly controlled. Typical highway projects which are normally several miles in length, contain much greater construction and material

variability, and hence show more variability in performance along the project in the form of localized failures. Projects designed using the Guide with average inputs would, therefore, have the tendency to show significant localized failures before the average project serviceability index drops to P_t, unless a level of reliability greater than 50 percent was selected for the design. Designing for a reliability greater than 50 percent can help overcome this limitation.

2. Limited materials and subgrade. The Road Test used a specific set of pavement materials and one roadbed soil. Other materials and soils will provide different performance. Part of this problem can be overcome through use of various inputs (k-value, F, drainage, loss of support).

3. Loss of Foundation Support. Many of the Road Test rigid sections showed severe pumping of the subbase. Therefore, Equations 1, 2 and 3 are heavily biased towards this condition. The effects of loss of support and drainage can now be adjusted to some extent for different conditions.

4. Short Road Test Performance Period. The number of years and heavy axle load applications upon which Equations 1, 2 and 3 are based represent only a fraction of the design age and load applications that exist on many pavements today over the design period (10 to 100 million 18-kip ESAL). Design periods under consideration usually range from 20 to 40 years. Even if these equations can be extrapolated for some additional number of load applications, there are several climatic effects that occur with time (as represented by age) to cause severe deterioration of the pavements even without heavy load applications (i.e., corrosion of steel, joint freeze-up, D-cracking, reactive aggregate, etc., some of which occurred on the AASHO sections left in service on I-80 until 1975). Therefore, in similar or more severe climates, the pavements would be expected to endure fewer load applications and fewer years than predicted by Equation 3 using average inputs. In mild climates, pavements would be expected to perform better than predicted.

5. Joint Design. Only one type of joint design was used at the Road Test. If other types are used, such as joints without dowels (as evidenced by the performance of the transverse cracks), or with some unusual type of load transfer devices, the pavement life would be significantly changed. The type of base would also affect load transfer and thus performance. Basic deficiencies in the joint design recommendations are little or no guidance for 1) joint spacing; 2) rational determination for dowel size and spacing 3) corrosive resistant dowels; 4) when mechanical LTDs are required; and 5) load transfer system other than dowels. Recommendations for considering load transfer by modifying the J factor are extremely rough and unsubstantiated.

6. Reinforcement Design. The mathematical expression used for longitudinal reinforcement design is a major simplification of the actual forces encountered. The most significant limitation arises

if the unrestrained slab length assumed in reinforcement design (i.e., distance between joints, L) is altered through a partial or complete sizing of one or more joints. This could cause a significant increase (double or more) in the steel stress, which may result in yielding or rupture of reinforcement at an intermediate crack between joints. Also, the loss of effective reinforcement through corrosion is not provided for in the procedure. It is expected, therefore, that long joint spacings in cold regions accompanied by joint seizure would result in rupture of the reinforcement with subsequent faulting and spalling of cracks.

7. Climate. Concrete pavement performance is not independent of climatic conditions, and there is evidence to indicate that climatic conditions could have a significant effect on pavement life (8). Since the Road Test was conducted over a period of only 2 years, climatic effects were not as significant as if the same traffic had been applied over a longer period of, say, 20 to 40 years. Steel corrosion requires several years to develop into a serious condition, so joint lockup and subsequent yielding of the steel reinforcement for JRCP pavements would logically not occur for at least several years after initial construction. Figure 4-2.26 shows the results of a life prediction model (8) developed from the Illinois COPES database where age and traffic data were available over both short and long time periods. An interaction between age and traffic can be observed in that at old ages and heavy traffic there is much greater pavement damage.

8. Load Equivalency Factors. The load equivalency factors relate specifically to the Road Test materials, pavement composition, climate, present serviceability index loss and subgrade soils. The accuracy of extrapolating them to other regions, materials and distresses, etc., is not known, but is questionable.

9. Limiting Criteria. The design procedure is based on terminal serviceability indices of 2.0 and 2.5. One study (10) has shown that it is very likely that jointed concrete pavement will require substantial maintenance by the time it reaches an average serviceability index of 2.5. Thus, higher terminal serviceabilities should be considered for higher traffic levels.

10. No Mixed Traffic. The AASHO Road Test accumulated traffic on each test section by operating vehicles with identical axle loads and axle configurations, as opposed to mixed traffic (different axle loads, configurations, etc.). The procedure of converting mixed traffic into equivalent 18-kip ESAL applications has never been field verified.

11. Lack Of Guidance On Some Design Inputs. The loss of support and drainage factors are very significant on influencing slab thickness, and there is very little guidance provided for their selection. The design reliability also has an extremely large effect on slab thickness and very little guidance is provided in selecting this factor.

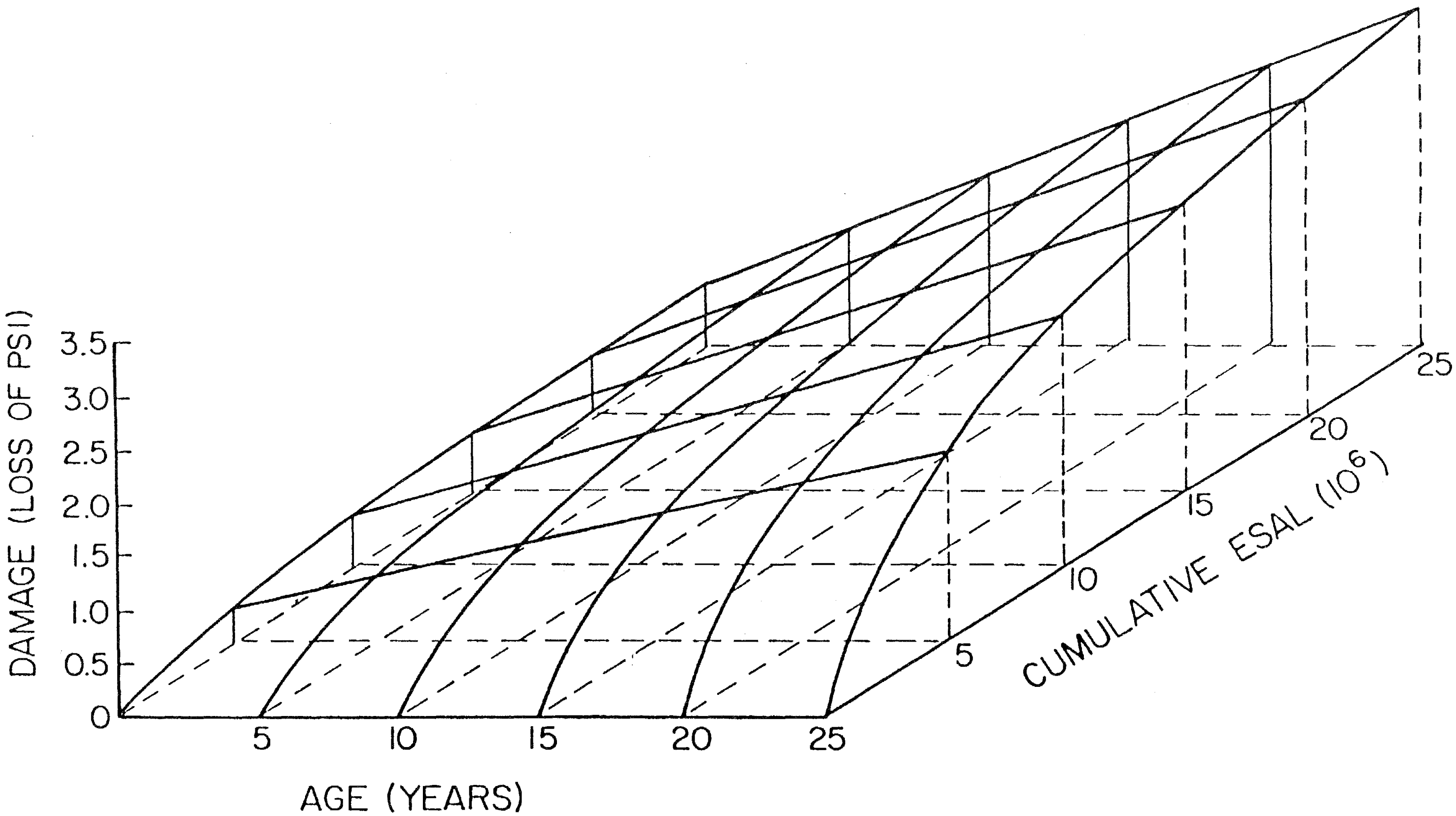

Figure 4 -2.26. Sensitivity of Illinois Damage (Serviceability) Model to Cumulative Load Repetitions and Age (Three-Dimensional Illustration) (8).

Successful use of the AASHTO Guide requires a lot of experience and knowledge of the assumptions and underlying basis for design. Pavement must be designed considering all features. Assumptions made during design should be clearly identified. It is strongly recommended that the resulting designs be checked using other procedures and mechanistic analyses.

11.0 SUMMARY

A brief summary of the development of the original rigid pavement design models from the AASHO Road Test was presented. This included discussion of the specific conditions that were present at the Road Test and how this was a contributing factor in the accuracy of the models obtained.

Recent changes and modifications to the original AASHO model was also given. This emphasized the recent revisions made to the AASHTO design procedure in 1986 by the AASHTO Task Force.

The various inputs required for design of rigid pavements with the AASHTO procedure was also discussed. This included definitions of the inputs as well as the procedure needed to obtain such inputs.

The design procedure for determining the required slab thickness was also described. This included considerations for roadbed swelling and frost heave.

As separate design procedures, the design of the joint and steel reinforcement were presented. For joint design, this included joint spacing, joint load transfer, joint forming, and reservoir dimensions. Reinforcement design was noted to be a function of slab length, steel working stress, and the friction factor for JRCP, and a function of concrete indirect tensile strength, concrete shrinkage, concrete thermal coefficient, reinforcing diameter, steel thermal coefficient, and design temperature drop for CRCP. However, for the latter, three limiting criteria also had to be considered: crack spacing, crack width, and steel stress.

A sensitivity analysis of a few key design inputs was presented and a detailed rigid pavement design was presented.

Finally, the limitations of the AASHTO rigid pavement design procedure was discussed in detail. It was noted that the use of the AASHTO design procedure requires much practical experience and much knowledge of the assumptions and limitations to ensure that a proper design is obtained.

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MODULE 4-3

OTHER RIGID PAVEMENT DESIGN METHODS

1.0 INSTRUCTIONAL OBJECTIVES

This module provides information on other selected rigid pavement design procedures, particularly those using the mechanistic approach. Upon completion of this module, the participants will be able to accomplish the following:

1. List available rigid pavement design procedures and determine if they utilize an empirical or a mechanistic approach to design.

2. List available rigid pavement structural analysis models and describe some of their capabilities.

3. For the PCA mechanistically based design procedure:

   a. List the basic assumptions and approach.
   b. List and describe the input variables including traffic, critical load placement, subgrade support, material properties and safety factor.
   c. Design the slab thickness for rigid pavements.
   d. Describe the recommendations provided for joint design and reinforcement design.
   e. List the advantages and disadvantages and state the weaknesses of the procedure.

4. Describe the advantages of conducting a design check of a rigid pavement design using one or more different procedures.

2.0 INTRODUCTION

2.1 Rigid Pavement Design and Analysis Procedures

Several rigid pavement design and analysis procedures have been developed, either through research studies or through experience derived by agencies over many years. Some design procedures that are of recent development or usage are:

DESIGN METHOD	REFERENCES	CURRENT VERSION	PAVEMENT TYPE
AASHTO Design Guide	9-10	1986	JPCP, JRCP, CRCP
Zero-Maintenance	12-15	1977	JPCP
California DOT	16	1985	JPCP
Portland Cement Assn.	17-20	1984	JPCP, JRCP
RPS-3 Texas SHDPT	21-22	1975	JRCP, CRCP
Associated Re-Bar Prod.	23-24	1981	CRCP
Illinois DOT	25-27	1982	CRCP
RISC	8	1982	JPCP, JRCP, CRCP
Jointed Conc. Shoulders	4	1978	JPCP

Some of the major rigid pavement analysis procedures that are of recent development or usage are:

ANALYSIS METHOD	REFERENCES	CURRENT VERSION	PAVEMENT TYPE
ILLISLAB	1-3	1986	JPCP, JRCP*, CRCP*
JSLAB	7	1984	JPCP, JRCP*, CRCP*
CRCP-3	5-6	1975	CRCP
RISC	8	1984	JPCP, JRCP*, CRCP*
PREDICT	11	1985	JPCP, JRCP

* Indirectly

A comprehensive evaluation of each of these design procedures and analysis procedures, which included both conceptual and analytical examinations, was recently conducted (28). Each of these procedures has strengths and weaknesses that contribute to or detract from its overall ability to adequately design and analyze a rigid pavement. A few of these design and analysis procedures have unique capabilities to analyze unusual design situations (e.g., widened lanes, concrete shoulders, joint spacing).

Some of the design procedures are empirically based and some are mechanistically based. (One procedure contains both empirical and mechanistic design approaches.)

1. Empirically based (design for serviceability):
   - AASHTO Design Guide
   - California DOT
   - Zero-Maintenance
   - RPS-3 Texas SHDPT
   - Associated Reinforcing Bar Producers
   - Illinois DOT

2. Mechanistically based (design for fatigue damage):
   - Zero-Maintenance
   - Portland Cement Association
   - RISC
   - PCA

A brief description of a few of the design and analysis procedures is given. A detailed description of the PCA mechanistic design procedure is presented to provide a comparison to the empirical AASHTO Design Guide approach.

2.2 Zero-Maintenance Design Procedure (12-15)

The term "zero-maintenance" refers to the concept of designing a pavement so that it would not need substantial structural maintenance over a certain design period. "Zero-maintenance" design is desirable for heavily trafficked pavements. This computerized procedure is based upon both empirical serviceability and mechanistic fatigue damage for JPCP only.

1. Serviceability. A predictive model was developed using data from the AASHO Road Test, the extended AASHO Road Test and 12 other

in-service pavement projects in different states in the four major climate zones. This model predicted the performance of those pavements better than the AASHO Road Test model developed from only two years of data.

2. Fatigue damage. A comprehensive fatigue damage analysis was developed that considers traffic axle load, type and number of applications, lateral variation of trucks in the traffic lane, proportion of trucks using the design lane, variability of the PCC modulus of rupture and its change over time, thermal curling stresses during the night and day and erodibility along the slab edge. The computed fatigue damage number using Miner's model was correlated with slab cracking as measured in the field. This provided some validation of the procedure and the capability to design for different levels of slab cracking.

2.3 RPS-3 Texas SHDPT Design Procedure (21-22)

This procedure is basically a computerized version of the 1972 version of the AASHTO Interim Guide, but contains a few differences and some important additions. The computer program can generate designs for JRCP and CRCP with AC or PCC overlays, and provides detailed cost information for economic comparison of alternatives. The output of the program is a summary table presenting up to 23 alternate pavement designs in order of increasing total overall present worth cost. The selection of the optimal design is based on the minimum total cost. Different design rehabilitates can be selected in a manner similar to that presented in the 1986 AASHTO Design Guide.

2.4 RISC Design and Analysis Procedure (8)

This mechanistic procedure is based upon coupling of a finite element slab resting on a multilayer elastic solid foundation of up to three discrete layers. The program considers up to three slabs in a row with or without shoulders, and such parameters as joint spacing, joint width, the effect of dowel bars and tie bars, load location, voids and partial contact between slab and supporting layers, and thermal curling stresses. The program requires a large amount of mainframe computer time and is expensive to use for certain types of designs.

2.5 ILLISLAB Finite Element Analysis Program (1-3)

This program computes stresses and deflections for rigid pavement slabs. It has the following overall capabilities:

1. Multiple wheel and axle loads in any configuration, located anywhere on the slab.

2. A combination of slabs such as multiple traffic lanes, traffic lanes and shoulders, or a series of transverse cracks such as in CRCP.

3. Jointed concrete pavements with longitudinal and transverse cracks with various load transfer systems.

4. Variable subgrade support including complete loss of support over any specified portion of the slab.

5. Concrete shoulders with or without tie bars.

6. Pavement slabs with a stabilized or lean concrete base, or asphalt or concrete overlay, assuming either perfect bond or no bond between the two layers.

7. Concrete slabs of varying thicknesses and moduli of elasticity, and subgrades with varying moduli of support.

8. A linear temperature gradient through the PCC slab for a single-layer pavement system.

2.6 JSLAB Finite Element Analysis Program (7)

JSLAB computes stresses and deflections in the PCC slab. The program has identical capabilities as ILLISLAB with the addition of the following:

1. Consideration of non-uniformly spaced dowels across the longitudinal and/or transverse joints.

2. Consideration of non-circular load transfer devices.

2.7 CRCP-3 Mechanistic Analysis Program (5-6)

This program can be used to determine the effects of drying shrinkage, uniform temperature drop and interior wheel load stress in CRCP. The program computes the following:

1. Final crack spacing and width.

2. Maximum concrete and steel stresses.

3. Changes in steel and concrete stresses, friction forces plotted along the horizontal stations of the slab.

4. Variations of concrete strength, concrete stress, steel stress, drying shrinkage, crack width and the changes of crack spacing with time.

3.0 PORTLAND CEMENT ASSOCIATION (17-20)

The Portland Cement Association's (PCA) thickness design procedure for concrete highways and streets was published in 1984, revising a procedure that has been used since 1966.

In this section the new design procedure is presented. One aspect of the new procedure is that an erosion analysis is applied in addition to the fatigue analysis. The erosion analysis recognizes that pavements can fail due to excessive pumping, erosion of the foundation, and joint faulting. The fatigue analysis recognizes that pavements can fail in fatigue due to excessive load repetitions.

The new design procedure is based on a comprehensive mechanistic analysis of concrete stresses and deflections at pavement joints, corners, and edges by a finite-element computer program. The program models joint load transfer provided by dowels or by aggregate interlock and the effects of concrete shoulders.

3.1 Analysis of Concrete Pavements

A finite element computer program called JSLAB (7), has been developed to analyze jointed concrete pavement sections. Joints may be modeled with load transfer systems of dowels, aggregate interlock, or keyways. JSLAB can also be used to evaluate the effect of joints with nonuniformly spaced load transfer devices.

For doweled joints, dowel properties such as diameter and modulus of elasticity are input directly. For aggregate interlock and keyway joints, a spring stiffness value is required. This value represents the load deflection characteristics of the joints. The stiffness value can be determined from field or laboratory tests.

Representative slab systems that can be analyzed are shown in Figure 4-3.1. A finite element representation of a jointed slab is shown in Figure 4-3.2.

The computer program JSLAB has been verified with closed form solutions (i.e., solutions that have exact mathematical expressions).

3.2 Design Considerations

3.2.1 Truck Load Placement

Based on data from the finite-element computer program analysis, as well as AASHO Road Test data, PCA researchers concluded that axle loads at the outside pavement edge created more severe conditions than any other load position. As the truck placement moves inward a few inches from the edge, the effects decrease substantially.

Only a small fraction of trucks run with their outside wheels placed at the edge, as shown by the data tabulated in Figure 4-3.3. The term "percent trucks at edge" is defined as the percent of trucks that are travelling with the outside edge of the contact area of the outside tire adjacent to or beyond the pavement edge. Most of the trucks travelling the pavement are driven with their outside wheel placed about two feet from the edge.

At increasing distances inward from the pavement edge, the frequency of the load applications increases while the magnitudes of stress and deflection decrease. Data on truck placement distribution and distribution of stress and deflection due to loads placed at and near the pavement edge are difficult to use directly in a design procedure. As a result, these distributions were analyzed and more easily applied techniques were prepared for design purposes.

For the fatigue analysis, fatigue is computed incrementally at fractions of an inch from the slab edge for different truck distributions. This gave the equivalent edge stress factors shown in Figure 4-3.4.

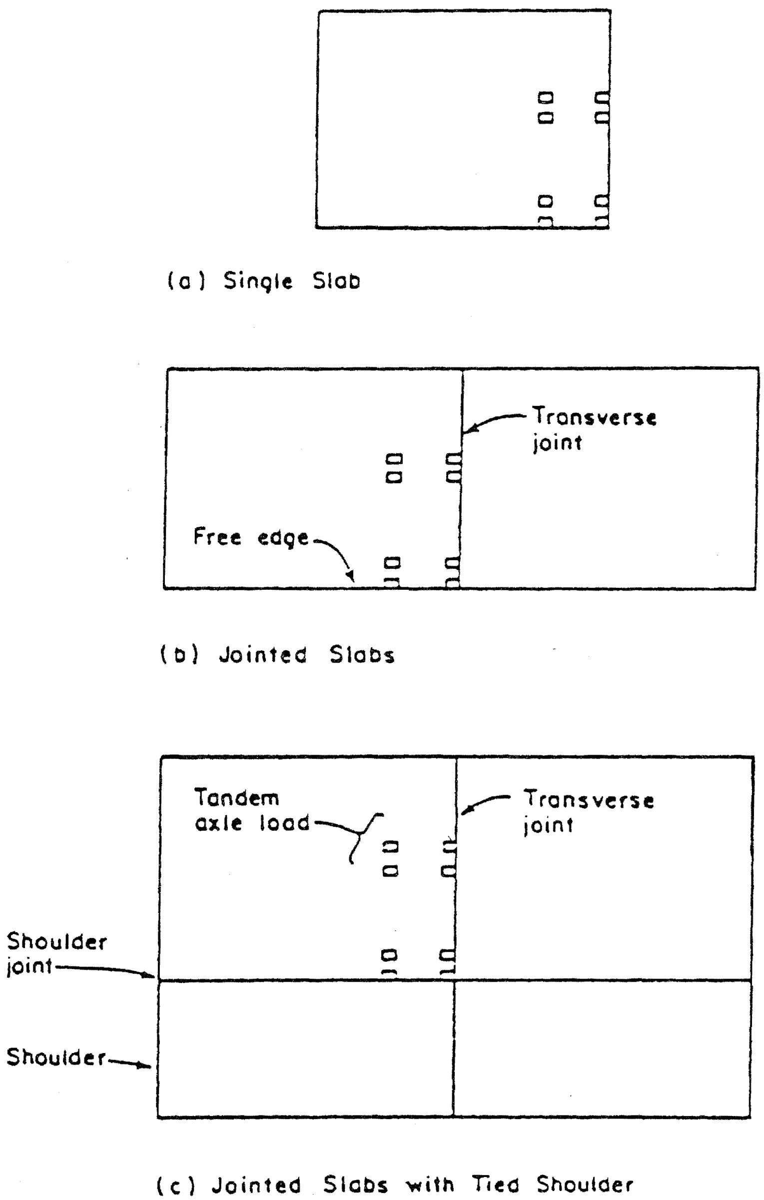

Figure 4-3.1. Typical Slab Systems.

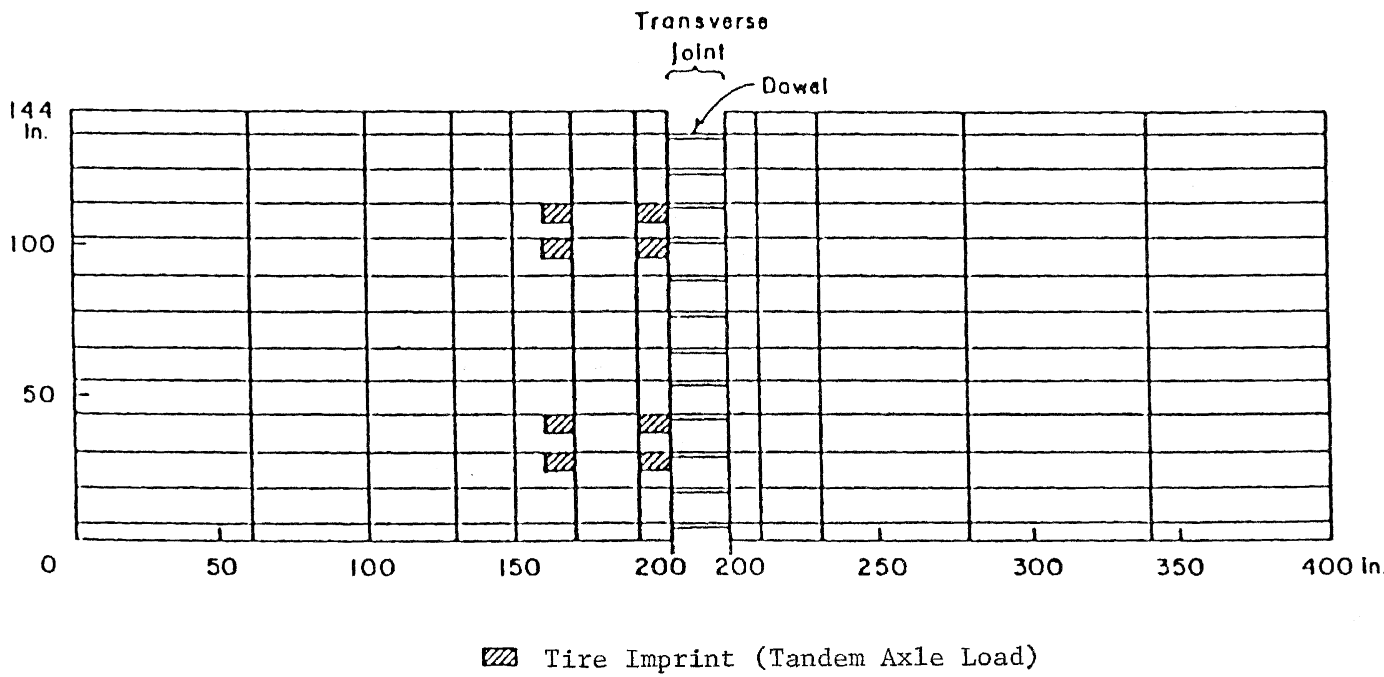

Figure 4-3.2. Finite Element Representation of Jointed Slab System.

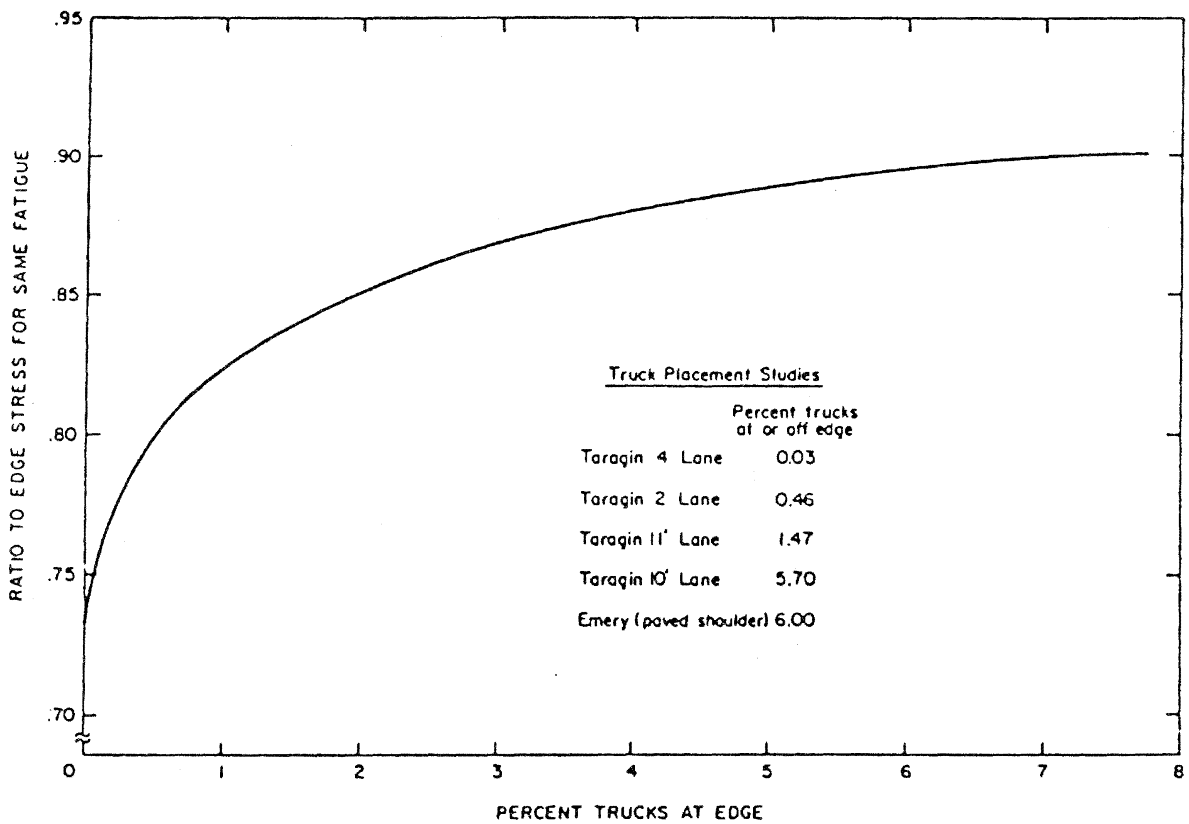

Figure 4-3.3. Relationship of Edge Stress Factor to Percent Edge Loadings.

Project	TST's per Day[1]	New Design Slab Thickness (in.)	Subbase	Drainage	SVF[2]	PSI[3]	Fault[4]	
							15 ft.	20 ft.[5]
One-A	1650	12.2	granular	good	6	2.8	3.4	4.4
		12.2	granular	poor	29	2.1	4.6	6.0
		10.5	cement tr.	good	NA[6]	NA	4.9	5.1
		10.5	cement tr.	poor	NA	NA	6.8	7.1
Urb-Hi	8750	14.0	granular	good	14	2.5	4.0	5.5
		14.0	granular	poor	83	1.5	5.2	7.3
		12.3	cement tr.	good	NA	NA	5.5	5.8
		12.3	cement tr.	poor	NA	NA	7.7	8.2

(1) Tractor-semi-trailer trucks
(2) Pavement roughness (slope variance due to faulting)
(3) Present Serviceability Index
(4) Joint faulting, 32nds of an inch
(5) Joint spacing
(6) Not applicable

Figure 4-3.4. Comparison of Erosion Analysis Results With Actual Faulting on Undowelled Pavements.

This factor, when multiplied by edge load stress, gives the same degree of fatigue consumption that would result from a given truck placement distribution. The factor, 0.894 for 6 percent truck encroachment, has been incorporated in the design tables.

For the erosion analysis, the worst cases involve erosion at the slab corner due to trucks placed either at the corner, when there is no concrete shoulder, or inward from the corner, when there is a concrete shoulder. For design purposes, the most severe case, 6 percent trucks at the edge, is assumed. This is incorporated in the design tables and charts.

3.2.2 Development of Erosion Analysis

Current mechanistic design procedures are based on the principle of limiting the flexural stresses in a slab to safe values. This is done so that fatigue cracking due to load repetitions is avoided. Safety factors or other aspects of design are selected so that design results agree with pavement performance as much as possible.

For some time it has been apparent that there is an important mode of distress, in addition to fatigue cracking, that needs to be addressed in the design process. This is the erosion of material beneath and beside the slab. Many repetitions of heavy axle loads at slab corners and edges cause pumping, erosion of subgrade, subbase, and shoulder materials, voids under and adjacent to the slab, and faulting of pavement joints, especially in pavements with undoweled joints.

These particular pavement distresses are considered to be more closely related to pavement deflections than to flexural stresses. Thus, it would seem that a starting point in the development of a mechanistic design procedure to control erosion should be based on the magnitude of repeated deflections at slab joints, corners, and edges.

Attempts to correlate deflections to AASHO Road Test performance and to faulting studies were not successful, even though the principal mode of failure of concrete pavements at the AASHO Road Test was pumping of granular subbase from under the slabs. It was found that, to be able to predict the AASHO Road Test performance, different values of deflection criteria would have to be applied to different slab thicknesses, and to a small extent, different foundation moduli (k-values).

Better correlation was obtained by multiplying the computed corner deflection values (w) by computed pressure values (p) at the slab-foundation interface.

Guided by the concept that power, or rate of work, with which an axle load pounded the slab might be the key, the above pw (in.-lb) was divided by a measure of the length of the deflection basin (radius of relative stiffness, in.). The concept is that a thin pavement with its shorter deflection basin receives a faster load punch than a thicker slab. That is, at equal pw's and equal truck speed, the thinner slab is subjected to a faster rate of work, or power (in-lb/sec). Successful correlation to road test performance was obtained with the following expression:

$$\text{Power} = 268.7\ (p^2/h)\ k^{(-0.73)}$$

where:

h = Slab thickness, inches
k = Modulus of subgrade reaction, pci
p = Estimated pressure at the slab foundation interface, psi

Included in the development of this erosion analysis were attempts to correlate the design results with studies on joint faulting. In these studies, the degree of joint faulting and serviceability of pavements was related to pavement thickness, heavy truck count, and other factors. These studies included pavements in Wisconsin, Minnesota, North Dakota, Georgia, and California, and included a range of variables not found at the AASHO Road Test. These include a greater number of trucks, undoweled pavements, a wide range of years of pavement service, and stabilized subbases. It was intended that, if possible, the erosion analysis be developed to fit the Road Test data and the results of these faulting studies.

This resulted in the development of the following equation:

$$\log N = 14.524 - 6.777 (C_L P - 9.0)^{0.103}$$

where:

N = Allowable load repetitions to the end of the design period
P = Power as defined above in terms of h and k
C_L = Subbase adjustment factor

The constants in the equation were selected to make the expression fitting the Road Test data also fit the faulting studies; C_L is an adjustment that has a value close to 1.00 for normal subbases and decreases to about 0.90 for high-strength subbases.

The equation for erosion damage is:

$$\text{Percent Erosion Damage} = 100 \Sigma n_i (C/N_i)$$

where:

n_i = Expected number of axle load repetitions in axle group i
N_i = Allowable number of repetitions in axle group i
C = (for 6 percent trucks at edge) 0.06 for pavements without shoulder, and 0.94 for pavements with concrete shoulder

Design charts show allowable N values as C/N_i, to save a design calculation step.

Figure 4-3.4 shows pavement thickness derived from the erosion criterion compared with terminal roughness due to faulting (SVF), terminal serviceability (PSI), and terminal joint faulting; the latter three items were determined from the two faulting studies. Generally accepted terminal values are: SVF of 11 to 14, PSI of 2.0 to 2.5, and faulting of 4 to 7 thirty-seconds of an inch.

It can be seen that the design thicknesses, when incorporated as inputs to the equations from the other studies, do approximately predict the appropriate terminal conditions.

The data in Figure 4-3.4 suggest that "good" and "poor" used in these studies refer to whether the pavement foundation is predominantly in a free-draining, dry condition, or in a poorly drained, wet condition. So far, this aspect of design has not been included in the design procedure but it deserves further study.

The erosion analysis is suggested for use as a guideline. It may be modified based on local experience, since climate, drainage, local factors, and new design innovations may have an influence. Accordingly, the 100 percent erosion damage criterion, an index number correlated to general performance experience, may be increased or decreased based on specific performance data gathered in the future for more favorable or more adverse conditions.

3.2.3 Variation in Concrete Strength

The variation of concrete strength is considered in the design procedure. Expected ranges of variations in the concrete's modulus of rupture have far greater effect than the usual variations in the properties of other materials (subgrade and subbase strength, layer thicknesses, etc.). Variation in concrete strength is introduced by reducing the modulus of rupture by one standard deviation.

For design purposes, a coefficient of variation of 15 percent is assumed and is incorporated into the design charts and tables. The user does not directly apply this effect. The value of 15 percent represents fair-to-good quality control, and, combined with other effects, was selected as being realistic and giving reasonable design results.

3.2.4 Concrete Strength Gain With Age

The 28-day flexural strength (modulus of rupture) is used as the design strength. However, the procedure incorporates the effect of concrete strength gain after 28 days. The user does not directly apply this factor but simply inputs the 28-day value as the design strength.

3.2.5 Fatigue

The flexural fatigue criterion used in the procedure is shown in Figure 4-3.5 and, except in the high load repetition range, is the same as that used in the previous PCA method. This change, which usually does not have much effect on design thickness, was made to eliminate the discontinuity in the old PCA curve that sometimes caused unrealistic effects.

The allowable number of load repetitions for a given axle load is determined based on the stress ratio (flexural stress divided by the 28-day modulus of rupture, ASTM C78). The designer does not need to determine the fatigue life from Figure 4-3.5 directly; the fatigue curve is incorporated into the design nomograph.

Use of the fatigue criterion is made on Miner's hypothesis that fatigue resistance not consumed by repetitions of one load is available for repetitions of other loads. Theoretically, the total fatigue consumed should not exceed 100 percent.

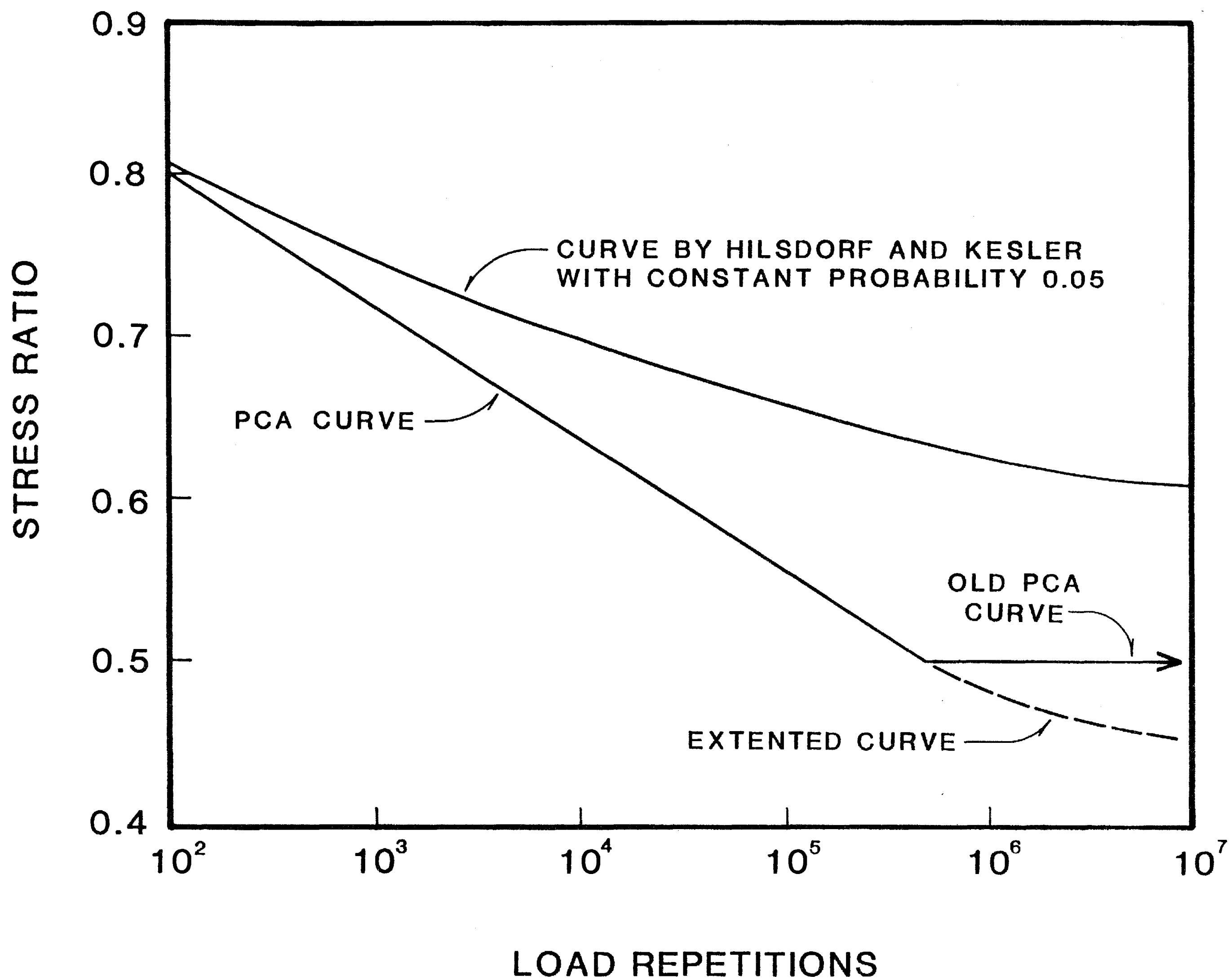

Figure 4-3.5. Old and New PCA Curves.

Combined with the effect of reducing the design modulus of rupture by one standard deviation, the fatigue criterion is considered to be conservative for design purposes.

3.2.6 Warping and Curling of Concrete

The PCA procedure does not include curling and warping stresses in its analyses. The exclusion of these stresses can produce a non-conservative design. Recent studies have shown that these stresses contribute to the wheel load stress developing under traffic which can produce a stress ratio for fatigue life that is greater than with the wheel load alone.

3.2.7 Tridem Axles

While the conventional single-axle and tandem-axle truck configurations are still the predominant loads on U. S. highways, use of triple axles is increasing. These are seen on some over-the-road trucks and on special haul roads for coal trucks.

The triple axle may be more damaging from an erosion (deflection) standpoint than from a fatigue standpoint. Analysis of these triple axles can be performed in the PCA procedure, using information provided in Appendix C of the PCA Manual (17).

3.2.8 Lean Concrete Subbase

Appendix B of the PCA manual (17) provides guidance on designing a concrete pavement with a lean concrete layer, either as a subbase or as the lower layer in a monolithic slab. Lean concrete is stronger than conventional subbase materials and is considered to be nonerodable. Use of lean concrete will therefore permit a reduction in the surface slab thickness design.

3.3 Design Procedure

Figure 4-3.6 shows the format for working design problems. It requires a projection of the weights and volumes of axle loads that will traffic the pavement over a selected design period.

The weights of axle loads are multiplied by a load safety factor (LSF = 1.0, 1.1, or 1.2) which is selected by the designer depending on the type of facility ranging from a light-traffic secondary road on up to a busy, multi-lane highway.

Both a stress-fatigue analysis and an erosion analysis are shown on the design worksheet. The fatigue analysis will usually control the design of light-traffic pavements (residential streets and secondary roads) with or without dowels, and medium-traffic pavements with dowels. The erosion analysis will usually control the design of medium- and heavy-traffic pavements with undowelled (aggregate interlock) joints and heavy-traffic pavements with dowelled joints.

Calculation of Pavement Thickness

Project Design 1A, four-lane Interstate, rural

Trial thickness 9.5 in.
Subbase-subgrade k 130 pci
Modulus of rupture, MR 650 psi
Load safety factor, LSF 1.2

Doweled joints: yes ✓ no
Concrete shoulder: yes no ✓
Design period 20 years

4 in. untreated subbase

Axle load, kips	Multiplied by LSF 1.2	Expected repetitions	Fatigue analysis: Allowable repetitions	Fatigue analysis: Fatigue, percent	Erosion analysis: Allowable repetitions	Erosion analysis: Damage, percent
1	2	3	4	5	6	7

8. Equivalent stress 206
9. Stress ratio factor 0.317
10. Erosion factor 2.59

Single Axles

30	36.0	6,310	27,000	23.3	1,500,000	0.4
28	33.6	14,690	77,000	19.1	2,200,000	0.7
26	31.2	30,140	230,000	13.1	3,500,000	0.9
24	28.8	64,410	1,200,000	5.4	5,900,000	1.1
22	26.4	106,900	Unlimited	0	11,000,000	1.0
20	24.0	235,800	"	0	23,000,000	1.0
18	21.6	307,200	"	0	64,000,000	0.5
16	19.2	422,500			Unlimited	0
14	16.8	586,900			"	0
12	14.4	1,837,000			"	0

11. Equivalent stress 192
12. Stress ratio factor 0.295
13. Erosion factor 2.79

Tandem Axles

52	62.4	21,320	1,100,000	1.9	920,000	2.3
48	57.6	42,870	Unlimited	0	1,500,000	2.9
44	52.8	124,900	"	0	2,500,000	5.0
40	48.0	372,900	"	0	4,600,000	8.1
36	43.2	885,800			9,500,000	9.3
32	38.4	930,700			24,000,000	3.9
28	33.6	1,656,000			92,000,000	1.8
24	28.8	984,900			Unlimited	0
20	24.0	1,227,000			"	0
16	19.2	1,356,000				
			Total	62.8	Total	38.9

Figure 4-3.6. PCA Design Worksheet.

The figures to be used with the design worksheet are:

Concrete Shoulder	Joint Load Transfer	Fatigue Analysis	Erosion Analysis
No	Dowels	8a, 9	10a, 11
No	Aggregate Interlock	8a, 9	10b, 11
Yes	Dowels	8b, 9	12a, 13
Yes	Aggregate Interlock	8b, 9	12b, 13

3.4 Example Problem

The following example will be done manually as well as demonstrate PCAPAV (29) a computerized version of the design procedure. Figure 4-3.7 is a blank worksheet provided for this example problem.

Given:

1. Traffic during design period: 5,000,000 ESAL (685/day).
2. Subgrade k = 125 pci.
3. Concrete flexural strength Mr : 700 psi.
4. Joint spacing = 40 ft.
5. Dowel bars: 1.25 in. diam. spaced at 12 in. intervals.
6. No shoulders.

Find:

Required concrete thickness by the PCA method.

Solution:

1. A rigorous traffic analysis requires the use of loadometer tables; however, assume that the road will be used by single axle trucks loaded to 18 kips/axle, so that the ESAL = 685 per day.
2. A load safety factor of 1.2 is recommended by the procedure for high volume roads.
3. Assume a trial thickness of 8 in.
4. Enter 18,000 in column 1, single axles on Figure 4-3.7.
5. Multiply 18,000 by the load safety factor (1.2) and enter 21,600 in column 2.
6. Enter 5,000,000 in column 3.

Calculation of Pavement Thickness

Project ______________________________

Trial thickness ____________ in.
Subbase-subgrade *k* ____________ pci
Modulus of rupture, MR ____________ psi
Load safety factor, LSF ____________

Doweled joints: yes ____ no ____
Concrete shoulder: yes ____ no ____

Design period ______ years

Axle load, kips	Multiplied by LSF	Expected repetitions	Fatigue analysis		Erosion analysis	
			Allowable repetitions	Fatigue, percent	Allowable repetitions	Damage, percent
1	2	3	4	5	6	7

8. Equivalent stress ________
9. Stress ratio factor ________

10. Erosion factor ________

Single Axles

11. Equivalent stress ________
12. Stress ratio factor ________

13. Erosion factor ________

Tandem Axles

			Total		Total	

Figure 4-3.7. Blank Worksheet for Example Problem.

7. Refer to Figure 4-3.8a (no concrete shoulders) and determine the equivalent stress in an 8 in. pavement due to a single axle load. Note that this table gives equivalent stress values for subgrade k of 100 and 200; it is therefore necessary to interpolate this value for subgrade k of 125. From Figure 4-3.8a the equivalent stress for k = 100 is 275 psi, and for k = 200 is 243 psi. A linear interpolation results in 267 psi for k = 125. Enter this value on line 8 of Figure 4-3.7.

8. Determine the stress ratio.
   SR = Equivalent stress/Modulus of rupture
   = 267/700
   = 0.381

9. Referring to Figure 4-3.9 locate 21,600 (from step 5) on the left most (single axle load) scale.

10. Locate 0.381 on the stress ratio factor scale.

11. Connect a line between the points determined in steps 9 and 10 and extend this line until it intersects the allowable load repetitions scale. Note that in this case the intersection occurs at approximately 6 million applications. Enter this value under column 4 of Figure 4-3.7.

12. Refer to Figure 4-3.10a (doweled joints, no shoulders) and determine the erosion factors for k = 100 and k = 200, i.e., = 2.82 and 2.80 respectively. Since the difference in these factors is small, an erosion factor = 2.82 can be used for k = 125. Enter this value on line 10 of Figure 4-3.7.

13. Referring to Figure 4-3.11 locate 22.6 kips on the leftmost single axle load) scale. Use Figure 4-3.12 and Figure 4-3.13 if shoulders are present.

14. Locate 2.82 on the erosion factor scale.

15. Draw a straight line through the points located in steps 13 and 14 and extend this line until it intersects the allowable load repetitions scale. This intersection occurs at approximately 5,000,000 repetitions. Enter this value in column 6 of Figure 4-3.7.

16. Calculate the fatigue and damage percents as:
   FP = Col 3/Col 4
   = (5,000,000/6,000,000) x 100
   = 83%
   DP = Col 3/Col 6
   = (5,000,000/5,000,000) x 100
   = 100

Slab thickness, in.	k of subgrade-subbase, pci						
	50	100	150	200	300	500	700
4	825/679	726/585	671/542	634/516	584/486	523/457	484/443
4.5	699/586	616/500	571/460	540/435	498/406	448/378	417/363
5	602/516	531/436	493/399	467/376	432/349	390/321	363/307
5.5	526/461	464/387	431/353	409/331	379/305	343/278	320/264
6	465/416	411/348	382/316	362/296	336/271	304/246	285/232
6.5	417/380	367/317	341/286	324/267	300/244	273/220	256/207
7	375/349	331/290	307/262	292/244	271/222	246/199	231/186
7.5	340/323	300/268	279/241	265/224	246/203	224/181	210/169
8	311/300	274/249	255/223	242/208	225/188	205/167	192/155
8.5	285/281	252/232	234/208	222/193	206/174	188/154	177/143
9	264/264	232/218	216/195	205/181	190/163	174/144	163/133
9.5	245/248	215/205	200/183	190/170	176/153	161/134	151/124
10	228/235	200/193	186/173	177/160	164/144	150/126	141/117
10.5	213/222	187/183	174/164	165/151	153/136	140/119	132/110
11	200/211	175/174	163/155	154/143	144/129	131/113	123/104
11.5	188/201	165/165	153/148	145/136	135/122	123/107	116/98
12	177/192	155/158	144/141	137/130	127/116	116/102	109/93
12.5	168/183	147/151	136/135	129/124	120/111	109/97	103/89
13	159/176	139/144	129/129	122/119	113/106	103/93	97/85
13.5	152/168	132/138	122/123	116/114	107/102	98/89	92/81
14	144/162	125/133	116/118	110/109	102/98	93/85	88/78

Figure 4-3.8a. Equivalent Stress - No Concrete Shoulder (Single Axle/Tandem Axle)

Slab thickness, in.	k of subgrade-subbase, pci						
	50	100	150	200	300	500	700
4	640/534	559/468	517/439	489/422	452/403	409/388	383/384
4.5	547/461	479/400	444/372	421/356	390/338	355/322	333/316
5	475/404	417/349	387/323	367/308	341/290	311/274	294/267
5.5	418/360	368/309	342/285	324/271	302/254	276/238	261/231
6	372/325	327/277	304/255	289/241	270/225	247/210	234/203
6.5	334/295	294/251	274/230	260/218	243/203	223/188	212/180
7	302/270	266/230	248/210	236/198	220/184	203/170	192/162
7.5	275/250	243/211	226/193	215/182	201/168	185/155	176/148
8	252/232	222/196	207/179	197/168	185/155	170/142	162/135
8.5	232/216	205/182	191/166	182/156	170/144	157/131	150/125
9	215/202	190/171	177/155	169/146	158/134	146/122	139/116
9.5	200/190	176/160	164/146	157/137	147/126	136/114	129/108
10	186/179	164/151	153/137	146/129	137/118	127/107	121/101
10.5	174/170	154/143	144/130	137/121	128/111	119/101	113/95
11	164/161	144/135	135/123	129/115	120/105	112/95	106/90
11.5	154/153	136/128	127/117	121/109	113/100	105/90	100/85
12	145/146	128/122	120/111	114/104	107/95	99/86	95/81
12.5	137/139	121/117	113/106	108/99	101/91	94/82	90/77
13	130/133	115/112	107/101	102/95	96/86	89/78	85/73
13.5	124/127	109/107	102/97	97/91	91/83	85/74	81/70
14	118/122	104/103	97/93	93/87	87/79	81/71	77/67

Figure 4-3.8b. Equivalent Stress - Concrete Shoulder (Single Axle/Tandem Axle)

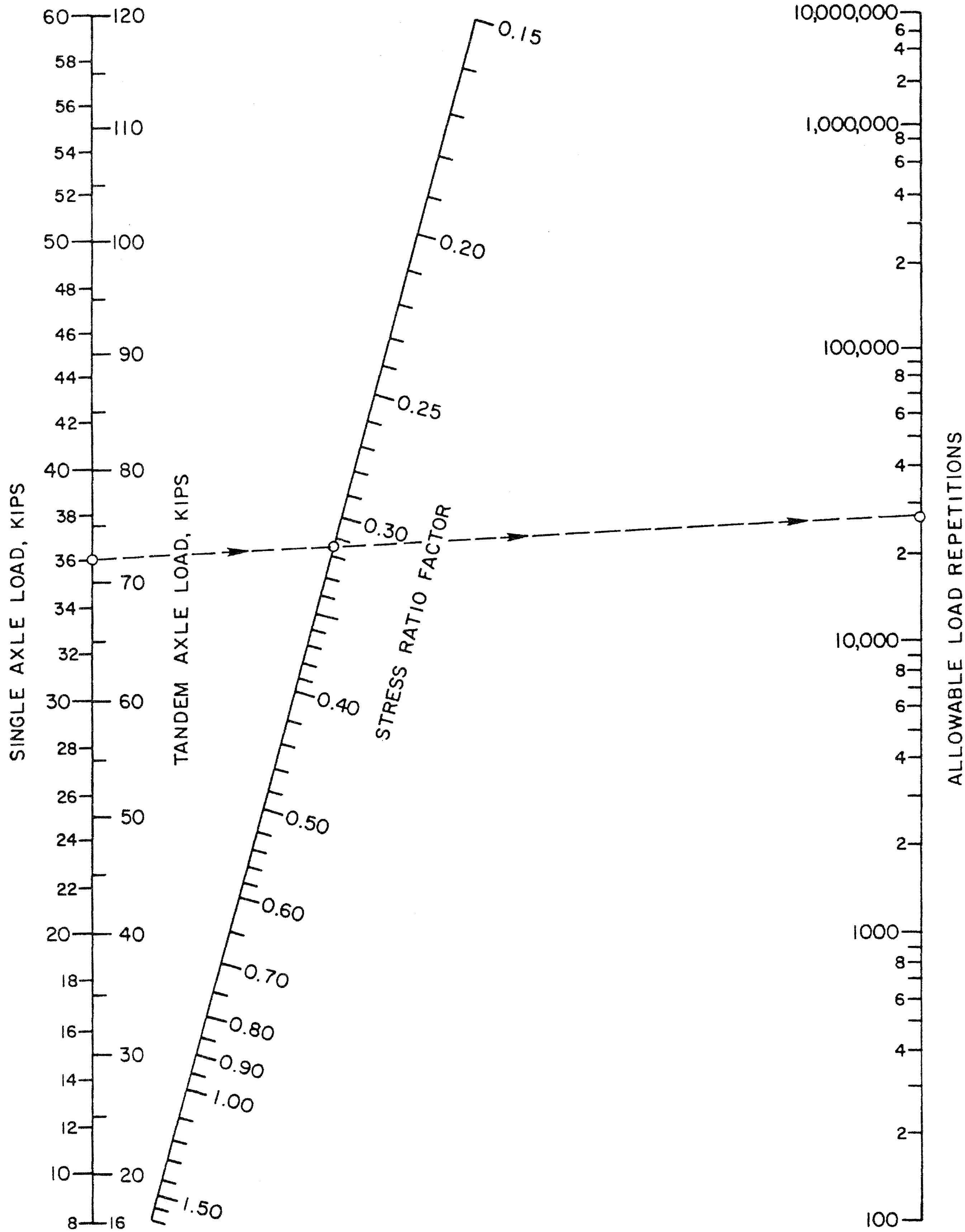

Figure 4-3.9. Fatigue Analysis Allowable Load Repetitions Based on Stress Ratio Factor (With and Without Concrete Shoulder).

Slab thickness, in.	k of subgrade-subbase, pci					
	50	100	200	300	500	700
4	3.74/3.83	3.73/3.79	3.72/3.75	3.71/3.73	3.70/3.70	3.68/3.67
4.5	3.59/3.70	3.57/3.65	3.56/3.61	3.55/3.58	3.54/3.55	3.52/3.53
5	3.45/3.58	3.43/3.52	3.42/3.48	3.41/3.45	3.40/3.42	3.38/3.40
5.5	3.33/3.47	3.31/3.41	3.29/3.36	3.28/3.33	3.27/3.30	3.26/3.28
6	3.22/3.38	3.19/3.31	3.18/3.26	3.17/3.23	3.15/3.20	3.14/3.17
6.5	3.11/3.29	3.09/3.22	3.07/3.16	3.06/3.13	3.05/3.10	3.03/3.07
7	3.02/3.21	2.99/3.14	2.97/3.08	2.96/3.05	2.95/3.01	2.94/2.98
7.5	2.93/3.14	2.91/3.06	2.88/3.00	2.87/2.97	2.86/2.93	2.84/2.90
8	2.85/3.07	2.82/2.99	2.80/2.93	2.79/2.89	2.77/2.85	2.76/2.82
8.5	2.77/3.01	2.74/2.93	2.72/2.86	2.71/2.82	2.69/2.78	2.68/2.75
9	2.70/2.96	2.67/2.87	2.65/2.80	2.63/2.76	2.62/2.71	2.61/2.68
9.5	2.63/2.90	2.60/2.81	2.58/2.74	2.56/2.70	2.55/2.65	2.54/2.62
10	2.56/2.85	2.54/2.76	2.51/2.68	2.50/2.64	2.48/2.59	2.47/2.56
10.5	2.50/2.81	2.47/2.71	2.45/2.63	2.44/2.59	2.42/2.54	2.41/2.51
11	2.44/2.76	2.42/2.67	2.39/2.58	2.38/2.54	2.36/2.49	2.35/2.45
11.5	2.38/2.72	2.36/2.62	2.33/2.54	2.32/2.49	2.30/2.44	2.29/2.40
12	2.33/2.68	2.30/2.58	2.28/2.49	2.26/2.44	2.25/2.39	2.23/2.36
12.5	2.28/2.64	2.25/2.54	2.23/2.45	2.21/2.40	2.19/2.35	2.18/2.31
13	2.23/2.61	2.20/2.50	2.18/2.41	2.16/2.36	2.14/2.30	2.13/2.27
13.5	2.18/2.57	2.15/2.47	2.13/2.37	2.11/2.32	2.09/2.26	2.08/2.23
14	2.13/2.54	2.11/2.43	2.08/2.34	2.07/2.29	2.05/2.23	2.03/2.19

Figure 4-3.10a. Erosion Factors - Doweled Joints, No Concrete Shoulder (Single Axle/Tandem Axle)

Slab thickness, in.	k of subgrade-subbase, pci					
	50	100	200	300	500	700
4	3.94/4.03	3.91/3.95	3.88/3.89	3.86/3.86	3.82/3.83	3.77/3.80
4.5	3.79/3.91	3.76/3.82	3.73/3.75	3.71/3.72	3.68/3.68	3.64/3.65
5	3.66/3.81	3.63/3.72	3.60/3.64	3.58/3.60	3.55/3.55	3.52/3.52
5.5	3.54/3.72	3.51/3.62	3.48/3.53	3.46/3.49	3.43/3.44	3.41/3.40
6	3.44/3.64	3.40/3.53	3.37/3.44	3.35/3.40	3.32/3.34	3.30/3.30
6.5	3.34/3.56	3.30/3.46	3.26/3.36	3.25/3.31	3.22/3.25	3.20/3.21
7	3.26/3.49	3.21/3.39	3.17/3.29	3.15/3.24	3.13/3.17	3.11/3.13
7.5	3.18/3.43	3.13/3.32	3.09/3.22	3.07/3.17	3.04/3.10	3.02/3.06
8	3.11/3.37	3.05/3.26	3.01/3.16	2.99/3.10	2.96/3.03	2.94/2.99
8.5	3.04/3.32	2.98/3.21	2.93/3.10	2.91/3.04	2.88/2.97	2.87/2.93
9	2.98/3.27	2.91/3.16	2.86/3.05	2.84/2.99	2.81/2.92	2.79/2.87
9.5	2.92/3.22	2.85/3.11	2.80/3.00	2.77/2.94	2.75/2.86	2.73/2.81
10	2.86/3.18	2.79/3.06	2.74/2.95	2.71/2.89	2.68/2.81	2.66/2.76
10.5	2.81/3.14	2.74/3.02	2.68/2.91	2.65/2.84	2.62/2.76	2.60/2.72
11	2.77/3.10	2.69/2.98	2.63/2.86	2.60/2.80	2.57/2.72	2.54/2.67
11.5	2.72/3.06	2.64/2.94	2.58/2.82	2.55/2.76	2.51/2.68	2.49/2.63
12	2.68/3.03	2.60/2.90	2.53/2.78	2.50/2.72	2.46/2.64	2.44/2.59
12.5	2.64/2.99	2.55/2.87	2.48/2.75	2.45/2.68	2.41/2.60	2.39/2.55
13	2.60/2.96	2.51/2.83	2.44/2.71	2.40/2.65	2.36/2.56	2.34/2.51
13.5	2.56/2.93	2.47/2.80	2.40/2.68	2.36/2.61	2.32/2.53	2.30/2.48
14	2.53/2.90	2.44/2.77	2.36/2.65	2.32/2.58	2.28/2.50	2.25/2.44

Figure 4-3.10b. Erosion Factors - Aggregate-Interlock Joints, (Single Axle/Tandem Axle)

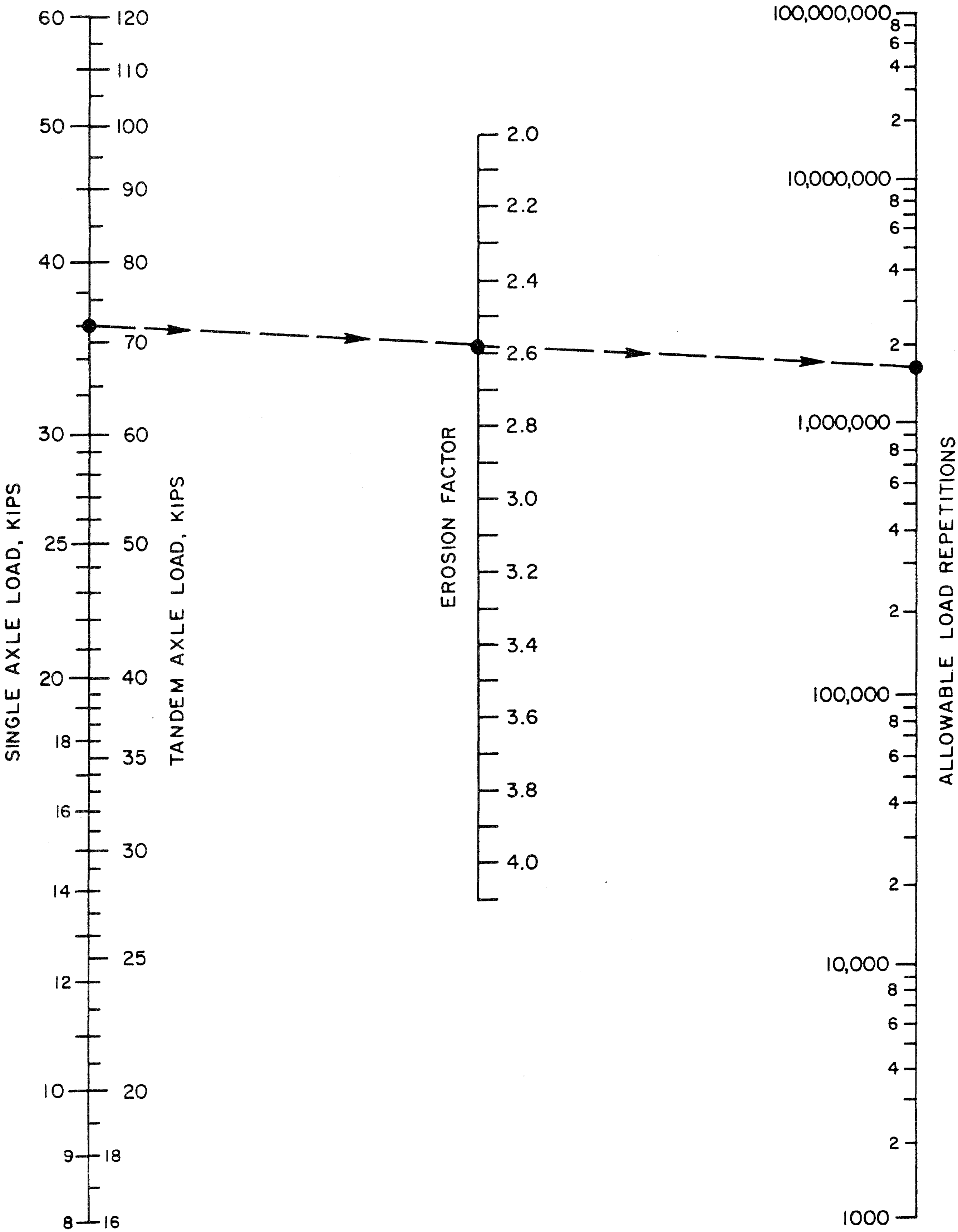

Figure 4-2.11. Erosion Analysis - Allowable Load Repetitions Based on Erosion Factor (without concrete shoulder)

Slab thickness, in.	k of subgrade-subbase, pci					
	50	100	200	300	500	700
4	3.28/3.30	3.24/3.20	3.21/3.13	3.19/3.10	3.15/3.09	3.12/3.08
4.5	3.13/3.19	3.09/3.08	3.06/3.00	3.04/2.96	3.01/2.93	2.98/2.91
5	3.01/3.09	2.97/2.98	2.93/2.89	2.90/2.84	2.87/2.79	2.85/2.77
5.5	2.90/3.01	2.85/2.89	2.81/2.79	2.79/2.74	2.76/2.68	2.73/2.65
6	2.79/2.93	2.75/2.82	2.70/2.71	2.68/2.65	2.65/2.58	2.62/2.54
6.5	2.70/2.86	2.65/2.75	2.61/2.63	2.58/2.57	2.55/2.50	2.52/2.45
7	2.61/2.79	2.56/2.68	2.52/2.56	2.49/2.50	2.46/2.42	2.43/2.38
7.5	2.53/2.73	2.48/2.62	2.44/2.50	2.41/2.44	2.38/2.36	2.35/2.31
8	2.46/2.68	2.41/2.56	2.36/2.44	2.33/2.38	2.30/2.30	2.27/2.24
8.5	2.39/2.62	2.34/2.51	2.29/2.39	2.26/2.32	2.22/2.24	2.20/2.18
9	2.32/2.57	2.27/2.46	2.22/2.34	2.19/2.27	2.16/2.19	2.13/2.13
9.5	2.26/2.52	2.21/2.41	2.16/2.29	2.13/2.22	2.09/2.14	2.07/2.08
10	2.20/2.47	2.15/2.36	2.10/2.25	2.07/2.18	2.03/2.09	2.01/2.03
10.5	2.15/2.43	2.09/2.32	2.04/2.20	2.01/2.14	1.97/2.05	1.95/1.99
11	2.10/2.39	2.04/2.28	1.99/2.16	1.95/2.09	1.92/2.01	1.89/1.95
11.5	2.05/2.35	1.99/2.24	1.93/2.12	1.90/2.05	1.87/1.97	1.84/1.91
12	2.00/2.31	1.94/2.20	1.88/2.09	1.85/2.02	1.82/1.93	1.79/1.87
12.5	1.95/2.27	1.89/2.16	1.84/2.05	1.81/1.98	1.77/1.89	1.74/1.84
13	1.91/2.23	1.85/2.13	1.79/2.01	1.76/1.95	1.72/1.86	1.70/1.80
13.5	1.86/2.20	1.81/2.09	1.75/1.98	1.72/1.91	1.68/1.83	1.65/1.77
14	1.82/2.17	1.76/2.06	1.71/1.95	1.67/1.88	1.64/1.80	1.61/1.74

Figure 4-3.12a. Erosion Factors - Doweled Joints, Concrete Shoulder (Single Axle/Tandem Axle)

Slab thickness, in.	k of subgrade-subbase, pci					
	50	100	200	300	500	700
4	3.46/3.49	3.42/3.39	3.38/3.32	3.36/3.29	3.32/3.26	3.28/3.24
4.5	3.32/3.39	3.28/3.28	3.24/3.19	3.22/3.16	3.19/3.12	3.15/3.09
5	3.20/3.30	3.16/3.18	3.12/3.09	3.10/3.05	3.07/3.00	3.04/2.97
5.5	3.10/3.22	3.05/3.10	3.01/3.00	2.99/2.95	2.96/2.90	2.93/2.86
6	3.00/3.15	2.95/3.02	2.90/2.92	2.88/2.87	2.86/2.81	2.83/2.77
6.5	2.91/3.08	2.86/2.96	2.81/2.85	2.79/2.79	2.76/2.73	2.74/2.68
7	2.83/3.02	2.77/2.90	2.73/2.78	2.70/2.72	2.68/2.66	2.65/2.61
7.5	2.76/2.97	2.70/2.84	2.65/2.72	2.62/2.66	2.60/2.59	2.57/2.54
8	2.69/2.92	2.63/2.79	2.57/2.67	2.55/2.61	2.52/2.53	2.50/2.48
8.5	2.63/2.88	2.56/2.74	2.51/2.62	2.48/2.55	2.45/2.48	2.43/2.43
9	2.57/2.83	2.50/2.70	2.44/2.57	2.42/2.51	2.39/2.43	2.36/2.38
9.5	2.51/2.79	2.44/2.65	2.38/2.53	2.36/2.46	2.33/2.38	2.30/2.33
10	2.46/2.75	2.39/2.61	2.33/2.49	2.30/2.42	2.27/2.34	2.24/2.28
10.5	2.41/2.72	2.33/2.58	2.27/2.45	2.24/2.38	2.21/2.30	2.19/2.24
11	2.36/2.68	2.28/2.54	2.22/2.41	2.19/2.34	2.16/2.26	2.14/2.20
11.5	2.32/2.65	2.24/2.51	2.17/2.38	2.14/2.31	2.11/2.22	2.09/2.16
12	2.28/2.62	2.19/2.48	2.13/2.34	2.10/2.27	2.06/2.19	2.04/2.13
12.5	2.24/2.59	2.15/2.45	2.09/2.31	2.05/2.24	2.02/2.15	1.99/2.10
13	2.20/2.56	2.11/2.42	2.04/2.28	2.01/2.21	1.98/2.12	1.95/2.06
13.5	2.16/2.53	2.08/2.39	2.00/2.25	1.97/2.18	1.93/2.09	1.91/2.03
14	2.13/2.51	2.04/2.36	1.97/2.23	1.93/2.15	1.89/2.06	1.87/2.00

Figure 4-3.12b. Erosion Factors - Aggregate-Interlock Joints, Concrete Shoulder (Single Axle/Tandem Axle)

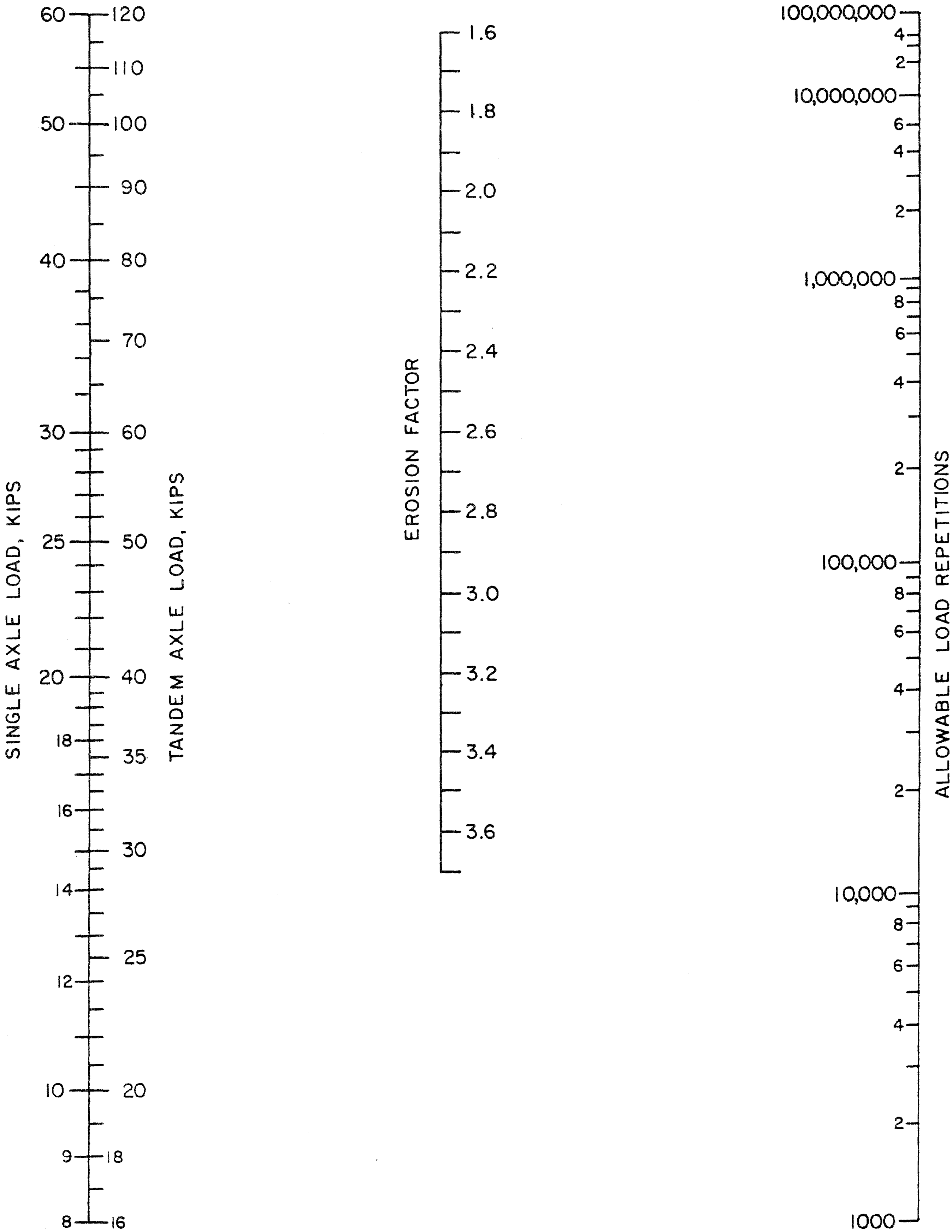

Figure 4-3.13. Erosion Analysis - Allowable Load Repetitions Based on Erosion Factor (With Concrete Shoulder).

Answer:

An 8-inch dowelled concrete pavement is just adequate to sustain 5,000,000 ESAL. The PCAPAV computer program was executed with the same input variables to verify the results. The input and output screens are shown in Figures 4-3.14 and 4-3.15.

3.5 Summary of the PCA Procedure

The thickness design procedure by the Portland Cement Association includes consideration of failure due to erosion of subbase materials as well as the traditional consideration of failure due to fatigue cracking.

The design is based on pavement stress and deflection data determined with a finite element computer program that models doweled and aggregate interlock joints, and pavements with and without concrete shoulders. The procedure also considers truck load placement, variation in concrete strength, different subbase materials, and concrete strength gain with age.

4.0 SUMMARY

There are several rigid pavement design and analysis procedures available. Some of these procedures offer many unusual capabilities in analyzing different design situations. Several of these procedures could be utilized by designers to develop designs for various components of the pavement such as shoulders (see Module 4-4), joints and reinforcement.

It is strongly recommended that no matter what design procedure is used by an agency or consultant to develop the pavement design, it should be checked using one or more other design procedures or analysis models or procedures. Each design procedure has its strengths and weaknesses, and only through the use of two or more design or analysis procedures will it be possible to determine the adequacy of the pavement design. Design checks using other available design and analysis procedures are common in all branches of engineering, and should also be used in pavement design.

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15:28:55                    PCAPAV(TM) 1.10                              Page 1
08-22-87   Proprietary Software of PORTLAND CEMENT ASSOCIATION
                              Pavement Design
                         ppppp      ccccc      aaaaa
                         p   p     c     c    a    a
                         p   p     c     c         a
                         p   p     c           aaaaa
                         ppppp     c     c    a    a
                         p         c     c    a    a
                         p          ccccc      aaaaa
           (C) Copyright Portland Cement Association 1985
                          All Rights Reserved
   This program is to be used as a design aid by experienced qualified
   ENGINEERS.  This program is not intended for use as a final design
   or a substitute for  sound  engineering  judgement.   The purchaser
   assumes all responsibilty for the use of this program in connection
   with any project.

 Input File: in.dat                      Output File: out.dat
 Project ID. pdto
 Engineer:   design
 Solution Options: Normal
                                          File not Found
move cursor   PgDn-Next Page
```

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15:31:00                    PCAPAV(TM) 1.10                              Page 2
08-22-87   Proprietary Software of PORTLAND CEMENT ASSOCIATION
                            Pavement  Design  Data
Modulus of Subg/Subb K  125.0 PCI       Axle Load Cat.  1.Light
Modulus of Rupture  MR  700.0 PSI                       2.Medium
                                                        3.Heavy
A D T T  685.00                                         4.Very Heavy
                                                        5.Input Axles
Design Life  20 Years                  Maximum Single axle load   22  KIPS
                                       Maximum Tandem axle load   36  KIPS
Load Transfer                                AXLE   LOADS
 At Joint         1.Dowel              SAL   Axles    TAL   Axles
                  2.Agg. Interlock     KIPS  /1000    KIPS  /1000
                                        22    0.00     36    0.00
 At Shoulder      1.Conc. Shoulder      20    0.00     32    0.00
                  2.No Conc. Shoulder   18  100.00     28    0.00
                                        16    0.00     24    0.00
 Load Safety Factor   1. 1.0            14    0.00     20    0.00
                      2. 1.1            12    0.00     16    0.00
                      3. 1.2            10    0.00     12    0.00
                                         8    0.00      8    0.00
 Estimated Pavement Thickness  8.0 IN    6    0.00      4    0.00
                                         4    0.00      0    0.00
move cursor   PgDn-Next Page
```

Figure 4-3.14. PCAPAV Input Screens (29).

15:31:54 PCAPAV(TM) 1.10 Page 1
08-22-87 Proprietary Software of PORTLAND CEMENT ASSOCIATION

Project: pdtc
Engineer: design
Input Data:
Subgrade / Subbase K 125.0 PCI
Modulus of Rupture MR 700.0 PSI
Avg. Daily Truck Traffic (2 way) ADTT 685.00
Design Life 20 years
Doweled Joints
No Concrete Shoulders
Load Safety Factor 1.2
Estimated Pavement Thickness 8.0 IN

A X L E L O A D S
Axles per 1000 Trucks

Single		Tandem	
22	0.00	36	0.00
20	0.00	32	0.00
18	100.00	28	0.00
16	0.00	24	0.00
14	0.00	20	0.00
12	0.00	16	0.00
10	0.00	12	0.00
8	0.00	8	0.00
6	0.00	4	0.00
4	0.00	0	0.00

Design Thickness =7.5 Inches

		Load Repetitions		---Fatigue Analysis-----			-----Erosion Analysis---		
SAL	*LSF	Axle/ 1000	Expected Reps	Stress Ratio	Allowable Reps	Fatigue Consump	Power	Allowable Reps	Erosion
22	26.4	0.00	0.	0.590	**********	0.00	41.225	**********	0.00
20	24.0	0.00	0.	0.539	**********	0.00	34.070	**********	0.00
18	21.6	100.00	250025.	0.489	1398539.	17.88	27.597	3887592.	6.43
16	19.2	0.00	0.	0.437	**********	0.00	21.805	**********	0.00
14	16.8	0.00	0.	0.386	**********	0.00	16.694	**********	0.00
12	14.4	0.00	0.	0.334	**********	0.00	12.265	**********	0.00
10	12.0	0.00	0.	0.281	**********	0.00	8.518	**********	0.00
8	9.6	0.00	0.	0.228	**********	0.00	5.451	**********	0.00
6	7.2	0.00	0.	0.174	**********	0.00	3.066	**********	0.00
4	4.8	0.00	0.	0.119	**********	0.00	1.363	**********	0.00

TAL	*LSF	Axle/ 1000	Expected Reps	Stress Ratio	Allowable Reps	Fatigue Consump	Power	Allowable Reps	Erosion
36	43.2	0.00	0.	0.428	**********	0.00	38.438	**********	0.00
32	38.4	0.00	0.	0.383	**********	0.00	30.371	**********	0.00
28	33.6	0.00	0.	0.338	**********	0.00	23.253	**********	0.00
24	28.8	0.00	0.	0.293	**********	0.00	17.084	**********	0.00
20	24.0	0.00	0.	0.246	**********	0.00	11.864	**********	0.00
16	19.2	0.00	0.	0.200	**********	0.00	7.593	**********	0.00
12	14.4	0.00	0.	0.152	**********	0.00	4.271	**********	0.00
8	9.6	0.00	0.	0.104	**********	0.00	1.898	**********	0.00
4	4.8	0.00	0.	0.054	**********	0.00	0.475	**********	0.00
0	0.0	0.00	0.	0.000	**********	0.00	0.000	**********	0.00

Total Fatigue Used = 17.88 Erosion Damage = 6.43

7.0 Inch Thickness Inadequate, Fatigue Used= 143.04 Erosion Damage = 12.24

Figure 4-3.15. PCAPAV Output Screen (29).

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MODULE 4-4

RIGID PAVEMENT SHOULDER DESIGN

1.0 INSTRUCTIONAL OBJECTIVES

This module deals specifically with the design of rigid pavement shoulders for conventional mainline rigid pavements (JPCP, JRCP and CRCP). The overall need for and the design of flexible shoulders is presented in Module 3-4, "Flexible Pavement Shoulder Design." The information presented in that module will not be repeated in this module, therefore, it is recommended that Module 3-4 be read prior to this module.

Upon completion of this module (and the additional related material in Module 3-4), the participant will be able to accomplish the following:

1. Briefly summarize the performance of rigid pavement shoulders and identify the effects that a properly designed tied rigid pavement shoulder should have on the adjacent rigid pavement traffic lane.

2. List the types of rigid shoulder pavements that could feasibly be used with each of the conventional mainline rigid pavement types.

3. Describe guidelines for tieing the rigid shoulder to the adjacent traffic lane, and for sealing the longitudinal joint.

4. Determine appropriate joint spacing for jointed concrete shoulders adjacent to JPCP, JRCP and CRCP mainline pavements.

5. Determine slab and base/subbase thicknesses for a rigid shoulder and provide subdrainage recommendations.

2.0 INTRODUCTION

Major topics addressed in this module include the performance of rigid shoulders, the effects of a rigid shoulder on adjacent lane performance, selection of the type of rigid shoulder pavement, longitudinal joint design and thickness design. Information related to shoulder design in general and design of flexible shoulders for rigid pavements can be found in Module 3-4.

3.0 PERFORMANCE OF RIGID SHOULDERS

PCC shoulders have been constructed for many years on some urban expressways, but only since the 1960's on rural highways. The first rural highway experimental concrete shoulders were built in Illinois in 1965 with several to follow around the U. S. The good performance of these pavements led to the design and construction of concrete shoulders in many states since that time. It has now become standard practice in many states and foreign countries to utilize concrete shoulders for rigid pavements (5,6,7,9,10,12).

4.0 EFFECTS OF A TIED RIGID PAVEMENT SHOULDER ON MAINLINE PAVEMENT

The major effects of a rigid shoulder pavement on the performance of the mainline pavement center around the improvement of structural capacity and the reduction of the effects of moisture.

4.1 Increase in Structural Capacity

The placement of a tied concrete shoulder next to the mainline traffic lane slab can substantially improve the structural load carrying capacity of the pavement. The tied concrete shoulder provides support to the edge which reduces deflections and stresses in the mainline slab. Figure 4-4.1 shows a reduction in measured PCC slab strains, and Figure 4-4.2 shows a reduction of measured edge deflections due to a tied PCC shoulder (11).

Figure 4-4.3 and Figure 4-4.4 show the effect of the load transfer of the lane/shoulder longitudinal joint on stresses and deflections, respectively (1,5). As an example of the effect of these results, performance data from the 1967 JPCP concrete shoulders in Illinois showed significantly less punchouts on CRCP mainline pavement where the JPCP tied shoulders were located than where asphalt concrete shoulders were located (1,5).

The width of the rigid shoulder is important as shown in Figure 4-4.5. A width of at least three feet is needed to provide the most effective stress reduction in the traffic lane (1,4,5).

4.2 Reduction in Surface Water Entering the Pavement Section

A sealed and tied longitudinal joint between the traffic lane and the rigid shoulder should reduce the amount of surface runoff water entering the pavement structure. Field studies by Dempsey (13) in Georgia and Illinois showed that sealing the longitudinal lane/shoulder greatly reduced the amount of inflow from rainfall into the pavement structure as shown in Figure 4-4.6.

4.3 Reduction of Pumping

Pumping of materials beneath the mainline slab should be reduced through the following effects:

1. Reduction of edge and corner deflections of the mainline slab.
2. Reduction of the infiltration of surface runoff water into the lane/shoulder longitudinal joint.

However, the pavement may still need a subdrainage system to remove rapidly that moisture that does enter the section (13).

4.4 Reduction in Edge Drop-off

Tied concrete shoulders have not shown (even after 20 years in Illinois) the tendency of most flexible shoulders to settle or deteriorate, resulting in a potential safety hazard along the longitudinal joint.

5.0 FEASIBLE TYPES OF RIGID SHOULDER PAVEMENT

Three types of rigid shoulder pavements have been constructed adjacent to rigid pavements. These include jointed plain concrete (JPCP), jointed reinforced concrete (JRCP) and continuously reinforced concrete pavements (CRCP). The use of these pavement types for rigid shoulders depends upon costs, constructability and the type of mainline pavement.

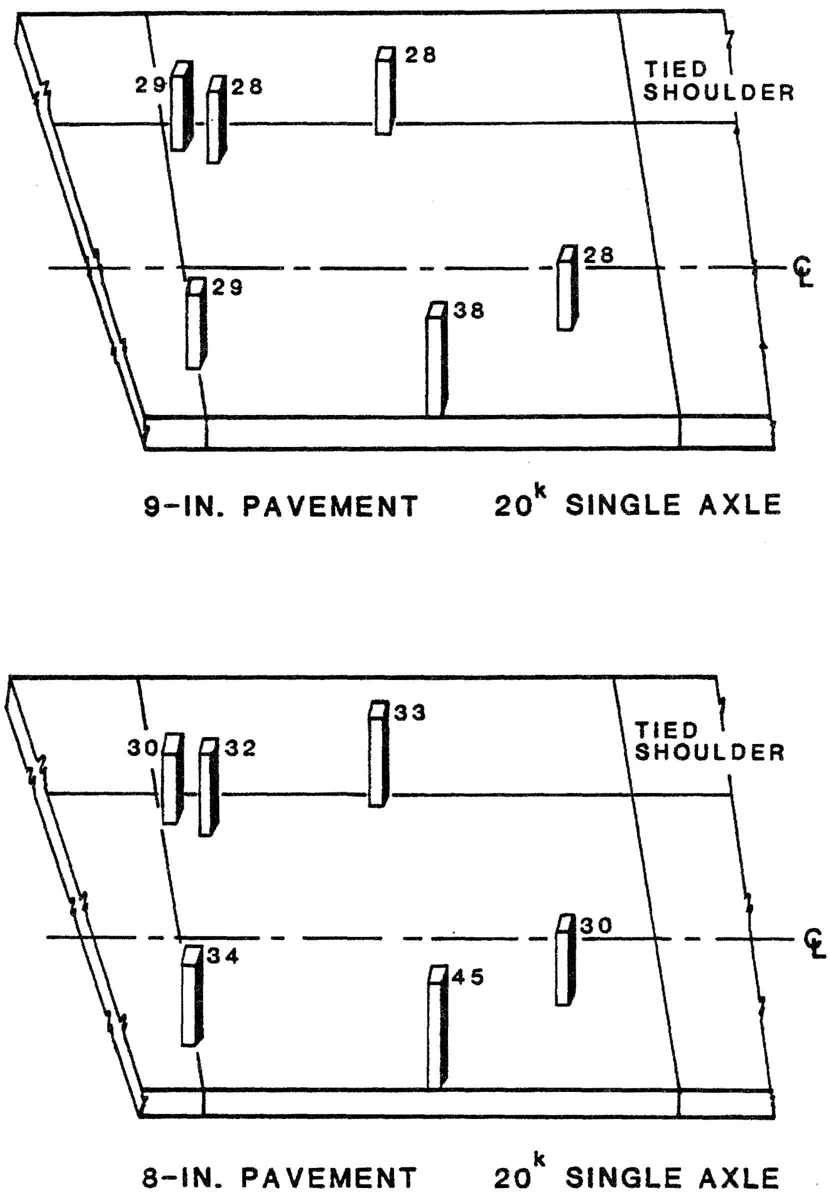

Figure 4-4.1. Measured Strains for Two Jointed Concrete Pavements Showing Effects of Tied PCC Shoulder (observe PCC shoulder edge vs free edge strains, 28 vs 38 units) (3).

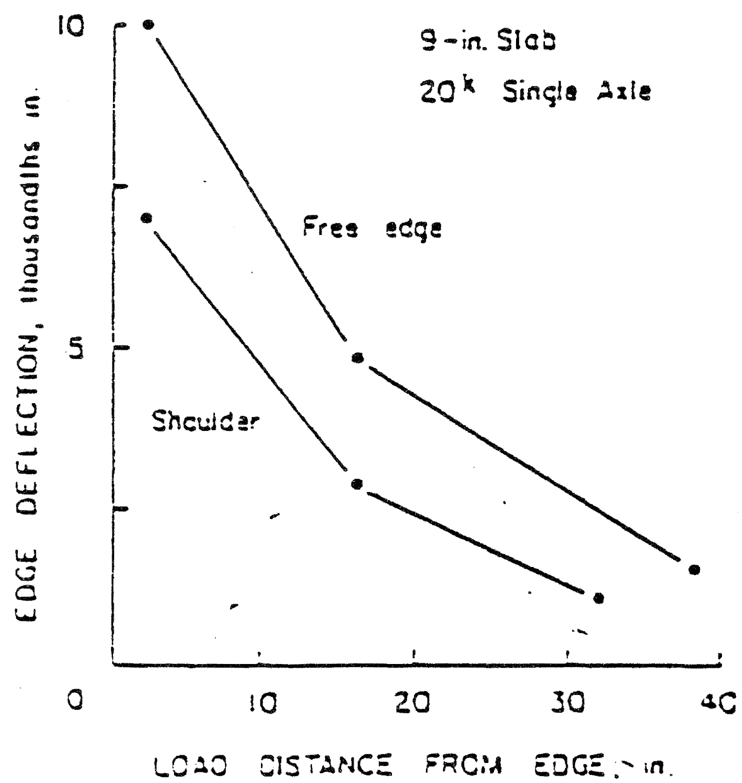

Figure 4-4.2. Measured Edge Deflection Reduction Due to Tied PCC Shoulder (3).

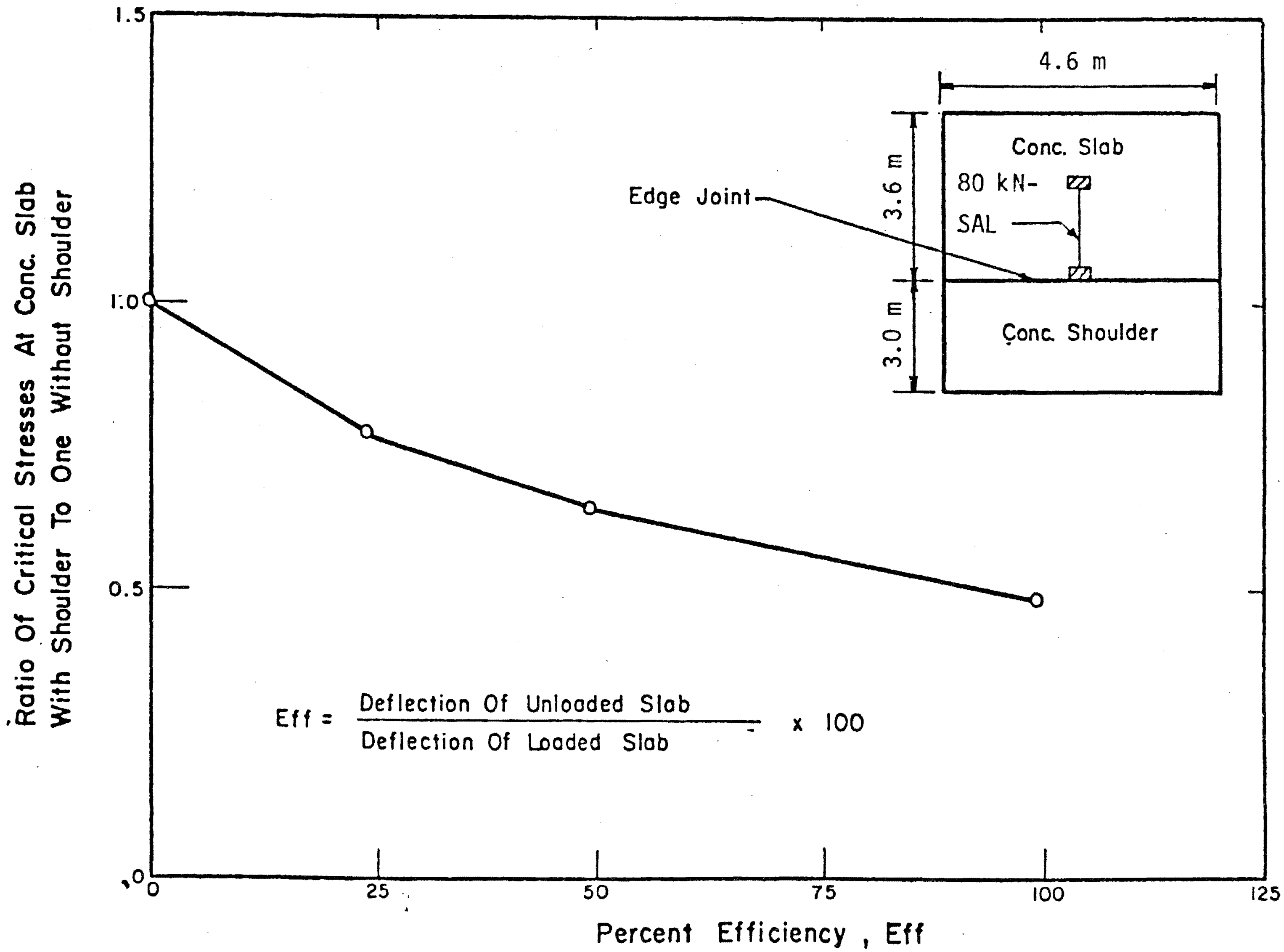

Figure 4-4.3. Effect of Concrete Shoulder on Critical Edge Stress for Varying Joint Load Transfer Efficiency (finite element solution) (5).

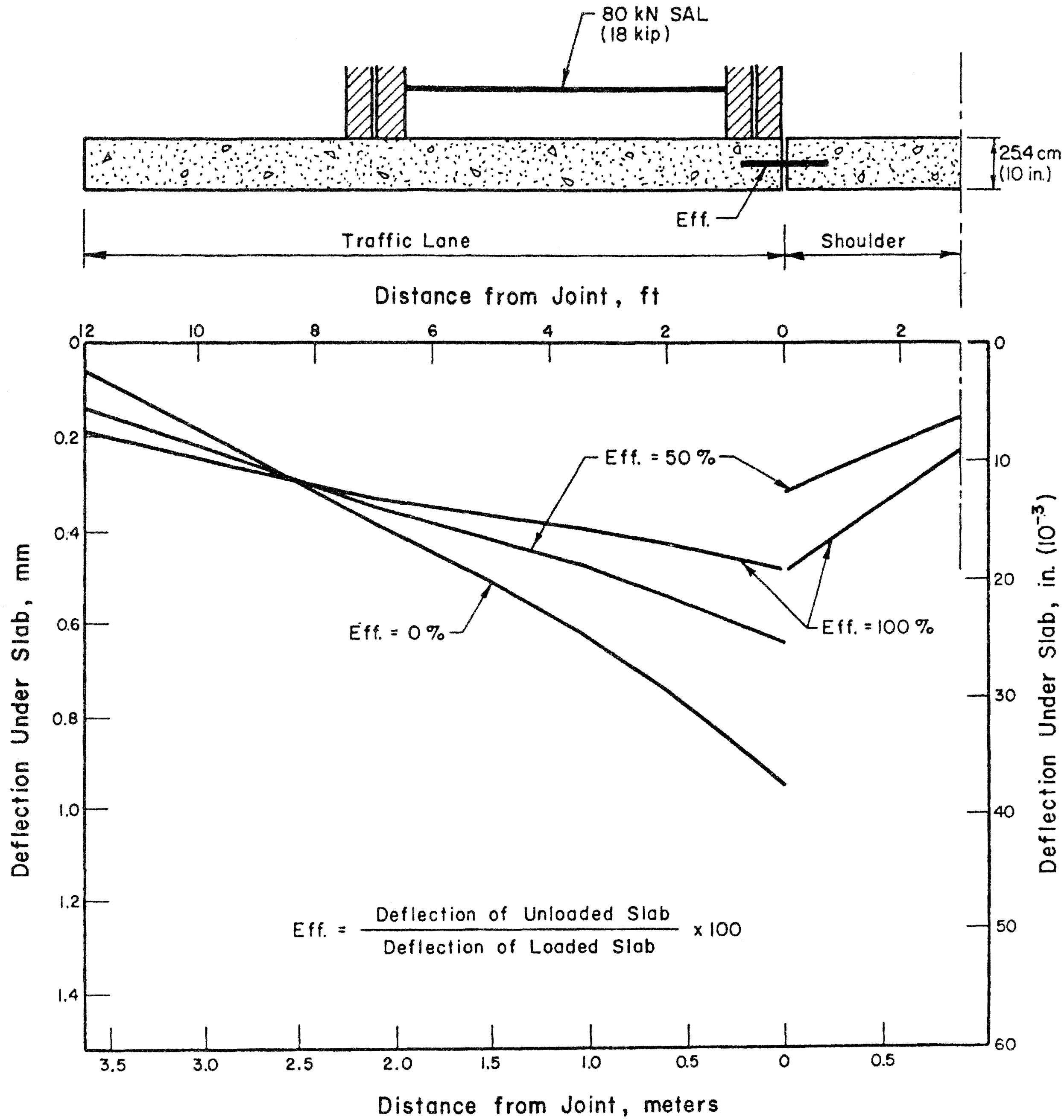

Figure 4-4.4. Effect of Lane/Shoulder Tie on Deflection of PCC Traffic Lane (finite element solution) (5).

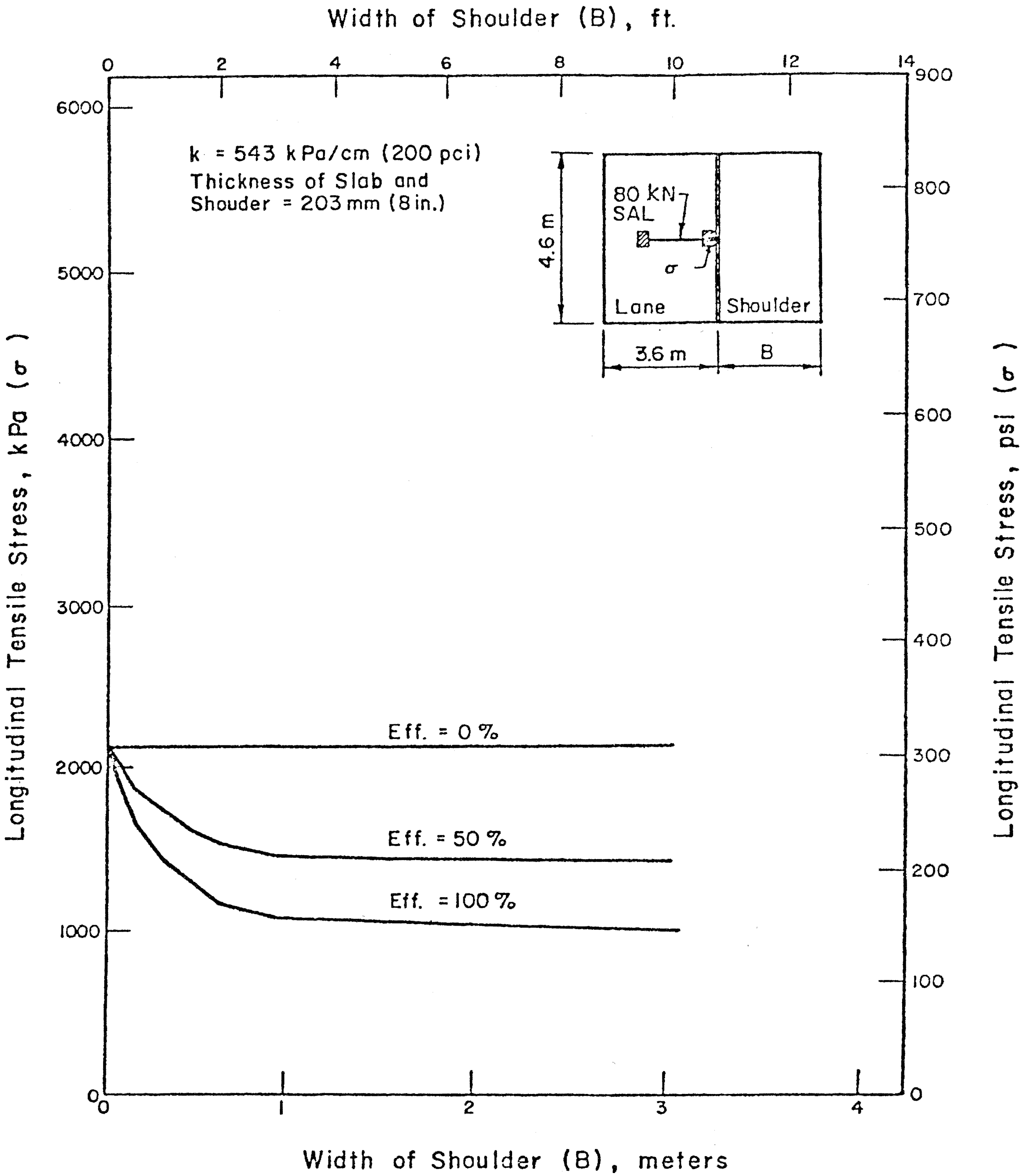

Figure 4-4.5. The Effect of Lane/Shoulder Tie and Width of PCC Shoulder on Tensile Stress of Traffic Lane (finite element solution) (5).

Pavement Test Section	Mean ($\bar{x}$) %	Coefficient of Variation (V) %	Standard Deviation(s) %
Georgia G1 Unsealed	30.6	61.1	18.7
Georgia G1 Sealed	0.7	194.0	1.4
Georgia G2 Sealed	--	--	--
Illinois I1 Unsealed	26.0	76.5	19.9
Illinois I1 Sealed	16.4	69.5	11.4
Illinois I2 Unsealed	52.1	36.7	19.1
Illinois I2 Sealed	11.6	28.4	3.3

$$\text{Percentage of Pipe Outflow} = \frac{\text{Pipe Outflow Volume}}{\text{Precipitation Volume}} \times 100$$

Figure 4-4.6. Statistical Data Based on Percentage of Pipe Outflow Showing Effect of Sealing Edge Joint (13).

Generally, the type of shoulder pavement should match the type of pavement used in the mainline to provide for the best performance and best cost. However, there may be some exceptions to this guideline as follows:

1. Mainline JPCP. Only JPCP type shoulders are recommended because of cost efficiencies involved. Also, if a longer jointed JRCP shoulder pavement were used, the excessive joint movements may cause problems in the traffic lane JPCP.

2. Mainline JRCP. A JRCP shoulder that matches the mainline in design could be placed at the same time as the outer traffic lane. However, it is also feasible to use a JPCP shoulder. JPCP may be more cost effective because no reinforcement would be placed in the shoulder and it could be placed at the same time as the JRCP mainline pavement by leaving out the reinforcing steel and cutting transverse joints at shorter intervals.

3. Mainline CRCP. A CRCP shoulder that matches the mainline in design could be placed at the same time as the outer traffic lane. However, it is also feasible to use a JPCP shoulder as long as the joint spacing is short so that joint movement will be minimal. This will reduce any potential effect of the movement of the transverse shoulder joint producing a crack in the mainline CRCP that ruptures the reinforcement. This has occurred where the shoulder has long joint spacing (e.g., 100 ft.). The elimination of reinforcement in the JPCP shoulder is a major cost reduction item.

6.0 DESIGN OF LONGITUDINAL JOINT BETWEEN LANE AND SHOULDER

The longitudinal joint between the traffic lane and the shoulder is very critical to pavement performance.

6.1 Effect of Longitudinal Lane/Shoulder Joint

6.1.1 Fatigue Damage From Edge Loads

Several studies have shown that the critical fatigue point for jointed concrete pavements is along the outer traffic lane edge (14,15). The presence of vertical shear support along the slab edge would reduce the stresses in the traffic lane, thus, reducing fatigue damage. Various studies have estimated that the adjacent lane slab thickness could be reduced by at least one inch due to the increased edge support (3,11).

6.1.2 Surface Water Entering The Pavement Section

The amount of surface runoff entering the pavement section from the longitudinal joint was studied by Dempsey (13). Results showed that up to 52 percent of precipitation volume that entered the pavement section entered through the longitudinal lane shoulder joint.

6.1.3 Pumping From High Corner and Edge Deflections

Corner and edge deflections depend largely upon load transfer at the transverse joint and also along the longitudinal joint as illustrated in

Figure 4-4.3. Any reduction in corner or edge deflection would reduce the amount of potential pumping.

6.2 Design of Load Transfer for the Longitudinal Joint

Provision of adequate load transfer can be accomplished through closely spaced tiebars. Tiebars can be inserted into the plastic concrete near the rear of the slip form paver. Bent bars can be installed by mechanical or manual means. The bent portion can be straightened later to tie the shoulder to the mainline.

Malleable tiebars of adequate diameter (#5 bars) and spacing (18 to 24 ins.) placed mid-depth in the slab depth are preferable to stiffer short bars with large spacing intervals (5). This will substantially reduce the possibility of stress concentrations above the tiebar which will cause the joint to spall in the vicinity of the bar.

A keyway can also be formed in addition to the tiebars to provide for additional load transfer capability, although there may be construction difficulties in forming the keyway.

Measured results of load transfer for several different longitudinal joints after about 10 years of heavy traffic are shown in Figure 4-4.7 and Figure 4-4.8. The design of the load transfer system definitely has a major effect on future load transfer.

The following recommendations are provided for longitudinal design efficiencies for various types of joints. These values represent approximate efficiencies after 10 years of heavy traffic loadings. The estimates of deflection joint efficiencies are based on the data from in-service measurements and engineering judgement (5).

Joint Type	Deflection Efficiency	Stress Efficiency
Tied and Keyed	70 - 100	30 - 100
Tied Butt		
30 ins. bar spacing	60 - 80	25 - 42
12 -24 ins. spacing	70 - 95	30 - 95
Non-tied	0 - 20	0 - 5

6.3 Sealing of the Longitudinal Joint

The longitudinal joint between the traffic lane and shoulder should be provided with a sealant reservoir and sealed with a high quality sealant. This will reduce the amount of surface water entering the pavement and also reduce the amount of chlorides from deicing salts entering and corroding the tiebars.

7.0 DESIGN OF TRANSVERSE JOINTS FOR SHOULDERS

When the shoulder pavement type matches the mainline pavement type, the transverse shoulder joint should match the mainline traffic lane joint. For example, if a JPCP mainline traffic lane has a 11 to 16 ft. random skewed joint spacing, the JPCP shoulder should exactly match the mainline joint spacing and skew.

Shoulder Design	Mean Edge* Deflection - Traffic Lane	Mean Edge* Deflection - Shoulder	Deflection Load Transfer Efficiency (%)
1 Tiebars, keyway, and Granular Subbase	0.1143 mm (0.0045 in.)	0.1118 mm (0.0044 in.)	97.8
2 Tiebars, keyway no subbase	0.1448 mm (0.0057 in.)	0.1016 mm (0.0040 in.)	70.2
3 No tiebars, keyway, with granular subbase	0.2108 mm (0.0083 in.)	0.0330 mm (0.0013 in.)	16.0

*Deflections measured with Benkleman Beam using 84.4 kN (19,000 lb) single axle. Procedure similar to that used at AASHO Road Test (Ref. 13) with outside of duals 7.5-15 cm (3-6 in.) from traffic lane slab edge and beam probe at traffic lane edge and at shoulder edge (creep speed deflection).

Figure 4-4.7. Field Measurement Data on 10-Year-Old I-74 Illinois PCC Shoulders to Determine Longitudinal Lane/Shoulder Joint Efficiency (5).

Shoulder Design	Mean Edge* Deflection- Traffic Lane	Mean Edge* Deflection- Shoulder	Deflection Load Transfer Efficiency (%)
Tiebars, and Granular Subbase (Intermediate)	0.2311 mm (0.0091 in.)	0.0889 mm (0.0035 in.)	38.5
Tiebars, Granular Subbase (Coarse)	0.2464 mm (0.0097 in.)	0.0762 mm (0.0030 in.)	31.0
Tiebars, with No Subbase	0.2159 mm (0.0085 in.)	0.1016 mm (0.0040 in.)	47.0

*Deflections measured with Benkleman Beam using 121.3 kN (27,300 lb) tandem axle. Procedure similar to that used at AASHO Road Test (Ref. 13) with outside of duals 7.5-15 cm (3-6 in.) from traffic lane slab edge and beam probe at traffic lane edge and at shoulder edge (creep speed deflection).

Figure 4-4.8. Field Measurement Data on 9-Year-Old I-80 Illinois PCC Shoulders to Determine Longitudinal Lane/Shoulder Joint Efficiency (5).

However, if the mainline pavement is a JRCP having a long joint spacing and a JPCP shoulder pavement is selected, the joint spacing for the JPCP shoulder should be much less. Figure 4-4.9 illustrates the effect of longer joint spacings on spalling of the transverse shoulder joints. Figure 4-4.10 shows the effect of joint spacing on transverse cracking of the shoulder. Therefore, the joint spacing for the JPCP shoulder should be no longer than normal good joint design spacing practice (Module 4-1).

General guidelines are that the maximum spacing should not be greater than 1.75 times the slab thickness in inches (a shoulder slab of 8 inches should not have a transverse joint spacing of greater than 1.75 x 8 = 14 ft.). The final shoulder joint spacing should be adjusted so that the joints in the mainline JRCP are matched with joints in the shoulder. For example, a 9 inch slab JRCP having a joint spacing of 45 ft. could have a JPCP shoulder having a joint spacing of 15 ft. (maximum joint spacing is 1.75 x 9 = 15.75 ft.). Any expansion joints in the mainline must also be matched with expansion joints in concrete shoulders.

Transverse joints should also be provided with an adequate reservoir and sealed similar to the mainline joints.

Transverse shoulder joints do not normally require dowels, even if the mainline pavement does contain dowels. The only exception to this would be where the shoulder is being designed to carry mainline traffic during rush hours or during an extensive time period to accommodate future major rehabilitation work.

8.0 SHOULDER THICKNESS DESIGN

8.1 Matching the Mainline pavement

It is recommended that the shoulder rigid pavement slab matches the adjacent traffic lane slab thickness. This thickness of shoulder is not required to carry encroaching truck loads (similar to the over design for the inner traffic lanes that do not carry as much truck traffic as the outer lane does). Experience has shown that a 6 inch thick concrete shoulder will perform without serious structural deterioration for over 15 years under heavy traffic in the main line. However, having the shoulder thickness equal to the adjacent traffic lane slab has the following advantages:

1. Construction is much easier.

2. Subdrainage of the cross-section may be better because water can seep along the slab/base interface without laying in a "bathtub" trench.

3. Differential frost heave is much less likely between the lane and the shoulder if they have the same cross-section.

Thus, it is recommended that at the longitudinal joint, the shoulder and mainline slabs be the same thickness. The other consideration of shoulder thickness is that it may be tapered to a thinner section at the outer edge of the shoulder as shown in Figure 4-4.11. The thickness required at the edge is a function of the number of parked trucks on the shoulder. This edge becomes a critical point for fatigue damage from parking trucks.

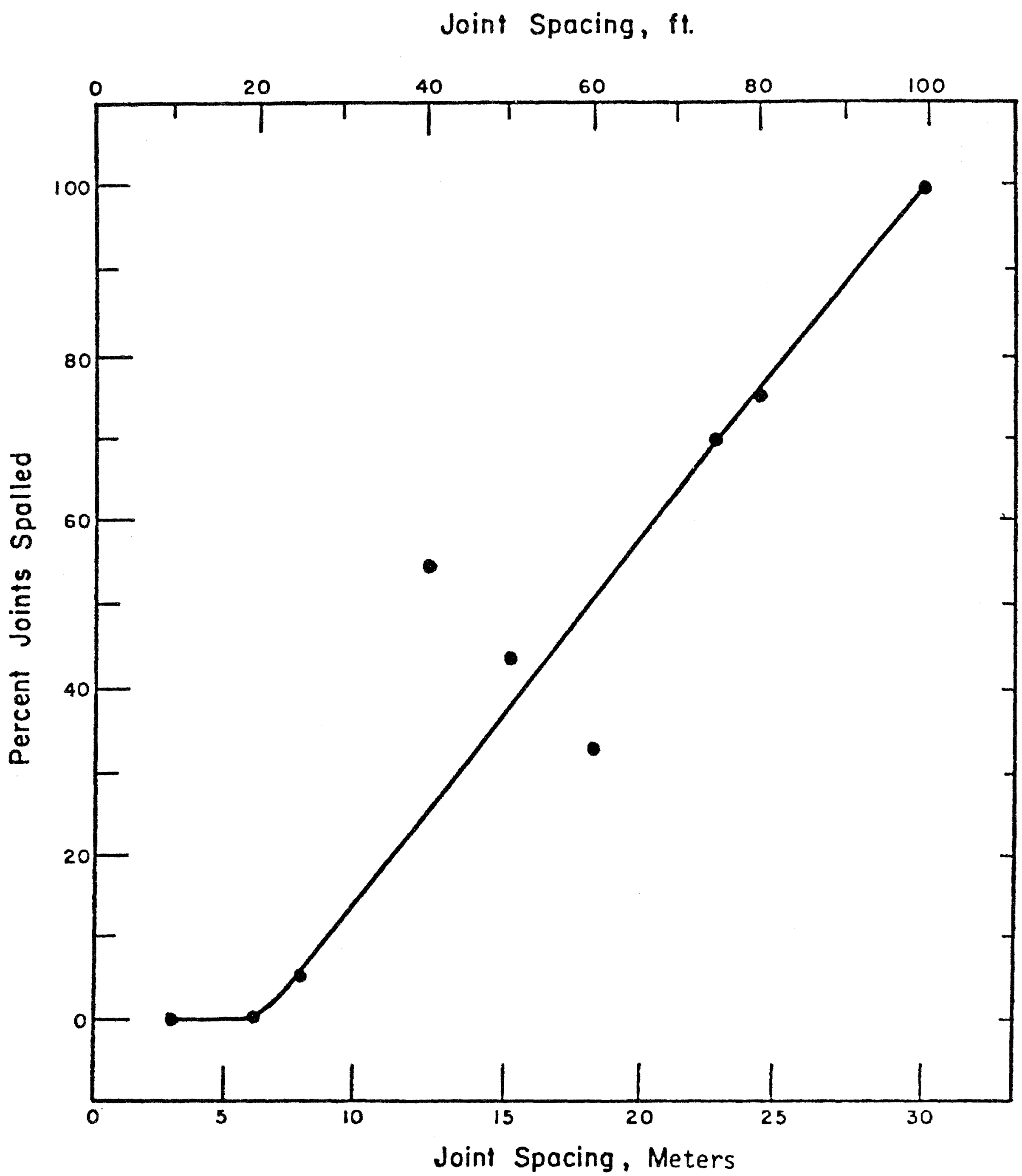

Figure 4-4.9. Effect of Joint Spacing of PCC Shoulder on Joint Spalling (data from 10-year-old Illinois projects) (5).

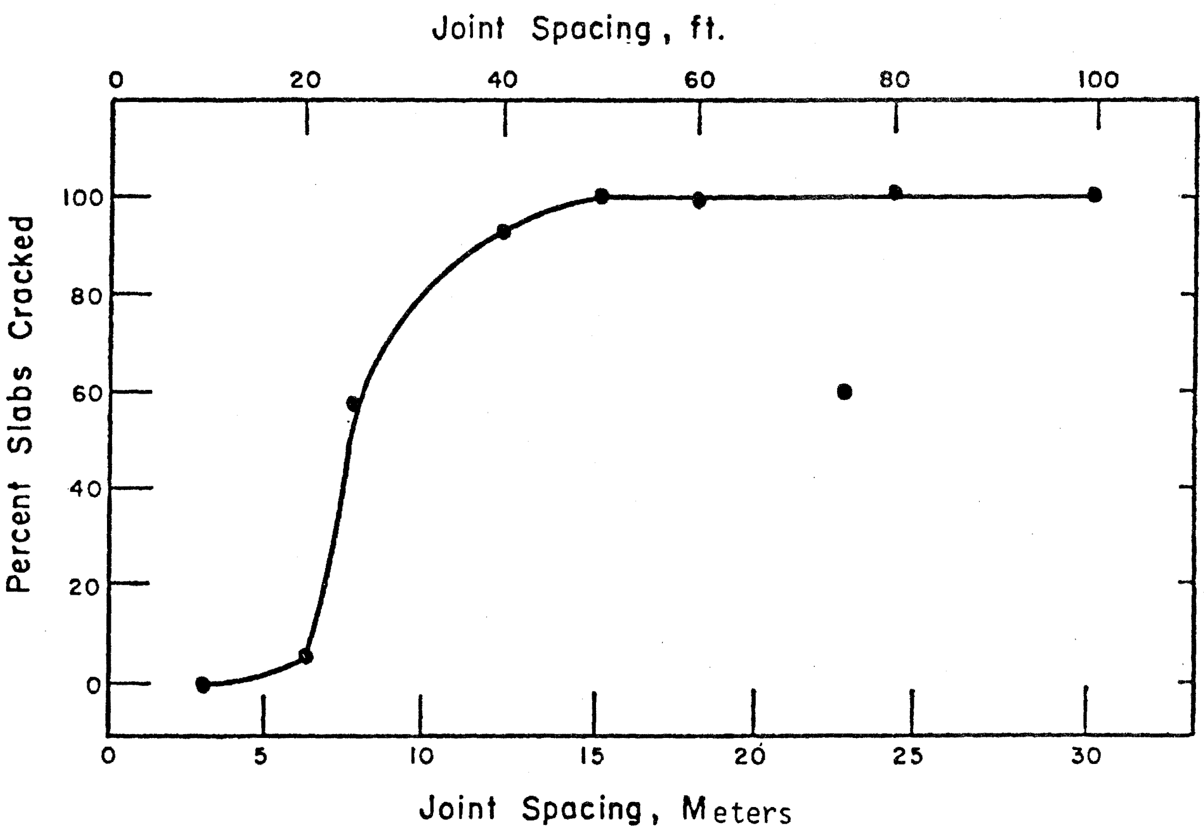

Figure 4-4.10. Effect of Joint Spacing of PCC Shoulder on Transverse Cracking (data from 10-year-old Illinois projects) (5).

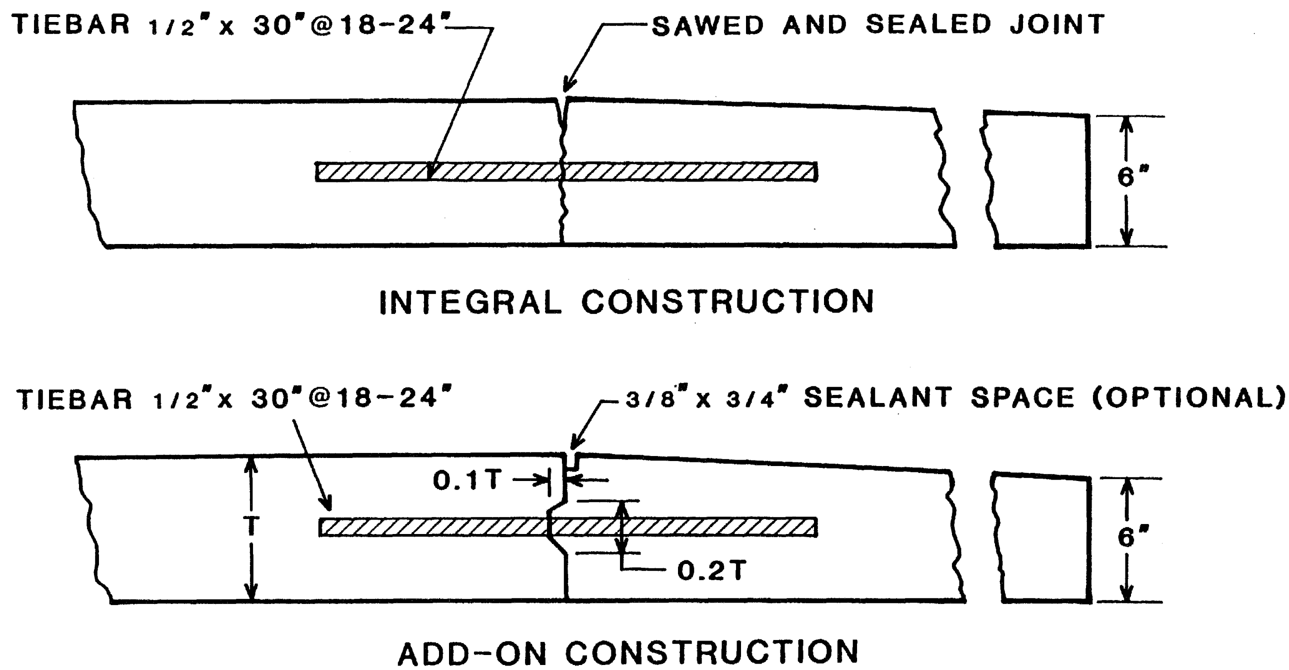

Figure 4-4.11. Examples of PCC Longitudinal Shoulder Joints and Taper of Shoulder Thickness.

8.2 Determining the Required Thickness for a Rigid Shoulder

Under certain circumstances it may be desirable to determine the thickness required for the shoulder considering the actual traffic that is expected to use the shoulder. The design of rigid pavement thickness varies somewhat from that of the mainline slab. Detailed design procedures have been developed for the Federal Highway Administration for determining the thickness of slab required under the following two loading conditions (5):

1. Encroaching traffic from the traffic lane (8).

2. Parking traffic (5).

The procedure involves the following steps. A computer program is available for the design of the shoulder (5).

8.2.1 Obtain Design Inputs

Design life, years

Slab properties
- Trial thickness of shoulder
- Slab width
- Mean modulus of rupture
- Coefficient of variation of PCC

Traffic
- ADT at beginning of design period
- ADT at end of design period
- Percent trucks
- Proportion of trucks in outer lane
- Percent directional distribution of trucks
- Mean axles per truck
- Percent trucks that use shoulder (see Mod. 3-4 or Ref. 5)
  - Encroached trucks
  - Parked trucks
  - Axle load distribution

Foundation support effective k-value (top base)

Traffic lane/shoulder tie, load transfer

Example input data are shown in Figure 4-4.12 for a shoulder design situation.

8.2.2 Conduct Fatigue analysis

The program conducts a fatigue damage analysis using the well known Miner's fatigue law at both the lane/shoulder longitudinal joint (for encroaching loads) and at the outer shoulder edge (from parking trucks). The example data were input to the Jointed Concrete Shoulder computer program (JCS-1) for shoulder thicknesses of 5, 6, 7, 8, and 9 inches. The output results are as follows:

SHOULDER DESIGN LIFE	20.0	YEARS
SLAB PROPERTIES		
SHOULDER THICKNESS	5,6,7,8,9	INCHES
TRAFFIC LANE THICKNESS	8.0	INCHES
SHOULDER WIDTH	10.0	FEET
MEAN PCC MODULUS OF RUPTURE (28 DAYS)	750	PSI
COEFFICIENT OF VARIATION OF PCC MODULUS	10	%
LOAD TRANSFER EFFICIENCY BETWEEN SHOULDER AND TRAFFIC LANE	50	%
FOUNDATION SUPPORT		
DESIGN MODULUS OF FOUNDATION SUPPORT (K)	200	PCI
ERODIBILITY OF FOUNDATION SUPPORT AT END OF DESIGN PERIOD	8.0	INCHES
TRAFFIC		
ADT AT BEGINNING OF DESIGN PERIOD	17100	EACH
ADT AT END OF DESIGN PERIOD	39100	EACH
PERCENT TRUCKS OF ADT	21	%
PERCENT TRUCKS IN DESIGN TRAVELED LANE	85.15	%
PERCENT DIRECTIONAL DISTRIBUTION	50	%
MEAN AXLES PER TRUCK	2.60	EACH
LENGTH OF SURVEYED STRETCH	10.0	MILES
AVERAGE LENGTH OF TOTAL ENCROACHMENTS PER TRUCK IN THE SHOULDER STRETCH	0.24	MILES
PERCENT TRUCKS THAT PARK ON THE SHOULDER	0.016	%
NUMBER OF SINGLE AXLE LOAD INTERVALS	13	EACH
NUMBER OF TANDEM AXLE LOAD INTERVALS	17	EACH

SINGLE AXLE LOAD DISTRIBUTION TABLE

WEIGHT RANGE (POUNDS)	PERCENT IN RANGE
0 - 3000	5.75
3001 - 7000	10.33
7001 - 8000	7.76
8001 - 12000	20.54
12001 - 16000	4.37
16001 - 18000	1.77
18001 - 20000	1.02
20001 - 22000	0.54
22001 - 24000	0.34
24001 - 26000	0.14
26001 - 30000	0.04
30001 - 32000	0.01
32001 - 34000	0.01

Figure 4-4.12. Parameters Assumed in Analysis for JCS-1 Program.

Figure 4-4.12 (Continued)

TANDEM AXLE LOAD DISTRIBUTION TABLE

WEIGHT RANGE (POUNDS)	PERCENT IN RANGE
0 - 6000	0.27
6001 - 12000	13.34
12001 - 18000	7.05
18001 - 24000	5.51
24001 - 30000	14.92
30001 - 32000	3.61
32001 - 34000	1.40
34001 - 36000	0.50
36001 - 38000	0.25
38001 - 40000	0.16
40001 - 42000	0.11
42001 - 44000	0.08
44001 - 46000	0.07
46001 - 50000	0.07
50001 - 52000	0.02
52001 - 54000	0.01
54001 - 56000	0.01

Figure 4-4.12. Parameters Assumed in Analysis for JCS-1 Program (Continued).

Total Fatigue Damage for Design Period

PCC Shoulder Thickness	Parked Traffic (outer edge)	Encroached Traffic (lane/shoulder jt.)
5 inches	0.418 E25 (>50)*	0.353 E04 (40)*
6 inches	0.574 E12 (>50)	0.695 E00 (13)
7 inches	0.334 E05 (>50)	0.652 E-02 (2)
8 inches	0.106 E01 (15)	0.316 E-03 (1)
9 inches	0.104 E-02 (5)	0.351 E-04 (<1)

* Note: Values in parenthesis is slab cracking in ft. per 1000 sq.ft. of traffic lane)

These fatigue damage values were calculated using Miner's Hypothesis. Computed damage values can range from near 0 to very large numbers. The greater the computed damage the greater the amount of slab cracking. A correlation was made between the computed fatigue damage number and actual slab cracking (5) and the resulting graph is shown in Figure 4-4.13. As shown, the greater the "damage" value, the greater the slab cracking. The designer, with the use of this curve, can select a limiting design fatigue damage value to limit the cracking of the shoulder slabs to a desirable level. For example, if 15 ft. of slab cracking per 1000 sq. ft. of surface was felt to be a limiting value for design (this would be about one slab cracked in every five slabs), the design fatigue damage value would be about 1.0.

8.2.3 Calculate Results

Using this value, the concrete shoulder thickness can be determined from the outputs shown above. A minimum slab thickness of 8 inches is needed for the outside edge of the shoulder due to parking traffic, and a minimum slab thickness of 6 inches is needed at the lane/shoulder longitudinal joint. Thus, the entire shoulder would likely be designed at 8 inches thick. However, this results indicates that if the longitudinal joint has substantial load transfer and there are substantial parked trucks, the outer edge could be critical.

The JCS-1 program is an easy to use tool that is tailored to the design of jointed plain concrete shoulders. It is available on both mainframe and IBM compatible personal computers.

The shoulder base and subbase should also match the mainline pavement to avoid problems such as a "bathtub" subdrainage situation and differential frost heave problems. Careful consideration should be given to subdrainage of the pavement structure to avoid blocking or holding of free moisture beneath the traffic lanes by the shoulder slab or base/subbase.

9.0 SUMMARY

This module has presented design concepts for rigid pavement shoulders. Several key design concepts must be considered in the design process to avoid performance problems in the future. Key design aspects for rigid shoulder design are as follows:

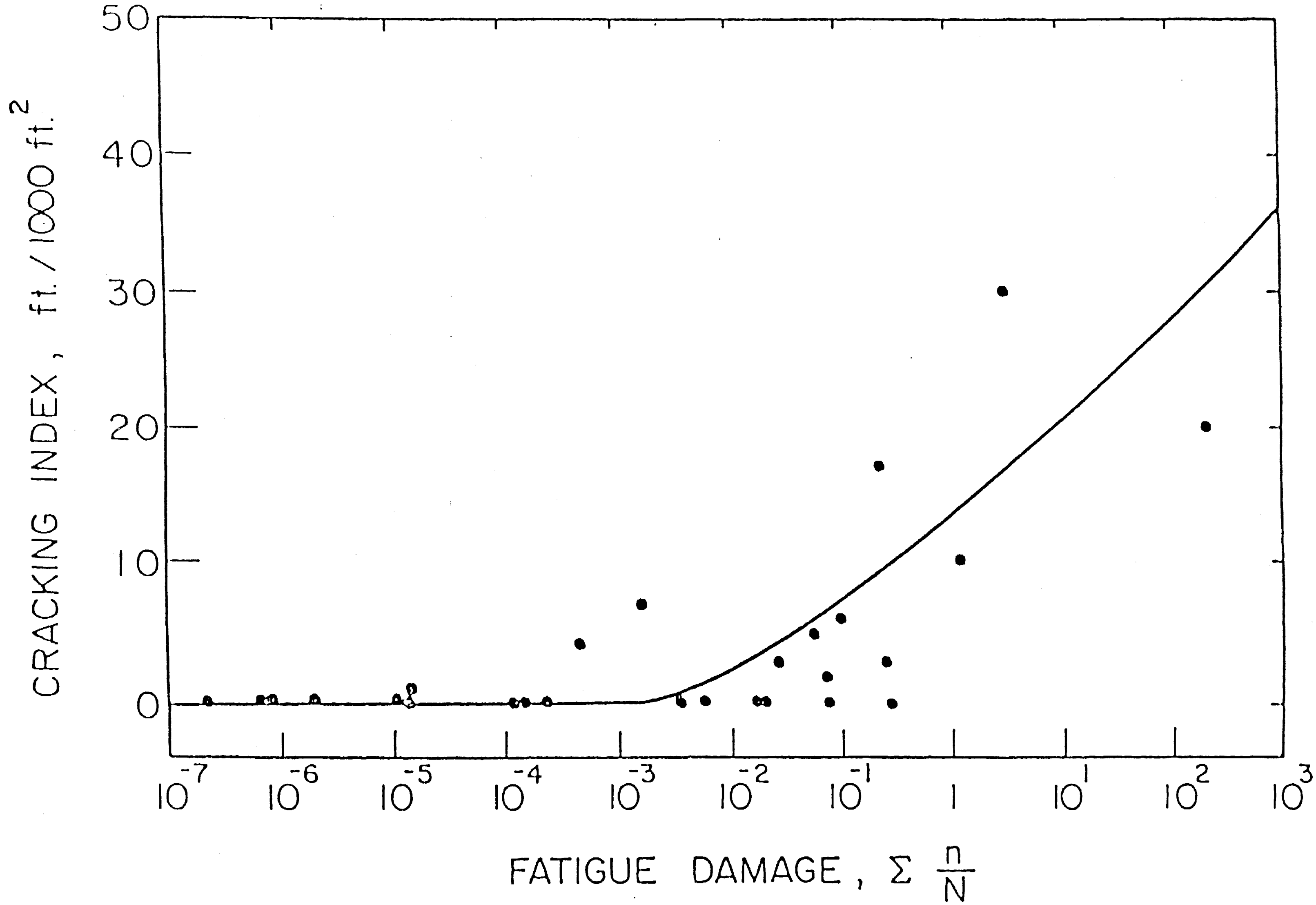

Figure 4-4.13. Cracking Index vs. Computed Fatigue Damage Developed for In-Service Pavements.

1. Adequate thickness of the shoulder slab and base must be provided to handle encroaching and parking truck traffic. Parking trucks may be more critical than encroaching trucks.

2. Provision of a uniform cross section and subdrainage beneath the shoulder to avoid differential frost heave and a bathtub design.

3. Provision of adequate load transfer across the longitudinal joint to increase support of the traffic lane edge, and to keep the shoulder tight with the adjacent traffic lane.

4. Selection of appropriate type of rigid shoulder based upon overall costs, compatibility between shoulder and traffic lane (joint opening/closings, potential cracks in either lane or shoulder).

5. Selection of adequate transverse joint spacing and load transfer in rigid shoulder.

6. Sealing of the longitudinal lane/shoulder joint to inhibit surface water from infiltrating.

10.0 REFERENCES

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BLOCK 5

Overlay Design

BLOCK 5 - OVERLAY DESIGN

Modules

5-1 SELECTION OF REHABILITATION STRATEGIES

5-2 TYPES OF OVERLAYS AND THEIR FUNCTIONS

5-3 DESIGN OF OVERLAYS ON FLEXIBLE PAVEMENTS

5-4 DESIGN OF OVERLAYS ON RIGID PAVEMENTS

This block introduces the design concepts and procedures for selecting structural thicknesses for overlays. A foundation is laid to assist the engineer in analyzing an existing pavement. Several rehabilitation strategies are discussed to help the engineer recognize that overlays may not always be the preferred solution. Design concepts for several different types of overlays are presented. Fundamental requirements to ensure adequate performance of the overlay are developed and considerations for reflection cracking are discussed. Structural thickness design procedures are presented for the various situations and overlay types which may be encountered.

Upon completion of this block the participants will be able to complete the instructional objectives listed for each module.

MODULE 5-1

SELECTION OF REHABILITATION TYPE

1.0 INSTRUCTIONAL OBJECTIVES

This module provides background information the engineer must consider in the rehabilitation selection process. Consideration is given to the development of the logical evaluation of a pavement to determine whether other forms of rehabilitation will be cost-effective as potential treatments.

Upon completion of this module, the participants will be able to accomplish the following:

1. Discuss the various types of rehabilitation techniques other than overlay.

2. Describe a pavement evaluation procedure required prior to rehabilitation selection.

3. List the factors which must be considered before selecting the preferred rehabilitation alternative.

4. Recognize the situations for which an overlay would be a feasible rehabilitation alternative.

2.0 INTRODUCTION

In recent years, the emphasis of highway construction has gradually shifted from new design and construction activities to maintenance and rehabilitation of the existing network (1). The need to maintain the already constructed network is essential to the economical operation of the overall transportation system.

Historically, overlays have been the most common rehabilitation technique utilized (2). However, these overlays have often been performed without regard to their applicability or cost-effectiveness. In many of these cases it may have been more practical or cost-effective to perform other types of rehabilitation or to perform routine maintenance on the pavement. To distinguish between these two activities, maintenance is defined as the preservation of the entire roadway, including surface, shoulders, roadsides, structures, and such traffic-control devices as are necessary for its safe and efficient utilization (3). Rehabilitation is defined as work undertaken to extend the service life of an existing facility, including the placement of additional surfacing material and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural or functional adequacy. This rehabilitation could include the complete removal and replacement of the pavement structure (3). The focus of this module will be on the selection of rehabilitation procedures to correct the observed problems.

To assist the engineer in selecting both the type and timing of pavement maintenance or rehabilitation, pavement performance must be systematically measured on a continuing basis. In this performance evaluation, both the functional and the structural performance of the pavement system should be considered.

Functional performance describes the adequacy of the pavement to meet its basic purpose of providing a safe and smooth riding surface. Functional adequacy is usually measured in terms of roughness and skid resistance. Structural performance is related to ability of the pavement to sustain traffic loading and is the primary input to all rational overlay design methods. Deflection testing is usually performed to predict future structural performance.

Although these two characteristics are intuitively related, there is currently no well-defined relationship between structural distress and functional performance. Thus, at present, judgment must be used in deciding when structural deterioration will lead to a level of functional performance below that considered reasonable by the user of the facility (which obviously will vary among vehicles, users, and types of facilities).

There are basic guidelines that should be followed in the determination of the appropriate rehabilitation strategy. These guidelines are illustrated in Figure 5-1.1. The selection process consists of 3 different phases: problem definition phase, potential problem solution phase and rehabilitation selection phase. It should be noted that this process is for a specific project and it assumes that the project was submitted for rehabilitation from some sort of network monitoring procedure.

These guidelines will be discussed briefly in the following sections. A complete analysis of alternative rehabilitation strategies (including their design) is found in Reference 4.

3.0 REHABILITATION STRATEGIES

Among some of the major rehabilitation methods available to the engineer are:

1. Overlay (asphalt or concrete).
2. Full-Depth Repairs.
3. Partial-Depth Repairs.
4. Joint/Crack Sealing.
5. Undersealing.
6. Grinding and Milling.
7. Subdrainage.
8. Pressure Relief Joints.
9. Load Transfer Restoration.
10. Surface Treatment.
11. Recycling.

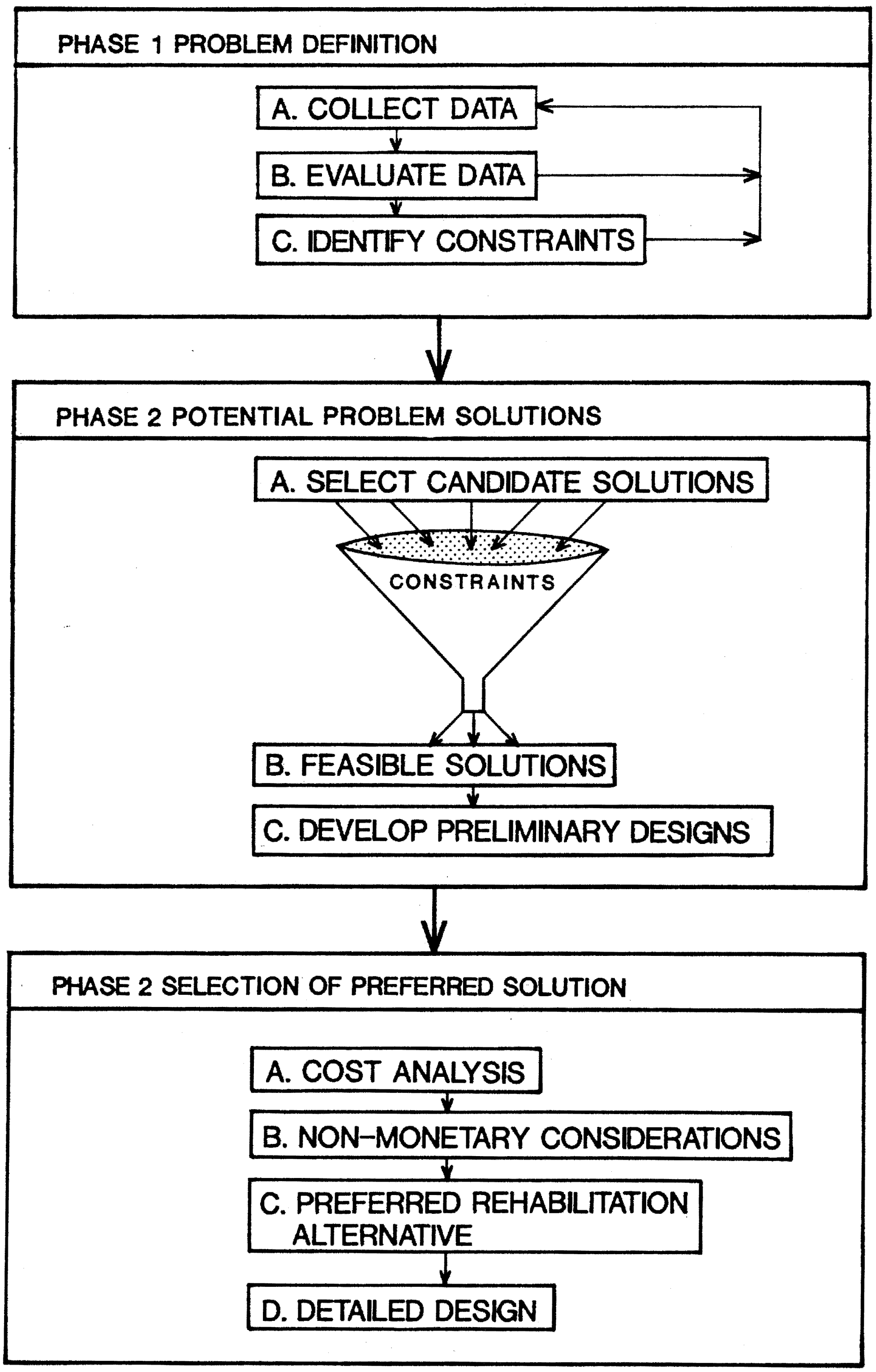

Figure 5-1.1. Pavement Rehabilitation Selection Process (1).

It is the desire of the engineer to choose the "preferred" rehabilitation strategy. The selected strategy must be cost-effective, must address the specific problems of the pavement, and it must meet any existing constraints of the project. This procedure is a complex one and will entail as much engineering judgment as engineering analysis. Compared to new pavement design, rehabilitation requires more of both items.

The following steps are fundamental in the decision-making process for selection of the preferred pavement rehabilitation strategy (1):

1. Determination of the cause of the distress(es) or problems of the pavement.

2. Development of a candidate list of solutions that will properly address, cure, and/or prevent future occurrences of the problem.

3. Selection of the preferred rehabilitation strategy, given economic and other project constraints.

3.1 Pavement Evaluation

Evaluation of the pavement is the first step of the above outlined procedure. It involves three steps: 1) data collection, 2) data evaluation, and 3) constraint identification. Many of the items are covered elsewhere in this course; therefore, they will be covered very briefly here. More information on these procedures is found in Reference 4.

3.1.1. Data Collection

A substantial data collection effort is required for a complete pavement evaluation. The majority of this data is obtained from a visual condition survey. Other items would be obtained from historical records, coring and boring, and testing (deflection, roughness, etc.).

At the network level, visual condition evaluation procedures are an integral part of the monitoring process and can serve as a guide to the type of maintenance or rehabilitation to be recommended. Also, these surveys can assist in the overall investment-programming process for maintenance and rehabilitation. The use of visual condition surveys is well established and should be a part of the maintenance and rehabilitation methodology of every organization that has responsibility for pavements.

Historical records provide original construction and design information, climate data, and traffic information. Coring and boring is required to determine the extent of the distress, and also for the determination of specific material properties (E values, etc.). Testing is necessary to determine specific pavement properties, such as load transfer efficiency, roughness, surface friction, etc.

The following is a list of items that would be required for the pavement evaluation (1):

1. Pavement Condition.

2. Shoulder Condition.

3. Pavement Design.

4. Geometric Design.

5. Materials and Soils Properties.

6. Traffic Volumes and Loadings.

7. Climate Conditions.

8. Drainage Conditions.

9. Safety Considerations.

The collection of specific data items will depend on the rehabilitation strategies being considered. Table 5-1.1 shows recommendations for the data required for the different rehabilitation technique being considered. Table 5-1.2 is an example overall pavement evaluation summary and checklist which follows this approach.

3.1.2. Data Evaluation

The data evaluation process involves the assessment and evaluation of the pavement from the very general to the very specific. This provides many pavement parameters which will be instrumental in the selection of the preferred rehabilitation strategy. Basically, this process takes the information collected in the first stage and transforms it into items which will have a direct influence on the rehabilitation selection. Thus, out of necessity, these two items (data collection and evaluation) are closely related. A sample procedure for the data collection/data evaluation process is shown below (1):

Step 1: Office Data Collection. Includes information such as location of the project, year constructed, year and type of major maintenance, pavement design, materials and soils properties, traffic, climate conditions, and any available performance data.

Step 2: First Field Survey. Includes items such as distress, drainage conditions, subjective roughness, traffic control options, and safety considerations.

Step 3: First Data Evaluation and the Determination of Additional Data Needs. Based on this first evaluation, a list of candidate rehabilitation alternatives may be developed to aid in assessing additional data needs.

Step 4: Second Field Survey. Detailed measuring and testing; includes such items as coring and sampling, roughness measurements, deflection testing, skid resistance, drainage tests, and vertical clearances.

Step 5: Laboratory Testing of Samples. Includes tests such as material strength, resilient modulus, permeability, moisture content, composition, density, and gradations (if felt to be necessary).

Table 5-1.1. Recommended Data for Various Rehabilitation Techniques (1).

DATA REQUIRED	FULL-DEPTH REPAIR	PARTIAL DEPTH PATCHING	GRINDING	RECYCLING	UNDERSEALING	SLAB JACKING	SUBDRAINS	JOINT RESEALING	PRESSURE RELIEF JOINTS	LOAD TRANSFER RESTORATION	SURFACE TREATMENT	OVERLAY
PAVEMENT DESIGN	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉
ORIGINAL CONSTRUCTION DATA	•		•	•			•	•	•	•		•
AGE	•	•	•	•			•				•	•
MATERIAL PROPERTIES	◉	•	•	◉	•		•			•		◉
SUBGRADE				◉	•	•	◉	•				◉
CLIMATE				•	•		•	•	•		◉	•
TRAFFIC LOADINGS AND VOLUMES	◉		◉	◉	•		◉	•	•	◉	◉	◉
DISTRESS	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉
NDT	•			•	◉					◉	•	◉
DESTRUCTIVE TESTING & SAMPLING	◉	◉	•	◉	•					•		◉
ROUGHNESS			•	•		•						
SURFACE PROFILE			◉			◉						•
DRAINAGE	◉		•	◉	•		◉	◉			•	◉
PREVIOUS MAINTENANCE	◉		•	•			•	•	•		•	•
BRIDGE PUSHING									◉			
UTILITIES	◉			◉	•	•	•					◉
TRAFFIC CONTROL OPTIONS	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉	◉
VERTICAL CLEARANCE				◉								•
GEOMETRICS				•								•

KEY ◉ DEFINITELY NEEDED • DESIRABLE

Table 5-1.2. Example Overall Pavement Evaluation Summary and Checklist (2).

STRUCTURAL EVALUATION

Existing Distress:
- Little or no load-associated distress
- Moderate load-associated distress
- Major load-associated distress

Structural Load-Carrying Capacity Deficiency:
- Yes, No

FUNCTIONAL EVALUATION

Roughness:
- Very Good, Good, Fair, Poor, Very Poor
- Measurement:__________
- Present Serviceability Index/Rating:__

Skid Resistance:
- Satisfactory, Questionable, Unsatisfactory

Rutting/Faulting Severity
- Low, Medium, High

VARIATION OF CONDITION EVALUATION

Systematic Variation Along Project:
- Yes, No

Systematic Variation Between Lanes:
- Yes, No

Localized Variation (very bad areas) Along Project:
- Yes, No

CLIMATIC EFFECTS EVALUATION

Climatic Zone
- Moisture Region: I Moisture Throughout Year
  - II Seasonal Moisture
  - III Very Little Moisture
- Temperature Region: A Severe Frost Penetration
  - B Freeze-Thaw Cycles
  - C No Frost Problems

Severity of Moisture-Accelerated Damage:
- Low, Medium, High
- Describe (asphalt stripping, pumping, etc.)____

Subsurface Drainage Capability-BASE:
- Satisfactory, Marginal, Unacceptable

Subsurface Drainage Capability-SUBGRADE:
- Satisfactory, Marginal, Unacceptable

Subsurface Drainage Capability:
- Acceptable, Needs Improvement
- Describe________

Table 5-1.2. (Continued).

PAVEMENT MATERIALS EVALUATION

Surface- Sound Condition, Deteriorated
Describe____________
Base - Sound Condition, Deteriorated
Describe____________
Subbase- Sound Condition, Deteriorated
Describe____________

SUBGRADE EVALUATION

Structural Support:
Low, Medium, High
Moisture Softening Potential:
Low, Medium, High
Temperature Problems:
None, Frost Heaving, Freeze-Thaw Softening
Swelling Potential:
Yes, No

PREVIOUS MAINTENANCE PERFORMED EVALUATION

Minor, Normal, Major
Has Lack of Maintenance Contributed to Deterioration?
Yes, No
Describe___________

RATE OF DETERIORATION EVALUATION

Long-Term:
Low, Normal, High
Short-Term:
Low, Normal, High

TRAFFIC CONTROL DURING CONSTRUCTION

Are Detours Available so That Facility Can be Closed:
Yes, No
Must Construction be Accomplished Under Traffic?
Yes, No
Could Construction be Done at Off-Peak Hours?
Describe_________

Table 5-1.2. (Continued).

GEOMETRIC AND SAFETY FACTORS

Current Capacity:
- Adequate, Inadequate

Future Capacity:
- Adequate, Inadequate

Widening Required Now:
- Yes, No

List High-Accident Locations:________
Bridge Clearances Problems:_________
Lateral Obstruction Problems:________
Utilities Problems:____________
Bridge Pushing Problems:__________

TRAFFIC LOADINGS

ADT (Two-Way):______________
ADTT (Two-Way):______________
Accumulated 18-kip ESAL:__________
Current 18-kip ESAL/year:__________

SHOULDERS

Pavement Condition:
- Good, Fair, Poor

Localized Deteriorated Areas:
- Yes, No

Step 6: Second Data Evaluation. Includes structural evaluation, functional evaluation, and determination of additional data requirements, if any.

Step 7: Final Field and Office Data Compilation. Preparation of a final evaluation report.

The collected data must be carefully evaluated and summarized in a systematic fashion.

3.1.3. Identification of Constraints

Often, there are several outside factors which play a major role in the selection of a rehabilitation strategy. These factors should be identified as early in the selection process as possible to ensure that they will be a part of the decision-making process. Some of the constraints which may affect the selection of alternatives are listed below (1).

1. Limited Project Funding.
2. Traffic Control Problems.
3. Minimum Desirable Life of Rehabilitation.
4. Geometric Design Problems.
5. Utilities.
6. Clearances.
7. Right-of-Way.
8. Available Materials and Equipment.
9. Contractor Expertise and Manpower.
10. Agency Policies.

The impact each of these may have on the rehabilitation strategy selection should be carefully considered. In addition, consideration should be given to how rehabilitation may affect the network as a whole, as opposed to just the individual project being rehabilitated. It may occasionally be necessary to select an alternative that is not optimal for a project because of overall network constraints (4).

3.2 Development of Candidate Solutions

Based on the data collected and evaluated and on the constraints of the project, candidate solutions must be developed for the project. Thus, all solutions which do not address the existing pavement distress and do meet the constraints of the project are not feasible alternatives. A feasible alternative is defined as one that addresses the cause of the distress and is effective in both repairing the existing deterioration and preventing its recurrence while satisfying all imposed constraints (1). A feasible alternative may encompass more than one repair technique. It is essential that the existing pavement distress be addressed to ensure that the rehabilitated pavements to not deteriorate further and make the completed rehabilitation worthless. Also, the rehabilitation should be considered only for those sections of the pavement with significant damage.

After all feasible alternatives have been selected, preliminary designs should be prepared, including cost estimates. Tables 5-1.3 and 5-1.4 provide a partial listing of candidate repair and preventive methods for rigid and flexible pavement distress.

3.3 Selection of the Preferred Rehabilitation Strategy

There is no set "method" that will enable the engineer to select the preferred rehabilitation strategy. Rather, it involves a combination of engineering judgment, creativity, and flexibility (1).

A major criterion used in identifying the preferred solution is cost analysis. This generally involves a life-cycle cost analysis to determine the cost-effectiveness of each alternative. This analysis requires inputs of costs and time, which, unfortunately, are subject to a large degree of uncertainty (1). Given this uncertainty, the best estimates for cost and time must be made, and the evaluation must be performed on those figures. It is also important to consider user costs in this analysis. More information on life-cycle cost analysis is found in Reference 5.

The common practice of selecting the alternative with the lowest initial construction cost is poor engineering practice that can lead to serious future pavement management problems for an agency.

There are many other factors which may influence the selection of alternate rehabilitation strategies; among these are (1):

1. Overall Project Management.

2. Service Life.

3. Duration of Construction.

4. Traffic Control Problems.

5. Reliability.

6. Constructability.

7. Maintainability.

8. Future Rehabilitation Options.

Overall project management may often be a project constraint, as mentioned previously. The service life of the rehabilitation strategy is also of particular importance, especially to agencies responsible for high volume roads for which lane closures and traffic delays pose considerable difficulties (1).

Table 5-1.3. Candidate Repair and Preventive Methods for Rigid Pavement Distress (1).

Joint/Crack Distress	Repair Methods	Preventive Methods
Pumping	1. Subseal	1. Reseal Joints 2. Restore Load Transfer 3. Subdrainage 4. Edge Support (PCC Shoulder/Edgebeam)
Faulting	1. Grind 2. Structural Overlay	1. Subseal 2. Reseal Joints 3. Restore Load Transfer 4. Subdrainage 5. Edge Support (PCC Shoulder/Edgebeam)
Slab Cracking	1. Full-Depth Repair 2. Replace/Recycle Lane	1. Subseal Loss of Support 2. Restore Load Transfer 3. Structural Overlay
Joint or Crack Spalling	1. Full-Depth Repair 2. Partial-Depth Repair	1. Subseal Loss of Support 2. Restore Load Transfer 3. Structural Overlay
Blowup	1. Full-Depth Repair	1. Pressure Relief Joint 2. Resealing Joint/Cracks
Punchouts	1. Full-Depth Repair	1. Polymer or Epoxy Grouting 2. Subseal Loss of Support 3. Rigid Shoulders

Table 5-1.4. Candidate Repair and Preventive Methods for Flexible Pavement Distress (1).

Joint/Crack Distress	Repair Methods	Preventive Methods
Alligator Cracking	1. Full-Depth Repair	1. Crack Sealing (may slow down alligator cracking)
Bleeding	1. Apply Hot Sand	
Block Cracking	1. Seal Cracks	
Depression	1. Level-up Overlay	
Polished Aggregate	1. Skid Resistant Surface Treatment 2. Slurry Seal	
Potholes	1. Full-Depth Repair	1. Crack Sealing 2. Seal Coat
Pumping	1. Full-Depth Repair	1. Crack Sealing 2. Seal Coat
Raveling and Weathering	1. Seal Coat	1. Rejuvenating Seal
Rutting	1. Level-up Overlay 2. Cold Milling With or Without Overlay	
Swell	1. Removal and Replacement	1. Paved Shoulder Encapsulation

The selection of the preferred alternative is conducted using the above items. If the cost-analysis yields no clear advantage for one solution, the other factors may play a larger role in the selection process (1). It is recommended that some sort of "weighing" procedure be used, in which all of the above factors carry a predetermined weight and the rehabilitation strategy is rated in each category. The strategy with the highest rating would be the preferred alternative.

After selection of the preferred alternative, the detailed design plans, specifications, and estimates should be prepared for that alternative. This may require further field and laboratory testing.

4.0 SELECTION OF OVERLAY AS APPROPRIATE

The selection process as outlined here produces the preferred alternative which best addresses the distresses of the pavement, while also being cost-effective and meeting the constraints of the project. This solution may or may not include the placement of an overlay as part of the strategy. While overlays have typically been the rehabilitation strategy most used by agencies, they have often been improperly designed (i.e., use of standard overlay design for all cases), and their relatively high cost has made other strategies more attractive. However, there are specific situations when overlays are a viable alternative and may be the preferred solution. Table 5-1.5 gives a partial listing of situations where overlays might possibly be warranted. It should be emphasized that this is only a listing of situations where an overlay could be a potential solution. A complete pavement analysis (as previously described) must be performed to determine if an overlay is the preferred solution.

In general, overlays are used to either:

1. Strengthen the existing pavement to support future traffic loadings.

2. Improve the surface characteristics of the pavement.

Improvement of the surface characteristics can address the functional condition of the pavement (rideability) or safety problems such as surface friction. It should be noted that pre-overlay repair of the distressed pavement is often required prior to the placement of the overlay.

5.0 SUMMARY

A brief description on the selection of the preferred rehabilitation strategy was presented. It was noted that the preferred strategy must address the pavement distress, yet be cost-effective and fit the constraints of the project. A logical procedure for rehabilitation selection was outlined, which included pavement evaluation, selection of candidate solutions, and selection of the preferred alternative. Finally, a brief overview of situations where overlays may be appropriate was presented.

Table 5-1.5. Possible Applications for Various Overlay Types.

OVERLAY TYPE	REASON FOR PLACEMENT
AC on AC	Structural Deficiency Severe Alligator Cracking Throughout Project Severe Rutting Throughout Project Substantial Loss of Support Throughout Project
AC on PCC	Structural Deficiency Severe Roughness Severe Faulting Severe Joint/Crack Deterioration
PCC on PCC (bonded)	Structural Deficiency Provide Adequate Surface Course
PCC on PCC (unbonded)	Structural Deficiency Severe Pavement Deterioration
PCC on AC	Structural Deficiency Severe Pavement Deterioration

6.0 REFERENCES

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MODULE 5-2

TYPES OF OVERLAYS AND THEIR FUNCTIONS

1.0 INSTRUCTIONAL OBJECTIVES

This module introduces the different types of overlays which are commonly used in pavement rehabilitation. The types of overlays are discussed in relation to the particular pavement for which they will be used. The requirements of the original pavement and how these factors relate to the suitability of a particular overlay are presented.

Upon completion of this module, the participant will be able to accomplish the following:

1. List the various reasons why overlays may be used in pavement rehabilitation.

2. State the effects of pre-overlay repair on overlay performance.

3. List the advantages and disadvantages of different overlay types.

4. List conditions for which each overlay type is best suited.

5. List the two major causes of reflection cracking and some potential solutions.

6. Discuss the impact that timing of overlay placement can have on the future performance of the pavement.

2.0 INTRODUCTION

Resurfacing is the most popular method of rehabilitation for existing pavements. Overlays may be used for many reasons, including the following (1):

2.1 Surficial Treatments

1. Increase skid resistance.
2. Improve pavement profile (level up).
3. Decrease roughness.
4. Improve surface drainage (increase slope).
5. Reduce water infiltration.
6. Retard environmental deterioration.
7. Enhance appearance.

2.2 Structural Treatments

1. Increase structural capacity.
2. Reduce rate of deterioration.

It is stressed that an evaluation similar to that described in Module 5-1 should be performed to ensure that an overlay is actually a feasible rehabilitation strategy. The type of overlay to be used for each pavement depends on the purpose of the overlay and the type and condition of the existing pavement.

3.0 TYPES OF OVERLAYS

There are two basic types of overlays: flexible overlays made with asphaltic materials (AC) and rigid overlays made with Portland cement concrete (PCC). There are several variations of each basic overlay type and each variation is most effective when used under specific conditions.

3.1 Asphalt Concrete Overlays

3.1.1 Types of Asphalt Concrete Overlays

Overlays made with asphaltic materials are used on both flexible and rigid pavements. For design purposes, the asphalt overlays are divided into the following categories according to the type of overlay materials used and the type of existing pavement:

1. Asphalt overlays on flexible pavements.

    a. Asphalt concrete overlay (AC) (dense graded hot mixed)
       - First application
       - Subsequent applications

    b. Surface treatments
       - Open graded porous friction course (PFC)
       - Chip seal course (CS)

2. Asphalt overlay on rigid (PCC) pavements.

    a. Asphalt concrete (AC)
       - First application
       - Subsequent applications

    b. Porous friction course (PFC)

Surface treatments, either the chip seal or the open-graded friction course, do not add to the structural capacity of the pavement. The major benefit of a chip seal coat is to waterproof the surface, whereas the major benefit of a porous friction course is to prevent hydroplaning (1). There is no thickness design procedure to use with this method of overlay; rather, the thickness of the treatment is governed by the size of the aggregate used.

Asphalt concrete overlays are assumed to be constructed as a hot mix with dense-graded aggregate and asphalt cement. State agencies will generally have different mixes for different overlay thicknesses (e.g. for base, binder, surfacing layers), as well as for certain friction requirements. State specifications should be consulted to select an appropriate material for the overlay.

3.1.2 Minimum Thickness Overlays

Minimum thickness overlays are used to correct surface deficiencies and correct non-load associated distresses. Other factors which affect the minimum thickness which can be applied include (1):

1. The minimum thickness of AC which can be placed with an acceptable surface is approximately one and one-half times the maximum particle size in the mix.

2. Thinner AC overlays cool much more quickly than thicker layers. To allow sufficient time for compaction to achieve the desired density, the required minimum thickness is greater in cold weather than in hot weather.

3. Thicker AC layers retard the development of reflective cracks and the severity of the cracks when they do form. Minimum thicknesses are often specified to ensure an adequate design life before these cracks reflect through to the surface and deteriorate.

4. The thinner the AC overlay, the greater the tendency for the overlay to pull free from the existing pavement and to develop potholes, or to develop slippage cracking and distortion.

Recommended minimum thicknesses for AC overlays are given in Table 5-2.1. The minimum thicknesses given are those judged by experience to provide adequate performance. These minimum values may need to be increased for unusual circumstances such as severe climatic conditions.

3.2 Portland Cement Concrete Overlays

Concrete overlays can be placed over both rigid and flexible pavements and are subdivided according to specific applications and the type of existing pavement. The most frequently used PCC overlays are listed below:

1. PCC overlay on existing PCC surface.

   a. Jointed concrete overlay (JPCP and JRCP)
      - unbonded
      - partially bonded
      - fully bonded

   b. Continuously reinforced overlays (CRC)
      - unbonded only

2. PCC on flexible pavement (unbonded only).

   a. Jointed concrete overlay (JRCP and JPCP)

   b. Continuously reinforced overlay

"Bonding" refers to the state of friction that is produced between the overlay and the existing surface and is an important design parameter.

3.2.1. Fully Bonded PCC Overlays

This type of overlay can be placed on JPCP, JRCP or CRCP. To achieve a fully bonded PCC overlay on a PCC surface, it is necessary to repair and carefully prepare the PCC surface of the existing pavement before placing the overlay. Repairs include full-depth repairs of deteriorated joints and cracks. Surface preparation includes removal of all oil, grease and surface contaminants, all paint, and all unsound concrete. This can be done by cold milling, sand blasting, or shot blasting. After cleaning the surface, a

Table 5-2.1. Sample Recommended Minimum Thickness (in inches) for Asphalt Concrete Overlays.*

Existing Pavement Surface Type	Traffic - ADT Both Directions			
	<750	750-2000	2000-3500	>3500
AC	1.0	1.5	2.0	2.5
PCC	1.5	2.0	3.0	4.0

*Assumes that all significant deterioration and localized failures are repaired prior to overlay.

grout made from sand and cement or cement and water should be placed on the clean dry surface just in front of the paver.

Fully bonded PCC overlays are most effective for the following situations:

1. The existing pavement does not exhibit many working cracks, or, if so, they will be full-depth repaired.

2. Joint deterioration is full- or partial-depth repaired.

3. Additional slab thickness is required to carry the anticipated future traffic.

4. The surface is rough.

Any working cracks will reflect through the overlay rapidly while tight cracks may take some years to reflect through the overlay and subsequently should not cause problems. As soon as possible after placement, joints in the overlay should be sawed directly above all joints (or working cracks) in the existing slab. The joint should be cut completely through the overlay to avoid secondary cracking. A minimum thickness of 2 to 3 inches is recommended for thin bonded overlay construction purposes. Reference 3 provides a detailed summary of the construction of bonded concrete overlays.

3.2.2. Partially Bonded PCC Overlays

This overlay is historically defined as the placement of new concrete over the existing slab without any special preparation of the surface. "Partial" bonding normally occurs whenever fresh concrete is placed directly on relatively sound, clean slabs. It is highly desirable to obtain as much bonding as possible; thus, the greater the cleaning of the existing surface the better. Repair of working cracks and deteriorated joints is again necessary to prevent reflection through the overlay.

3.2.3. Unbonded PCC Overlays

Unbonded overlays are achieved only if steps are taken to prevent bonding of the overlay to the existing PCC slab. Asphalt concrete has been commonly used for debonding the slabs. There is evidence, however, that AC layers of less than one inch do not provide an adequate bond breaker to ensure complete independent action of the slabs. The unbonded overlay is most applicable for existing concrete pavements that are badly deteriorated. Repair to the existing PCC pavement should be minimal.

When used on an existing flexible pavement, the PCC overlay thickness is designed as a new PCC pavement using the existing pavement as a subbase. The support k-value should be measured directly on the AC surface or estimated from deflection data.

A number of CRCP overlays have been constructed since 1959 with overall performance fairly good (4). The major problems have occurred where the existing slab had poor joint load transfer which resulted in a reflection crack in the CRCP overlay. This crack then deteriorated into a punchout requiring repair. For construction purposes, a minimum thickness requirement of 6 inches was recommended for CRC overlays by Illinois and Georgia DOT's.

4.0 PRE-OVERLAY TREATMENT/REPAIR

Since the existing pavement condition seriously affects the performance of the overlay, the amount and type of pre-overlay restoration (treatment and/or repair) needed on the existing pavement must be carefully determined for each overlay type being considered. Required pre-overlay repair is determined by the type of overlay, the structural adequacy of the existing pavement, the distress exhibited by the existing pavement, the thickness of the required overlay, costs of the pre-overlay repair, and other constraints. Treatments to reduce reflection cracking should also be considered as will be subsequently discussed in this module.

Condition surveys play an important role in the identification of the existing pavement distress. Other testing (such as deflection) may be required to help determine the extent of structurally weakened areas and existing joint/crack load transfer. Much of this pavement condition information can be obtained from the rehabilitation selection procedure outlined in Module 5-1.

Overlays add structure on top of the existing surface. Hence, if the cause of the deterioration lies beneath the surface, one of two general approaches must be followed (1). The first approach is to repair the distress prior to overlaying. A PCC pavement which is faulting and developing corner breaks normally has developed voids near the joints. Poor load transfer may also exist at the transverse joints/cracks. Before the pavement is overlaid, the void problem must be addressed. Cement grout subsealing to fill the voids and subsurface drainage to remove free moisture are treatments which should be considered.

An asphalt pavement having very high deflections in areas of alligator cracking has "base failure," and full-depth repairs are normally required. If the subbase or base has deteriorated significantly, reconstruction, reworking of the foundation materials, recycling, or some other strategy may be required (1).

The second approach is merely to add sufficient thickness to protect the weakened areas in the lower layers. If it is found that a stabilized subbase/base layer has deteriorated, the reduced strength of that layer should be considered in the design of the overlay. The thickness of the overlay is thereby increased to account for the decreased strength of the deteriorated layer and protect it from excessive stresses. As upper layer thicknesses are increased, the loads are distributed over larger areas in the lower pavement layers, decreasing the stress and deflection imposed on the deteriorated layer. However, the thickness required to adequately protect very weak layers can become so large that this approach is not usually an economically feasible alternative.

Regardless of the type of overlay or the type of base pavement, one factor that should always be addressed is drainage, both surface and subsurface. If the present pavement is deteriorating because of poor drainage, adding an overlay will not correct the problem. Overlay design procedures generally assume that the existing pavement has adequate drainage. A drainage survey and evaluation to determine deficiencies should be a part of a pre-overlay analysis (2).

4.1 Localized Repair

Many pavements have areas of localized distress caused by material and subgrade variability, such as alligator-cracked areas in asphalt pavements, and shattered or broken slabs in concrete pavements. Deflections in these areas are generally much higher than in other areas. Failure to repair these areas normally results in a decreased service life of the overlay.

Some full-depth and/or partial-depth repair is normally required to achieve an economical solution. An analysis of repair costs and the resulting change in overlay cost should be made, as is illustrated conceptually in Figure 5-2.1.

An asphalt overlay must be designed for the mean deflection plus some level of variation (such as two standard deviations) based on desired design reliability. As alligator-cracked areas in asphalt pavements are patched, the overall deflections and variability will be reduced. Thus, more repair will decrease required overlay thickness to near the level required in areas which do not need repair. Thus, as the repair cost increases, the overlay costs will decrease to some minimum level. The lowest initial total cost is therefore a balance between repair costs and the cost of the overlay (1). However, it must be pointed out that any deteriorated areas that are not repaired will eventually reflect through AC overlays and bonded PCC overlays and considerably shorten the life of the overlay.

4.2 Surface Leveling

Transverse surface irregularities in flexible pavements (rutting, crown problems, etc.) can sometimes be corrected by an overlay; however, it has been found that rutting will often reappear in an overlaid pavement depending on the initial cause of the rutting. This has been attributed to the difficulty of adequately compacting the asphalt concrete in the rutted areas. This problem of differential compaction can be corrected by milling the surface to remove the irregularities prior to overlaying, or filling the ruts with a stable leveling course that is properly compacted using rubber tired rollers prior to placement of the overlay. An overlay will not correct a rutting problem that is caused by a deficiency in the asphalt concrete mixture.

4.3 Joint and Corner Voids

When voids (or loss of support) exist under the corners of concrete slabs and adjacent to joints, the deflection at the joints will be higher than normal. This can lead to rapid formation and deterioration of reflective cracks in asphalt overlays. Voids are usually associated with poor drainage which allows the pavement to remain exposed to saturation for long periods of time. The removal of free water by subdrainage accompanied with filling the voids by subsealing should be considered. Subsealing alone is not adequate to prevent future pumping.

5.0 SELECTION OF OVERLAY TYPE

The purpose of an overlay design procedure is to determine the overlay thickness in conjunction with pre-overlay repair required to provide a

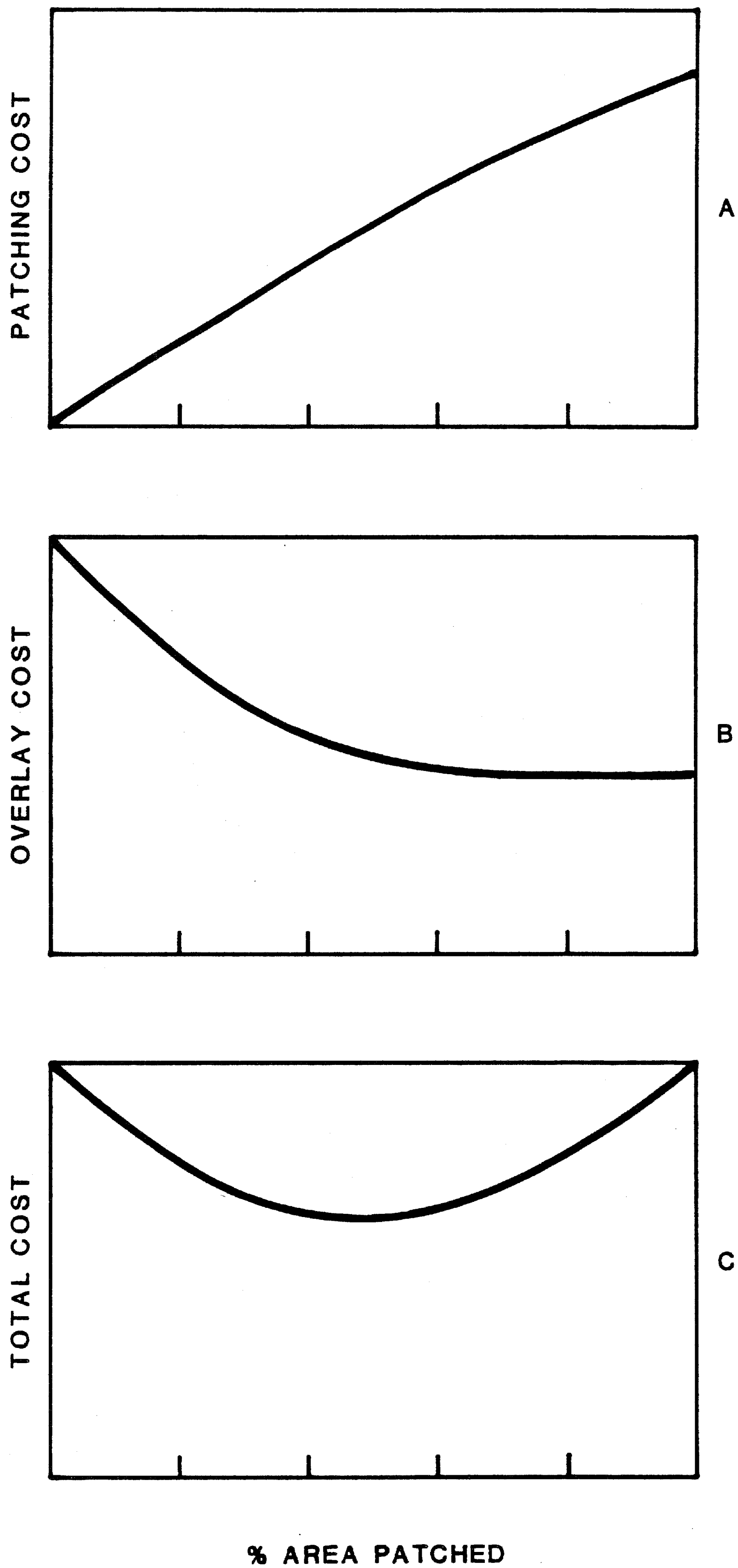


Figure 5-2.1. Repair Cost, Overlay Cost, and Total Cost vs. Percent Area Repaired (1).

serviceable pavement over the design period. If the existing pavement is structurally inadequate for the expected traffic over the design period, the overlay thickness must be sufficient to increase the structural capacity of the pavement to carry the traffic for the design period.

An overlay design procedure must contain several features to adequately provide the capabilities for which it was developed. Two basic requirements are (2):

1. Differentiate among areas of the existing pavement which have performed differently and should receive different treatment.

2. Determine different overlay thicknesses for various sections, thus providing equivalent performance for all sections.

To provide these results, the following features should be included in the overlay design procedure (2):

1. The technique used to measure performance and determine the need for an overlay must be clearly defined.

2. The method must account for the many possible different conditions of the pavement prior to the time of the overlay.

3. The method must define required repair to the existing surface prior to placing the overlay.

4. The information needed for the design procedure to determine the overlay thickness must be clearly defined. This should include a description of all tests and measurements required and provide a method for accounting for daily and seasonal influences on test results. In addition, the procedure should clearly define the location at which any tests should be taken on the pavement and the frequency of those tests.

5. The design procedure should account for subgrade support conditions as well as material properties in each of the pavement layers.

6. The method should account for magnitude, type, and number of traffic loadings.

7. The method should account for the environmental influences on the performance of pavements.

8. Any additional constraints established by the procedure must be clearly defined.

9. It is desirable that the method be able to use different design lives in terms of both time and traffic and performance.

5.1 Design Approaches

There are four basic approaches used in the design of overlays:

1. Engineering judgment.

2. Structural deficiency.

3. Deflection.

4. Mechanistic fatigue damage.

Each of these approaches is summarized briefly below.

5.1.1 The Engineering Judgment Approach

The engineering judgment approach is based upon the experience and judgment of the engineer in determining the thickness and materials required. This approach is based upon observations of performance of similar pavements. Even with engineering judgment, it may be necessary to conduct testing to characterize the existing materials and pavement structure. This is not a requirement, but it may aid the engineer in making an informed judgment.

5.1.2 The Structural Deficiency Approach

The structural deficiency approach is based on the concept that the required overlay thickness is the difference between the effective structural thickness of a newly designed pavement over the existing roadbed soil and the effective structural thickness of the existing pavement. Much of the required data can be obtained from the "as-built" documents and from a distress evaluation of the existing pavement, but coring, testing, and/or nondestructive testing (NDT) may be required to determine existing pavement layer properties and moduli.

5.1.3 The Deflection Approach

The deflection approach is based upon the concept that the required overlay thickness is that necessary to reduce the "as is" measured maximum deflection to a tolerable (limiting) deflection to carry future traffic over the design period. The limiting deflections are determined from an empirical relationship developed between the number of 18-kip equivalent axle loads to terminal serviceability and deflection. Deflection testing is required to develop the maximum deflection profile along the existing pavement.

5.1.4 Mechanistic Approach

The mechanistic approach requires some testing, varying from a few tests to an extensive testing program. The overlay design in this approach is based on the mechanistic analysis of multilayer elastic systems for fatigue damage (cracking) and permanent deformation (rutting). Several computer programs are available for this type of analysis.

5.2 Inputs for Overlay Design

There are many inputs required for the design of overlays. These are discussed below.

5.2.1 Visual Pavement Condition

As previously discussed, the condition of the existing pavement is a major factor influencing the design and selection of the overlay type. Consideration should be given to the type, level of severity, and amount of pavement distress. If a pavement exhibits more than one type of distress, the distress which requires the largest overlay thickness governs. However, if the distress requiring the largest overlay thickness occurs only in isolated areas, then it may be more economical to repair the distressed area and use the overlay thickness required for the most extensive, but less serious, distress.

Many pavements that are still structurally sound may require an overlay because the surface has deteriorated due to scaling, studded tire rutting, ravelling, or reduced friction resistance. A visual condition survey can identify these surficial problems and some structural problems as described below:

1. Identify surficial problems of the existing pavement (e.g. cross slope, surficial distress, hydroplaning from ruts immediate surface repairs needed).

2. Identify structural load-associated distress (e.g. fatigue cracking, rutting, corner breaks).

3. Provide guidance in performing a structural analysis to determine if a structural overlay or is needed.

4. Determine the adequacy of surface and subsurface drainage.

5. Evaluate the type and extent of pavement distress.

6. Determine the amount and type of additional needed diagnostic investigation.

5.2.2 Diagnostic Investigation

One possible additional diagnostic investigation is structural evaluation to provide data for the design of overlays. Overlay design can be accomplished using a series of tests which are statistically representative of pavement components through coring, deflection measurements at the pavement surface, or a combination of both.

A knowledge of the structural condition of the pavement along its length and across lanes for multilane facilities provides valuable information for selecting feasible overlay alternatives and aiding in their design. A structural evaluation can be developed by considering existing distress, nondestructive testing (NDT), and destructive testing (coring). Existing distress caused primarily by traffic loadings along the project provides information on the structural effect of previous traffic. There are, however, limitations as to the amount of information that can be obtained from this source, and it may be desirable to conduct NDT and/or destructive testing to obtain further knowledge of the structural condition.

NDT can be used in the following capacities:

1. Assist in the design of overlays.

2. Provide a measure of deflection variability along the project that can be used to select distinct design sections.

3. Assist in determination of causes of distress; locate inadequate base, subgrade or voids, and determine load transfer efficiency at joints and cracks (see Figure 5-2.2).

4. Evaluate the effect of seasonal changes on the pavement (see Figure 5-2.3).

5. Determine current pavement support and in situ stiffness values of each layer.

Used with or without NDT diagnostic measurements, destructive testing accomplished through coring, test pits, sampling and testing can provide the following:

1. Representative material samples of each pavement layer and subgrade can be obtained and tested for various properties, including layer thickness, strength, repeated load behavior, elastic properties, water content, density, gradation, asphalt content, asphalt cement properties, steel corrosion, etc.

2. Detailed investigations of localized conditions to determine the cause or extent of deterioration near cracks, joints, wheelpaths, etc.

Roughness is also often used as an input to indicate how the traveling public regards the condition of a pavement. There are several devices which can be used to measure roughness and it can also be rated subjectively by a panel. Each device yields a different output and the using agency generally develops some procedure to convert the readings into a standard rating such as present serviceability index (PSI).

Material properties are often inputs to overlay design procedures. These properties can be determined in a number of ways using different test procedures. For instance, California bearing ratios (CBR) for subgrade soils can be determined for material compacted in different ways and resistance values (R-values) can be conducted at different pressures. Indirect tensile strengths and resilient properties can also be determined from different tests.

Environmental inputs are necessary to adjust the expected life of the overlay for nonload related deterioration. Most environmental factors are used in determining the required thickness to resist expected traffic loadings during different seasons. However, a structurally sound overlay can fail from strictly environmentally induced distresses. When the overlay procedure is to be used in more than one environmental zone, the important environmental factors such as precipitation, maximum and minimum temperatures, freezing index, number of freeze-thaw cycles, Thornthwaite Moisture Index, etc., should be included as design inputs.

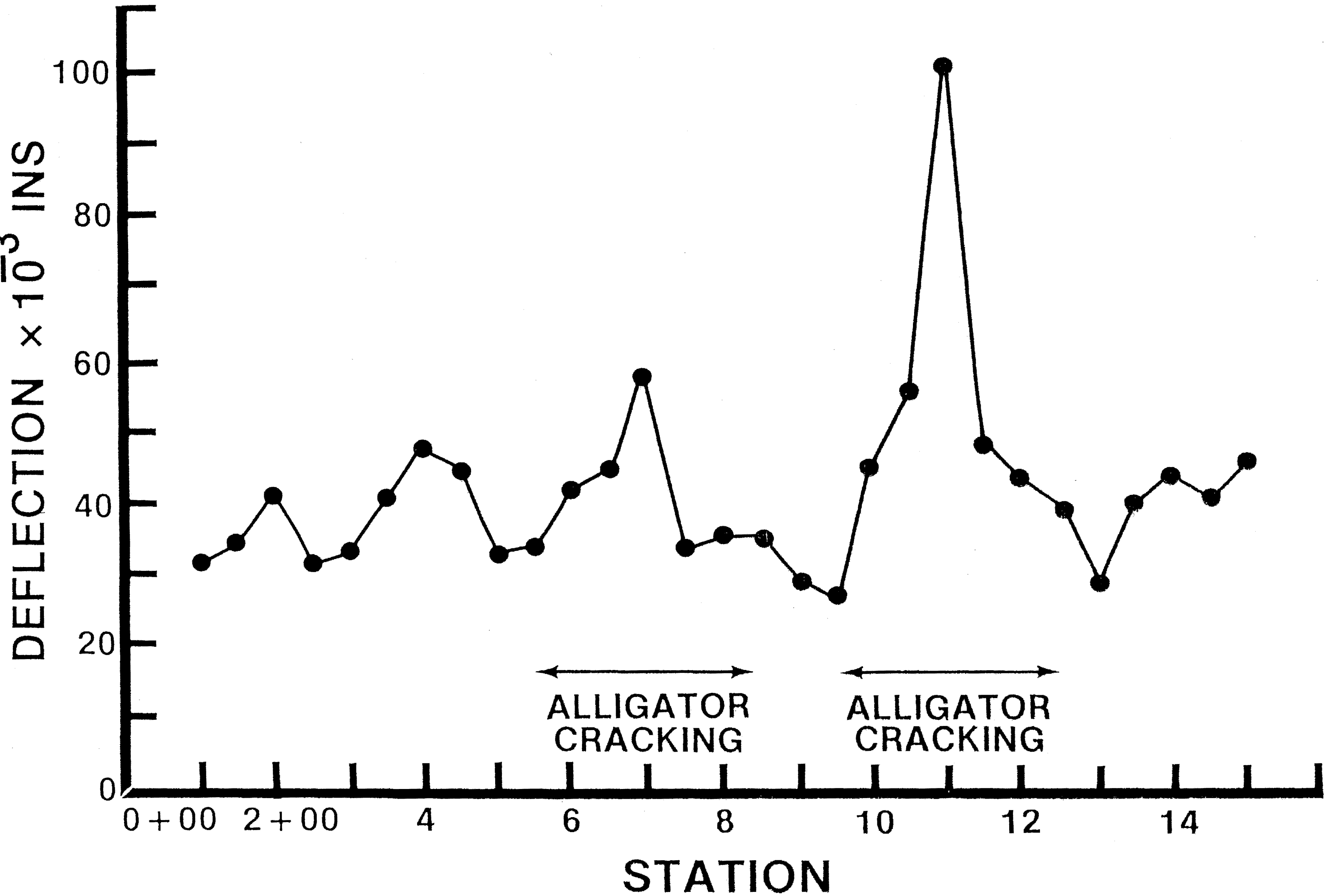


Figure 5-2.2. Variation of Deflection Along Project (1).

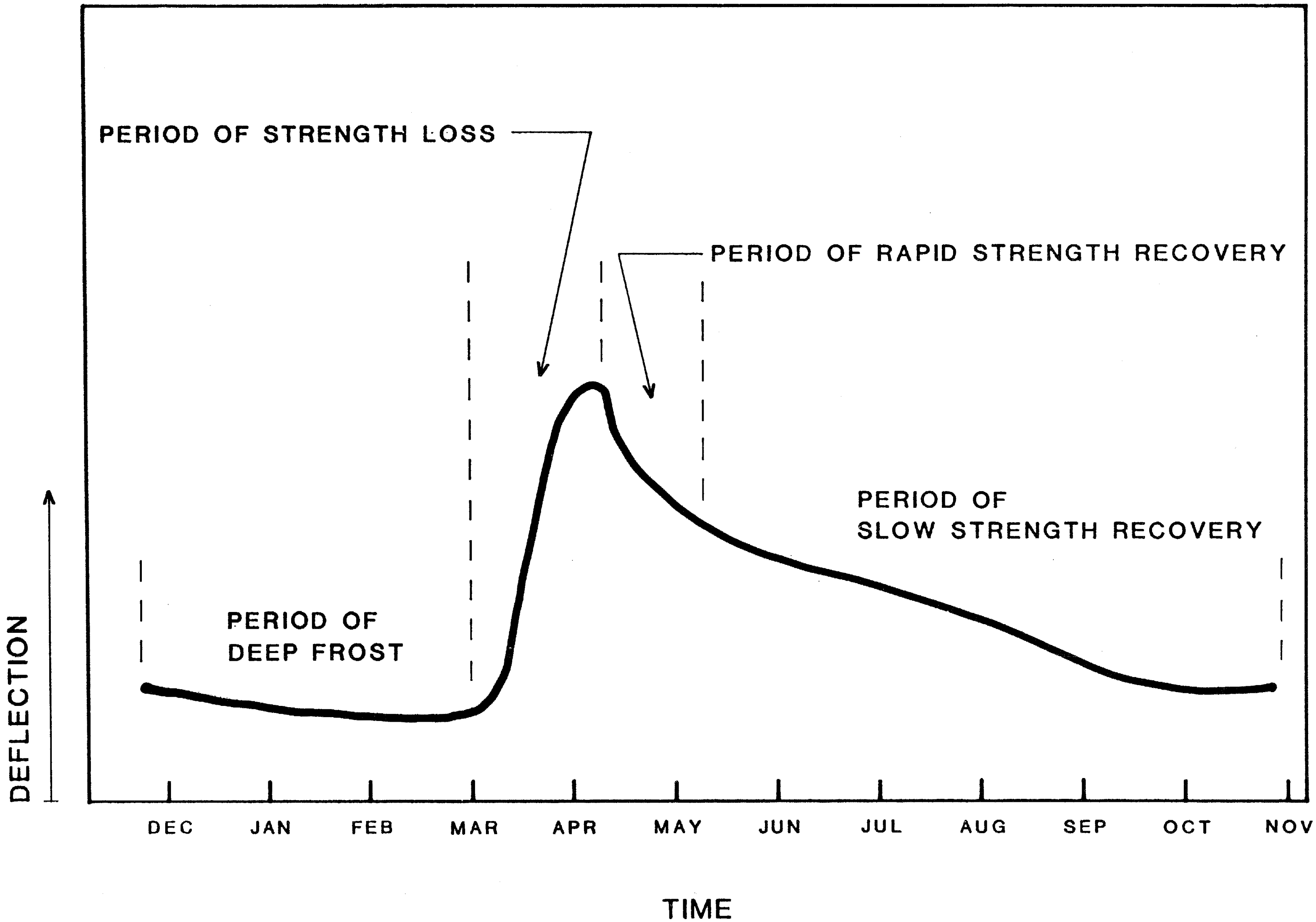


Figure 5-2.3. Influence of Season on Pavement Deflection (1).

5.3 Selection of Feasible Overlay Types

The selection of feasible overlay types to be considered should be based on an analysis of the information gathered during the project survey and evaluation (as described in Module 5-1). A list of factors to be considered for selection are given below.

1. The first factor to be considered is the existing pavement type (AC, AC over PCC, JPCP, JRCP or CRCP). The following are considered feasible:

Existing Pavement	Feasible Overlay Type (condition)
AC (Flexible)	AC (good-fair) Any PCC (any)
AC/PCC	AC (good-fair) Unbonded PCC (any)
JPCP	AC (good-fair) Bonded JPCP (good) Partially Bonded JPCP (good-fair) Unbonded - Any PCC (poor)
JRCP	AC (good-fair) Bonded JPCP (good) Partially Bonded JRCP (good-fair) Unbonded Any PCC (poor)
CRCP	AC (good-fair) Bonded PCC (no joints or reinf.) (good) Unbonded - Any PCC (poor)

2. The structural adequacy of the pavement must be considered. If it is structurally adequate, then only surface correction may be needed. Structural adequacy can be based on visual identification of load-associated distress, deflection analysis, or coring and analytical methods. If the pavement is not structurally adequate, then a structural overlay design will be required. The load transfer of cracks and joints in the existing pavement should also be included in this part of the evaluation.

3. The distress of the existing pavement should be considered next. If surface distress can be rated as good to fair, then most overlay options are available and life-cycle cost should be the primary factor. If the distress is rated as poor, the options become more limited. For instance, bonded and partially bonded PCC overlays should only be placed on existing PCC pavements which have relatively few cracked slabs. If all cracked slabs are replaced, a bonded or partially bonded PCC overlay can be used. Otherwise, only unbonded PCC or thick asphalt overlays should be considered. The list above under Item 1 can be consulted for recommendations for varying conditions.

4. As previously discussed, the amount of pre-overlay repair or type of pre-overlay treatment should also be considered. If all areas of medium- and high-severity alligator cracking are repaired with full-depth patching prior to the overlay of an asphalt pavement, then a thinner asphalt overlay can be used. If they are not repaired, then a thicker asphalt or a PCC overlay will be necessary.

There are numerous other factors (as discussed in Module 5-1) that should be considered, including:

1. Initial Cost.
2. Life-Cycle Cost.
3. Overhead Clearance Problems.
4. Curb Clearance.
5. Traffic Control Constraints During Construction.
6. Design Reliability and Expected Performance.
7. Future Maintenance Options.
8. Material Availability.
9. Energy and Environmental Constraints.

5.4 Milling and Recycling as an Option

As part of the overlay selection process, the use of a combination of milling, recycling and overlay should be considered. Milling refers to the removal of some of the existing pavement. Recycling refers to the operation of reusing (usually after some processing) some of the existing material from the project or other projects. Depending on conditions, a combination of milling and overlay or milling, recycling and overlay may be a feasible alternative for good performance.

5.4.1 Consideration Factors for Selection of Recycling

Among some of the factors contributing to the process of selecting recycling as a component of the overlay for any specific project are the following:

1. Location and Size of Project.
2. Existing Pavement Cross Section.
3. Pavement Cross-Section Slope, Distortion, Rutting.
4. Geometrics.
5. Pavement Section.

In summary, all known information about the pavement materials and background needs to be summarized and used in the decision process. Surprises at the time of construction can usually be avoided by proper testing, evaluation, planning, and design.

5.4.2 Flexible Pavement Recycling/Overlay

There are three basic flexible pavement recycling methods that could be used in combination with overlay on AC pavement:

1. Surface.

2. In-place.

3. Central plant, hot or cold mix processing.

There are several combinations of methods and equipment that could be utilized in a project, in which case, specific advantages and disadvantages of each process should be carefully considered. Only surface recycling is discussed in combination with overlaying, as it offers a great amount of flexibility to work with the existing pavement.

Surface recycling is defined in the NCHRP 1-17 study (9) as the reworking of the surface of a pavement to a depth of less than about an inch by heater-planer, heater scarifier, hot-milling, cold-planing, or cold milling device. It is a continuous, single- or multiple-pass operation that can involve the use of new AC overlay materials, including aggregates, rejuvenators or mixtures.

Surface recycling has specific advantages, including the reduction of thermal reflection cracking, improvement of skid resistance, reduction of localized roughness, and treatment of a variety of distress types (minor raveling, minor flushing, minor corrugation, slight rutting, oxidation, minor faulting) in a cost-effective manner. It promotes a bond between the old pavement and thin overlay. This procedure should not be used on severely flushed pavements or pavements with pronounced structural problems.

The remaining recycling options are considered to be a normal process to furnish material for an asphalt concrete mixture. If their use is economical in an overlay project, and the thickness reduction from removal is economical, recycling should be included.

6.0 TIMING OF RESURFACING

After pavements are constructed, they generally perform in a satisfactory manner for some period of time with a slow rate of deterioration. At some point they begin to deteriorate quickly, as illustrated in Figure 5-2.4(a). The time at which an overlay is placed may have significant impact on the cost-effectiveness of the overlay. User costs, pre-overlay restoration, overlay thickness, construction costs, and overlay life are all affected by the condition of the existing pavement prior to the overlay. As the pavement serviceability decreases below a critical level, user costs generally increase rapidly. These user costs include vehicle maintenance and operation costs, delay, and discomfort. The change in user cost is illustrated in Figure 5-2.4(b).

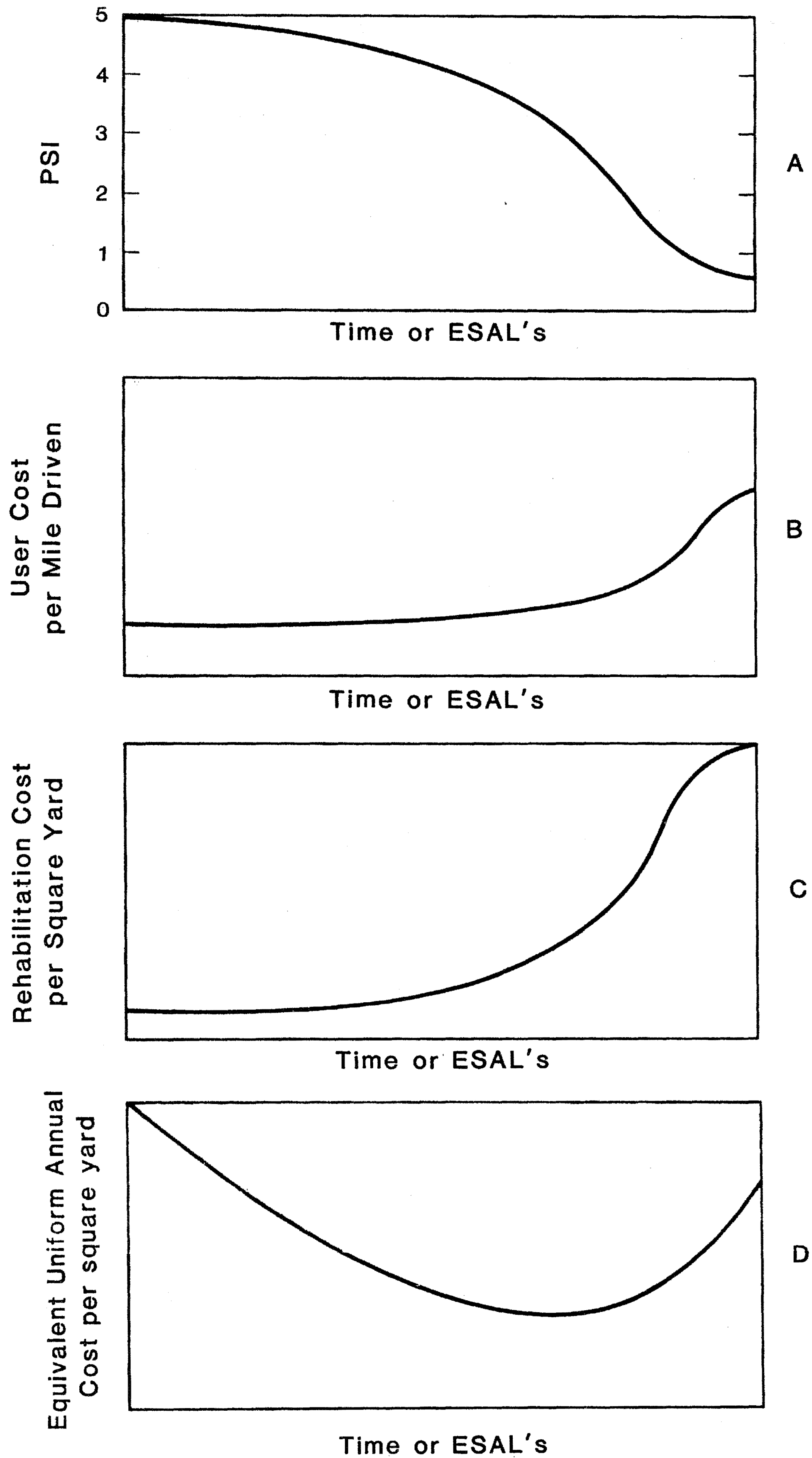


Figure 5-2.4. PSI, User Cost, Rehabilitation Cost and Equivalent Uniform Annual Cost vs Cumulative Time or Traffic Loadings (2).

As the pavement condition deteriorates, additional distress is also developing. This additional distress requires additional repair, in terms of patching, subsealing, and joint repair prior to placement of the overlay. Even with these repairs, deflection analysis may indicate that additional thickness is required for the overlay. All of these factors lead to increased rehabilitation costs as illustrated in Figure 5-2.4(c). Not only are costs affected by the time of placement, but the service life of the overlay may also be decreased, even with increased thickness (due to "unseen" deterioration in the pavement structure, such as softening of pavement layers from excessive free moisture, contamination of layers from fines pumped from the subgrade and micro-cracks in AC or PCC from fatigue damage).

The life-cycle cost including user and rehabilitation costs will increase as rehabilitation is delayed. Figure 5-2.4(d) shows how life-cycle costs could vary if an overlay were placed at any year after construction. The life-cycle costs would be high if an overlay was placed shortly after construction because it would be an unnecessary expenditure. As the pavement ages and is subjected to traffic, its serviceability level decreases which increases user costs, pre-overlay repair requirements and subsequently the required overlay thickness. The low point in the curve of total annual costs indicates the optimum time to perform rehabilitation. Any delay in placing the overlay beyond that time will result in increased life-cycle and rehabilitation costs, often referred to as "deferred maintenance costs" (1).

Overlays should be planned so that they can be placed at the optimum time. Since it can take several years of advance planning to get an overlay constructed, the agency needs to monitor the pavement to detect deterioration levels which indicate the need for an overlay in the near future.

7.0 ADVANTAGES/DISADVANTAGES OF ALTERNATE OVERLAYS

Each overlay type has distinct advantages and disadvantages. Flexible overlays often allow easier traffic control because only one lane needs to be closed at a time. Rigid overlays often do not have that advantage although unique construction and traffic control techniques have been used to alleviate this problem.

All types of overlays have the capability to provide a smooth wearing surface and to increase serviceability to a level as good or better than that produced during the initial construction.

Nearly all types and designs of overlays have the disadvantages of decreasing overhead clearances, requiring overlaying of the shoulders (which may not need it), raising guard rails, placing additional fill material adjacent to shoulders, decreasing curb height, and disturbing drainage patterns. It has been found that in some instances, successive overlays provide shorter and shorter life extensions (e.g., first overlay fails after 15 years, second one after 9 years) (10). This would, of course, depend greatly upon the pre-overlay repair and design.

In general, thick AC overlays and unbonded PCC overlays are most practical when the existing pavement is in poor condition and is more suited as a "support" for the overlay rather than as a part of the pavement structural system. Thus, these are two solutions that are generally acceptable for all conditions, although these solutions may not always be the most cost-effective.

If the existing pavement is structurally sound or can be economically restored to a structurally sound condition, then other alternatives are available to make maximum use of the existing pavement structure. These alternatives would include minimum thicknesses of AC overlays with a variety of reflective crack control possibilities for flexible pavements repaired to good to fair condition. Bonded or partially bonded PCC overlays or minimum thickness of AC overlays with reflective crack control are possible for existing PCC pavements.

8.0 PROBLEMS/LIMITATIONS OF CURRENT OVERLAY DESIGN

A number of important problems that must be solved to improve evaluation and overlay design techniques are listed below (11):

1. More effort should be directed toward developing comparisons between stiffness properties estimated by deflection or other nondestructive measuring techniques and laboratory-determined stiffness values from cores and bulk samples. This would greatly assist in the improvement of overlay design procedures.

2. Considerable effort should be directed toward solving the reflection cracking problem in overlays. Reflection cracking is the major problem with overlays and should be considered in the design process.

3. There is a lack of performance data for overlays. Because this type of information requires time to accumulate, efforts should be directed toward this aspect of performance evaluation.

9.0 REFLECTION CRACKING

The overlay design procedures in use today provide an overlay thickness that will prolong the life of the pavement from either a serviceability viewpoint (by reducing roughness), or from a structural viewpoint (by reducing deflections). A comprehensive and practical design procedure considering the problem of reflection cracking is not yet available to the highway industry. Numerous researchers are currently investigating this problem which may lead to design considerations available in the near future (2). This section provides a brief overview of the causes and potential solutions for reflection cracking.

9.1 Causes

A reflection crack is a crack in a new surface that appears as a result of the presence of a crack in the old surface. Reflection cracking occurs in all types of pavements but is pronounced in resurfaced jointed concrete pavements and in flexible pavements with low temperature thermal cracks. The movements at these joints or cracks cause a physical tearing of the overlay material right at the discontinuity produced by the joint or crack (2).

There are different reasons for the movement at the existing joints or cracks:

1. Large seasonal temperature variation.

2. Daily temperature cycles.

3. Traffic loads.

Lower seasonal temperatures produce a horizontal opening due to the contraction of the original surface (see Figure 5-2.5). A thermally induced tensile stress is also produced in the overlay due to the contraction that the overlay material experiences, as shown in Figure 5-2.6. The combination of these stresses will be most severe directly over the joint or crack.

Daily temperature cycles also produce thermal tensile stresses in the overlay. In a concrete pavement, the temperature cycling produces temperature gradients in the slab that result in curling at the joints. When the temperature drops and the top of the slab is cooler than the bottom, the slab will curl upward (see Figure 5-2.7). This causes an opening that, while less severe than a low temperature opening, occurs far more often (2).

Traffic loading results in a completely different type of deformation as shown in Figure 5-2.8. This differential vertical deflection across the joint causes a shearing action in the overlay rather than the opening mode produced by the other causes. This type of deformation occurs much more often than the others but is typically much smaller which reduces the rate of propagation (2).

9.2. Factors Affecting Propagation

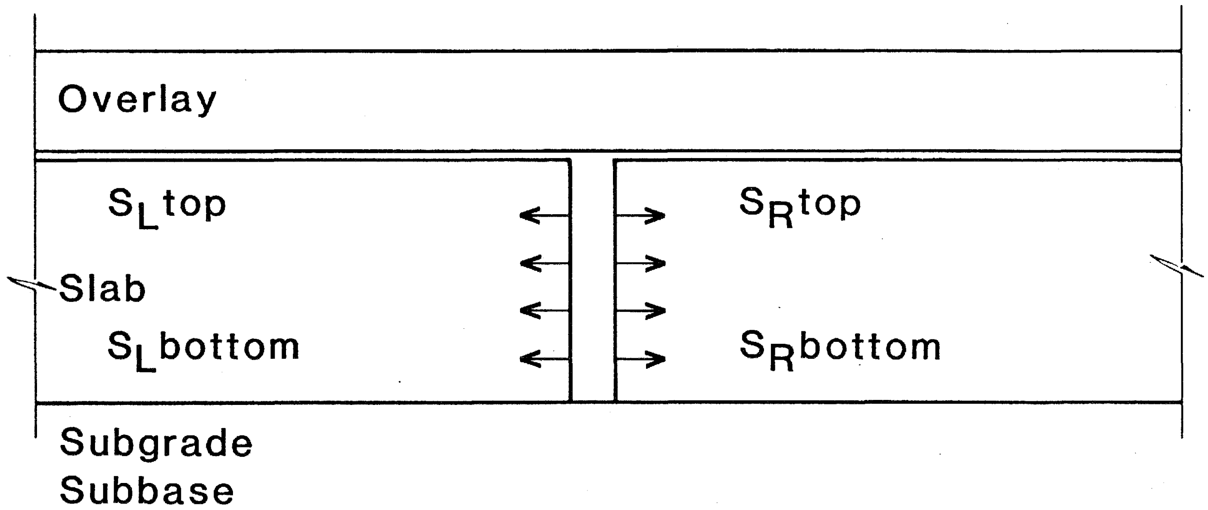
Each cycle of low temperature and traffic causes damage to the overlay and pushes the crack further up into the overlay. The crack does not propagate equally with each type of deformation, even for the same magnitude, and the number of loads by itself is not a criterion for predicting the rate of reflection cracking. The rate of growth of the crack depends upon many things, including the amount of load transfer that exists across the crack or joint in the old pavement. The less load transfer that exists, the quicker the crack will propagate. When designing an overlay to resist reflection cracking it is important to determine the amount of load transfer exhibited by the old pavement (2).

Once a reflection crack breaks through the overlay, the rate at which further deterioration (such as spalling) occurs is dictated by the materials and the load transfer which affects the amount of relative movement from one side of the crack to the other as a load passes over (2). The mechanics of reflection cracking will not be covered here, but a complete analysis on this subject is given in Reference 12.

The key to eliminating reflection cracking would be to eliminate the deformations and stresses produced over the existing discontinuity. Since this cannot be accomplished, efforts must focus on reducing the rate of appearance and subsequent rate of deterioration of reflection cracking.

9.3 Influence of Deflections

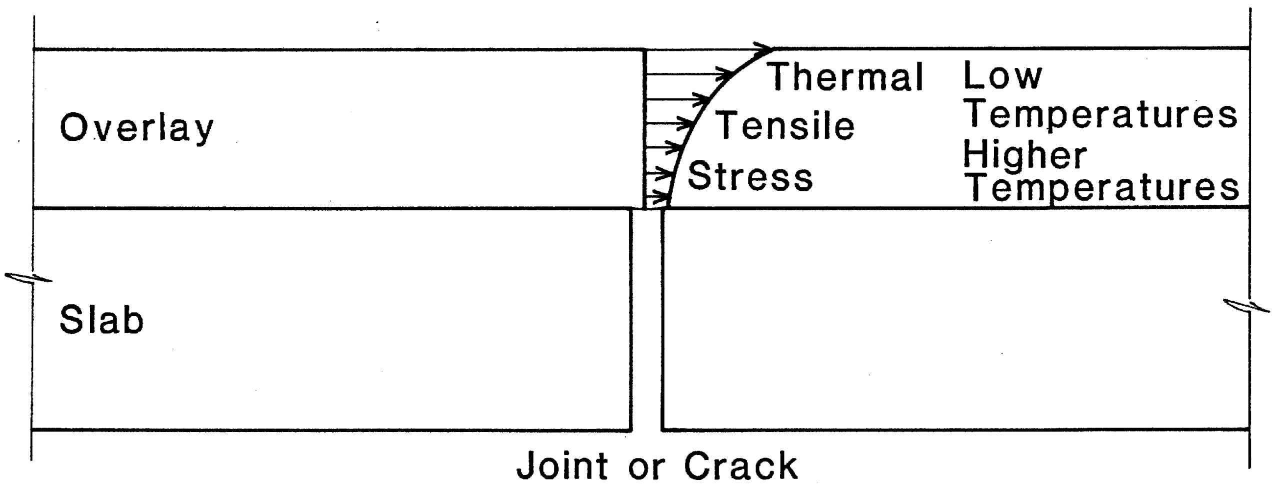
A Virginia study yielded the data presented in Table 5-2.2, showing the influence of different vertical deflections on the rate of reflection cracking. The horizontal deformations are assumed to be the same for all



Note:

1. S_L is not necessarily equal to S_R if joint or crack spacing varies from one slab to another.
2. S_{top} may not be equal to S_{bottom} which will also produce curling.
3. The horizontal opening at the top of the slab will vary daily with short term temperature cycling as well as seasonally due to long term temperature cycling.

Figure 5-2.5. Horizontal Stresses Due to Temperature Change in Existing Pavement (2).



Note:

Thermal crack will start from top. With a reflection crack starting at the bottom, the stress distribution will actually propagate a crack from both directions.

Figure 5-2.6 Action of Thermal Stresses in Overlay Above Joint or Crack (2).

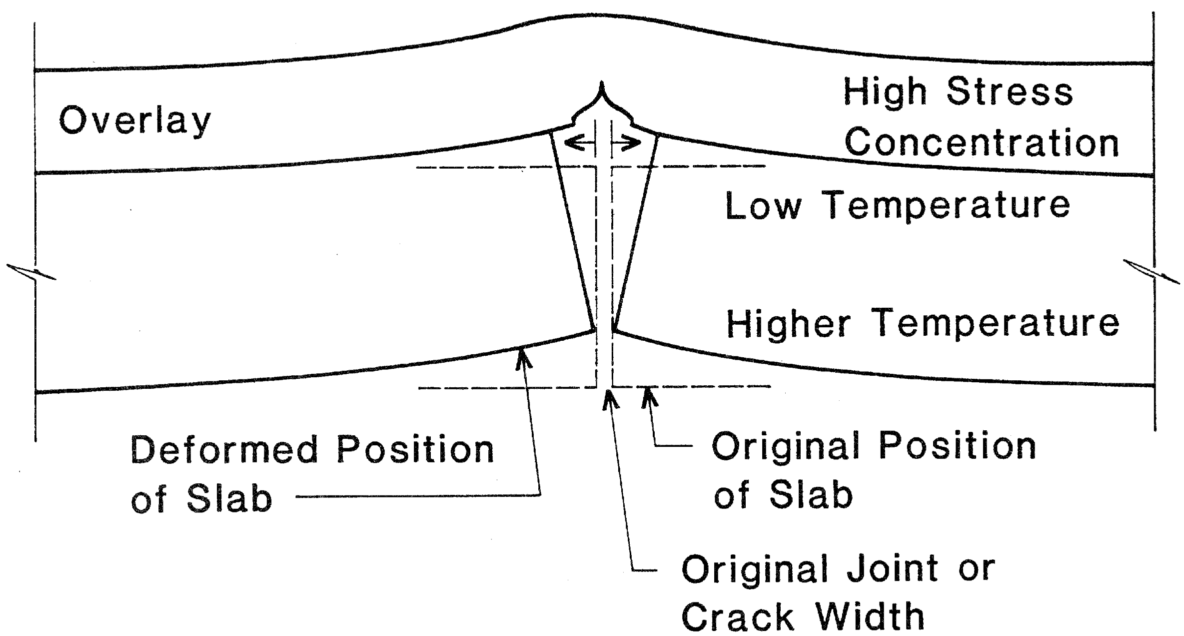


Figure 5-2.7. Stress Caused by Thermal Curling of the Pavement Slab Due to Temperature Differential Through the Pavement Slab (2).

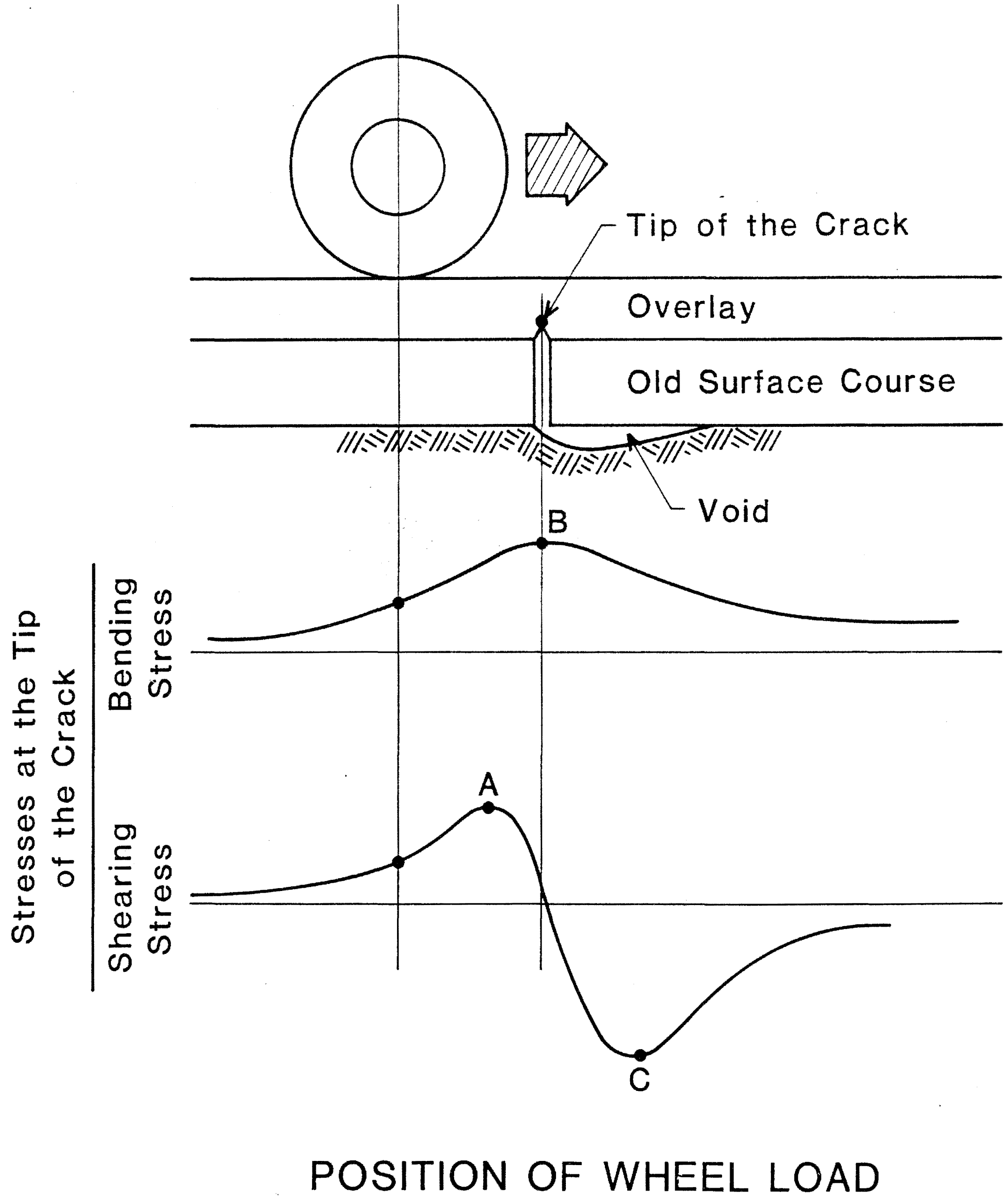


Figure 5-2.8. Traffic Pulse Induced Stresses and Crack Growth in Overlay (2).

Table 5-2.2. Influence of Differential Vertical Deflection on Rate of Reflection Cracking (2).

Differential Vertical Deflection		% Of Joints Cracked	
in	(mm)	Fabric	Control
0.000	(0.00)	0	44
0.002	(0.05)	29	54
0.004	(0.10)	74	88
0.006	(0.15)	88	100
0.008	(0.20)	100	100

sections. The Asphalt Institute (13) calls for limiting differential deflection to 0.002 inches under a 9000 lb wheel load for a good performing overlay and also specifies that the maximum opening resulting from low temperatures should be limited to 0.05 inches. Since typical joints and cracks in PCC or AC pavements exhibit movements several times greater than these values it is unlikely that these limits on deformation levels can be economically accomplished in most circumstances.

9.4 Treatments

Present concerns with reflection cracking focus on three areas:

1. The rate of reflection cracking through the overlay.

2. The severity of the crack after reflection cracking occurs.

3. The amount of water that can infiltrate through the crack.

The treatments discussed in this section are directed toward one or more of these areas. Procedures used in overlays to reduce reflection cracking attempt to serve one or more of the following functions (2):

1. Eliminate the cracks completely.

2. Reduce the rate of appearance of the reflection cracking.

3. Reduce the severity of the reflection cracks when they eventually do occur.

4. Provide a waterproof barrier that will prevent moisture from penetrating into the foundation materials once the overlay does crack.

Several treatment methods have been used in an attempt to reduce the rate or severity of reflection cracking. These methods are summarized below.

9.4.1 Fabric Stress Relieving Interlayer

Geotextile treatments employ fabrics that may be woven or nonwoven synthetic materials made of polypropolene, polyester, fiberglass, nylon, or combinations of these materials. Some of the more commonly used fabrics include Petromat, Typar, Bidim, Mirafi, and others (15).

The purpose of the fabric is fourfold:

1. Provide a stress-relieving interlayer.

2. Provide physical restraint to the movement of the crack as the underlying joint or crack opens.

3. Keep the width of the crack small once it has reflected through.

4. Retard or prevent the infiltration of water through the reflection crack into the sublayers.

Fabrics have experienced a mixed success rate. They have been used with both flexible and rigid pavements, with the best results coming from flexible pavement applications. Even if the overlay cracks, these treatments do provide some measure of waterproofing, although the extent of waterproofing has not been substantiated. Best results occur when the fabric is placed within the overlay, which will require two layers exceeding a minimum thickness.

9.4.2 Reinforcing Fabrics and Grids

The major difference between fabrics used for stress relief and those used for reinforcing is their in-place modulus relative to the asphaltic concrete of the overlay. Reinforcing fabrics or grids have a higher modulus than that of the surrounding overlay. Typical reinforcing fabrics are woven fiberglass such as Roadbond/Roadglass, Burlington glass fabric, and Bay Mills Glasgrid; reinforcing grids include Tensar and others (2). It is only with reinforcing fabrics and grids that the reflection crack may be stopped below the reinforcing layer and turned to travel horizontally.

Although fabrics and grids may provide some benefit to the overlay in retarding load-induced reflection cracking, their principal benefit is in retarding thermally-caused reflection cracking. Thus it is wise before using any fabric or grid to provide as much load transfer and as good a supporting base layer below the old pavement surface as possible. This can be accomplished in PCC pavements through restoration of load transfer across joints and cracks with dowel bars placed in slabs and through subsealing if loss of support exists.

9.4.3 Stress-Relieving Interlayer

This treatment differs from stress-relieving fabric in that it is constructed on the existing pavement surface. This treatment can cover the entire pavement surface and generally involves a spray application of rubber asphalt with aggregate chips. Its function is the same as the manufactured treatments, i.e., to absorb the deformations from the existing pavement.

One such treatment is called the Stress Absorbing Membrane Interlayer, (SAMI), which is a surface treatment made with rubber asphalt and stone chips and may be up to 0.5 inch thick. It has worked effectively over flexible pavements with fatigue cracking, but provides no benefit for vertical deformation produced by traffic over a transverse joint or transverse working crack.

9.4.4 Crack-Arresting Interlayer

The crack-arresting interlayer is a thick granular layer that arrests or stops the development of the reflection crack by providing a large void space that effectively blunts the cracks. Open-graded granular layers about 3 1/2 inches thick have been used with low fines content and large-sized aggregates. They are often stabilized with asphalt cement.

Crack-arresting interlayers have proven to be effective when properly designed and constructed. Their effectiveness is reduced when not properly compacted, resulting in an unstable mix. Due to the large top size of the aggregate, the layer is generally 3 to 4 inches thick plus an AC surfacing on top, causing clearance and shoulder elevation problems. In addition, even with 25% air voids, these asphalt stabilized mixes have sometimes stripped after being in service.

9.4.5 Pre-Overlay Rehabilitation

Any form of rehabilitation or restoration that reduces the movement at the joints or cracks will reduce reflection cracking. For PCC pavements this includes cement grouting to fill voids and load transfer restoration with dowels to reduce the vertical differential deflection.

Breaking and seating the slabs is a method that reduces the opening deformation by reducing the slab length. Aggregate interlock must be maintained and the size of the broken pieces is critical to overlay performance. If reinforcement exists, it must be broken or the cracks will not open sufficiently to reduce joint opening. The broken pieces must be firmly seated into the foundation prior to overlay. If the foundation is weak, the pieces may rock and result in significant reflection cracking surrounding the pieces.

The slabs must not be broken so badly that aggregate interlock is lost. This is needed to prevent the pieces from rocking, which will cause a reflection crack. Various agencies have had success with such broken piece sizes as 3 x 3 feet maximum down to 1 x 1 foot. The smaller sizes require a very solid foundation.

9.4.6 Reflection Cracking Severity Control

In addition to methods that try to reduce the rate of cracking, some agencies allow the cracks to occur but have tried to reduce their severity and rate of deterioration. Several northeastern states mark the pavement prior to overlay and then saw a joint reservoir in the asphalt concrete overlay directly over the old joint or crack. This new reservoir is then sealed and treated as a joint. This technique greatly reduces the spalling that occurs at the reflection crack over many years. However, care must be taken to ensure that the new joint is placed <u>directly</u> over the old.

The states using such techniques have long-jointed reinforced pavements. These pavement types experience the largest amount of joint opening and are more susceptible to reflection cracking than a plain concrete pavement with 15 foot joint spacing. The shorter jointed pavements require a proportionately larger amount of sawing and, because of this added work, other methods may prove more cost-effective. This technique has been used effectively by some states for over 20 years.

Increased overlay thickness will reduce severity of reflection cracks but is not normally cost-effective. Thickness increases 3 to 6 inches have shown a great influence on both rate of propagation and severity; however, overlays over 6 inches have shown less influence on the rate of reflection cracking.

9.4.7 Treatment Comparison

The results of a comprehensive study by the Georgia DOT (16), illustrated in Figures 5-2.9 through 5-2.11, show the influence of different treatments and the effect of overlay thickness. The unexpected influence of adding subdrains cannot be interpreted without knowing the quality of the installations and whether the foundation and moisture conditions varied appreciably. None of the treatments illustrated have any influence on crack propagation developing from the differential deflections under traffic. More work is needed to quantify these deflections before a design procedure for reflection cracking can be formulated.

Reference 14 contains a summary of the methods of design and practice to reduce reflection cracking in bituminous overlays used by highway agencies across the United States. From that study, the following conclusions were drawn:

1. Design criteria is not available for use in the design of fabric overlays. However, the research projects currently ongoing may lead to the development of such a method.

2. The experimental project experiences in many states have not resulted in clearly defined results. While some agencies report successful use, other have had failures. However, construction techniques that appear to have practical validity have been developed in those states reporting successful use of fabrics.

3. There is evidence suggesting that the waterproofing capabilities of fabric treatment are the most promising attributes of fabrics in an asphalt overlay system. The capability of impermeable fabrics to prevent the pumping of fines from AC or PCC pavement base layers also offer benefits that should contribute to extending roadway serviceability.

4. There is evidence that fabrics offer benefits in delaying reflection cracking only when the existing roadway provides structurally adequate foundations, in which case, the maintenance/rehabilitation requirement focus on rideability.

A similar study by an NCHRP research panel (17) that included reflection cracking in the AC overlay over flexible or rigid pavements was performed at the same time as the NEEP 10 project (14). The conclusions of that study were (17):

1. Overlay systems that have retarded reflection cracking of asphalt concrete overlays on old asphalt concrete pavements are low-viscosity asphalt (in the overlay or as an interlayer), surface recycling of the old surface, asphalt-rubber interlayer, certain fabrics (for other than thermally induced cracks), and thick overlays (> 2 inches).

2. Overlay systems that have retarded reflection cracking of asphalt concrete overlays on old Portland cement concrete pavements are 6 inch thick overlays where vertical movement is not excessive, and stress-relieving layers such as a prefabricated fabric membrane strips or a 2.5 inch layer of open-graded asphalt concrete base.

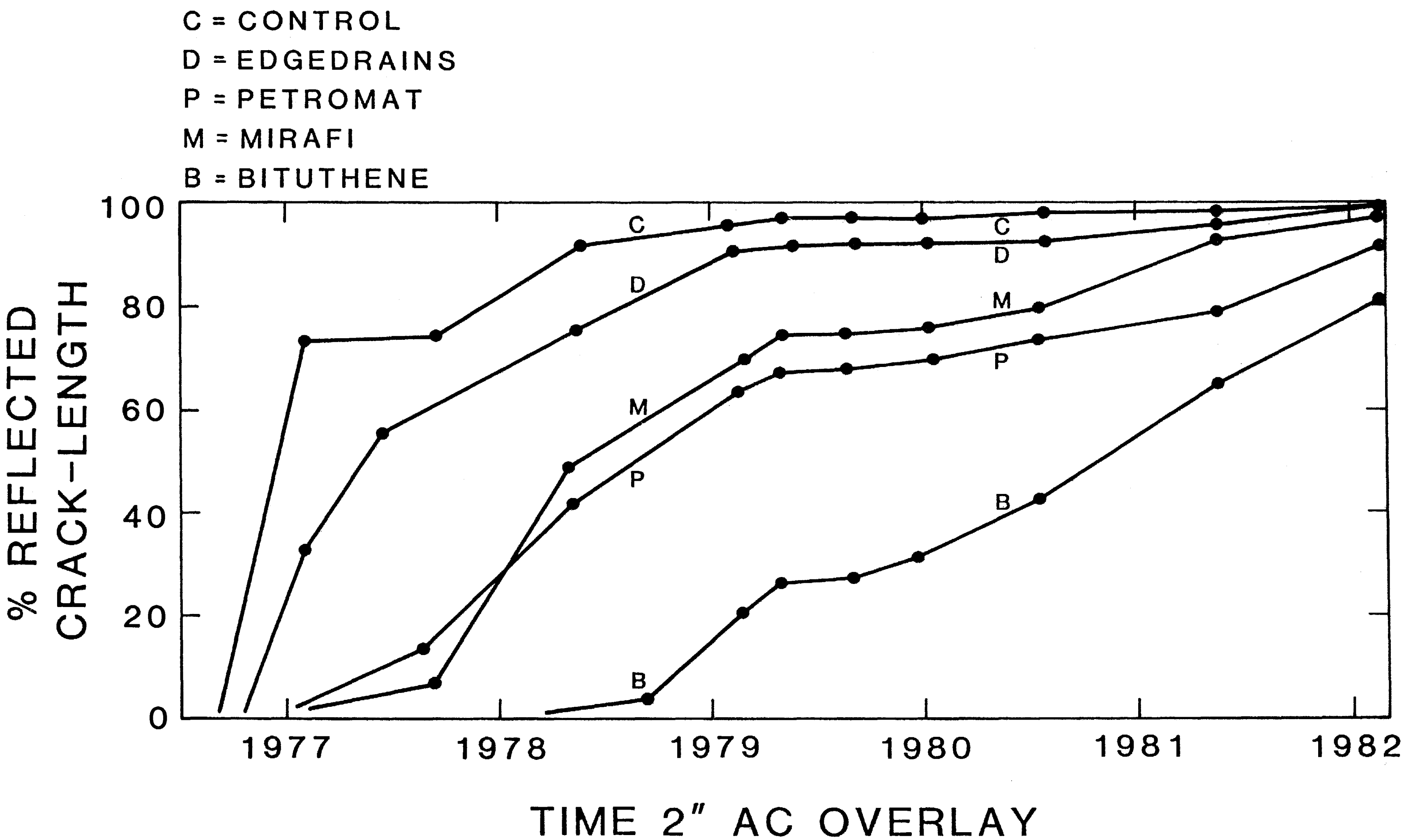


Figure 5-2.9. Reflective Cracking in 2 inch Overlay with Various Treatments (1).

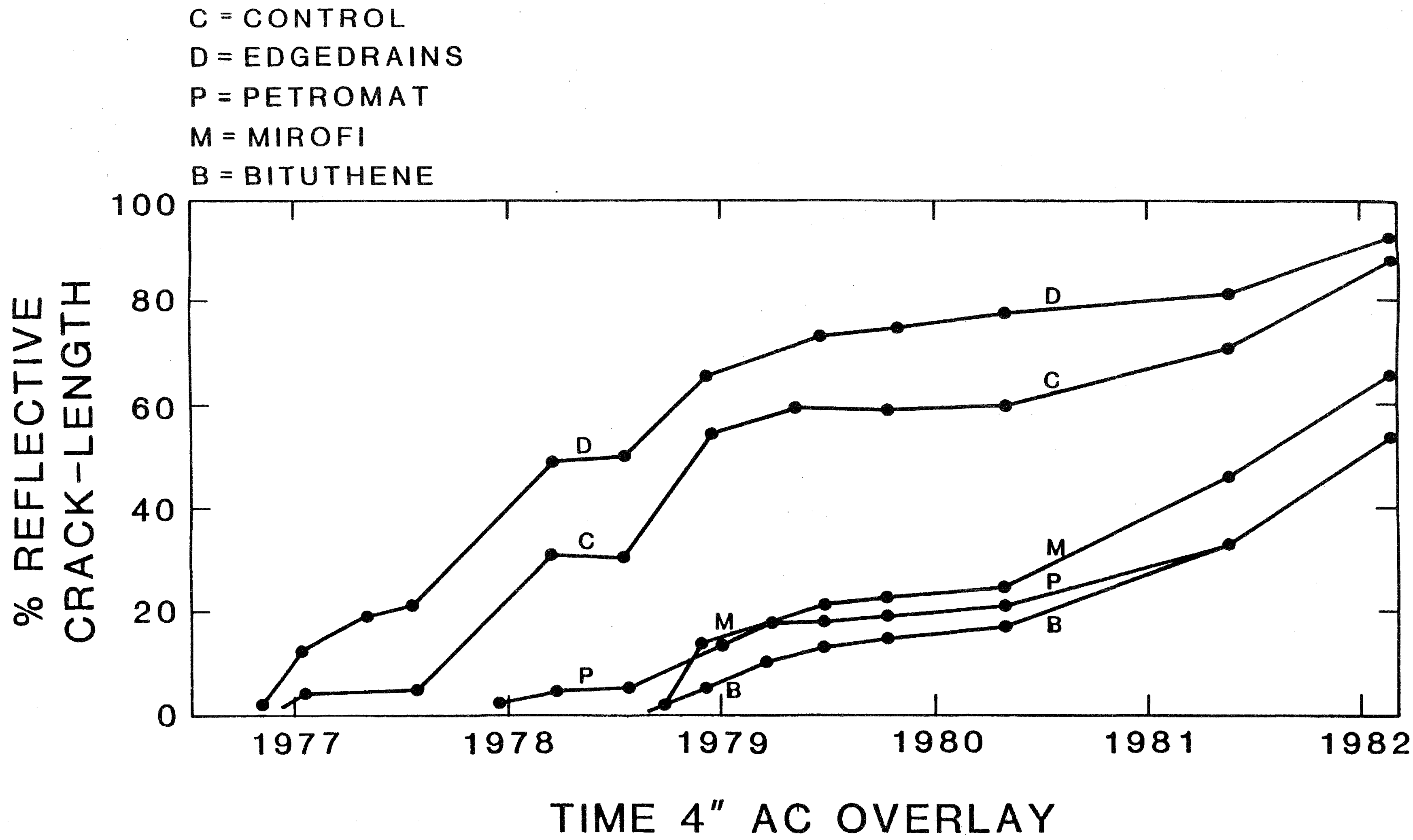


Figure 5-2.10. Reflective Cracking in 4 inch Overlay with Various Treatments (1).

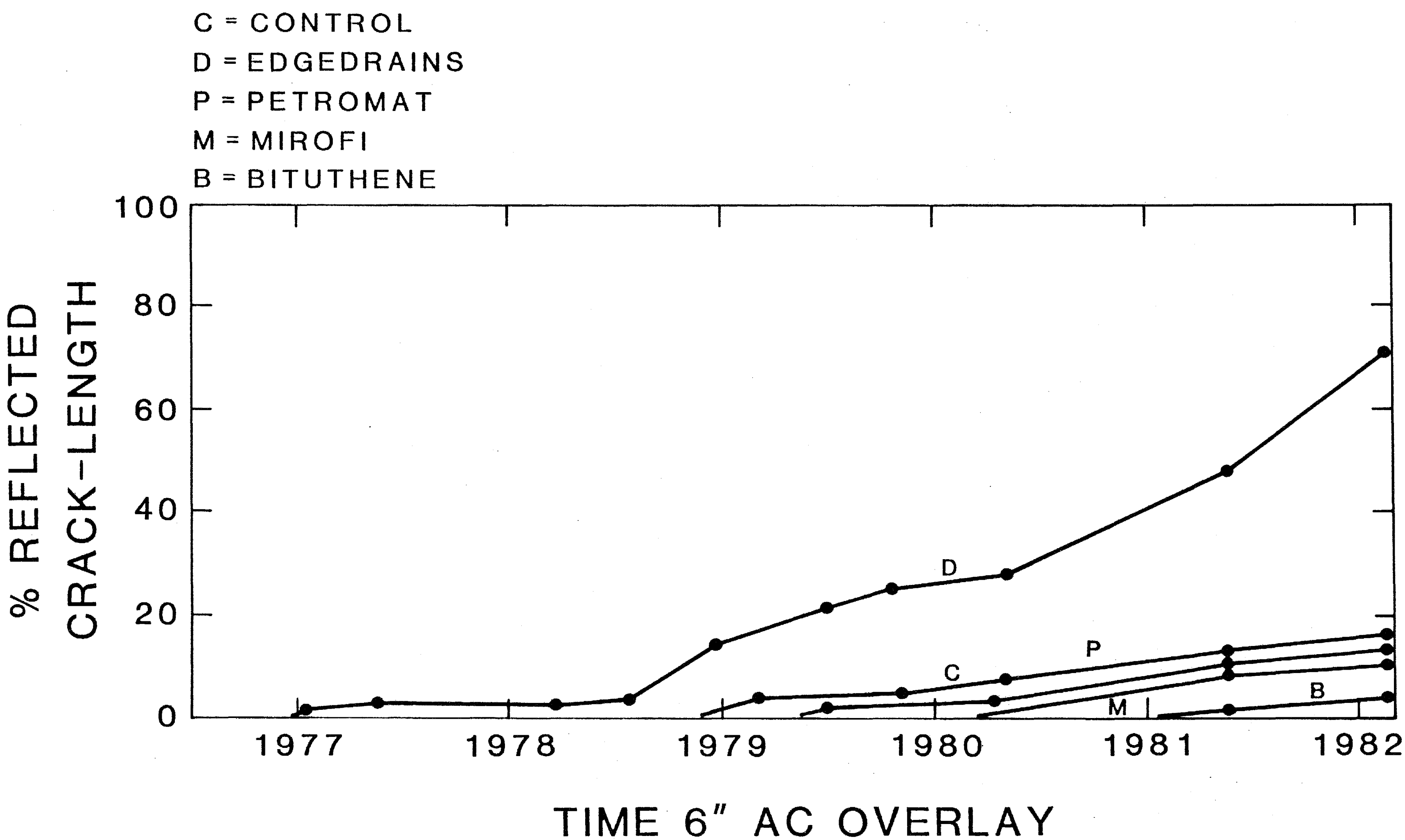


Figure 5-2.11. Reflective Cracking in 6 inch Overlay with Various Treatments (1).

3. For bonded and partially-bonded Portland cement concrete overlays, new joints should be placed over old joints; unbonded overlays do not require matching joints.

4. Additional research and field testing are necessary to verify theoretical approaches as practical design systems.

10.0 SUMMARY

Resurfacing is the major rehabilitation alternative used in the U.S. The type of overlay to be used for each pavement depends on the function of the overlay and the type and condition of the existing pavement. A detailed pavement survey and evaluation must be performed to ensure the selection of a feasible overlay. Overlays are used to correct pavement problems such as: reduced skid resistance, irregular pavement profile, inadequate surface slopes for drainage, existence of surface deterioration, impaired appearance, and structural deficiency.

There are two basic types of overlay: flexible overlays made with asphaltic materials and rigid overlays made with PCC concrete. There are many variations of each overlay type and each variation is best suited for specific situations.

Pre-overlay treatment and repair is essential to the performance of the overlay. An analysis of the patching cost (amount of patching) versus overlay cost (thickness) should be performed to arrive at the optimum solution for both. However, any seriously deteriorated area not repaired will propagate through most overlays.

There are four approaches for the design of overlays, each with a different underlying philosophy. Inputs for overlay design were also discussed, including the use of diagnostic investigations (e.g., NDT and coring). Factors were also presented for the selection of the overlay type. A major point that requires consideration in every overlay design is the use of combinations of milling and recycling with overlays as a rehabilitation alternative.

Advantages and disadvantages of different overlay types were discussed, as well as problems/limitations of current overlay design.

Finally, a brief overview of reflection cracking was given, including the causes and different treatments available.

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MODULE 5-3

DESIGN OF OVERLAYS FOR FLEXIBLE PAVEMENTS

1.0 INSTRUCTIONAL OBJECTIVES

This module presents current design practices for overlays of flexible pavements. The AASHTO design procedure (structural deficiency approach) is emphasized and other design procedures are discussed, including the Asphalt Institute procedure (deflection-based approach). The underlying principles of each method are presented to develop a basic understanding of the procedure and to recognize the limitations of each procedure.

Upon completion of this module, the participant will be able to accomplish the following:

1. State the basic approach of the AASHTO overlay design procedure.

2. Design a flexible overlay for a flexible pavement using the AASHTO procedure.

3. State the basic design approach of the Asphalt Institute procedure.

4. Design a flexible overlay for a flexible pavement using the Asphalt Institute method.

5. List the limitations of each overlay design method.

6. State the approach for the design of rigid overlays of flexible pavements.

2.0 INTRODUCTION

In this section, several different procedures are discussed for the design of overlays for flexible pavements. These procedures enable the engineer to determine overlay thickness requirements for the given pavement condition and traffic levels. As discussed in Module 5-2, there is no universally accepted method for the design of overlays. Therefore, the procedures presented here should be used as a guide and adjusted to fit local conditions as needed. In addition, it is assumed that pre-overlay repair and reflective crack control actions are taken. If this is not the case, then the overlay will likely fail prematurely.

3.0 TYPES OF OVERLAYS OVER FLEXIBLE PAVEMENTS

There are two major types of overlays for flexible pavements with many subtypes:

1. Asphalt Concrete Overlay.

2. Portland Cement Concrete Overlay.

The selection of overlay type depends largely upon the life-cycle costs generated for each overlay type. However, engineering factors such as the condition of the existing pavement, the current and future traffic levels, service life and traffic control constraints must also be considered.

Asphalt concrete overlays are the predominant overlay type for both flexible and rigid pavements. They may be placed for either structural improvements or for surficial improvements. The use of rigid overlays on flexible pavements is somewhat uncommon; however, they have been used very successfully in the U.S. and other countries. This alternative may be most cost effective when severely distressed flexible pavements (and no severe vertical clearance problems) are encountered.

4.0 AASHTO DESIGN METHOD FOR FLEXIBLE OVERLAYS ON FLEXIBLE PAVEMENTS

The AASHTO overlay methodology is based upon the serviceability-performance relationships previously discussed. The AASHTO procedure makes use of the concept of remaining pavement life by direct consideration of the damage in the existing pavement as well as the acceptable level of damage (design terminal serviceability level) for the overlay.

The AASHTO procedure incorporates and encourages the use of nondestructive testing (NDT) as a major input into the overall design methodology. The major use of NDT is the characterization of the in-place roadbed soil and pavement layer material properties. This leads to the determination of the effective structural capacity of the existing pavement.

Figure 5-3.1 shows the conceptual relationship between traffic loadings, pavement serviceability and structural capacity. This relationship serves as the basis for the AASHTO overlay design procedure. The original pavement is assigned an initial serviceability of p_o and an initial structural capacity of SC_o. With traffic loadings the pavement deteriorates, gradually reaching the selected terminal serviceability of P_{t1} after x load repetitions of traffic. At this time the existing pavement has a structural capacity designated as SC_{xeff}.

To elevate the condition of the pavement to a serviceability near the original value (P_o), additional structural capacity in the form of an overlay (designated as SC_{OL}) is required. This structural capacity and the effective structural capacity of the existing pavement should be equivalent to the structural capacity (SC_y) of an overlaid pavement designed for the given conditions and projected traffic loadings (y). The values N_{fx} and N_{fy} represent the amount of traffic loadings required to deteriorate the pavement to its "failure" serviceability (P_f), where it is assumed that no remaining life exists in the pavement (i.e., $SC_{eff} = 0$).

4.1 Fundamentals of the AASHTO Overlay Design Procedure

The AASHTO design for flexible pavements is an empirical procedure derived from the 1958-60 AASHO Road Test data. The basic design equation gives the relationship between a pavement structural requirement and the number 18-kip equivalent single axle loads the pavement can carry to a selected terminal serviceability. As discussed in previous modules, the pavement structural requirement is defined by the structural number as follows:

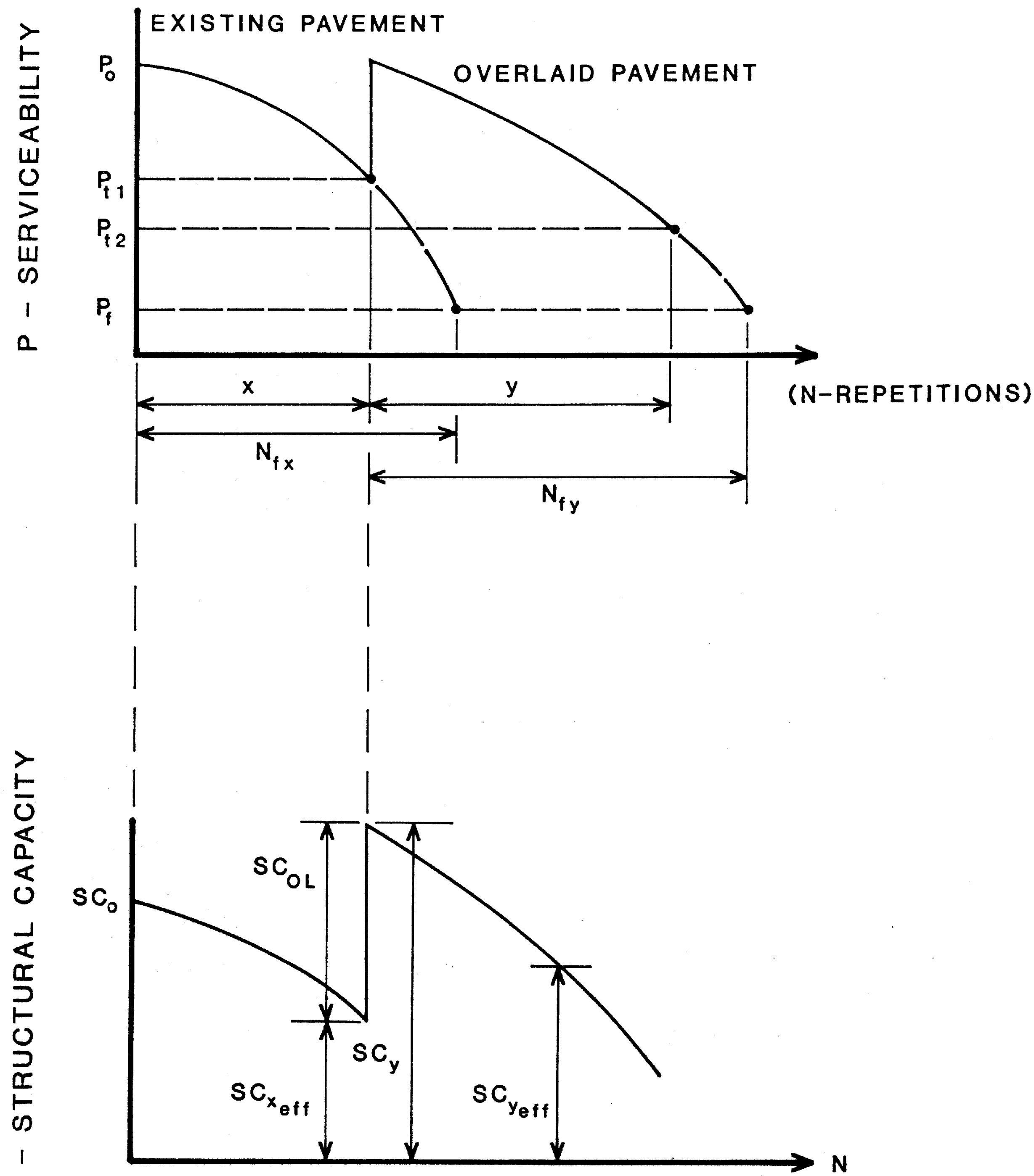


Figure 5-3.1. Influence of Traffic Loading on Pavement Serviceability and Structural Capacity (1).

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

where:

SN = pavement structural number, a number (weighted thickness) representing the required strength of the entire pavement structure.

a_1, a_2, a_3 = coefficients to represent the material quality of the surface, base, and subbase layers of the pavement, respectively.

D_1, D_2, D_3 = the thicknesses, in inches, of the surface, base, and subbase layers, respectively.

m_2, m_3 = drainage coefficients to represent the drainage characteristics of the base and subbase layers, respectively.

For flexible pavements, the structural number (SN) is equivalent to the structural capacity. A satisfactory design is achieved whenever the calculated SN (using the above equation) is equal to or greater than the required SN. The required SN is a function of traffic, subgrade support, environmental factors and the initial and terminal serviceability required for the pavement. The required SN for a given set of conditions is determined using the nomograph in Module 3-2.

The AASHTO flexible overlay design approach is based on the following structural deficiency concept (1):

$$SN_{OL} = SN_y - F_{RL}(SN_{xeff})$$

in which $SN_{OL} = a_{OL} * h_{OL}$

where:

SN_{OL} = Structural Number of the overlay

h_{OL} = required AC overlay thickness (in inches)

a_{OL} = layer coefficient assigned to the AC overlay material

SN_{xeff} = Effective Structural Number of the existing pavement

SN_y = Structural Number of a new pavement designed to carry the specified traffic over the new design period and for the given subgrade support, and environmental conditions and serviceability loss.

F_{RL} = Remaining Life Factor accounting for the damage of the existing pavement and the desired degree of damage to the overlay at the end of the overlay traffic (always ≤ 1).

Thus, the required structural number of the overlay is equal to the difference between the structural number of a new design (for the projected traffic and the existing soil and drainage conditions) and the (weighted) effective structural number of the existing pavement.

4.2 Overlay Design Process

This section presents the specific steps in the AASHTO overlay design process for flexible overlays of flexible pavements. It is emphasized that the approach is applicable to all possible combinations of overlay and existing pavement types.

There are seven steps in the AASHTO overlay design procedure (1):

1. Analysis Unit Delineation.
2. Traffic Analysis.
3. Materials and Environmental Study.
4. Effective Structural Capacity Analysis.
5. Future Overlay Structural Capacity Analysis.
6. Remaining Life Factor Determination.
7. Overlay Design Analysis.

Procedures to conduct each of these steps follows.

4.2.1 Analysis Unit Delineation

The first step in the overlay design process is the delineation of basic analysis units. The objective is to determine boundaries along the project length that subdivide the rehabilitation project into statistically homogenous pavement units possessing uniform pavement cross sections, subgrade support, construction histories and pavement condition. If more than one analysis unit is identified, practical construction and cost considerations must be used to decide whether separate overlay designs should be developed for each analysis unit or which units should be combined (1).

In determining analysis units, there are two extreme cases which may be encountered (1):

Case I. Accurate Historic Data Unavailable

If a review of historic data discloses little useful information, an NDT deflection study should be conducted first along with a visual distress survey. Deflection testing should be done in the outer wheel path of the lane adjacent to the outer shoulder for multilane facilities. For two-lane highways, testing in the outer wheel path in one direction is normally sufficient, unless there are obvious differences in conditions in each section. If that is the case, then a similar test pattern should be conducted in each direction. When prior information concerning unit boundaries is unavailable, the testing should be conducted with deflections taken at equal intervals of 300 to 500 feet. This pavement response data should be analyzed (statistically or plotted on a profile) to delineate the boundaries of the units. The engineer should then make an evaluation regarding the practicality of these unit lengths for constructability and cost-effectiveness. A general guideline for a minimum unit (construction) length is a half mile.

The analysis units determined through the combined use of NDT and engineering judgment are then used as the basis for conducting any destructive testing (coring) necessary to determine pavement layer material and subgrade type and thickness. In turn, this information is then used to verify and/or modify the units previously established.

Case II. Accurate Historic Data Available

When accurate historic traffic construction and design information regarding a specific pavement is available, the engineer has a relatively good idea as to unit boundaries prior to any field testing. In this case, nondestructive deflection testing (10-15 test points randomly selected in each unit) can be conducted to verify (and modify if needed) the preliminary units selected. If necessary, a destructive sampling plan can be developed to further examine the appropriateness of the selected analysis units.

4.2.2 Traffic Analysis

The purpose of the traffic analysis step is twofold. First, it is necessary to determine the cumulative 18-kip equivalent single-axle load (ESAL) applications anticipated for the overlay. Future traffic projections are an estimate and the further one projects ahead, the less reliable the estimate. Such difficulties notwithstanding, traffic projections must be made as part of the overlay design process.

Second, it is necessary to determine the cumulative 18-kip ESAL applications sustained by the existing pavement to date since its last major construction or reconstruction. There may be some difficulty in obtaining accurate traffic histories, but this information is needed to help estimate the remaining life of the existing pavement. The above traffic computations are discussed in detail in Module 2-5.

4.2.3 Materials and Environmental Study

Design values for layer materials used in the rehabilitation process may be categorized into three major groups:

1. Existing pavement layer properties.

2. Existing pavement subgrade properties.

3. Design properties of overlay layers (including the use of recycled materials).

The primary material property of concern for all three categories listed above is the elastic modulus.

Prediction of Pavement Layer Moduli (Existing Pavement)

Deflection testing is used to estimate the in-place properties (elastic modulus) of each layer within the pavement structure. The underlying assumption of NDT is that a unique set of layer moduli (E_1, E_2, . . . E_n) exists such that the predicted deflection basin under the dynamic load-pavement combination yields values equal to the measured deflection basin. Multilayer elastic theory is used in a "backcalculating" technique to arrive at the layer moduli values satisfying the measured and predicted deflection basin values. Backcalculation should only be done when layer thicknesses are known from coring results. NDT surveys should be conducted at the time of year the pavement is at its weakest due to seasonal environmental conditions (1).

A limitation of this approach is that the mathematical complexities require a computerized solution. However, several such solutions are available on mainframe and microcomputer systems. More information on determination of the pavement layer moduli is found in Reference 1.

The determination of reasonable moduli values is by no means an easy task. Considerable experience and engineering judgment is required. Values obtained from the computer backcalculation analyses that appear to be unrealistic should be reevaluated in terms of the type of materials identified from coring.

The extent of pre-overlay repair of the existing pavement will have a major effect on overlay performance (as discussed in Module 5-2). The AASHTO design procedure assumes that significantly deteriorated areas are repaired. Failure to repair such areas will result in premature failure of the overlay.

Prediction of Subgrade Layer Moduli (Existing Pavement)

The resilient modulus of the subgrade is obtained from the same NDT deflection testing conducted for the structural layers. However, because the subgrade modulus will vary throughout a typical year, and because some subgrade soils exhibit a degree of nonlinear behavior, additional analysis of the subgrade modulus is required. When a subgrade behaves in a nonlinear fashion, it exhibits "stress sensitivity" (i.e., modulus decreases as stress increases). Therefore, adjustments must be made to the values obtained from NDT to account for these factors. More information on determination of the subgrade modulus (and any required adjustments) is found in Reference 1.

Deflection equipment that does not produce loadings near the design 9-kip wheel loads may not produce moduli that exist under traffic loads of this magnitude. Therefore, it is essential that the deflection equipment be capable of providing adequate loads.

Design Properties of Overlay Materials

The selection of appropriate design modulus and/or strength parameters for the AC materials to be used in the overlay are treated as new materials. Their properties are discussed in Module 2-3.

4.2.4 Effective Structural Capacity Analysis

The fourth step in an overlay analysis is to estimate the effective (in-place) structural capacity of the pavement to be overlaid. Information regarding material properties derived in the previous step (step 3) is used to arrive at this parameter. As previously mentioned, the effective structural capacity is equivalent to the effective structural number (SN_{xeff}) for flexible pavement systems (1).

The following steps are performed in the determination of the effective structural number:

1. Using the modulus values obtained in step 3 (described above), the existing layer coefficients are determined from the appropriate tables presented in Module 2-3.

2. Based on the conditions of the existing pavement, an estimate is made on the drainage coefficients (as described in Module 2-4).

3. SN_{xeff} is then calculated by using the equation

$$SN_{xeff} = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

where:

SN_{xeff} = Effective Structural Number

a_1, a_2, a_3 = layer coefficients for surface, base, and subbase, respectively

D_1, D_2, D_3 = layer thicknesses for surface, base, and subbase, respectively

m_2, m_3 = drainage coefficients to represent the drainage characteristics of the base and subbase layers, respectively.

It is important to note that the predicted moduli of any asphalt layer must be adjusted to a standardized temperature of $70^{o}F$ before computing the effective structural number (1).

It is also possible to determine the effective structural number using only the maximum deflection obtained from NDT testing, provided the characteristics of the NDT device are known. More information on this alternative procedure is found in Reference 1.

The determination of the effective structural number of the existing pavement (SN_{xeff}) is the most critical step in the design procedure. Overestimating SN_{xeff} will result in an overlay which is inadequate for the existing conditions, whereas underestimating SN_{xeff} will result in over-design of the overlay.

4.2.5 Future Overlay Structural Capacity Analysis

The fifth step in an overlay design analysis is to determine the future overlay structural capacity (i.e., structural number). The objective of this step is to determine the total structural number of a new pavement designed to carry the anticipated traffic loading (y repetitions) in the overlay period to a terminal serviceability of p_{t2} using the existing subgrade support. This concept was illustrated in Figure 5-3.1. This step in the overlay process is in essence a new pavement design for a flexible system. It should also be noted that p_{t1} (serviceability of the existing pavement prior to overlay) need not be equal to p_{t2} (serviceability of overlaid pavement at the end of the design period of the overlay) (1).

This step is accomplished by using Module 3-2 for new pavement design. The following inputs are required:

1. R = Reliability Level (As discussed in Module 2-6)

2. S_o = Standard Deviation (As discussed in Module 2-6)

3. W_{18} = Future (Projected) 18-kip ESAL applications (As discussed in Module 2-5)

4. E_{RS} = Resilient Modulus of Roadbed Soil (As discussed in Module 2-2).

5. PSI = Change in Pavement Serviceability over Future Life

   PSI = $p_{o2} - p_{t2}$, where

   p_{o2} = initial serviceability of overlay (4.2 for new pavements at AASHO Road Test)

   p_{t2} = terminal serviceability of overlay (generally 2.0 to 3.0)

4.2.6 Remaining Life Factor Determination

Step six in the overlay design analysis is to determine the remaining life factor, F_{RL}. This is an adjustment factor applied to the effective capacity parameter (SN_{xeff}) to reflect a more realistic assessment of the weighted effective capacity during the overlay period. This factor depends upon the remaining life value of the existing pavement prior to overlay (R_{Lx}) and on the remaining life of the overlaid pavement system after the overlay traffic (and subsequent serviceability) has been reached (R_{Ly}). As a consequence, both of these values (R_{Lx} and R_{Ly}) must be known (1).

Remaining Life of Existing Pavement

The remaining life of the existing pavement (R_{Lx}) prior to overlay is a difficult parameter to accurately determine. There are five methods that can be used to evaluate the remaining life of the existing pavement. A brief overview of the methods is given below. More information on the determination of R_{Lx} is found in Reference 1.

The five methods used to estimate the remaining life of the existing pavement are (1):

1. NDT Approach.

   The percent remaining life is found by comparing the effective structural number of the existing pavement (SN_{xeff}, determined from NDT) to the original pavement structural number (SN_o), i.e.,

   $$C_x = SN_{xeff}/SN_o$$

   where C_x is the pavement condition factor. With this factor determined, Figure 5-3.2 can be used to obtain R_{Lx}.

2. Traffic Approach.

   When reasonably accurate traffic information is available, the predicted traffic loading to failure (N_{fx}, generally taken as $p_f = 2.0$) is computed and compared to the actual cumulative traffic loading over the life of the existing pavement:

   $$R_{Lx} = (N_{fx} - x)/N_{fx}$$

3. Time Approach.

   The remaining life factor is estimated based on the time the section has been in service (t), its expected life before requiring an overlay (T_f), and projected traffic growth rate (r_g). Figure 5-3.3 is used with the above inputs to obtain R_{Lx}.

4. Serviceability Approach.

   A knowledge of the Present Serviceability Index (PSI) and of the original pavement structural number (SN_o) is used to estimate the remaining life by using Figure 5-3.4.

5. Visual Condition Survey Approach.

   An overall pavement condition factor (C_x) is developed for the individual pavement layers using layer condition factors (from Table 5-3.1). The condition factor is calculated using the following procedure:

   $$C_x = \frac{(h_1 C_{v1} + h_2 C_{v2} + \ldots + h_n C_{vn})}{h_1 + h_2 + \ldots + h_n}$$

   Figure 5-3.2 is then used to determine R_{Lx}.

While each of these approaches is theoretically equivalent, rarely will all approaches yield the same value. This is due to estimation errors,

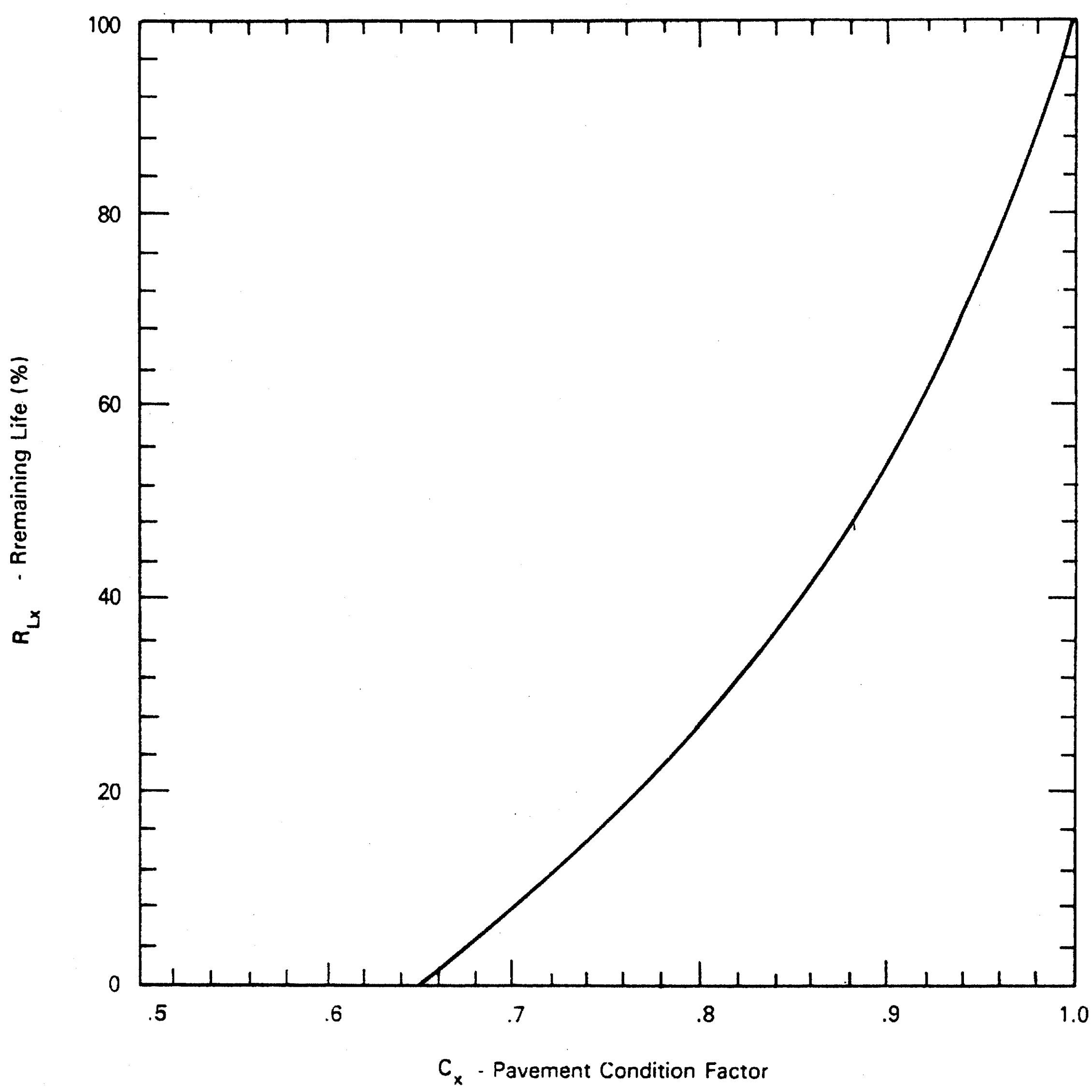


Figure 5-3.2. Remaining Life Estimate Predicted from Pavement Condition Factor (1).

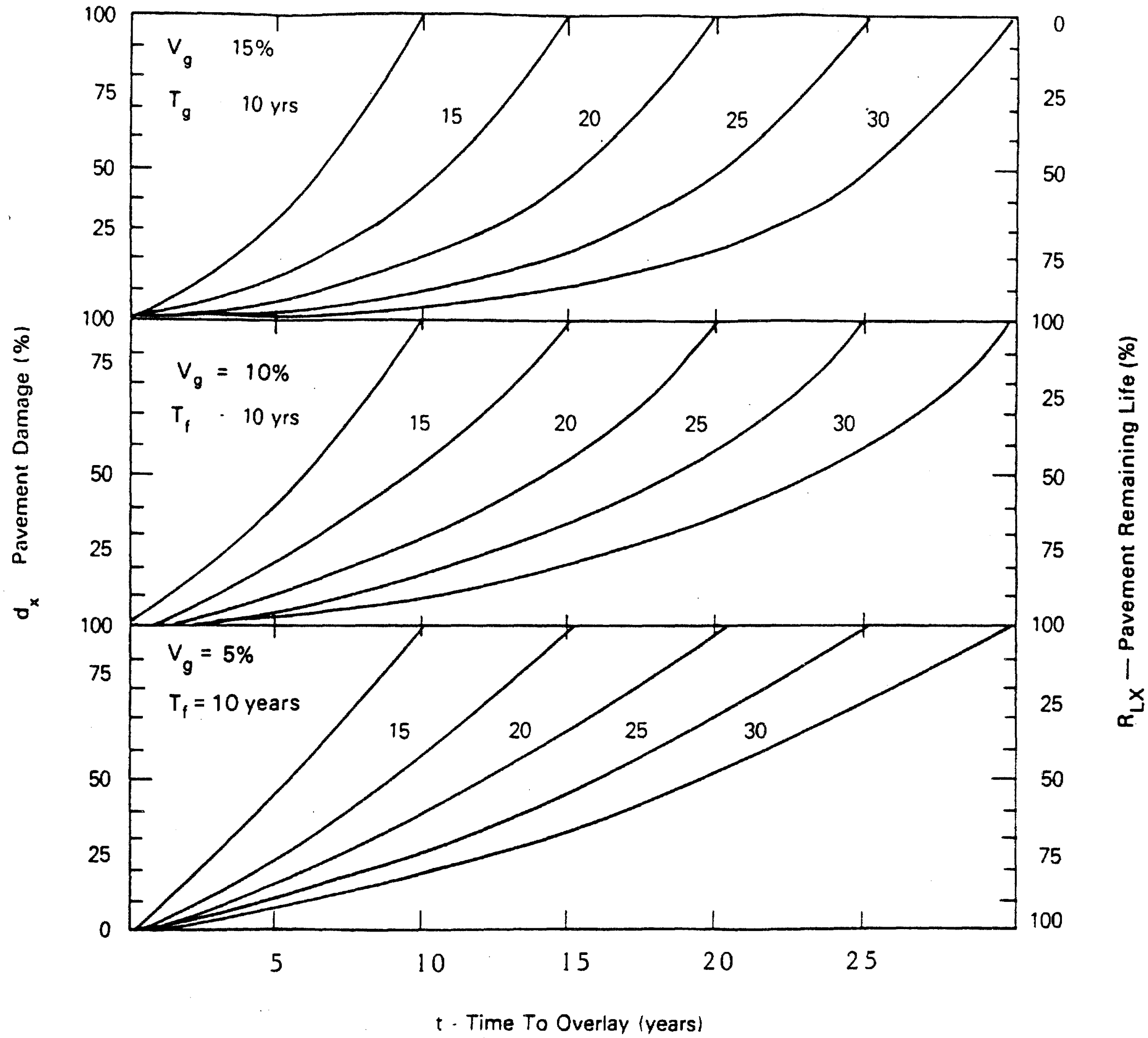


Figure 5-3.3. Remaining Life Estimate Based on Time Considerations for Various Traffic Growth Rates (1).

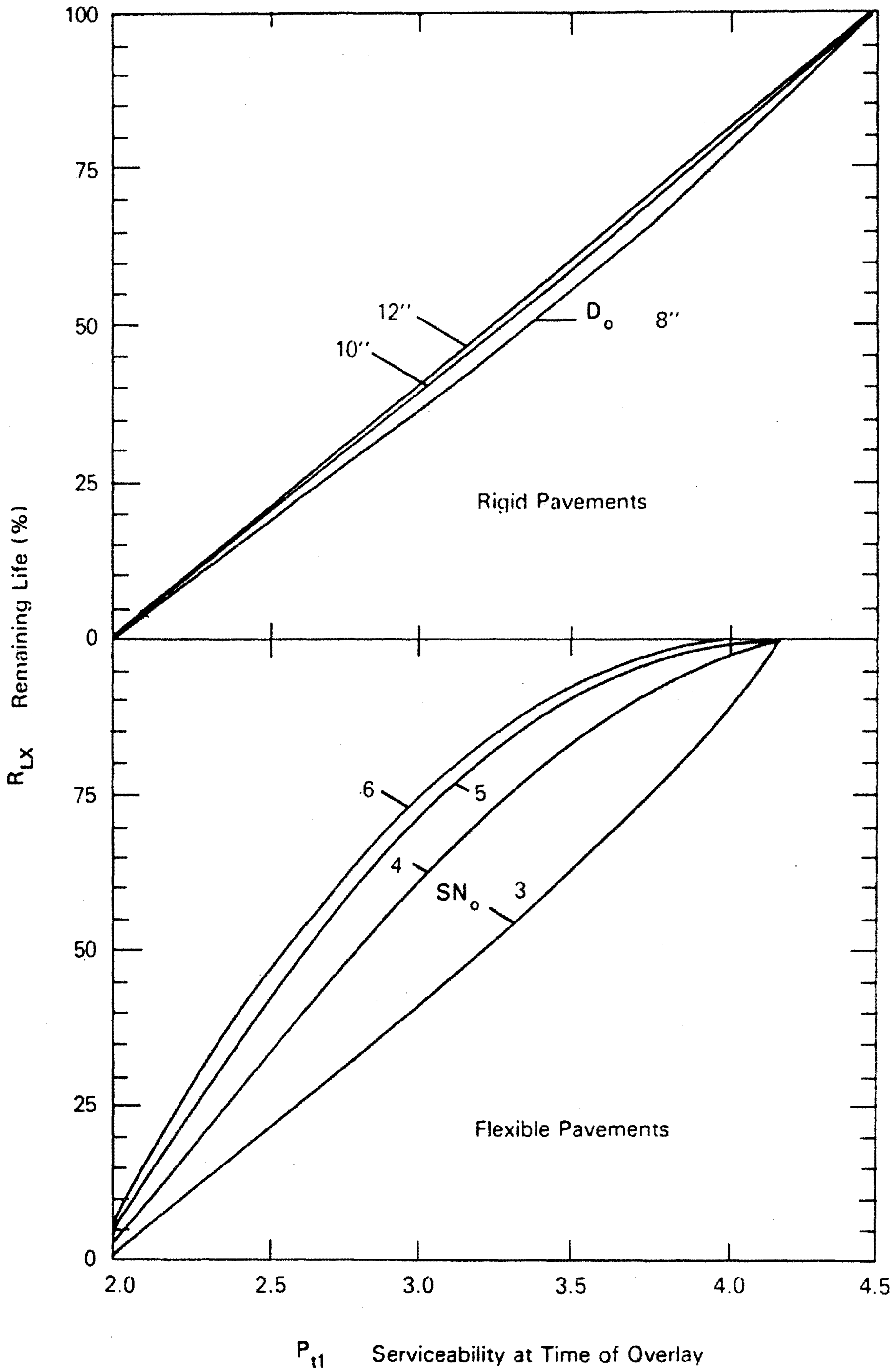


Figure 5-3.4. Remaining Life Estimate Based on Present Serviceability and Pavement Cross Section (1).

Table 5-3.1. Summary of Visual (C_V) and Structural (C_X) Conditions Values (1).

Layer Type	Pavement Condition	C_v Visual Condition Factor Range	C_X Struct Cond Factor Value
Asphaltic	1. Asphalt layers that are sound, stable, uncracked and have little to no deformation in the wheel paths	0.9-1.0	.95
	2. Asphalt layers that exhibit some intermittent cracking with slight to moderate wheel path deformation but are still stable.	0.7-0.9	.85
	3. Asphalt layers that exhibit some moderate to high cracking, have ravelling or aggregate degradation and show moderate to high deformations in wheel path	0.5-0.7	.70
	4. Asphalt layers that show very heavy (extensive) cracking, considerable ravelling or degradation and very appreciable wheel path deformations	0.3-0.5	.60
PCC	1. PCC pavement that is uncracked, stable and undersealed, exhibiting no evidence of pumping	0.9-1.0	.95
	2. PCC pavement that is stable and undersealed but shows some initial cracking (with tight, non working cracks) and no evidence of pumping	0.7-0.9	.85
	3. PCC pavement that is appreciably cracked or faulted with signs of progressive crack deterioration: slab fragments may range in size from I to 4 sq.yds., pumping may be present	0.5-0.7	.70
	4. PCC pavement that is very badly cracked or shattered into fragments 2-3 ft. in maximum size	0.3-0.5	.60
Pozzolanic Base/ Subbase	1. Chemically stabilized bases (CTB, LCF...) that are relatively crack free, stable and show no evidence of pumping	0.9-1.0	.95
	2. Chemically stabilized bases (CTB, LCF...) that have developed very strong pattern or fatigue cracking, with wide and working cracks that are progressive in nature: evidence of pumping or other causes of instability may be present	0.3-0.5	.60
Granular Base/ Subbase	1. Unbound granular layers showing no evidence of shear or densification distress, reasonably identical physical properties as when constructed and existing at the same "normal" moisture - density conditions as when constructed	0.9-1.0	.95
	2. Visible evidence of significant distress within layers (shear or densification), aggregate properties have changed significantly due to abrasion, intrusion of fines from subgrade or pumping, and/or significant change in in situ moisture caused by surface infiltration or other sources	0.3-0.5	.60

Special Notes:

I. The visual condition factor, C_v is related to the structural condition factor, C_x, by:

$$C_v + C_x^2$$

2. The structural condition factor, C_x, and *not* the C_v value, is the variable used in the structural overlay design equation (for all overlay-existing pavement types). It is defined by:

$$SC_{xeff} + C_x SC_o$$

deviations between predicted and actual field behavior, and complexities involved with obtaining precise input values. Current state of the art technology does not permit recommendation as to which of the five procedures noted is superior in estimating the R_{Lx} value. However, the procedure utilizing NDT deflection studies results in a better quantitative assessment of the existing in-place structural capacity and should usually be relied on more heavily than other approaches. Nonetheless, the engineer should assess the reasonableness of several results rather than relying on only one approach (1).

The engineer must also recognize the limitations associated with each approach, particularly with respect to the estimation error involved in the remaining life of the subject pavement. Generally speaking, a larger error for R_{Lx} will be made for a relatively new pavement than for a more distressed and damaged pavement.

Remaining Life of Overlaid Pavement

Determination of the remaining life of an overlaid pavement system (R_{Ly}) is directly set by the engineer in the selection of the desired terminal serviceability for the pavement designed to carry the expected traffic, SN_y. Knowledge of the future design overlay traffic (y) and the structural number (SN_y) required to yield p_{t2} after y repetitions allows for the determination of the ultimate number of repetitions to failure (N_{fy}) that the pavement could be subjected. The nomograph in Module 3-2 is used to determine N_{fy} (read as W_{18}). The following inputs are required:

1. R = Reliability Level (same as in Step 5)

2. S_o = Standard Deviation (same as in Step 5)

3. SN_y = Structural Number of "new" pavement system (obtained in Step 5)

4. E_{RS} = Resilient Modulus of Roadbed Soil (same as in Step 5)

5. PSI = Change Pavement Serviceability over Future Life

   PSI = $p_{o2} - p_f$, where

   p_{o2} = initial serviceability of overlay (4.2 for new pavements at AASHO Road Test)

   p_f = serviceability of overlay until failure (generally 2.0)

Thus, both y and N_{fy} are known and R_{Ly} can be computed from the following equation:

$$R_{Ly} = (N_{fy} - y)/N_{fy}$$

where:

N_{fy} = number of 18-kip ESAL loadings to failure (where p_f = 2.0)

y = future (anticipated) 18-kip ESAL loadings

Calculation of Remaining Life Factor

Having obtained estimates of R_{Lx} and R_{Ly} as outlined above, the remaining life factor, F_{RL}, is obtained from Figure 5-3.5.

4.2.7 Overlay Thickness Design

Steps 4, 5 and 6 provide the necessary variables to calculate the required overlay thickness, h_{OL}, from the equation:

$$h_{OL} = SN_{OL}/a_{OL} = [SN_y - F_{RL}(SN_{xeff})]/a_{OL}$$

where a_{OL} is the structural layer coefficient of the overlay material. A required overlay thickness is computed for each identified analysis unit. The final step is to determine if the results obtained are practical and cost-effective or if certain analysis units should be combined.

4.3 Limitations

The most serious limitation of the AASHTO approach is the difficulty in determining several of the needed inputs for the overlay design equation. The calculation of the effective structural number is dependent on assigning layer and drainage coefficients for the existing pavement. The layer coefficients are determined from the modulus values backcalculated from NDT. While the use of NDT has provided a more accurate measurement of the moduli values and hence more accurate layer coefficients, it must be emphasized that NDT measurements are a function of climate, season, temperature and method of analysis. Therefore, care must be taken to ensure that the obtained moduli values are reasonable. In addition, due to mathematical complexities, a computer solution is required to determine the moduli values from NDT data. The assignment of the drainage coefficients is also a major source of error since this is a difficult parameter to estimate.

The calculation of the remaining life factor depends on the remaining life of the existing pavement and the remaining life of the overlaid (future) pavement system. While the determination of the remaining life of the overlaid pavement system is straightforward, there may be some difficulty in accurately calculating the remaining life of the existing pavement. Since there are five methods available for this computation, it is recommended that several of these methods be employed to ensure that the best estimate of the remaining life of the existing pavement is obtained.

Finally, it must be noted that the AASHTO flexible overlay design method is an empirical procedure developed under one set of conditions as described in Block 3. Therefore, care should be taken in extrapolating its use to other conditions.

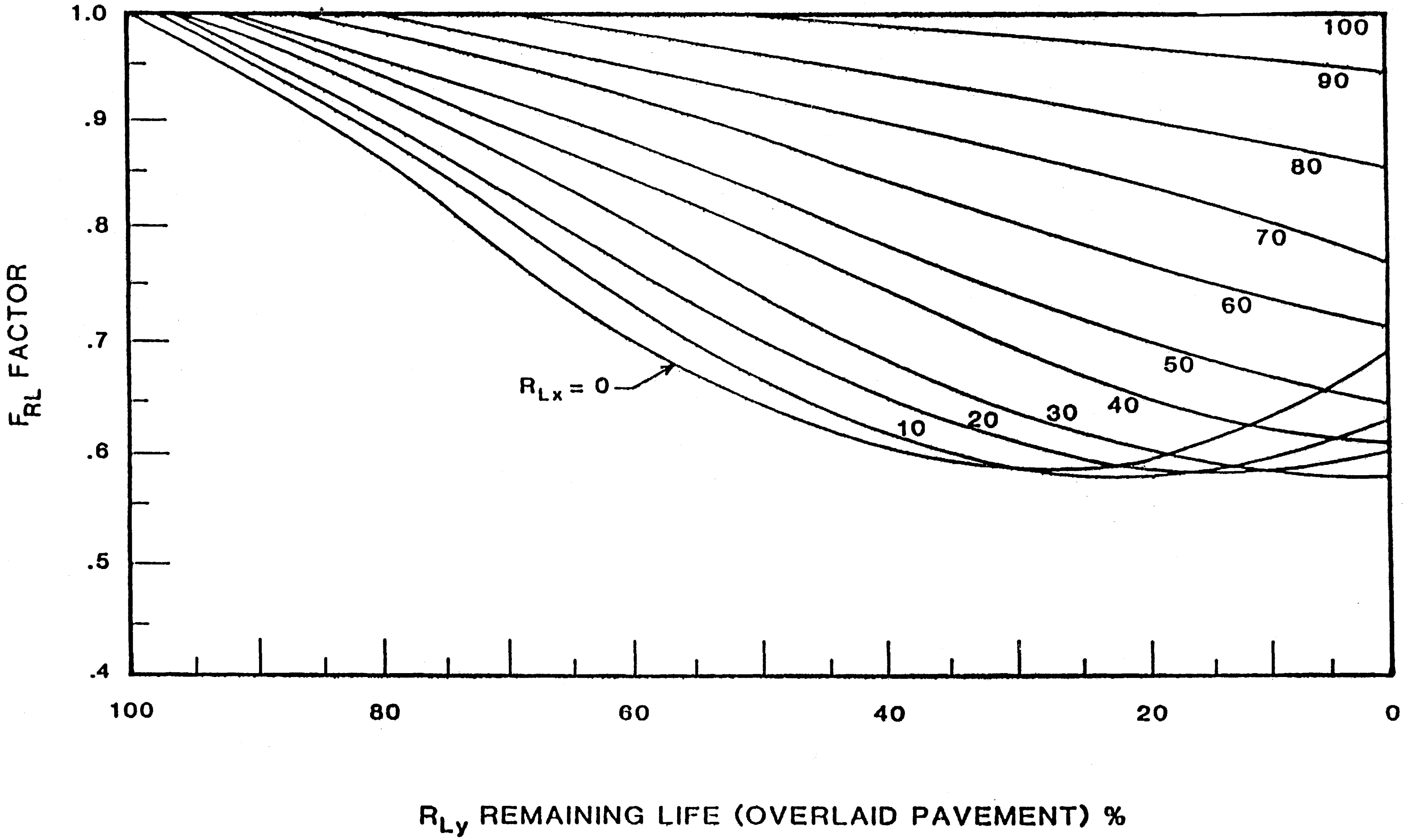


Figure 5-3.5. Remaining Life Factor as a Function of Remaining Life of Existing and Overlaid Pavements (1).

5.0 OTHER DESIGN METHODS FOR OVERLAYS OF FLEXIBLE PAVEMENTS

Other methods available to the engineer for the design of flexible overlays on flexible pavements include many deflection-based approaches (Asphalt Institute, California Department of Transportation, Virginia Department of Highways and Transportation, etc.) and many mechanistically-based procedures (Shell Research, FHWA/Resource International, Inc., Kentucky Department of Highways, etc.). The Asphalt Institute deflection-based approach is presented here due to its widespread use and basic conceptual approach. More information on the above procedures and other overlay design methods is in Reference 2.

In designing an overlay or any new pavement structure, it is good engineering practice that a method of checking design thicknesses be instituted to ensure that an adequate design is achieved. Another design procedure may be used as a means of checking the overlay thickness given by the AASHTO procedure.

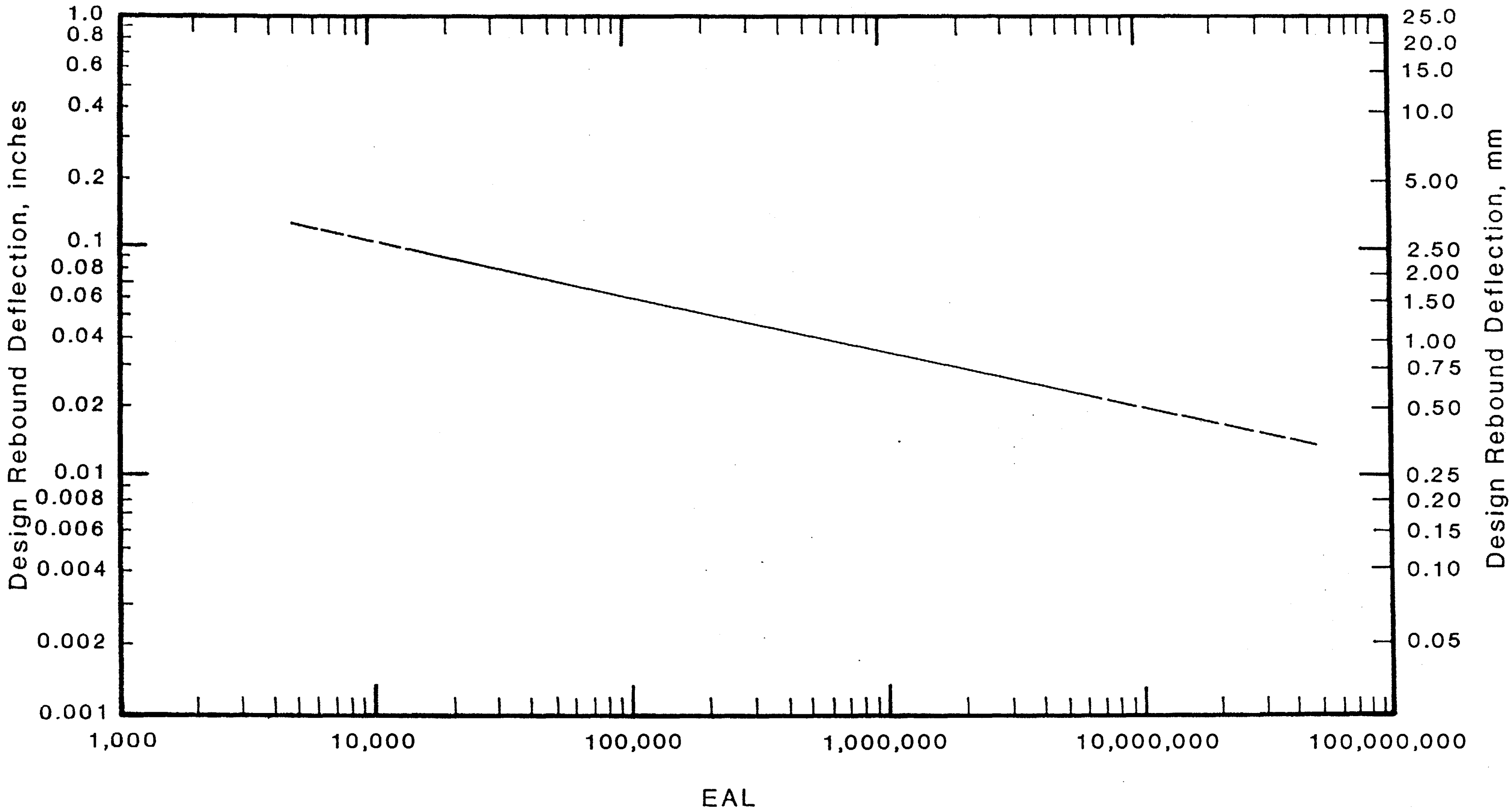
5.1 Asphalt Institute Method

The Asphalt Institute approach is based on two concepts. The first is the relationship between deflection and number of load applications to "failure" (i.e., terminal serviceability). Pavements with lower deflections can withstand many times more load applications than pavements with high deflections. For example, a pavement with a measured deflection of 0.080 inches can withstand approximately 22,000 18-kip ESAL applications to failure, while a pavement with a measured deflection of 0.040 inches can withstand approximately 560,000 ESAL applications to failure. The pavement with the lower deflection will last <u>twenty-five times</u> longer than the pavement with a deflection only twice as large.

This concept is illustrated in Figure 5-3.6, which is the curve used for overlay design by the Asphalt Institute. It must be realized that this is a general relationship, with a large scatter of data about the best-fit line. In recognition of the data scatter, the Asphalt Institute design curve is the lower bound of data from many projects, and, hence, is very conservative (<u>3</u>). A pavement structure with a cement-treated base, for example, may have a much different deflection-ESAL relationship.

The second concept in the Asphalt Institute overlay design procedure is that as the overlay thickness increases, the deflection on top of the overlay is decreased (Figure 5-3.7) according to elastic layer theory. The procedure utilizes two-layer elastic theory to compute the reduction in deflection for increased overlay thickness. The upper layer is the AC overlay and the lower layer is the "composite" of all layers of the existing pavement. The lower layer is characterized by a single modulus computed from Benkelman Beam deflections measured on the existing pavement. The overlay thickness is increased until the deflection is reduced to that required from Figure 5-3.6 for the future traffic loadings (<u>3</u>).

The Asphalt Institute procedure utilizes deflections obtained with a Benkelman Beam, which probably has the most widely developed data base from its long history of use in such measurements. The Benkelman Beam device measures pavement response to an actual wheel load applied at very low



Design Rebound Deflection Chart

Figure 5-3.6. Influence of Deflections on Number of 18-kip ESAL Applications to Failure (4).

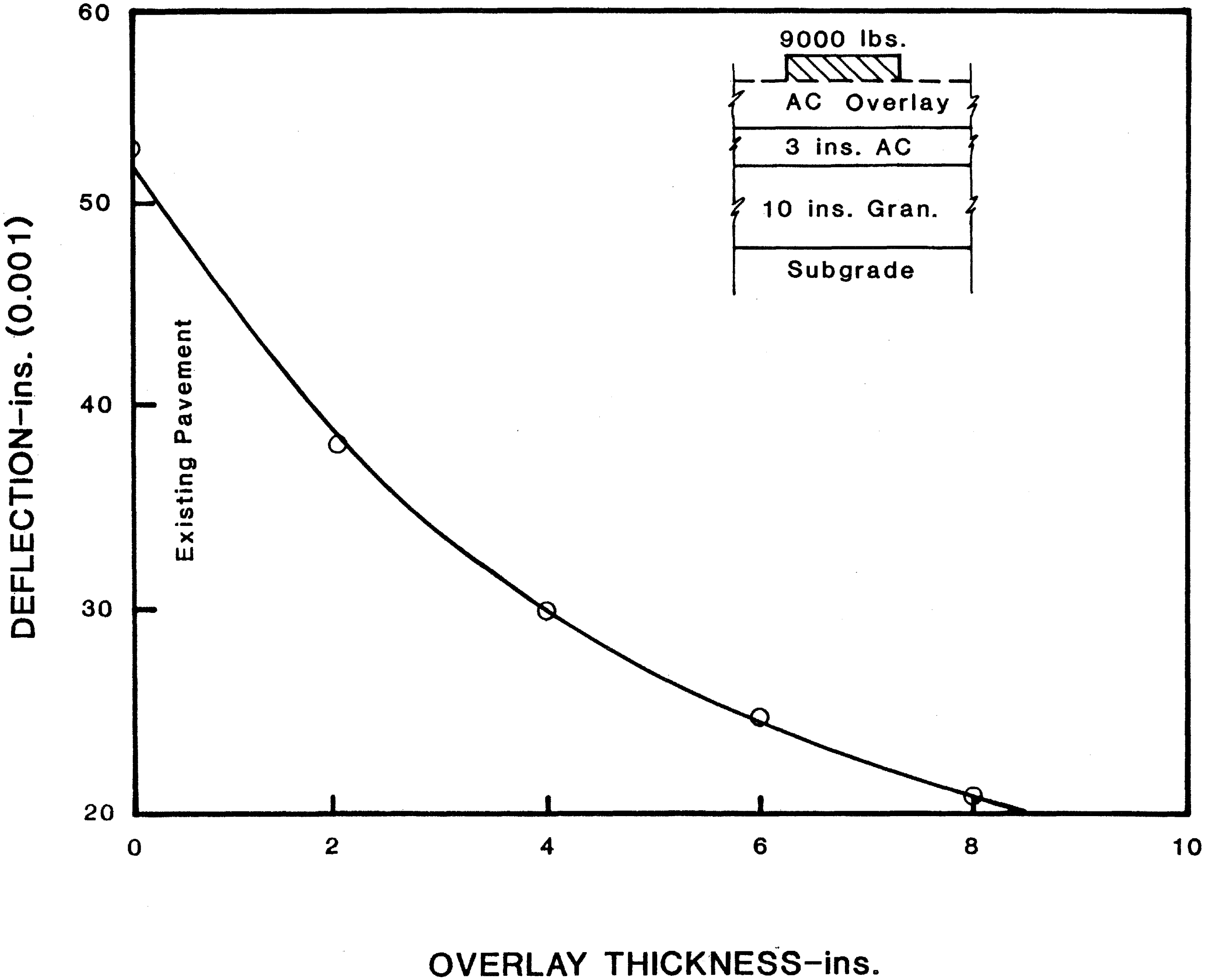


Figure 5-3.7. Effect of Overlay Thicknesses on Maximum Deflection Using Elastic Layer Theory (3).

speeds. The Benkelman Beam is a narrow 12 foot long beam that is placed on the pavement between the dual tires of the rear axle of a loaded truck. The beam is pivoted at a point 8 foot from the probe end. The truck moves ahead at creep speed (5 mph) and the total pavement rebound deflection is read on a dial gauge (rebound deflection is the amount of vertical rebound of a surface that occurs when a load is removed from the surface). It is possible to measure deflections with a device other than a Benkelman Beam and still use the Asphalt Institute design method, provided a correlation is established between the other deflection device and the Benkelman Beam.

The steps required for the Asphalt Institute method of flexible overlay design for flexible pavements are described below (4):

5.1.1 Pavement Condition Evaluation

A pavement condition evaluation consisting of a visual condition survey and deflection testing is performed. The condition survey is performed to establish functional adequacy and the Benkelman Beam is used to obtain pavement deflections which indicate structural adequacy. At least ten measurements are recommended for a specific test section, or a minimum of twenty measurements per mile.

5.1.2 Representative Rebound Deflection

When the deflection tests on the pavement section are completed, the recorded pavement rebound deflections are used to determine a Representative Rebound Deflection (RRD) for the design section. The RRD is determined by the equation:

$$RRD = (x + 2\sigma)\, c \cdot f$$

where:

x = Mean of the Benkelman Beam Deflection, inches

σ = Standard Deviation of Deflection Along Project

c = Critical Period Adjustment Factor

f = Temperature Adjustment Factor

Standard Deviation

Variation in the rebound deflection along the project is taken into account by designing for the mean deflection plus two standard deviations. It is important to plot the deflection profile to see if it varies substantially along the project. If this occurs, then it may be cost-effective to divide the project into two or more design sections, rather than apply the same thickness throughout the entire project.

Critical Period Adjustment Factor

Due to thermal and moisture changes, the deflection of a given pavement will usually vary from season to season. The overlay design is made for the critical damage period, thus it is recommended that deflection of the existing pavement be measured during the critical climatic period. The critical period is defined as the interval during which the pavement is most likely to be damaged by heavy loads. In frost areas, this will normally be the period directly after the spring thaw.

If deflections are not measured in the critical period, it is essential that the measured deflections be adjusted to the critical period. Adjustments are then made to account for seasonal variations in deflection.

These adjustments can be determined from a continuous record of measured rebound deflection values for a similar pavement. This will indicate the critical period, as shown in Figure 5-3.8 and Figure 5-3.9. The type of subgrade is also very important as shown. This allows the rebound deflection measurements to be made during the critical period, in which case the adjustment factor (c) equals 1.0. If the rebound deflection measurements cannot be made during the critical period, an adjustment factor (c) equal to the ratio of the critical period deflection to the deflection of the test date can be determined from the plot. If no record of comparable deflection data is available, the rebound deflection measurements can be made at any time and adjustments should be selected using engineering judgment.

Temperature Adjustment Factor

The RRD must also be corrected to account for the mean pavement temperature at the time of the testing, since the Asphalt Institute methods assumes a standard temperature of 70^{o}F. It is recommended that deflections be repeated at selected reference points throughout the time of testing. In addition, pavement temperature must also be recorded.

A temperature adjustment curve can be developed for the actual pavement during testing since the adjustment factor varies considerably with different pavement types. If actual data is not available, Figure 5-3.10 can provide temperature adjustment factors (f), given the mean pavement temperature and the base thickness. Recommendations for determining the mean pavement temperature, and details and justification for the critical period and temperature adjustment factors, are found in Reference 4.

5.1.3 Traffic Analysis

The Asphalt Institute method requires the computation of the future (anticipated) traffic loadings in the form of 18-kip ESAL applications. A standard analysis procedure for determining the design 18-kip ESAL applications was discussed in Module 2-5.

5.1.4 Overlay Thickness Design

With the Representative Rebound Deflection (RRD) and the future traffic loadings, Figure 5-3.11 is used to determine the required overlay thickness. For example, if the RRD = 0.080 and the design ESAL = 1 million, the AC overlay thickness required is approximately 3 inches.

It is also possible to estimate the remaining life of the existing pavement or to determine the length of time before an overlay is required using the Asphalt Institute Method. More information on these computations is given in Reference 4.

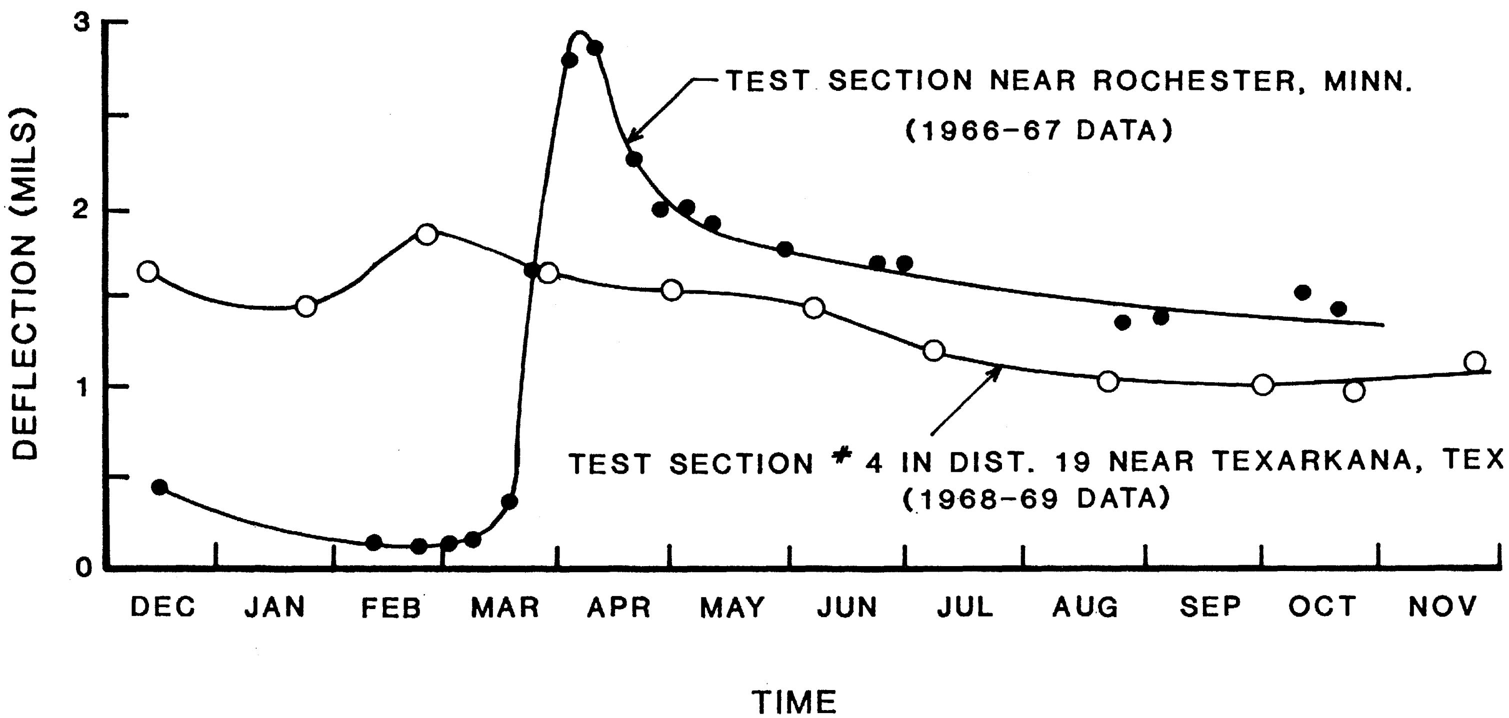


Figure 5-3.8. Illustration of the Effect of Geographical Location on Seasonal Deflections (3).

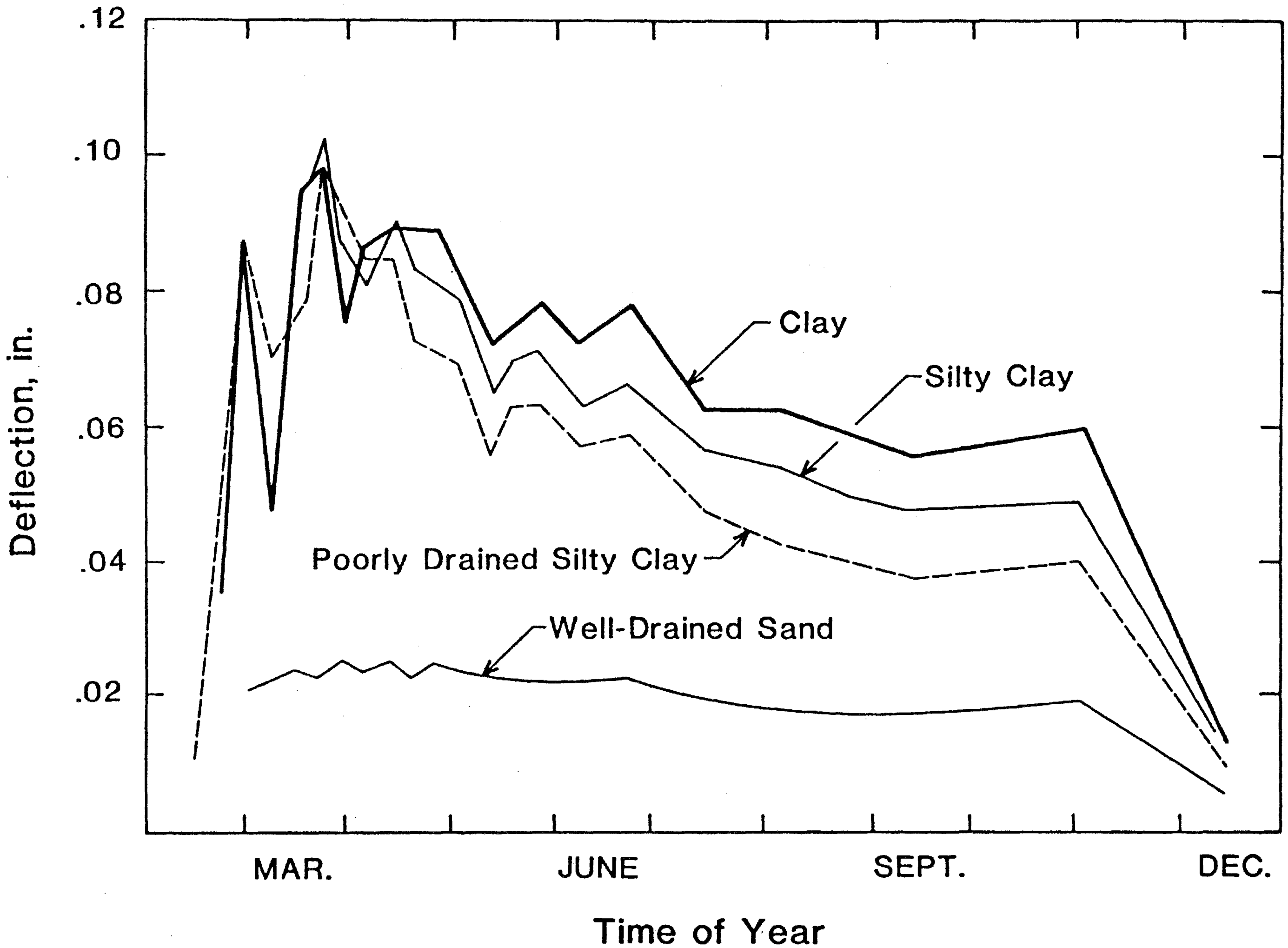


Figure 5-3.9. Influence of Subgrade Type on Seasonal Pavement Deflection Variations (3).

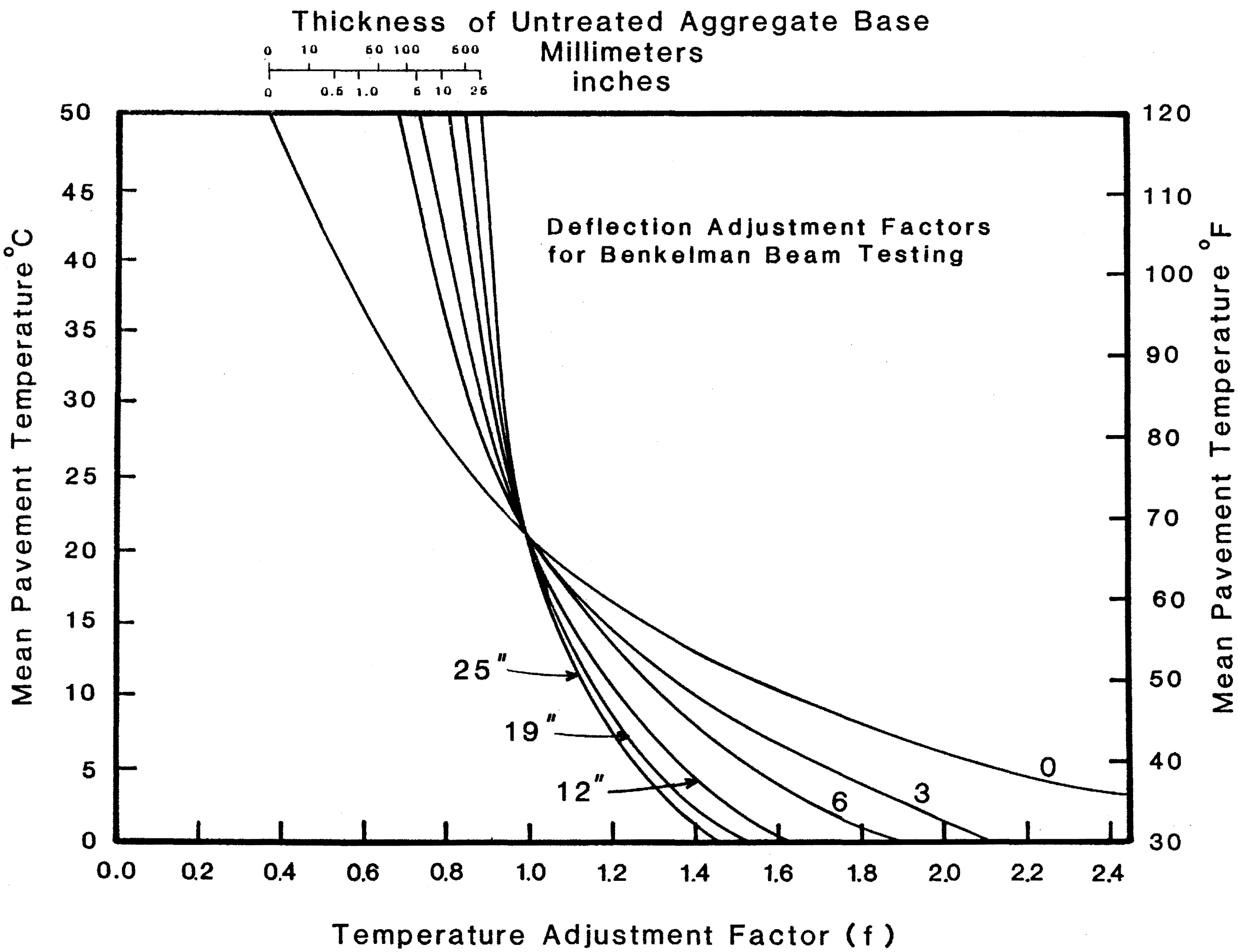


Figure 5-3.10. Temperature Adjustment Factors for Benkelman Beam Deflections (4).

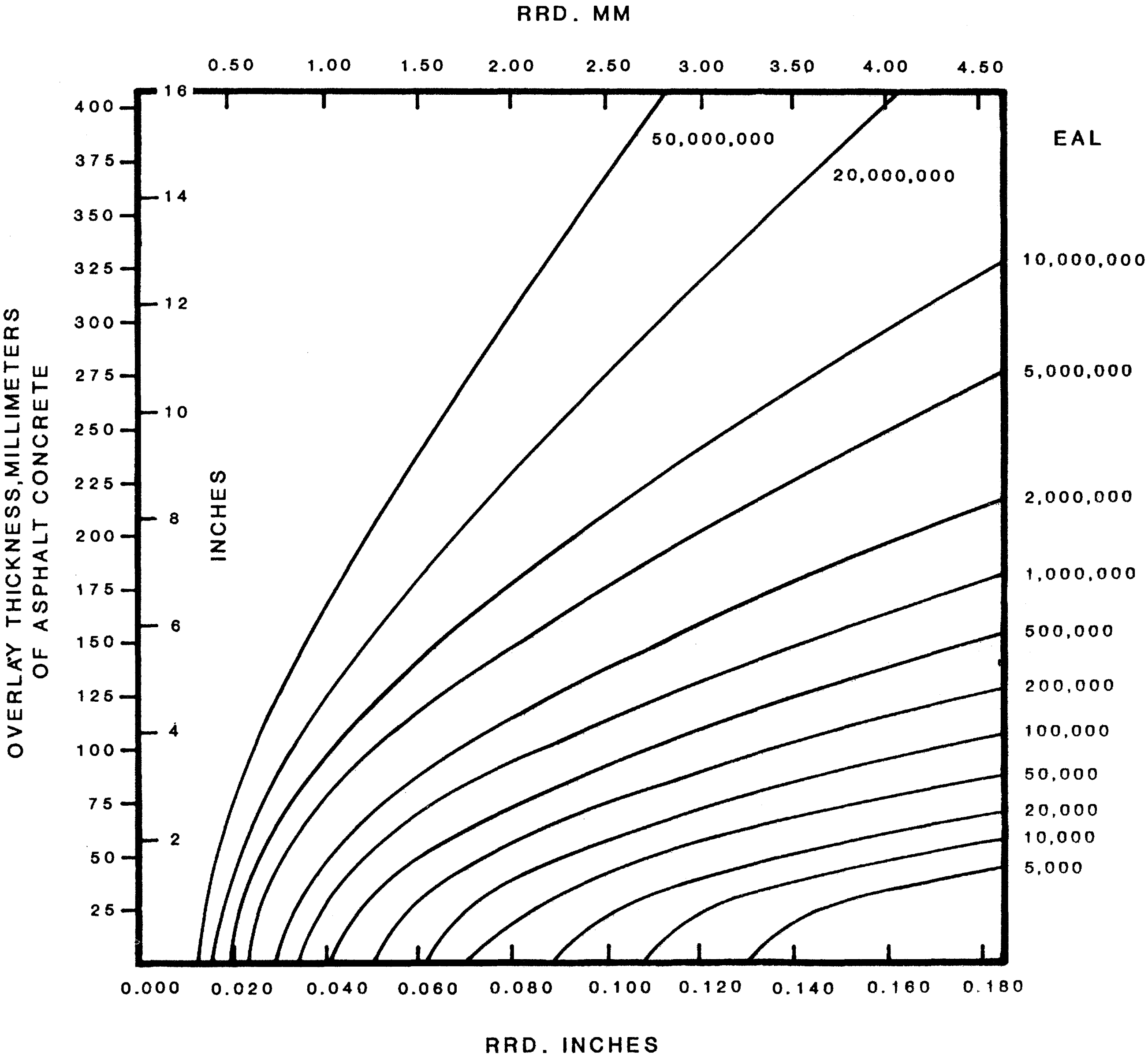


Figure 5-3.11. Overlay Thickness Design Chart (4).

5.1.5 Limitations

There are problems with the Asphalt Institute method of flexible overlay design for flexible pavements which limits the applicability of the procedure. The major limitation is the accuracy of Figure 5-3.6 for predicting the life of the overlaid pavement. This curve is based on new pavement performance and its accuracy for overlays is greatly questionable. This method utilizes the maximum measured deflection without any regard to the shape of the deflection basin which determines the actual state of stresses and strains in the pavements. Also, different "types" of flexible pavements have been shown to have different deflection-traffic curves (e.g., cement-treated base, full-depth AC, lean concrete base). Different climates also result in different performance (e.g., contrast the AASHTO Road Test's (Illinois') wet-freeze climate to that of Southern California's dry-nonfreeze climate). Finally, this method does not directly consider the material properties of the existing pavement or of the proposed overlay.

6.0 RIGID OVERLAYS FOR FLEXIBLE PAVEMENTS

The structural overlay analysis procedure for this overlay method uses the existing flexible pavement as the composite foundation support for a new rigid pavement. Some milling and/or level up of the existing AC pavement may be necessary if severe distortions exist. The design analysis consists of determining the design (composite) modulus of reaction (k_c) of the existing pavement and then following the procedures noted in Block 4 to design the PCC overlay as a new rigid pavement.

NDT deflection testing is the most efficient means to evaluate the composite modulus of reaction (k_c) of the existing pavement. In this situation, it is not necessary to examine the entire deflection basin as only the maximum deflection under the load is used to establish the k_c value. The maximum deflection value is corrected for temperature and used to determine the composite modulus of reaction. Details for determining k_c are discussed in Reference 1. Once the k_c value has been determined, the overlay design is treated as a new rigid pavement design (see Block 4).

7.0 EXAMPLE PROBLEMS

This section gives example problems for both the AASHTO and Asphalt Institute methods of flexible overlay design for flexible pavements.

7.1 AASHTO Design Procedure

Step 1. Collect basic information and design criteria.

<u>ORIGINAL PAVEMENT</u>

Two-lane highway, age = 15 years

Existing pavement structure: 4 inch AC surface course
10 inch crushed stone base course
10 inch gravel sand subbase course

Roadbed soil: Silty-clay

Existing Serviceability Index: 2.6
Terminal Serviceability Index: 2.5

Accumulated 18-kip ESAL: 1.3 million - one direction
Original design 18-kip ESAL: 2.0 million - one direction

OVERLAY REQUIREMENTS

Design Period: 10 years

Design 18-kip ESAL: 5 million - one direction

Step 2. Determine the required SN_y for 10 year design period.

Use the AASHTO thickness design nomograph to determine SN_y.

Design Reliability: 95 percent
Overall Standard Deviation: 0.35

Roadbed soil effective resilient modulus: 5000 psi

Loss of serviceability: 4.4 (after overlay) - 2.5 (terminal) = 1.9

SOLUTION: $SN_y = 5.0$

Step 3. Determine the effective SN of the existing pavement

$$SN_{xeff} = a_1D + a_2D_2m_2 + a_3D_3m_3$$

where:
D_i = Layer thickness determined from coring/boring
a_i = Structural coefficient determined from deflection testing and E backcalculation
m_i = Drainage coefficient determined from:
- drainage time in each layer
- percent time layer is saturated

Corings and borings through the depth of the pavement and into the roadbed soil were taken along the project. Falling Weight Deflectometer (FWD) tests were also conducted along the project and the deflection basins were analyzed. The moduli of the pavement layers and roadbed soil were backcalculated using an elastic layered procedure. Results of these tests are as follows (all average values were obtained):

Layer	Core/Bore Thickness	Modulus	Backcalculated Estimated a_i
AC	4.1 inches	405,000 psi	0.42*
Crushed Stone	9.8 inches	30,000 psi	0.14*
Gravel Sand	10.5 inches	11,000 psi	0.08*
Roadbed Soil		7,500 psi	

* - The a_i values were determined by the recommended modulus correlations.

The deflection data were taken during the early summer and only an adjustment in the resilient modulus of the roadbed soil is deemed necessary. The effective resilient modulus was determined to be 5000 psi using the procedure recommended in the AASHTO Guide.

Drainage coefficients were estimated for the base and subbase layers as follows:

Layer	Drainage	Saturation	m_i
Crushed Stone	Fair (time to drain 1 week)	> 25 %	0.8
Gravel Sand	Poor (time to drain 1 month)	> 25 %	0.6

$$SN_{xeff} = 0.42 * 4.1 + 0.14 * 9.8 * 0.8 + 0.08 * 10.5 * 0.6$$
$$= \underline{3.3}$$

Step 4. Determine the remaining life factor F_{RL}

The F_{RL} value depends on:

R_{Lx} = percent remaining life of the existing pavement (0 - 100)

R_{Ly} = percent remaining life of the overlaid pavement (0 - 100)

The remaining life of the original pavement can be calculated from the traffic data as:

$$R_{Lx} = (2.0 - 1.3)/2.0 = 0.35$$

The calculation of remaining life for the overlay requires an assumption of the serviceability at the time of the next rehabilitation and the traffic used.

$$R_{Ly} = (6.3 - 5.0)/6.3 = 0.21$$

The remaining life factor, F_{RL}, is determined as 0.64.

Step 5. Computation of the final overlay design thickness

$$SN_{ol} = 5.0 - 0.64 * 3.3$$
$$= 2.9$$

$$SN_{ol} = a_{ol}D_{ol}$$

The overlay thickness is computed assuming its elastic modulus is 450,000 psi at 68^{O}F. The structural coefficient is 0.44 for asphalt concrete.

$$D_{ol} = 2.9/0.44 = 6.5 \text{ inches}$$

While this is a very thick AC overlay, it must be noted that the traffic has greatly increased over time. The 18-kip ESAL over the future 10 years is 5/1.3 = 3.8 times greater than over the entire past 15 years. A design reliability level of 95% also results in a greater overlay design thickness.

7.2 Asphalt Institute Design Procedure

Given: Two-Lane Highway in Northern Climate

Existing Pavement:

4-inch AC Surface over 12-inch Crushed Stone Base

Deflections:

Obtained with Benkelman Beam

Deflections measured in Spring

Mean Deflection = 0.034 inches

Standard Deviation = 0.00411 inches

Mean Pavement Temperature = $80^{o}F$

Traffic:

Overlaid Pavement Anticipated 20-Year Traffic Loadings = 10×10^6 18-kip ESALs

Determine: Required AC Overlay Thickness for 20-Year Design Life.

Solution: The Representative Rebound Deflection (RRD) is given as:

$$RRD = (x + 2o)\, c \cdot f$$

The values x (mean deflection) and o (standard deviation of deflection measurements) are known quantities. The correction factor c (critical period correction) and f (temperature correction) need to be identified. The following steps are required for the determination of the overlay thickness design.

1. Critical Period Adjustment Factor (c) Determination.

   Since the measurements were obtained in the Spring, it is assumed that c = 1.0.

2. Temperature Correction Factor (f) Determination.

   With inputs of a 12-inch crushed stone base course and a mean pavement temperature of $80^{o}F$, Figure 5-3.10 gives

   f = 0.92

3. RRD Determination.

$$RRD = [0.034 + 2(0.00411)] \times 1.0 \times 0.92$$

$$RRD = 0.0388 \text{ inches}$$

4. Overlay Thickness Determination.

Figure 5-3.11 is used to find the required overlay thickness, with the following inputs:

RRD = 0.0388 inches

20-Year Anticipated 18-kip ESALs = 10×10^6

The design chart yields:

h_{OL} = 3.70 inches

For practical purposes, use a 4.0 inch AC overlay.

8.0 AASHTO COMPUTER PROGRAM FOR OVERLAY DESIGN

A computerized version of the AASHTO pavement design procedure has been developed and is available for microcomputers. The computer program (DNPS86) incorporates all of the inputs used in the non-computerized method of pavement design.

The computer program is also capable of determining overlay thicknesses for any overlay type and any existing pavement type. However, the program does not directly consider the existing pavement condition or the remaining life in its analysis. That is, an overlay cannot be designed for a given pavement system under given conditions. Rather, the only means of overlay design is accomplished by considering staged construction.

The staged construction involves choosing an analysis period and dividing this time period into two or more performance periods for each stage of construction. For example, if a 20-year analysis period is selected, and an initial flexible pavement is chosen for a 12-year performance period, then an 8-year performance period is required for any subsequent overlays. This is illustrated in Figure 5-3.12, which are actual DNPS86 screens for such a design. Figure 5-3.12 (a) and 5-3.12 (b) are the only additional input screens needed for the overlay design analysis; the other required input screens are the same as those shown in Block 3. Figure 5-3.12 (c) is the output screen providing not only the design of the initial pavement, but also the design of the flexible overlay.

Thus, when conducting an overlay analysis with DNPS86, the program will determine required thicknesses (given the projected performance periods) for the "new" pavement and the subsequent overlay.

9.0 SUMMARY

This module outlined procedures and concepts for the design of overlays on flexible pavements. In the design of any pavement structure, whether new

```
              * * *  FLEXIBLE OVERLAY DESIGN INPUTS  * * *

   PERFORMANCE PERIOD FOR FIRST OVERLAY (YEARS)  . .         8.0

   SERVICEABILITY INDEX AFTER OVERLAY CONSTRUCTION .         4.50

   OVERLAY STANDARD DEVIATION (LOG REPETITIONS)  . .         0.490

   OVERLAY CHARACTERISTICS
     Material Type . . . . . . . . . . . . . . . . . .     Asphalt
     Layer Coefficient . . . . . . . . . . . . . . . .       0.44
     Minimum Thickness (inches)  . . . . . . . . . . .       2.00



F1: HELP  F2: IMPORT/STORE  F3: ANALYZE/PRINT/EXIT  F4: DISPLAY RESULTS
```

a

```
              * * *  FLEXIBLE OVERLAY COST INPUTS  * * *

   OVERLAY CONSTRUCTION COSTS AND SALVAGE VALUE
     Unit Cost of Overlay Material ($/CY)  . . . . .       0.00
     Salvage Value (percent) . . . . . . . . . . . .          0
     Shoulders, If Different Than Overlay ($/lin ft)       0.00
     Mobilization and Other Fixed Costs ($/lin ft) .       0.00

   OVERLAY MAINTENANCE COST
     Initial Year Costs Begin to Accrue  . . . . . .          0
     Yearly Increase ($/lane mile/year)  . . . . . .       0.00



F1: HELP  F2: IMPORT/STORE  F3: ANALYZE/PRINT/EXIT  F4: DISPLAY RESULTS
```

b

```
        * * *  SOLUTION FOR INPUT DATA FILE: acol.dat          * * *

FLEXIBLE PAVEMENT STRUCTURAL DESIGN           LIFE CYCLE COSTS ($/SY)

  Performance Life (yrs)       12.0             Initial Pavement
  18-kip ESAL Repetitions     751289.             Construction           .00
                                                  Maintenance            .00
  Layer        Layer          Required            Salvage Value          .00
   No.      Description       Thickness
                              (inches)          First Overlay
   1        AC Surface          5.34              Construction           .00
   2        Base Course        12.00              Maintenance            .00
   3                                              Salvage Value          .00
   4
   5                                            Second Overlay
                                                  Construction           .00
DESIGN FOR PROJECTED FUTURE OVERLAY(S)            Maintenance            .00
                                                  Salvage Value          .00
                          First      Second                        ----------
  Overlay Type           Asphalt                Net Present Value        .00
  Req'd Thick (in)         4.39
  Perf Life (yrs)          8.0
  18-kip ESAL Reps       737613.                Press Any Key to Continue ...
```

c

Figure 5-3.12. Example DNPS86 Flexible Overlay Design Input Screen, Flexible Overlay Cost Input Screen, and Pavement Design Output Screen (5).

or overlay, it is essential that initial designs be checked to ensure that reasonable results are obtained.

For flexible overlays of flexible pavements, the AASHTO method (structural-deficiency approach) and the Asphalt Institute method (deflection-based approach) were presented. For rigid overlays of flexible pavements, the recommended approach is to treat the problem as a new rigid pavement design after the calculation of the composite modulus of reaction, k_c. This is discussed in more detail in Block 4.

The limitations of the AASHTO approach were found to be in the assigning of layer and drainage coefficients, the determination of remaining life, and the limited empirical basis of the procedure.

The limitations of the Asphalt Institute approach were noted to be the large potential error in the deflection-ESAL curve used to project the life of the overlaid pavement, the use of the maximum deflection only (and not the entire deflection basin), and the lack of more adequately considering material properties in the design procedure.

10.0 REFERENCES

1. "AASHTO Guide for Design of Pavement Structures," Published by the American Association of State, Highway and Transportation Officials, Washington, D. C., 1986.

2. Finn, F. N., and C. L. Monismith, "Asphalt Overlay Design Procedures," NCHRP Synthesis of Highway Practice No. 116, Washington, D. C., 1984.

3. Darter, M. I., S. H. Carpenter, M. Herrin, R. E. Smith, M. B. Snyder, E. J. Barenberg, B. J. Dempsey, and M. R. Thompson, "Techniques for Pavement Rehabilitation," Participants Manual, National Highway Institute, Federal Highway Administration, Washington, D. C., 1984.

4. "Asphalt Overlays for Highway and Street Rehabilitation," Manual Series No. 17 (MS-17), The Asphalt Institute, College Park, MD, 1983.

5. "DNPS86 - Design of New Pavement Systems," Microcomputer Program Developed by Austin Research Engineers, Inc. for the National Cooperative Highway Research Program, Transportation Research Board, National Research Council, 1986.

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MODULE 5-4

DESIGN OF OVERLAYS FOR RIGID PAVEMENT

1.0 INSTRUCTIONAL OBJECTIVES

This module presents current design procedures for asphalt and concrete overlays of rigid pavements. The AASHTO design procedure is emphasized. The Asphalt Institute and the Portland Cement Association procedures are also presented. The underlying principles of the procedures are presented to develop a basic understanding of their strengths and limitations.

Upon completion of this module the participant should be able to accomplish the following:

1. Describe the types of conventional overlays for rigid pavements.

2. Identify major factors that must be considered in rigid pavement overlay design.

3. State the basic concepts of the AASHTO overlay design procedure.

4. Design an AC overlay over a rigid pavement using the AASHTO method.

5. Design different types of concrete overlays over a rigid pavement by the AASHTO and PCA method.

6. List limitations of each design method studied.

2.0 INTRODUCTION

There is no universally accepted overlay design method. The three major approaches to overlay design of rigid pavements are described below. Each agency is encouraged to establish its own method which rationally considers the important factors.

1. Engineering Judgement. A number of agencies rely on the judgement and experience of their engineers in determining the thickness and bonding condition required. Some agencies have monitored the performance of previous overlays and have an approximate estimate of how selected standard overlays will perform. This is particularly true for AC overlays of rigid pavements. Some agencies have set up standards such as 2-inch AC overlays for certain classes of roads, 3-inch AC overlays for other classes, etc. There are obvious deficiencies to this approach because very few engineers have adequate experience to determine the required overlay thickness for a given traffic and design life. The development of an overlay design procedure that quantitatively considers the important design factors is strongly recommended.

2. Structural Deficiency. The basic concept is that the required overlay is equal to the difference between a newly designed pavement structure over the existing subgrade with the desired life, and the structure of the existing pavement. The difference

in structure represents the theoretical structural deficiency that must be met by the overlay. If both pavement structures are made of the same material, the difference is in thickness of concrete or asphalt directly, but if not, equivalency factors must be used to convert different materials into one material, or the AASHTO structural number approach must be used. This is by far the most common approach for the design of overlays over rigid pavements. The AASHTO, Corps of Engineers and other agencies utilize the structural deficiency approach (1,12,13,18).

3. Mechanistic Fatigue Damage Approach. The basic concept is to consider past and future fatigue damage in the existing pavement and overlay. The existing pavement is characterized as a finite element slab layer or an elastic layered system and overlay thickness is increased until traffic load stresses are reduced to provide an acceptable fatigue damage over the design period. Several procedures have been developed by research agencies and are currently under trial evaluation (12,13). None have been utilized widely for overlay design.

Overlay thickness design procedures either assume that adequate pre-overlay repair and reflective crack control actions are taken, or they permit different levels of repair to be considered. If pre-overlay repair and reflective crack control is not adequate, then the overlay will likely fail prematurely and not provide the designed structural service.

3.0 TYPES OF OVERLAYS OVER RIGID PAVEMENTS

There are two major types of overlays for rigid pavements with many subtypes:

1. Asphalt concrete
2. Portland cement concrete

The selection of overlay type and design should depend primarily on life-cycle costs. However, engineering factors such as the condition of the existing pavement, the current and future traffic levels, service life and traffic control constraints must also be considered.

Asphalt concrete overlays have been used extensively for rigid pavements. They may be placed for either structural improvements or for surficial improvements. Problems can develop, including, rutting and reflective cracking, which have led to many premature failures of AC overlays on rigid pavements.

Portland cement concrete (PCC) overlays have been used with success on rigid pavements, including fully bonded, partially bonded and unbonded types. The attainment of adequate bond has been a problem on several bonded concrete overlays.

3.1 Asphalt Concrete Overlays Over Rigid Pavements

The thickness of asphalt concrete (AC) overlays for PCC pavements has typically been determined by using engineering judgement or by modifying empirical flexible pavement overlay design procedures (usually structural deficiency type of approach). Some mechanistic-empirical approaches have also been developed (12,13).

The engineering judgment approach to AC overlay design for rigid pavement is typically a specification of minimum thickness requirements for different highway classes or traffic levels. Minimum thickness requirements for AC overlays on rigid pavements are based on the need to provide level up, prevent localized debonding and potholing of the AC surface and to retard the rate of reflective cracking.

Thin overlays provide only temporary improvement in rideability. AC overlays in excess of 2.5 inches are needed to provide any structural improvement. This represents the approximate effect of a one-inch increase of slab thickness based on equivalent stress in the slab from traffic loads.

As the thickness of the AC overlay increases, two performance trends develop. Rutting increases substantially, and reflection cracking (and its severity) decreases substantially. Both distress types must be considered in AC overlay design to achieve acceptable performance.

A variety of techniques have been used for reflection crack control for AC overlays. These include several different AC overlay layer types and combinations. These are described in Module 5-1.

3.2 Portland Cement Concrete Overlays on Rigid Pavements

There are three types of bonding conditions for concrete overlays over concrete pavements:

1. Fully bonded
2. Partially bonded
3. Unbonded

There are also three different types of conventional types of concrete overlays:

1. Jointed plai
2. Jointed reinforced
3. Continuously reinforced concrete

The most effective type of concrete overlay and bonding condition to be used is mainly a function of costs and the existing pavement condition. Costs are primarily a function of pre-overlay repair needed and traffic control problems to build the concrete overlay.

3.2.1 Fully Bonded Concrete Overlays

Through specific construction procedures, a complete permanent bond may be achieved between the overlay and the existing pavement. An all out effort must be made to achieve permanent full bond between overlay and existing slab. If the existing pavement is in fair to good condition and a structural improvement is needed, a fully bonded concrete overlay may be cost effective. The damaged slabs must be replaced. The thickness required for a fully bonded overlay is much less than for the other concrete overlay types and generally ranges from 2 to 4 inches.

3.2.2 Partially Bonded Concrete Overlays

Partially bonded concrete overlays require repair and/or replacement of damaged slabs. The surface should at least be cleaned by sweeping and water blasting or sand blasting (or equivalent) so that as much bonding as possible occurs without a large expenditure of funds as with the fully bonded overlay. Typical thicknesses range from 5 to 7 inches, which is thicker than a fully bonded overlay because a full bond is not guaranteed and the PCC overlay slab must be capable of performing satisfactory if substantial debonding occurs.

3.2.3 Unbonded Concrete Overlays

Unbonded concrete overlays are normally used to improve the structural capacity of an existing concrete pavement in relatively poor condition. A "bond breaking" level-up layer is placed between the existing slab and the overlay to prevent any bond between layers and absorb movement of the base slab which would otherwise crack the PCC overlay. Unbonded overlays are thicker than the other concrete overlay types (8 to 10 inches) because of complete lack of bond between the layers.

4.0 AASHTO OVERLAY DESIGN METHODOLOGY

The AASHTO overlay design procedure is based on the serviceability-performance relationships presented in Block 2. Also implicit in this approach is the remaining pavement life concept which considers both the damage within the existing pavement as well as the desired level of damage (terminal serviceability level) for the overlaid pavement. The AASHTO procedure strongly recommends the use of nondestructive testing as part of overlay rehabilitation for the characterization of the in-place slab, subbase and subgrade material properties to determine the structural adequacy of the existing pavement (1).

The basis of the AASHTO design procedure is shown in Figure 5-4.1. In comparing serviceability to traffic repetitions several serviceability values must be considered:

p_o = Initial serviceability of the existing pavement surface (when constructed or when last overlaid).

p_{t1} = Terminal serviceability of the existing pavement immediately prior to the overlay.

p_{t2} = Terminal serviceability desired from the overlaid pavement after the overlay design traffic has been applied.

p_f = The ultimate failure serviceability for any pavement type corresponding to a completely damaged (failed) pavement.

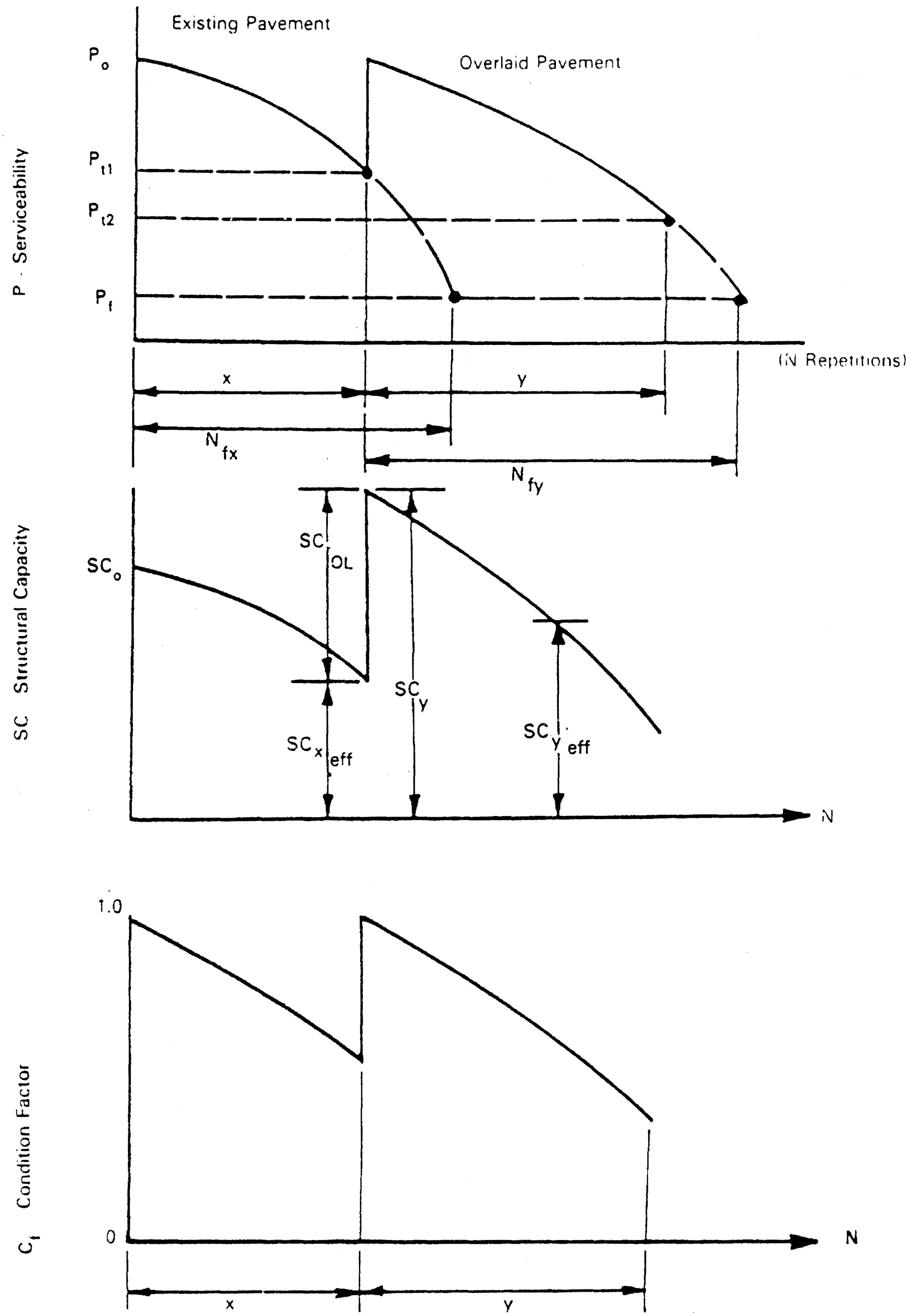


Figure 5-4.1. Relationship Between Serviceability-Capacity Condition Factor and Traffic (1).

It is not essential that p_{t1} equal p_{t2} since serviceability at the time of the overlay and serviceability at the end of the overlay period are input considerations of the designer. For this overlay method, "failure" serviceability is P_f=2.0. Therefore, when a pavement serviceability reaches a p_f=2.0, the pavement is said to be 100 percent damaged with no remaining life. Since this is a subjective value, pavements with more than 100 percent damage or negative remaining life may exist.

In Figure 5-4.1, "x" represents the actual ESAL repetition on the original pavement up to the desired time of the overlay. The "y" represents the future equivalent repetitions within the overlay period. The values N_{fx} and N_{fy} represent the total number of repetitions necessary for the original pavement and the overlay to reach "failure" serviceability, respectively.

In terms of structural capacity, Figure 5-4.1 demonstrates that increases in traffic repetitions gradually reduce the initial structural capacity, SC_o to some "effective capacity" prior to the overlay. The overall pavement condition factor, C_x, is related to the effective capacity by the following:

$$SC_{xeff} = C_x \, SC_o$$

The structural capacity necessary to support the overlay traffic during the overlay period is noted by SC_y.

The difference in structural capacity between what is needed for the future overlay period and what is effectively available in-place at the time of the overlay represents the additional structural capacity required of the overlay.

When the concept of remaining life is considered, the general structural deficiency overlay design equation becomes:

$$SC_{overlay}^{\,n} = SC_y^{\,n} - F_{RL} \, (SC_{xeff})^n$$

In this equation, the power "n" is a constant dependent upon the type of overlay pavement. The three primary factors which determine the amount of additional structural capacity required from the overlay are:

SC_y = The total structural capacity required to support the overlay design traffic over existing subgrade conditions.

Note: For AC overlay of rigid pavement, this value is the new SN required. For PCC overlay this value is the new slab thickness required.

SC_{xeff}= The effective structural capacity of the existing pavement immediately prior to the time of the overlay reflects the damage to that point in time.

Note: For AC overlay of rigid pavement, this value is the effective SN of the exiting slab plus base. For PCC overlay of rigid pavement, this value is the effective slab thickness (reduced for cracking).

F_{RL} = The remaining life factor which accounts for damage of the existing pavement as well as the desired degree of damage to the overlay after the application of the overlay traffic. It is always less than or equal to a value of 1.0.

n = A constant depending upon the type of bonding condition between the existing slab and overlay (1 to 2).

Once these three variables are determined, the required overlay thickness can be directly computed. Sections 5.0 and 8.0 present how these variables are determined for AC and PCC overlays, respectively using the AASHTO procedure.

5.0 DESIGN OF ASPHALT CONCRETE OVERLAYS OVER RIGID PAVEMENTS USING THE AASHTO PROCEDURE

This section summarizes the AASHTO design process for flexible overlays over existing rigid pavements. It should be emphasized that the approach presented below is applicable to all overlay types and existing pavement types.

There are seven steps in the overlay design procedure (1).

1. Analysis Unit Delineation
2. Traffic Analysis
3. Materials and Environmental Study
4. Effective Structural Capacity Analysis (SC_{xeff})
5. Future Overlay Structural Capacity Analysis (SC_y)
6. Remaining Life Factor Determination (F_{RL})
7. Overlay Design Analysis

A discussion of these steps follows.

5.1 Analysis Unit Delineation

The first step in the overlay process is the clear delineation of basic analysis units. The objective is to determine boundaries along the project length that subdivide the rehabilitation project into statistically homogenous pavement units possessing uniform pavement cross sections, subgrade support, construction histories, and subsequent pavement condition. If more than one analysis unit exists, the engineer must use his/her best judgement to decide whether a variable overlay design along the project length is practical or whether a uniform overlay design should be developed (1).

Case I. Accurate Historic Data Unavailable.

If a review of historic data has disclosed little useful information, an NDT deflection study should be conducted first along with a visual distress survey. The pavement response data and distress data thus obtained should be analyzed to delineate the boundaries of the analysis units. The engineer should then make an evaluation regarding the practicality of these unit lengths. A general guideline for a minimum unit (construction) length is 0.5 miles.

The analysis units are then used as the basis for conducting any destructive tests (coring) necessary to determine pavement layer material type and layer thickness. In turn, this information is then used to verify and/or modify the analysis units previously established.

Case II. - Accurate Historic Data Available.

When accurate historic construction and design information regarding a section of pavement is available, the engineer has a relatively good idea concerning the location of unit boundaries prior to any field testing. Nondestructive deflection testing and visual surveys should be conducted to verify or modify the preliminary units selected. Ten to fifteen NDT test points should be randomly selected within each unit. If necessary, a destructive sampling (i.e. coring and boring) plan may be then developed to further examine the appropriateness of the analysis units selected.

5.2 Traffic Analysis

The purpose of the traffic analysis step is to determine the cumulative 18 kip equivalent single-axle load (ESAL) applications along a pavement length from the date the PCC slab was originally constructed through the end of the anticipated overlay period. Traffic computations are presented in Module 2-6.

5.3 Materials and Environmental Study

Design values for layer materials used in the rehabilitation process may be categorized into three major groups:

1. Existing pavement layer properties.

2. Existing pavement subgrade properties.

3. Design properties of overlay layers (including the use of recycled materials.

The primary material property of concern for all three categories listed above is the elastic or resilient modulus. Nondestructive testing is a valuable tool to determine the in-place modulus values of each layer within the pavement structure. Coring and lab testing is another, but more costly method, to estimate modulus values.

5.3.1 Modulus Prediction of Existing Pavement Layers

The underlying assumption of NDT is that a set of layer moduli exists such that the predicted deflection basin under the dynamic load-pavement combination yields values equal to the measured deflection basin. Calculation of modulus values for concrete pavements may be done using "backcalculation" techniques, similar to those used for flexible pavement modulus backcalculation. Analysis procedures are presented in Module 5-3 and References 1 and 15.

The interior center slab loading position is used for all backcalculation analyses. Testing at concrete edges or corners is not recommended for backcalculation purposes because of asymmetrical support conditions and the possibility of voids beneath the slab.

There are two different conditions for which backcalculation of the modulus of the slab may be required:

1. Noncracked slab: The load plate is not placed near any crack in the slab, the E_{pcc} represents the actual concrete modulus of elasticity.

2. Cracked slab: The load plate is placed directly adjacent to a transverse working crack, the E_{pcc} represents the modulus of elasticity at a typical working crack which is representative of a damaged slab.

If a pavement has numerous working transverse cracks and these will not be repaired with a full-depth doweled or tied repair, then the loss of structural capacity caused by these cracks must be considered in the evaluation of the existing rigid pavement.

5.3.2 Prediction of Subgrade Strength

Most rigid pavement systems will have a subbase-subgrade foundation requiring the development of a composite modulus of reaction term, k_c. The NDT-derived moduli must be converted to this rigid pavement design parameter. As with flexible pavements, the influence of environmental factors upon NDT-derived values must be considered. Once the subbase and subgrade values are determined, the "effective modulus of subgrade reaction" must be determined using Figure 5-4.2, or through direct backcalculation (15).

NDT equipment used for backcalculation must produce loads of similar size and duration of those produced by heavy trucks. This insures the calculation of modulus values that are similar to the effective modulus values under heavy truck loading.

5.3.3 Design Properties of Overlay Material

The selection of appropriate design modulus and/or strength parameters for the overlay material should be examined. The desirable and typical properties have been discussed in Module 5-3.

5.4 Effective Structural Capacity Analysis

The fourth step in an overlay analysis is to estimate effective in-place structural capacity of the pavement to be overlaid. This parameter is determined by use of the material properties derived in the previous step. When existing rigid pavements are evaluated, the resulting effective structural capacity SC_{xeff}, is equal to the effective thickness D_{xeff}, which includes the PCC slab and subbase.

5.4.1 NDT Method

The predicted layer moduli of the existing PCC slab (E_{pcc}) is the important parameter used to define the D_{xeff} value. This value can be determined with the use of Figure 5-4.3. Knowing the E_{pcc} from the NDT analysis and the total PCC thickness of the existing slab, D_o, the D_{xeff}

Example:

D_{SB} = 6 inches

E_{SB} = 20,000 psi

M_R = 7,000 psi

Solution: k_∞ = 400 pci

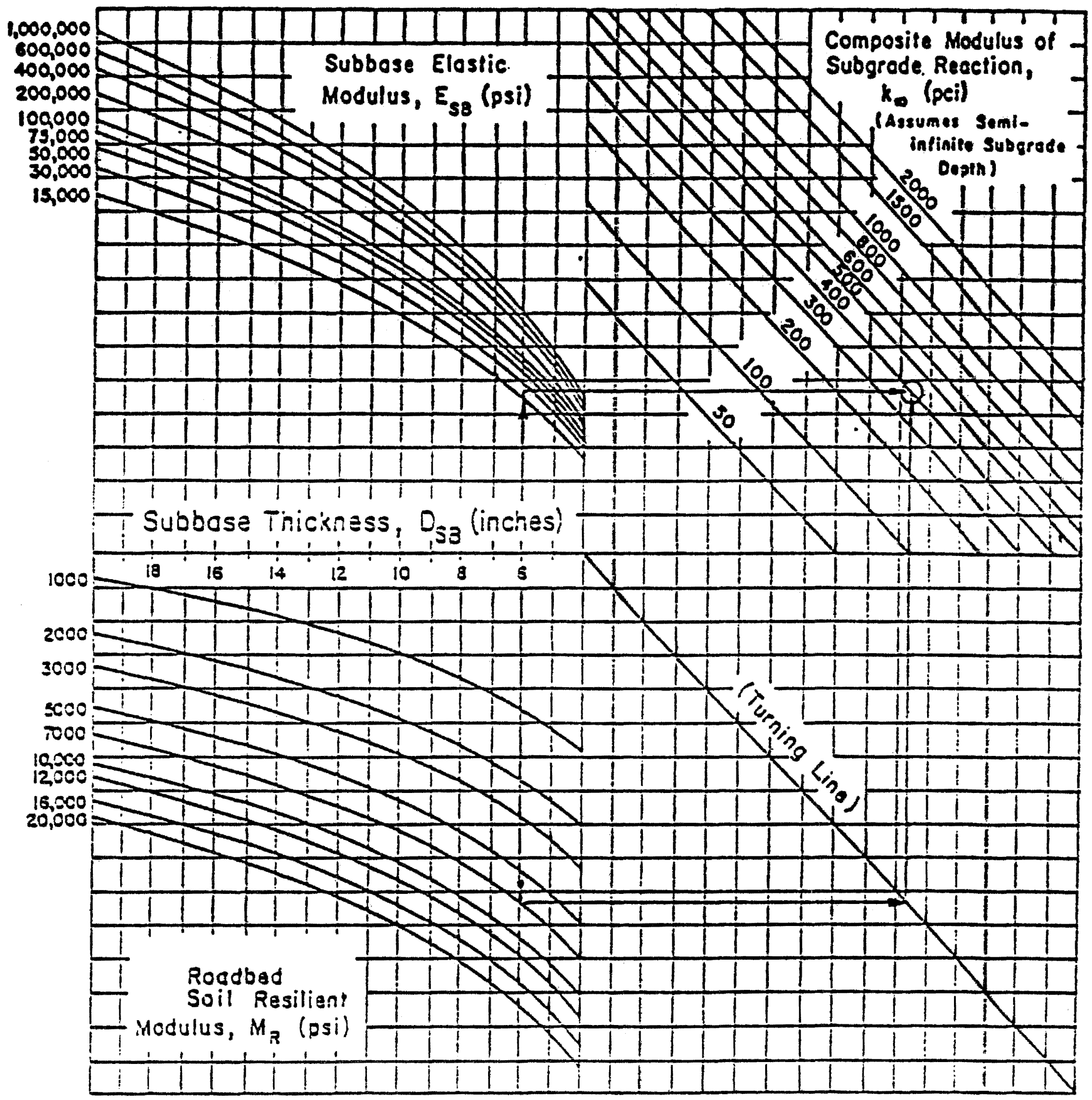


Figure 5-4.2. Chart for Estimating Composite Modulus of Subgrade Reaction, Assuming a Semi-Infinite Subgrade Depth. (For Practical Purposes, a Semi-Infinite Depth is Considered to be Greater than 10 feet Below the Surface of the Subgrade.)

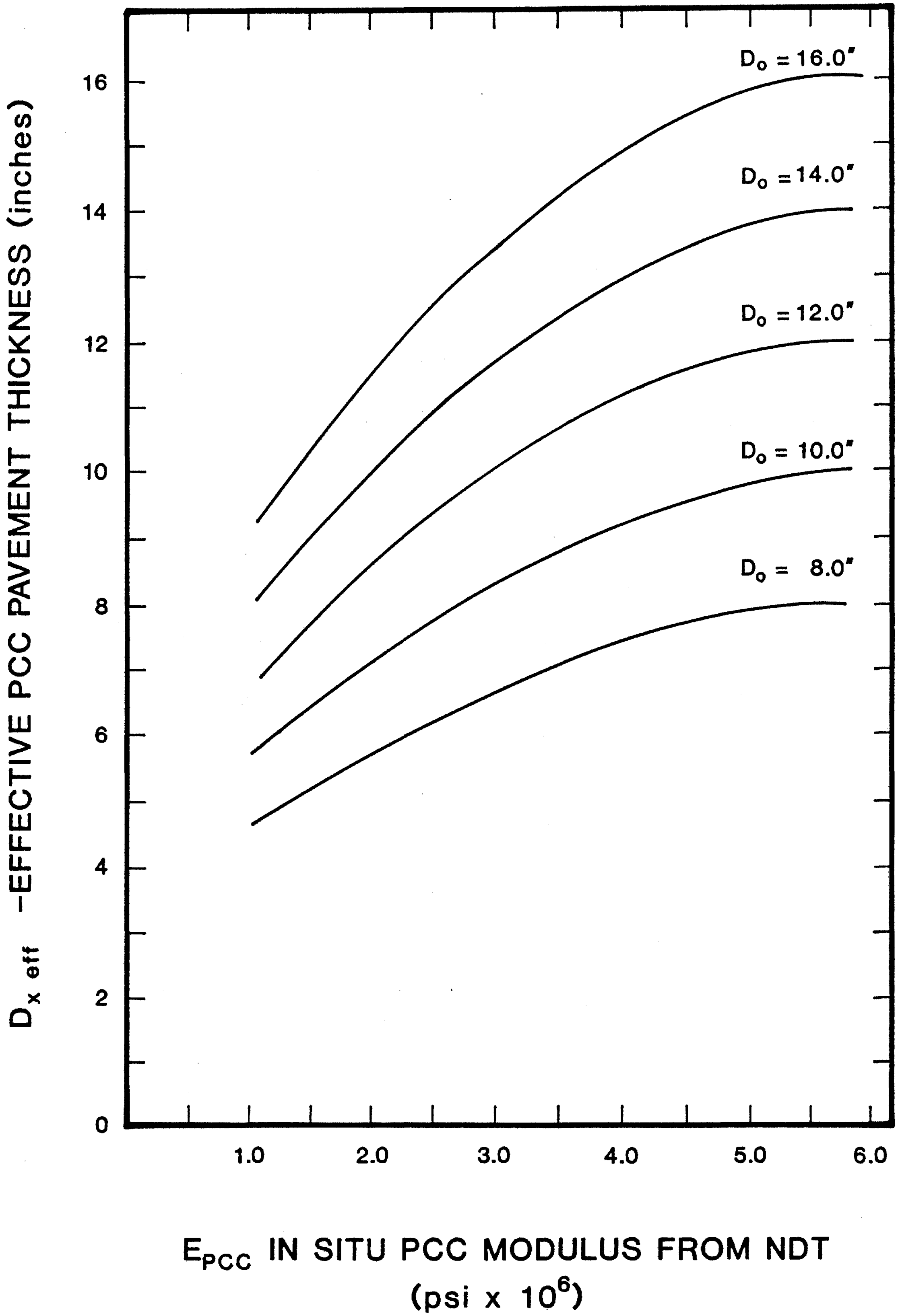


Figure 5-4.3. Determination of Effective PCC Structural Capacity for NDT-Derived PCC Modulus (1).

value can be directly read from the Figure. For example, if D_o=10.0 inches and the effective PCC modulus determined from NDT was $E_{pcc} = 3.0 \times 10^6$ psi, then $D_{xeff} = 8.4$ inches.

5.4.2 Non-NDT Approximate Procedure

In the event that it is not possible to utilize NDT measurements for overlay evaluation, three alternative techniques are recommended.

1. Visual Condition Factor Approach. Figure 5-4.4 shows the approximate relationship between the C_y value (visual condition factor reflecting the degree of slab cracking) and the cracked PCC modulus percentage, E_R. The E_R value multiplied by a undamaged PCC value of 5 million psi, gives an estimate of the in situ (or effective) PCC modulus, E_{pcc}. Once this is determined, Figure 5-4.2 is used to determine the D_{xeff} value.

2. Nominal Size of PCC Slab Fragments. Figure 5-4.5 depicts the approximate relationship between the E_R ratio and the nominal size of slab fragments of a cracked PCC pavement. When an estimate of the slab fragment size is made, the approach for determining the D_{xeff} is identical to that in the visual condition factor approach.

3. Remaining Life Approach. The remaining life of the existing pavement (R_{Lx}, percent remaining life from initial construction to a serviceability of 2.0) can be estimated using any one of six different methods as described in Section 4.2.6. Once the percent remaining life is estimated (say 10 percent), the relationship shown in Figure 5-4.6 is used to determine the C_x value (pavement condition factor). The D_{xeff} value is then computed as:

$$D_{xeff} = C_x D_o.$$

5.5 Future Overlay Structural Capacity Analysis

The fifth step is to determine the future required overlay structural capacity (SC_y, which is actually SN_y for AC overlay of rigid pavement, or D_y for PCC overlay of rigid pavement). This is the total structural capacity of a new pavement required to carry y load repetitions in the overlay design period to a terminal serviceability of p_{t2} (Figure 5-4.1) using the existing subgrade support for the design value.

The analysis assumes (for the moment) that the existing pavement (SN_{xeff} or D_{xeff}) does not exist over the roadbed soil. Consequently, this step in the overlay process is simply a new pavement design (SN_y) for a flexible pavement system or a new slab design for a rigid pavement system. The design nomograph to determine a new flexible pavement SN_y using the AASHTO procedure is presented in Module 5-3.

5.6 Remaining Life Factor Determination

Step six in the overlay design analysis is to determine the remaining life factor, F_{RL}. This factor is an adjustment factor (about 0.5 to 1)

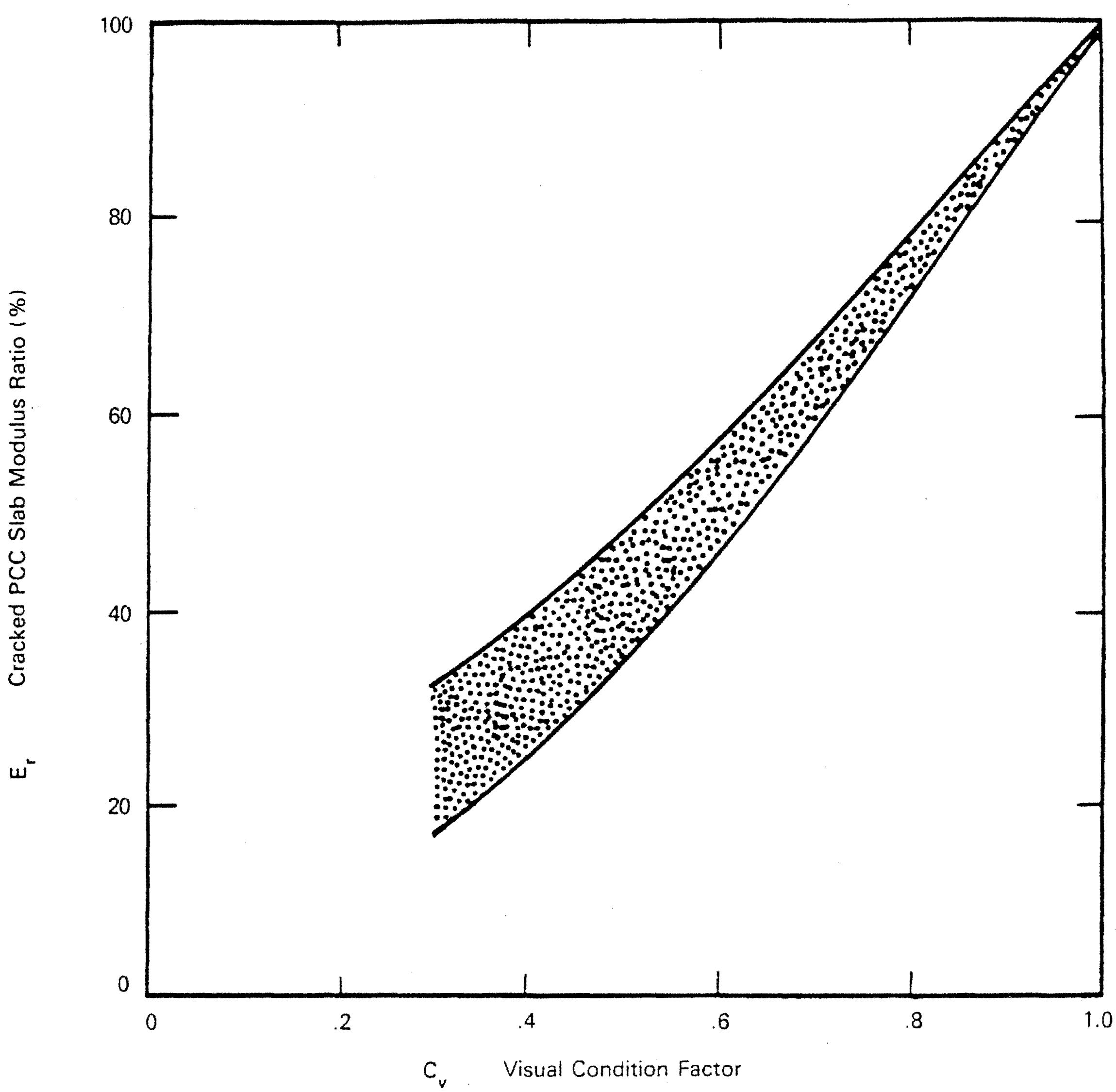


Figure 5-4.4. Relationship of Visual Condition Factor to PCC Cracked Modulus Ratio (1).

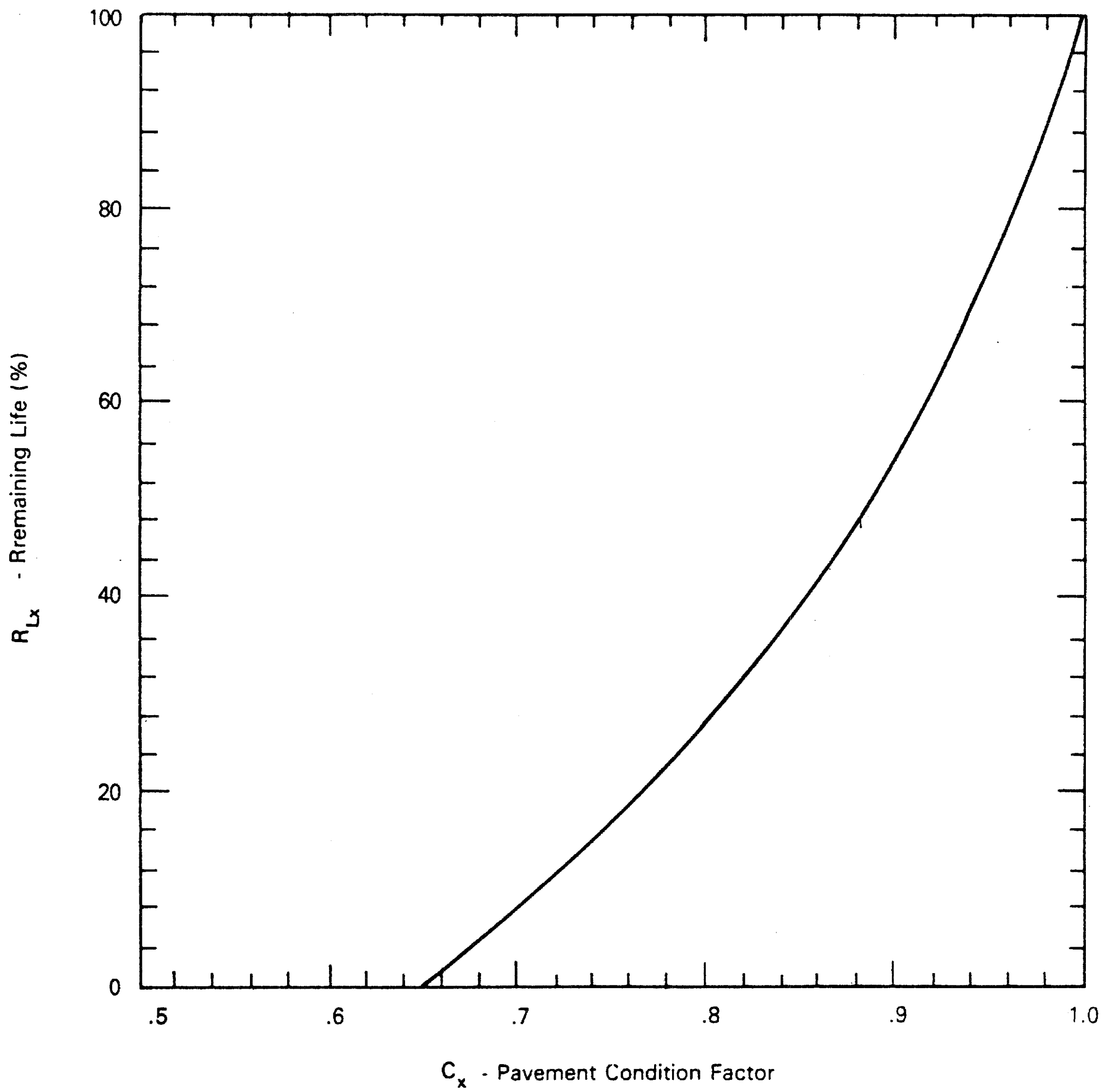


Figure 5-4.5. Remaining Life Estimates Predicted from Pavement Condition Factor (1).

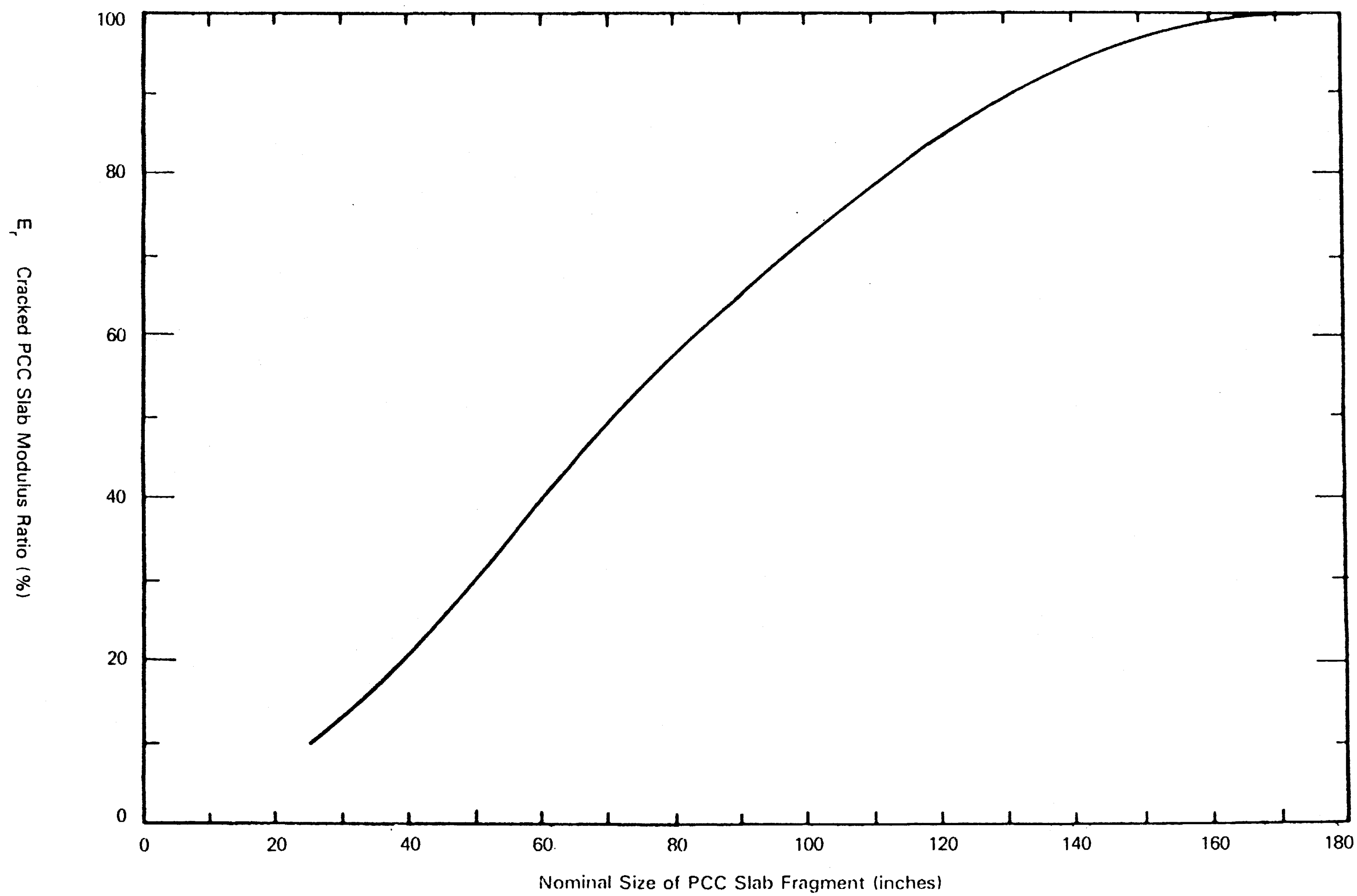


Figure 5-4.6. Relationship of Slab Fragment Size to PCC Cracked Modulus Ratio (1).

multiplied by the effective thickness (D_{xeff}) to reflect a more realistic assessment of the weighted effective capacity during the overlay period. The F_{RL} reduces the existing slab thickness. This factor depends upon the following two factors:

R_{Lx} = percent remaining life value of the existing pavement prior to overlay (0 to 100).

R_{Ly} = percent remaining life of the overlaid pavement system after the overlay design traffic has been applied and terminal serviceability has been reached (0 to 100).

5.6.1 Remaining Life of the Existing Pavement

The remaining life of the existing pavement prior to overlay is a difficult parameter to accurately determine. There are five possible methods to estimate the R_{Lx} value. Each of the methods are presented in detail in Reference 1. They are summarized below:

1. NDT Approach. The percent remaining life is found by comparing the effective structural capacity (D_{xeff}) of the existing pavement to the initial (new pavement) structural capacity (D_o).

   $$C_x = D_{xeff}/D_o$$

   The C_x is then used in Figure 5-4.6 to determine the percent remaining life (R_{Lx}).

2. Traffic Approach. When reasonably accurate traffic information is available, the predicted traffic loading to a specified failure level (generally taken as $p_f = 2.0$) is computed and compared to the actual cumulative traffic loading over the life of the existing pavement.

   $$R_{Lx} = ((N_{fx} - x)/N_{fx})\ 100$$

3. Time Approach. The remaining life factor is estimated based on the time the section has been in service (t), its expected life before requiring an overlay (T_f), and projected traffic growth rate (r_g). Figure 5-4.7 is used to determine the R_{Lx} value.

4. Serviceability Approach. The Present Serviceability Index (PSI) and engineering judgment are used to give an estimation of the remaining life using Figure 5-4.8.

5. Visual Condition Survey Approach. An overall pavement condition factor (C_x) is computed from the individual pavement layer condition factors (C_v) (see Table 5-4.1). C_x is computed and the percent remaining life is determined from Figure 5-4.6.

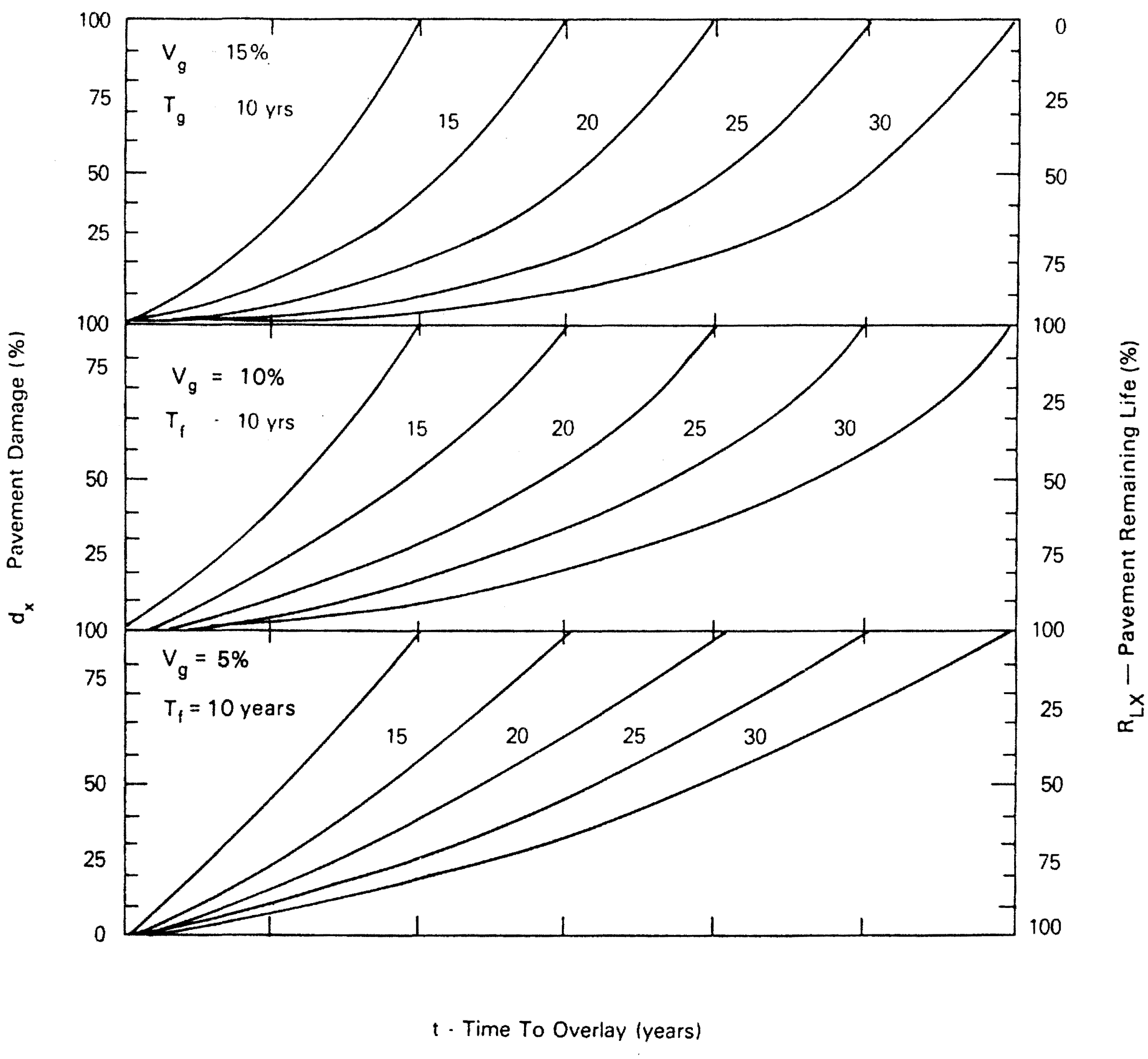


Figure 5-4.7. Remaining Life Estimate Based on Time Considerations for Various Traffic Growth Rates (1).

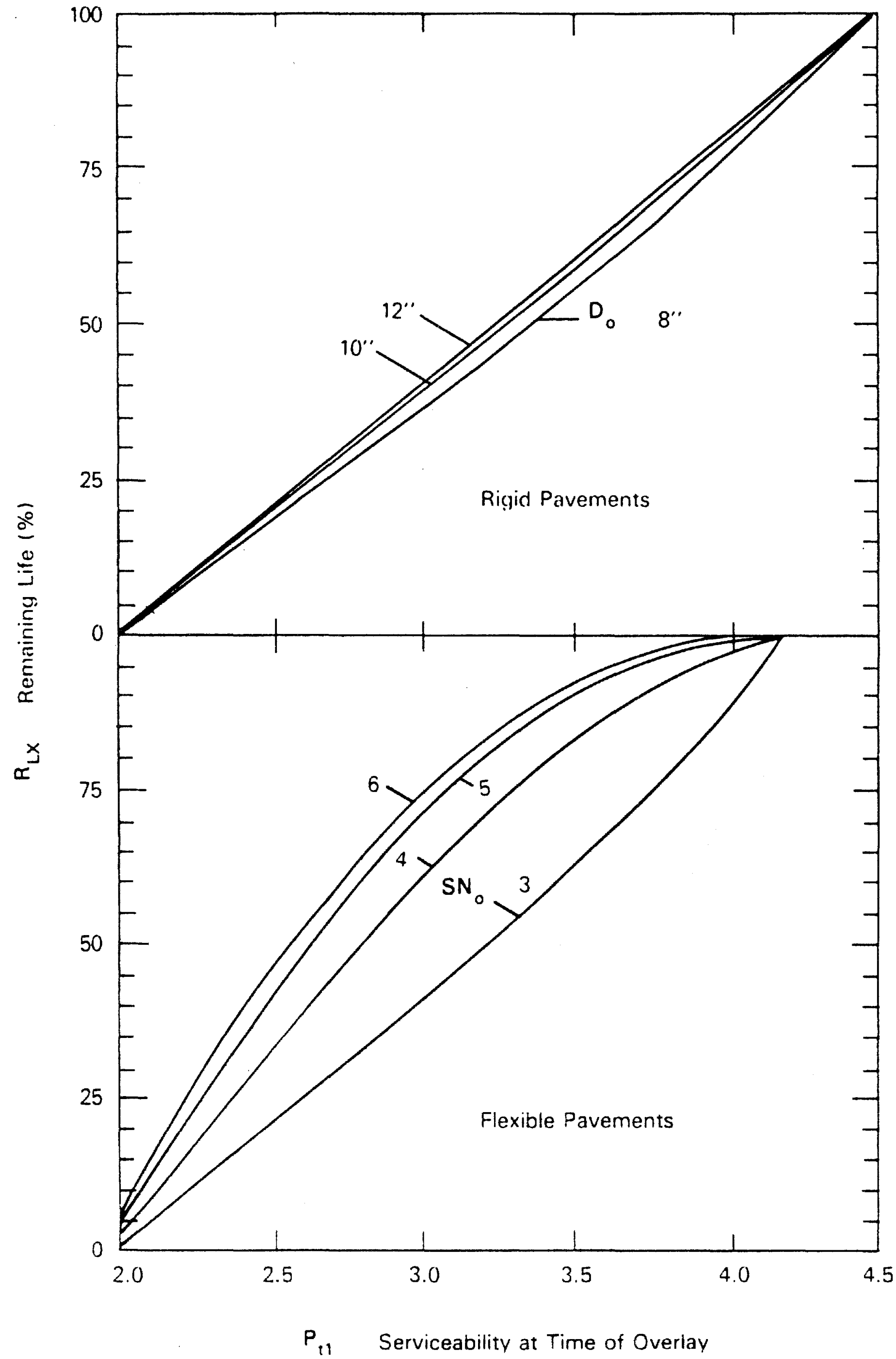


Figure 5-4.8. Remaining Life Estimate Based on Present Serviceability Value and Pavement Cross Section (1).

Table 5-4.1. Summary of Visual (C_V) and Structural (C_X) Condition Values (1).

Layer Type	Pavement Condition	C_v Visual Condition Factor Range	C_x Struct Cond Factor Value
Asphaltic	1. Asphalt layers that are sound, stable, uncracked and have little to no deformation in the wheel paths	0.9-1.0	.95
	2. Asphalt layers that exhibit some intermittent cracking with slight to moderate wheel path deformation but are still stable.	0.7-0.9	.85
	3. Asphalt layers that exhibit some moderate to high cracking, have ravelling or aggregate degradation and show moderate to high deformations in wheel path	0.5-0.7	.70
	4. Asphalt layers that show very heavy (extensive) cracking, considerable ravelling or degradation and very appreciable wheel path deformations	0.3-0.5	.60
PCC	1. PCC pavement that is uncracked, stable and undersealed, exhibiting no evidence of pumping	0.9-1.0	.95
	2. PCC pavement that is stable and undersealed but shows some initial cracking (with tight, non working cracks) and no evidence of pumping	0.7-0.9	.85
	3. PCC pavement that is appreciably cracked or faulted with signs of progressive crack deterioration: slab fragments may range in size from 1 to 4 sq.yds., pumping may be present	0.5-0.7	.70
	4. PCC pavement that is very badly cracked or shattered into fragments 2-3 ft. in maximum size	0.3-0.5	.60
Pozzolanic Base/ Subbase	1. Chemically stabilized bases (CTB, LCF...) that are relatively crack free, stable and show no evidence of pumping	0.9-1.0	.95
	2. Chemically stabilized bases (CTB, LCF...) that have developed very strong pattern or fatigue cracking, with wide and working cracks that are progressive in nature: evidence of pumping or other causes of instability may be present	0.3-0.5	.60
Granular Base/ Subbase	1. Unbound granular layers showing no evidence of shear or densification distress, reasonably identical physical properties as when constructed and existing at the same "normal" moisture - density conditions as when constructed	0.9-1.0	.95
	2. Visible evidence of significant distress within layers (shear or densification), aggregate properties have changed significantly due to abrasion, intrusion of fines from subgrade or pumping, and/or significant change in in situ moisture caused by surface infiltration or other sources	0.3-0.5	.60

Special Notes:

1. The visual condition factor, C_v is related to the structural condition factor, C_x by:

$$C_v + C_x^2$$

2. The structural condition factor, C_x and *not* the C_v value, is the variable used in the structural overlay design equation (for all overlay-existing pavement types). It is defined by:

$$SC_{xeff} + C_x SC_o$$

None of the five procedures noted is consistently superior to the others in estimating the percent remaining life R_{Lx} value. However, the procedure utilizing NDT deflection studies often results in a better quantitative assessment of the existing in-place structural capacity, and should usually be relied on more heavily than other approaches. Nonetheless, the engineer should must the reasonableness of several results rather than rely on one approach (1).

5.6.2 Remaining Life of Overlaid Pavement

Determination of the percent remaining life of the overlaid pavement system (or R_{Ly}, which is also a percent from 0 to 100) uses the design input parameters selected by the engineer. The remaining life is directly set by the engineer in his selection of the desired terminal serviceability for the SC_y. The following equation can be used to calculate the percent remaining life of the overlaid pavement.

$$R_{Ly} = (N_{fy} - y)/N_{fy}$$

y: Future overlay design 18-kip ESAL traffic to terminal serviceability.

SC_y: Structural capacity of pavement to yield the terminal serviceability after y load repetitions (new pavement design either SN_y for AC overlays or D_y for rigid overlays).

N_{fy}: Number of 18-kip ESAL load repetitions to failure serviceability, 2.0.

5.7 Remaining Life Factor

The final step is the determination of the "remaining life factor" (F_{RL}) (a number from about 0.5 to 1.0). This value is determined using Figure 5-4.9 and depends upon R_{Ly} and R_{Lx}.

5.8 Asphalt Concrete Overlay Thickness Determination

The equations to be used for conventional AC overlays over rigid pavements are shown in Table 5-4.2. There are also equations to be used for the break and seat technique, which is discussed in Module 5-1.

The following definitions apply to the equations presented in Table 5-4.2:

D_o = Existing PCC layer thickness.

D_{xeff} = Effective thickness of the in-place (cracked) PCC layer reflecting its reduced modulus value.

SN_{xeff} = The total effective structural number of the existing pavement structure above the subgrade.

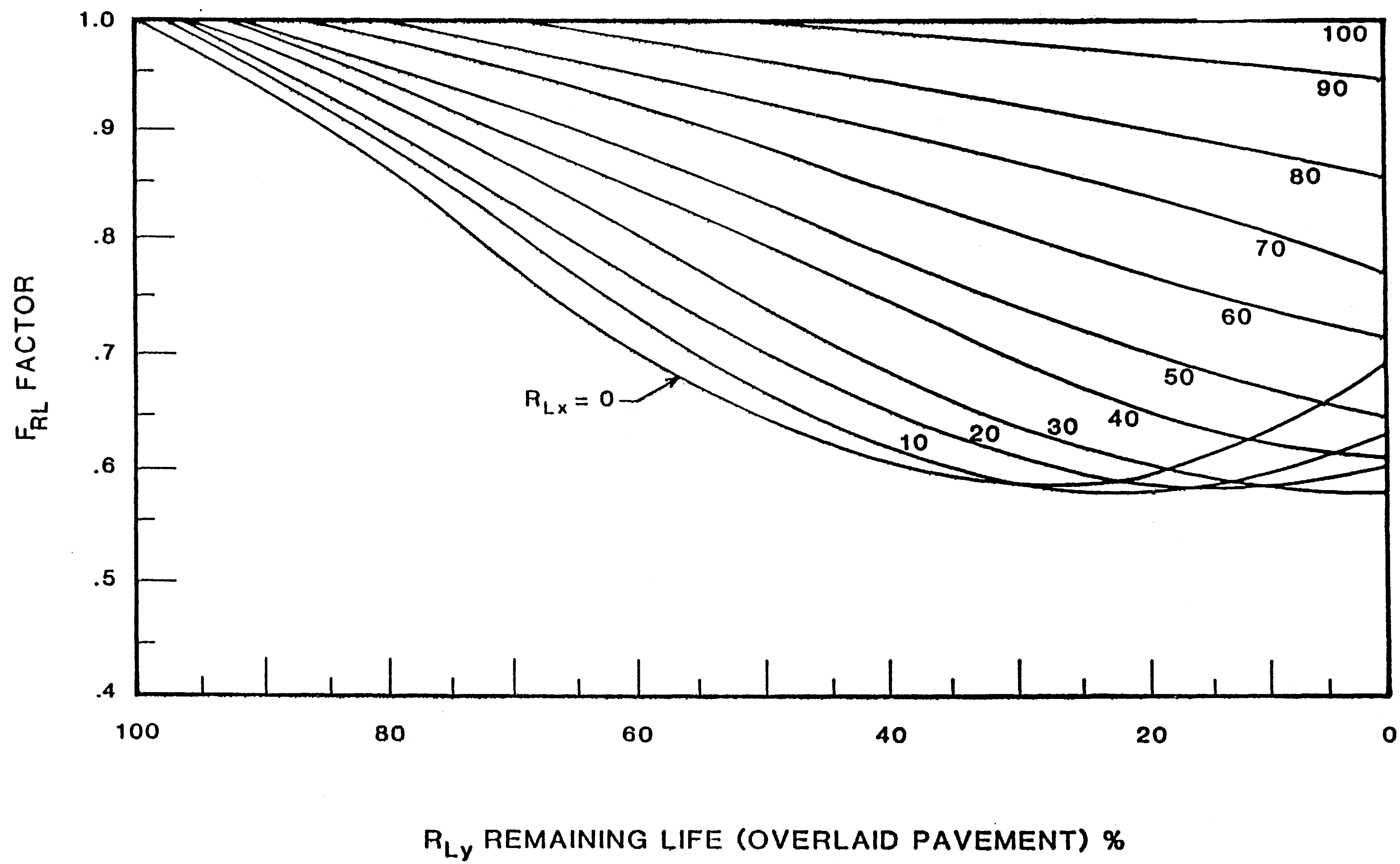


Figure 5-4.9. Determination of Remaining Life Factor, F_{RL} (1).

Table 5-4.2. Summary of Overlay Equations Used in Flexible Overlays over Existing Rigid Pavements.

Type Overlay	Specific Method Used	SN_{ol} Equation
Normal Structural Overlay:		
	NDT Method	$SN_{ol}=SN_y-F_{RL}(0.8D_{xeff}+SN_{xeff-rp})$
	Visual Condition Factor Method	$SN_{ol}=SN_y-F_{RL}(a_{2r}D_o+SN_{xeff-rp})$
Break-Seat Overlay:		
	Estimating Nominal Crack Spacing Method	$SN_{ol}=SN_y-0.7(0.4D_o+SN_{xeff-rp})$*
	Post Cracking NDT Method	$SN_{ol}=SN_y-0.7(a_{bs}D_o+SN_{xeff-rp})$

Special Note: The coefficient on D_o(i.e.,0.8, 0.4, etc.) actually varies from 0.35 for a nominal crack spacing of approximately 2.0 ft. to a value of 0.45 for a nominal crack spacing of approximately 3.0 ft.

$SN_{xeff\text{-}rp}$ = The effective (in-place) structural capacity of all remaining pavement layers above the subgrade except for the existing PCC layer.

a_{2r} = The structural layer coefficient of the existing cracked PCC pavement layer. This value is used in a normal structural overlay analysis and has been related to the value of the visual condition factor, C_v.

a_{bs} = The structural layer coefficient of the PCC pavement layer after it has been broken (cracked) during the break-seat approach. This value is related to the in-place (broken) PCC modulus.

5.8.1 Normal AC Structural Overlay Approach

The required thickness of the asphalt overlay is determined from

$$h_{ol} = SNa_{ol},$$

with a_{ol} being the structural layer coefficient of the overlay material.

The general solution methodology follows all of the steps discussed in Section 5.0. Solution Steps (1), (2), (5), and (6) are all considered straightforward in the analysis. The primary difference between the various methods shown in Table 5-4.2 concerns Steps (3) and (4) relative to how the material properties and the Effective Structural Number of the existing pavement is considered. These parameters may be evaluated in three ways:

1. NDT Method. In this approach, the prediction of all in-place layer properties (E_i) is determined from the backcalculation technique using the measured NDT deflection basin. This is the preferred approach.

2. Visual Condition Factor. If it is impossible to utilize NDT, estimates of the visual condition factor, C_v, can be used to obtain the effective structural number of the cracked PCC layer.

After using one of the approaches noted, the structural overlay thickness, h_{o1}, determined in this procedure is called the normal structural overlay thickness. However, this value must now be evaluated in consideration of the recommended reflective cracking techniques and adjusted as needed.

5.8.2 Use of Thick Overlays to Minimize Reflective Cracking

Reflective cracking of asphalt overlays over existing rigid pavements is a complex phenomena, as described in Module 5-1. These cracks may be caused by both differential vertical pavement movement under traffic and horizontal slab movements caused by temperature and moisture changes. For horizontal movement, the influence of the existing PCC slab length, as well as the maximum annual temperature differential is important.

Since reflection cracking from the existing rigid pavement has caused many AC overlays to fail prematurely, it is important to consider some means for control. One procedure recommended by AASHTO is to utilize the minimum overlay thicknesses shown in Table 5-4.3 which are a function of the existing PCC slab length and maximum temperature difference expected within a year. The normal structural overlay approach should be used as a final design thickness if it is thicker than this minimum. A major deficiency of this table, however, is the lack of consideration of traffic loadings and vertical load transfer. However, if the AC overlay thickness shown in Table 5-4.3 is much thicker than the normal AC overlay thickness, some other means of reflection crack control may be much more cost-effective as described in Module 5-1.

5.8.3 Break and Seat Approach to Minimize Reflective Cracking

A rehabilitation technique being used to reduce the problem of reflective cracking is the break and seat (crack and seat) approach. This technique uses special slab fracturing equipment to break the slab into nominal pieces 24 to 42 inches in size. Then a heavy roller is used to ensure the slabs are firmly "seated" before the asphalt overlay is placed. Because the effective slab length is greatly reduced, reflective cracking may be reduced. However, a recent evaluation of many such projects by the FHWA has led to the conclusion that "there generally is a reduction in the amount of reflective cracks through the overlay during the first few years following construction of a C&S project. However, after 4 to 5 years the C&S sections exhibited approximately the same amount of reflective cracks as the control sections." (16). The only projects that showed reduced reflection cracking had cement treated bases, small changes in seasonal temperatures and were non-reinforced jointed concrete pavements (16).

When the break and seat technique is used, the design of the asphalt overlay follows the equations noted in Table 5-4.2. Because the broken pavement is transformed into a common state of "damage", the F_{RL} is relatively constant for all R_{Ly} values. Thus, step (6) in the overlay design process is not necessary, and a value of 0.7 is used (1). In addition, it is only necessary to determine the "y" (future traffic) in step (2), since information regarding the "x" (previous traffic) is meaningless.

Two alternatives may be used to design asphalt structural overlay for a cracked and seated pavement using the AASHTO procedure. The first involves a design assumption regarding the nominal size of the slab fragments after the breaking has occurred. The other approach is a post-cracking design using NDT to determine the actual in-situ properties of the broken pavement.

1. Estimating the Nominal Crack Spacing. This design method assumes a nominal slab fragment size of approximately 30 inches will be obtained in the breaking phase. The effective (in-situ) structural number of the broken slab is given by 0.40 x D_o (original slab thickness). The information obtained from NDT is then used to determine the in-situ modulus values of the existing pavement layers. Normally the NDT deflection basin study is performed before the breaking process. Modulus values for all pavement layers, except the existing PCC layer, are used. The $SN_{xeff-rp}$ value can be determined using the modulus values backcalculated from NDT and the structural layer coefficient (Module 5-3).

Table 5-4.3. Minimum asphalt concrete structural overlay thickness for PCC Pavements (from the Asphalt Institute MS-17 (1).

Existing PCC Slab Length (ft.)	h (min - inches) Maximum Annual Temperature Differential (^{o}F)					
	30	40	50	60	70	80
10	4	4	4	4	4	4
15	4	4	4	4	4	4
20	4	4	4	4	5	5.5
25	4	4	4	5	6	7
30	4	4	5	6	7	8
35	4	4.5	6	7	8.5	*
40	4	5.5	7	8	*	*
45	4.5	6	7.5	9	*	*
50	5	7	8.5	*	*	*
60	6	8	*	*	*	*

* Alternate other than thickness of AC overlay should definitely be considered to minimize reflective cracking.

2. Post Cracking Design. In some cases, it may be desirable to perform NDT after the breaking operation has occurred. After the modulus values of all of the layers (including the broken PCC surface) have been determined through backcalculation procedures, SN_{eff} can be determined. Figure 5-4.10 can be used to determine the in-situ structural layer coefficient, a_{bs}, of the broken layer knowing the modulus of that layer. The product of a_{bs} and D_o yields the effective structural number of the broken PCC slab. The SN_{eff-rp} value can be determined from the modulus values and structural coefficients of the remaining layers as shown in Module 5-3.

6.0 DESIGN OF ASPHALT CONCRETE OVERLAYS OVER RIGID PAVEMENTS USING OTHER METHODS

There are several methods available to the design engineer for the design of flexible overlays on rigid pavements, such as the Asphalt Institute, Illinois Department of Transportation, Mississippi DOT, etc. Mechanistic-empirical approaches have also been developed, such as, POD (by FHWA/Austin Research Engineers) and OAR (by FHWA/Resource International) (12).

In the design of an overlay, or any pavement structure, it is essential that the design thickness is checked to assure adequate design. Any of the above and other methods could be used as a "check" on the AASHTO method. The "checking" of an overlay design is considered very important.

6.1 Asphalt INstitute Method

The Asphalt Institute method for the design of asphalt concrete overlays on Portland cement concrete pavements uses a "component analysis." Portland cement concrete structural components must be evaluated so that a representative effective thickness can be assigned and used in assessing current adequacy. The effective thickness (T_e) of an existing pavement must be converted to an equivalent thickness of asphalt concrete using the proper conversion factor (See Table 5-4.4).

In the design of overlays of rigid pavements several design features must be considered, such as joints and cracks, unstable slabs, and broken slabs. The concrete pavement should be examined to determine its condition, thickness, and support. Among the signals of distress to look for are pumping, cracking, spalling, faulting of joints, and slab movement under traffic. Also, if the pavement is to be undersealed or broken and seated prior to overlay, this should be considered in selecting the conversion factors from Table 5-4.4.

Evaluating the condition of the concrete pavement layers is a largely subjective determination and its effectiveness, to considerable degree, depends upon the experience of the observer (4).

6.1.1 Overlay Design by Component Analysis

The following steps outline the procedure for designing asphalt concrete overlays of rigid payments based on component analysis of the existing pavement prior to the overlay. The recommended minimum thicknesses

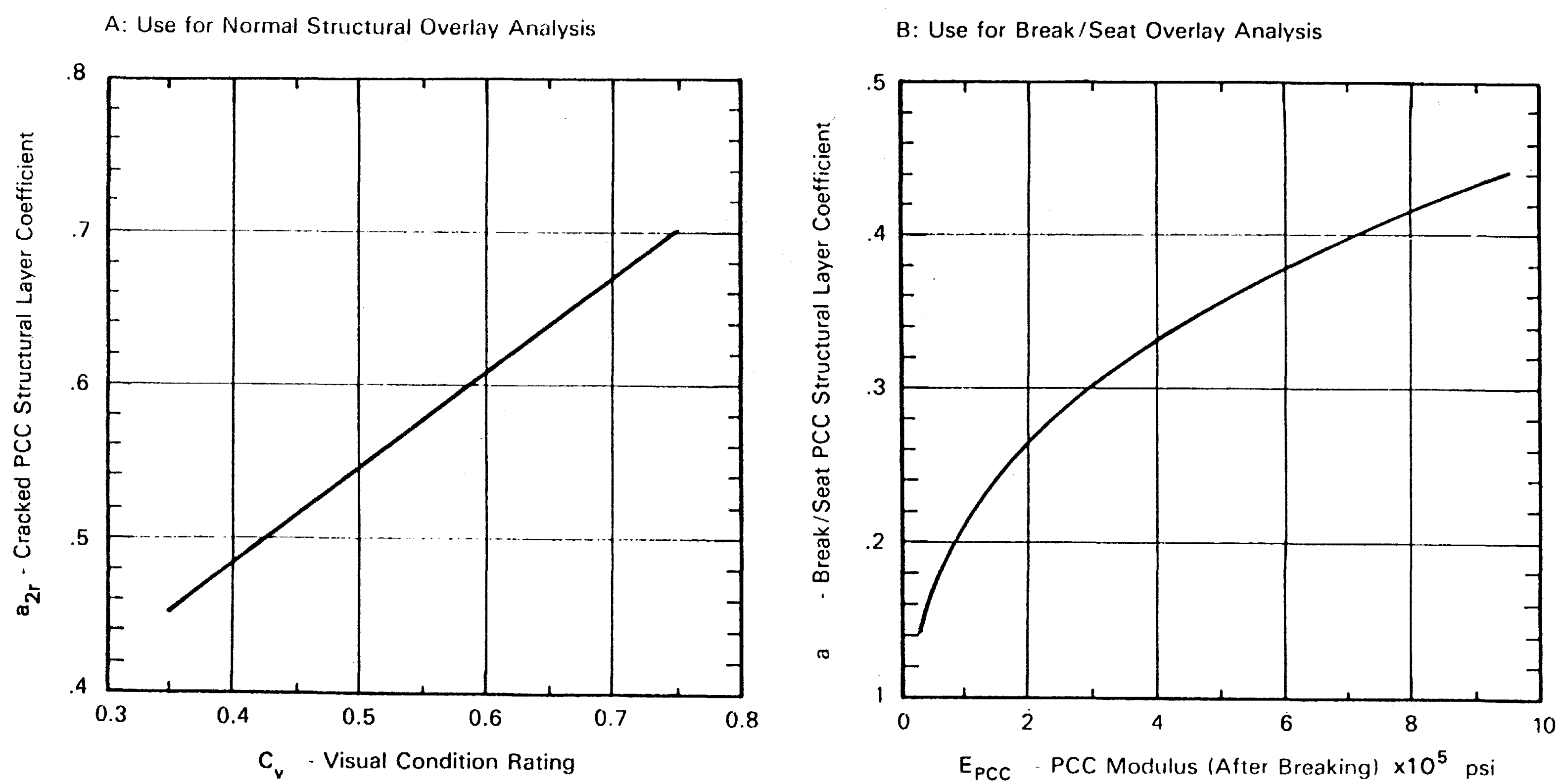


Figure 5-4.10. Structural Layer Coefficients for PCC Layer Used With AC Overlays (Normal Structural and Break/Seat Conditions) (1).

Table 5-4.4. Conversion Factors for Converting Thickness of Existing Pavement Components to Effective Thickness (T_e) (4).

(These conversion factors apply ONLY to pavement evaluation for overlay design. In no case are they applicable to original thickness design.)

Classification of Material	Description of Material	Conversion Factors*
I	a) Native subgrade in all cases	0.0
	b) Improved Subgrade**—predominantly granular materials—may contain some silt and clay but have P.I. of 10 or less	
	c) Lime modified subgrade constructed from high plasticity soils—P.I. greater than 10.	
II	Granular Subbase or Base—Reasonably well-graded, hard aggregates with some plastic fines and CBR not less than 20. Use upper part of range if P.I. is 6 or less; lower part of range if P.I. is more than 6.	0.1-0.2
III	Cement or lime-fly ash stabilized subbases and bases** constructed from low plasticity soils—P.I. of 10 or less.	0.2-0.3
IV	a) Emulsified or cutback asphalt surfaces and bases that show extensive cracking, considerable raveling or aggregate degradation, appreciable deformation in the wheel paths, and lack of stability.	0.3-0.5
	b) Portland cement concrete pavements, (including those under asphalt surfaces) that have been broken into small pieces 0.6 metre (2 ft) or less in maximum dimension, prior to overlay construction. Use upper part of range when subbase is present; lower part of range when slab is on subgrade.	
	c) Cement or lime-fly ash stabilized bases** that have developed pattern cracking, as shown by reflected surface cracks. Use upper part of range when cracks are narrow and tight; lower part of range with wide cracks, pumping or evidence of instability.	

*Values and ranges of Conversion Factors are multiplying factors for conversion of thickness of existing structural layers to equivalent thickness of asphalt concrete.

**Originally meeting minimum strengths and compaction requirements specified by most state highway departments (See Article 1.02 DEFINITIONS).

Table 5-4.4. Conversion Factors for Converting Thickness of Existing Pavement Components to Effective Thickness (T_e) (continued).

Classification of Material	Description of Material	Conversion Factors*
V	a) Asphalt concrete surface and base that exhibit appreciable cracking and crack patterns.	0.5-0.7
	b) Emulsified or cutback asphalt surface and bases that exhibit some fine cracking, some raveling or aggregate degradation, and slight deformation in the wheel paths but remain stable.	
	c) Appreciably cracked and faulted portland cement concrete pavement (including such under asphalt surfaces) that cannot be effectively undersealed. Slab fragments, ranging in size from approximately one to four square metres (yards), and have been well-seated on the subgrade by heavy pneumatic-tired rolling.	
VI	a) Asphalt concrete surfaces and bases that exhibit some fine cracking, have small intermittent cracking patterns and slight deformation in the wheel paths but remain stable.	0.7-0.9
	b) Emulsified or cutback asphalt surface and bases that are stable, generally uncracked, show no bleeding, and exhibit little deformation in the wheel paths.	
	c) Portland cement concrete pavements (including such under asphalt surfaces) that are stable and undersealed, have some cracking but contain no pieces smaller than about one square metre (yard).	
VII	a) Asphalt concrete, including asphalt concrete base, generally uncracked, and with little deformation in the wheel paths.	0.9-1.0
	b) Portland cement concrete pavement that is stable, undersealed and generally uncracked.	
	c) Portland cement concrete base, under asphalt surface, that is stable, non-pumping and exhibits little reflected surface cracking.	

for a structural asphalt overlay on a concrete pavement are given in Table 5-4.3.

1. Determine the subgrade strength condition, (M_r). When original design records are available, some limited testing should be performed to analyze that state of the subgrade. When original records are not available, subgrade strength must be established through soil sampling. To avoid biasing the results, random sample locations should be selected in each soil type encountered. The subgrade soil samples are tested in the laboratory to determine their strength values using the resilient modulus test procedure.

2. Determine the design traffic ($ESAL_D$) using the procedures shown in Module 2-6.

3. Use the full-depth asphalt concrete design chart (Figure 5-4.11) to determine the thickness of a new pavement, T_n, required for the expected traffic and the subgrade conditions (M_r).

4. The thickness of asphalt concrete overlay, T_o, required is equal to $T_n - T_e$.

7.0 JOINTED CONCRETE OVERLAYS OVER RIGID PAVEMENTS

PCC overlays of existing PCC pavements can be either jointed plain, jointed reinforced or continuously reinforced. Continuously reinforced overlays musts be unbonded, while jointed PCC overlays can be either unbonded, partially bonded, or fully bonded. A few bonded concrete overlays have been placed directly on existing CRCP and have performed well (14). A summary of the performance of existing concrete overlays is presented in References 5, 8, 9, 10, 11, 13 and 14. The AASHTO and PCA design procedures are presented in detail. Other procedures are available (11,12,13).

8.0 DESIGN OF CONCRETE OVERLAYS OVER RIGID PAVEMENTS USING THE AASHTO PROCEDURE

8.1 Development of Design Input Factors

The seven steps in the overlay design procedure were discussed in detail in Section 5.0 for AC overlays.

1. Analysis Unit Delineation
2. Traffic Analysis
3. Materials and Environmental Study
4. Effective Structural Capacity Analysis (SC_{xeff})
5. Future Overlay Structural Capacity Analysis (SC_y)
6. Remaining Life Factor Determination (F_{RL})
7. Overlay Design Analysis

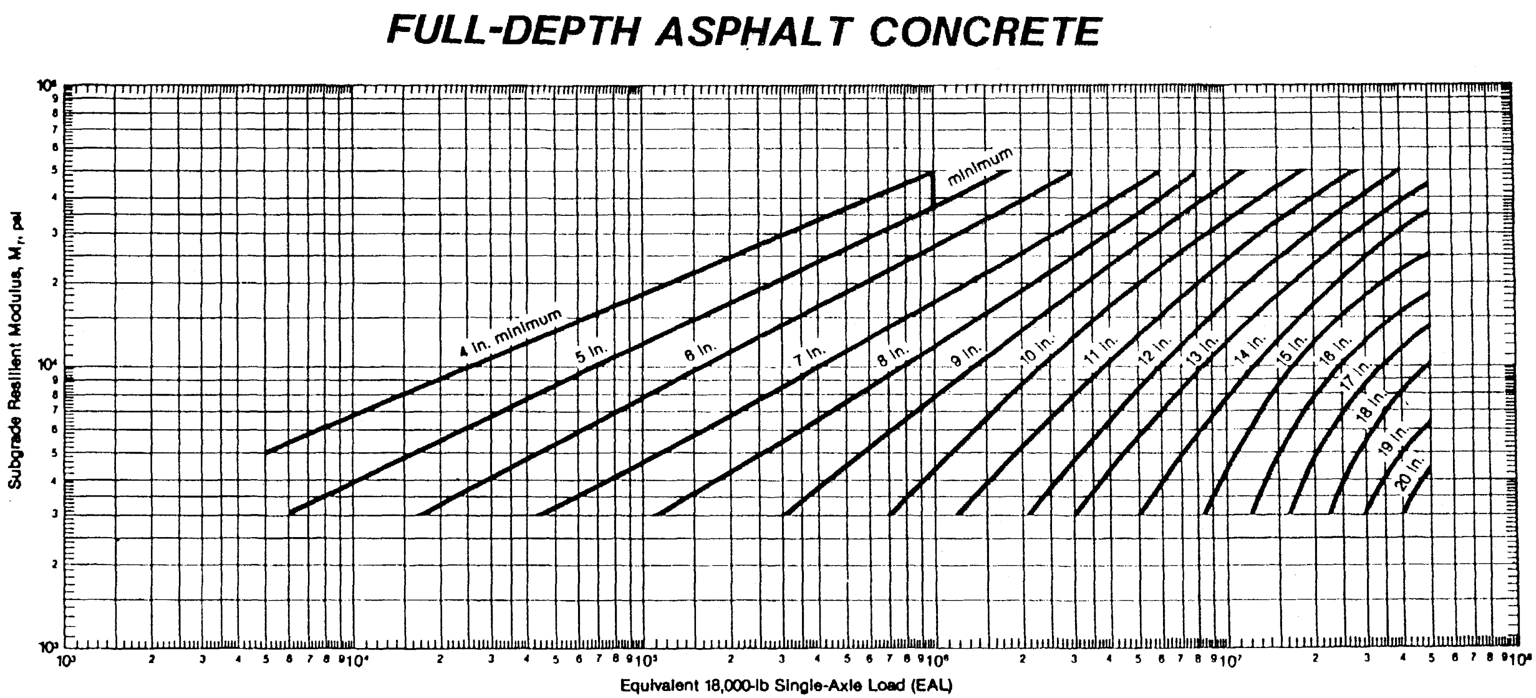


Figure 5-4.11. Design Chart for Full-Depth Asphalt Concrete (4).

While the design concepts are the same, there are some differences for concrete overlays, which will be discussed in Section 8.0.

8.2 Overlay Methodology for Concrete Overlays Over Rigid Pavements

Three potential types of rigid overlays may be considered: full bond, partial bond, and unbonded. The remaining life and structural condition of the existing pavement play a very important role in determining the relative applicability and cost-effectiveness of a particular PCC overlay type.

The six steps leading to the structural overlay analysis are identified in Section 8.1. Figure 5-4.12 (1) summarizes the concrete overlay design procedures, some inputs and the conditions of use for the different jointed concrete overlay conditions. Some application constraints and limitations for the use of concrete overlays are discussed below:

1. Joints in the existing pavement must be matched in location and type with joints in the PCC overlay. This is critical for fully and partially bonded PCC overlays. When unbonded PCC overlays of significant thickness are used, it is not necessary to match the joints, but it is still good practice (12).

2. Working cracks in the existing pavement will to reflect through fully and partially bonded concrete overlays. If the partially bonded overlay is of significant thickness, the seriousness of the reflective crack can be mitigated by using reinforcing steel in the overlay across the cracks in the existing pavement (11,12,14).

3. It is recommended that severely distressed slabs be replaced before overlaying. A thinner overlay will be required and the incidence of reflective cracking will be reduced (14).

4. Rocking slabs which are pumping and faulted must be stabilized prior to overlaying.

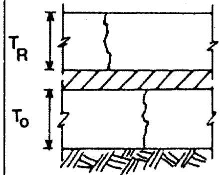
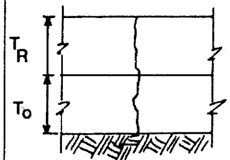
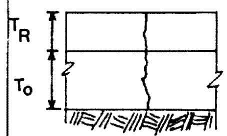
5. To determine the thickness of rigid pavement required for a new design, the drainage coefficient must be determined. This is done based on the conditions of the existing pavement structural section.

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	Water will not drain

Table 5-4.5 shows the recommended drainage coefficients (C_d) which depend on the quality of drainage and the time the pavement structure is exposed to moisture levels approaching saturation.

The design equation for determining the required PCC overlay thickness over an existing rigid pavement is shown below:

$$(D_{OL})^n = [(D_y)^n - F_{RL}(D_{xeff})]^{1/n}$$

	UNBONDED OR SEPARATED OVERLAY	PARTIALLY BONDED OR DIRECT OVERLAY	BONDED OR MONOLITHIC OVERLAY
TYPE OF OVERLAY			
PROCEDURE	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL PLACE SEPARATION COURSE-PLACE OVERLAY CONCRETE	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL AND REMOVE EXCESSIVE OIL AND RUBBER-PLACE OVERLAY CONCRETE	SCARIFY ALL LOOSE CONCRETE, CLEAN JOINT, CLEAN AND ACID ETCH SURFACE-PLACE BONDING GROUT AND OVERLAY CONCRETE
MATCHING OF JOINTS IN OVERLAY PAVEMENT — LOCATION	NOT NECESSARY	REQUIRED	REQUIRED
MATCHING OF JOINTS IN OVERLAY PAVEMENT — TYPE	NOT NECESSARY	NOT NECESSARY	REQUIRED
REFLECTION OF UNDERLYING CRACKS TO BE EXPECTED	NOT NORMALLY	USUALLY	YES
REQUIREMENT FOR STEEL REINFORCEMENT	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT. OR CONDITION OF EXISTING PAVEMENT	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT. STEEL MAY BE USED TO CONTROL CRACKING WHICH MAY BE CAUSED BY LIMITED NON-STRUCTURAL DEFECTS IN PAVEMENT	NORMALLY NOT USED IN THIN OVERLAYS. IN THICKER OVERLAY STEEL MAY BE USED TO SUPPLEMENT STEEL IN EXISTING PAVEMENT.
T_R SHOULD BE BASED ON THE FLEXURAL STRENGTH OF	OVERLAY CONCRETE	OVERLAY CONCRETE	EXISTING CONCRETE
MINIMUM THICKNESS	6"	5"	1"
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVEMENT — NO STRUCTURAL DEFECTS C=1.0 *	YES	YES	YES
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVEMENT — LIMITED STRUCTURAL DEFECTS C=0.75 *	YES	ONLY IF DEFECTS CAN BE REPAIRED	ONLY IF DEFECTS CAN BE REPAIRED
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVEMENT — SEVERE STRUCTURAL DEFECTS C ≤0.35 *	YES	NO	NO
APPLICABILITY OF VARIOUS OVERLAY TYPES — SURFACE CRACKS SCALLING, SPALLING AND SHRINKAGE CRACKS — NEGLIGIBLE	YES	YES	YES
APPLICABILITY OF VARIOUS OVERLAY TYPES — SURFACE CRACKS SCALLING, SPALLING AND SHRINKAGE CRACKS — LIMITED	YES	YES	YES
APPLICABILITY OF VARIOUS OVERLAY TYPES — SURFACE CRACKS SCALLING, SPALLING AND SHRINKAGE CRACKS — EXTENSIVE	YES	NO	YES

* C VALUES APPLY TO STRUCTURAL CONDITION ONLY AND SHOULD NOT BE INFLUENCED BY SURFACE DEFECTS

Figure 5-4.12. Summary of Concrete Overlay on Existing Pavements (1).

Table 5-4.5. Recommended Values of Drainage Coefficient, C_d, For Rigid Pavement Design (1).

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1 - 5%	5 - 25%	Greater Than 25%
Excellent	1.25 - 1.20	1.20 - 1.15	1.15 - 1.10	1.10
Good	1.20 - 1.15	1.15 - 1.10	1.10 - 1.00	1.00
Fair	1.15 - 1.10	1.10 - 1.00	1.00 - 0.90	0.90
Poor	1.10 - 1.00	1.00 - 0.90	0.90 - 0.80	0.80
Very Poor	1.00 - 0.90	0.90 - 0.80	0.80 - 0.70	0.70

where:

D_{OL} = Required thickness of the PCC overlay, inches
D_Y = Design thickness for a new PCC slab for the specified traffic and existing subgrade conditions, inches
F_{RL} = Remaining life factor
D_{xeff} = Effective thickness of existing PCC slab, inches
n = 1.0 - Fully Bonded Overlay,
1.4 - Partially Bonded Overlay,
2.0 - Fully Bonded Overlay.

A detailed example is provided in Section 11 showing the design of a concrete overlay.

8.2.1 Fully Bonded Concrete Overlay

To achieve a fully bonded PCC overlay it is necessary to carefully prepare the surface of the existing pavement before placing the overlay. This preparation should include removal of all oil, grease, and surface contaminants, all paint, and all unsound concrete. This can be done by cold milling, shot blasting, sand blasting and other techniques (12).

In addition to cleaning the surface, a thin layer of cement grout should be placed on the cleaned dry surface just in front of the concrete paver. The grout can be broomed or applied with a high-pressure sprayer.

The grout must not be allowed to dry or set prior to placement of the concrete for the overlay. Also, the grouts should be applied only to completely dry pavement surfaces (12,19). Research is underway in Texas and Iowa to determine the necessity of grouting.

Field and laboratory tests should be conducted to ensure that the bonding techniques specified will provide a good bond. It may be desirable to remove a portion of a slab from the field for testing in the laboratory prior to overlay. The slab portion should be cleaned and overlaid in the lab and then cored and the bond determined through direct shear testing. Bonded overlays should not be placed during the times of high temperature changes (e.g. early spring and late fall) or it may experience debonding problems early in its life (14,19).

Fully bonded PCC overlays should be used only when the existing pavement is in good condition or where the serious distress has been repaired (11,14,19).

Joints in the overlay should be sawed directly above all of the joints in the existing slab (including full depth repair joints and longitudinal joints) as soon as possible. Any delay in sawing will result in cracks forming near the joint which will ultimately results in spalling and deterioration of the joint. The joint should be cut completely through the overlay to avoid secondary cracking. Pressure relief joints are normally not required except at fixed structures. Reference 18 contains guidelines on the use of pressure relief joints.

A minimum overlay thickness of 3 inches is recommended for construction purposes (12), and for keeping horizontal shear stresses during curing to a minimum. Curing is absolutely critical to proper bonding of the overlay; thus, careful control of the curing methods should always be performed (14,19).

Use of dowel bars in bonded and partially bonded overlays is not recommended, because they may produce localized failures in the overlay directly above the dowels. They may also cause the overlay to debond.

8.2.2 Partially Bonded Concrete Overlays

Partially bonded PCC overlays result whenever fresh concrete is placed directly on relatively sound, clean concrete slabs. Unless steps are taken to prevent bond, it is usually assumed some degree of bond will be achieved between the overlay and the existing pavement, so the overlay is designed slightly thinner to take advantage of the resulting stress reductions. Several of these overlays have been placed on highways and airports with success (12). There exists some questions as to the reliability of this design due to the fact that a full bond is not achieved. It is preferable to use either a fully bonded or unbonded overlay.

It is very important to keep the joint spacing of a partially bonded overlay as short as possible, due to the stiff underlying PCC slab. Joint spacing should not exceed 1.5 to 1.75 times the slab thickness for a JPCP overlay (e.g., a 6 inch overlay should have a joint spacing not greater than 10.5 feet).

8.2.3 Unbonded Concrete Overlays

Unbonded overlays are typically placed over pavements that are badly cracked to prevent the reflection of cracks and joints through the overlay. Bonding must be prevented through the use of a bondbreaking material. A typical bondbreaker layer consists of AC of less one to two inches (19).

Joints do not need to be matched to joints in the underlying pavement. This is a major advantage over the bonded and partially bonded overlays. Dowel bars must be placed in unbonded JRCP overlays, even if the joints are not matched. For JPCP unbonded overlays, results from a recent survey showed there was little faulting for undowelled joints.

9.0 DESIGN OF CONCRETE OVERLAYS OVER RIGID PAVEMENTS USING OTHER DESIGN METHODS

There are several design methods available to the design engineer for design of rigid overlays over rigid pavements. The Corps of Engineers, Concrete Reinforced Steel Institute (CRSI), American Concrete Institute (ACI) and AASHTO methods are all very similar in that they assign a "C" value to the existing pavement based on a visual condition survey. There are several mechanistic design procedures that may be used by the design engineer, such as the Portland Cement Association (13) and POD (by FHWA/Austin Research Engineers) (3). The PCA method will be presented herein in some detail.

9.1 PCA Method

New thickness design procedures have recently been developed by the Portland Cement Association for concrete overlays of existing rigid pavement systems (13).

9.1.1 Evaluation of the Existing Pavement

As part of the design process for concrete resurfacings, a comprehensive evaluation of the existing pavement should be made. The evaluation program should consist of the following items.

1. Pavement Condition Survey - As discussed in Module 5-1, the condition survey should identify the type, quantity, and severity of all pavement distresses. The visual survey should be performed for the entire project.

2. Nondestructive Deflection Load Testing - The need for load testing is based on the results of the visual survey. If the condition survey indicates the existence of, or potential for, load related distress, then load testing should be carried out to determine the severity of the problem. Load testing should be conducted at joints and cracks to determine the absolute deflections at slab corners and relative deflections across joints and cracks. Results of load testing can be used to determine if loss of support exists and if load transfer across the joints and cracks is adequate.

   Load testing should be performed with a nondestructive testing device that imparts an 8,000 to 10,000 pound load on the pavement. Use of lighter loads is not recommended for reasons discussed earlier.

3. In-Situ Materials Evaluation - For bonded resurfacing projects, a detailed material testing program is recommended to evaluate the engineering properties of the existing pavement materials. The strength of the subsurface pavement layers is determined through backcalculation procedures similar to those presented in Section 5.3. For the existing concrete pavement, it is necessary to obtain representative values of the flexural strength and the modulus of elasticity. Because it is not usually practical to obtain beam specimens from the pavement, it is recommended that split tensile tests be performed on pavement cores.

   The design flexural strength (f_r) is determined by use of the following equation:

$$f_{te} = f_t - 1.65 * s_f$$

where:

f_{te} = Effective split tensile strength, psi.
f_t = Average value of split tensile tests, psi.
s_f = Standard deviation of the strength values, psi.

The design flexural strength is calculated as follows:

$$f_r = A * B * f_{te}$$

where:

f_r = Design flexural strength, psi.
f_{te} = Effective split tensile strength, psi.
A = Regression constant.
B = Damage factor = 0.90.

The values of A range form 1.35 to 1.55. When available a value of A based on local experience should be used. In the absence of local experience a value of 1.45 should be used (13).

The constant B, equal to 0.90, is used to relate the strength of a concrete specimen obtained about 2 feet away from the outside lane edge to that of concrete at the outside lane edge. It is assumed that concrete at the outside edge experiences higher stresses than concrete away from the edge. Thus, the concrete at the outside lane edge will be more highly fatigued and will exhibit lower strength.

The modulus of elasticity of the existing pavement may be determined by testing concrete cores in accordance with ASTM C469 or may be estimated from the following equation:

$$E_c = D * f_r$$

where:

E_c = Design modulus of elasticity, psi.
D = Constant = 6000 to 7000.
f_r = Design flexural strength, psi.

9.1.2 Development of Design Procedure for Unbonded Resurfacing

Stress data developed using JSLAB, a finite element program (similar to ILLISLAB) developed by Construction Technologies Laboratories for FHWA (13), were used to prepare design design charts for the determination of unbonded resurfacing thickness. These charts are applicable to existing concrete pavements having effective modulus of elasticity values ranging from 3,000,000 to about 4,000,000 psi. Design charts are presented for three cases of existing pavement conditions. These cases are:

Case 1. Existing pavement exhibits a large amount of midslab and corner cracking and poor load transfer at the joints and cracks.

Case 2. Existing pavement exhibits a small amount of midslab and corner cracking. It exhibits reasonably good load transfer at the joints and cracks. Localized repairs were performed to correct distressed slabs.

Case 3. Existing pavement exhibits a small amount of midslab cracking and good load transfer at the cracks and joints. The loss of support was corrected by undersealing.

The design chart for Case 1 was developed using data from analysis of resurfacing sections containing a crack in the existing pavement directly under an edge load on the overlay. The design chart for Case 3 was developed using data from the analysis of resurfacing sections with no cracking in the existing pavement. The design chart of Case 2 was developed through interpolation between Case 1 and Case 3 conditions. The design charts are given in Figures 5-4.13 through 5-4.16.

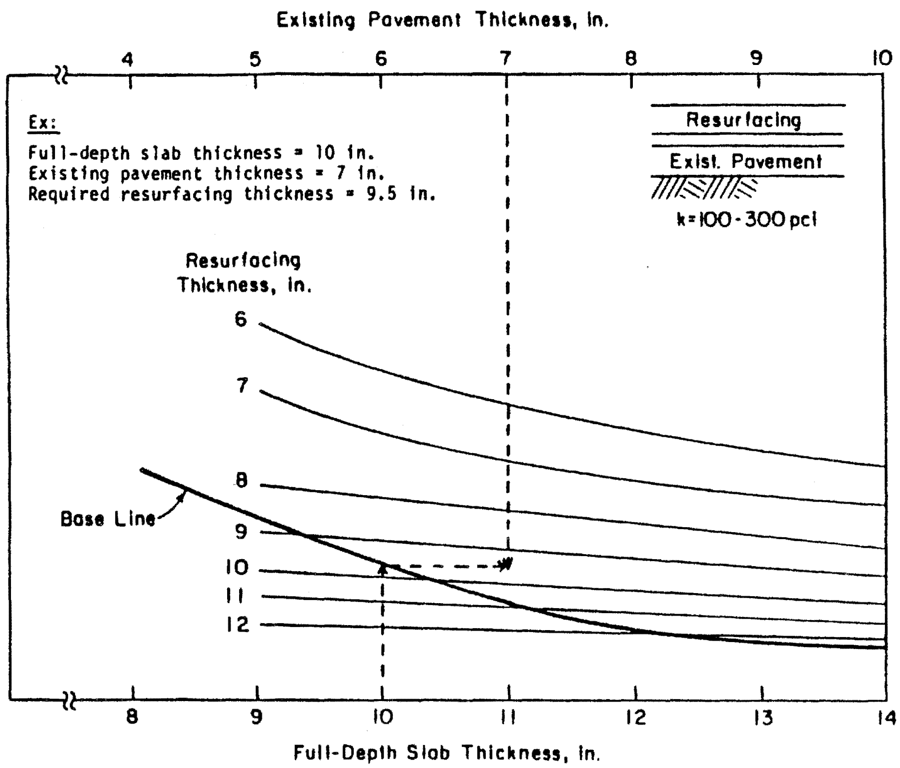


Figure 5-4.13. Design Chart for Case 1 Condition of Existing Pavement (13).

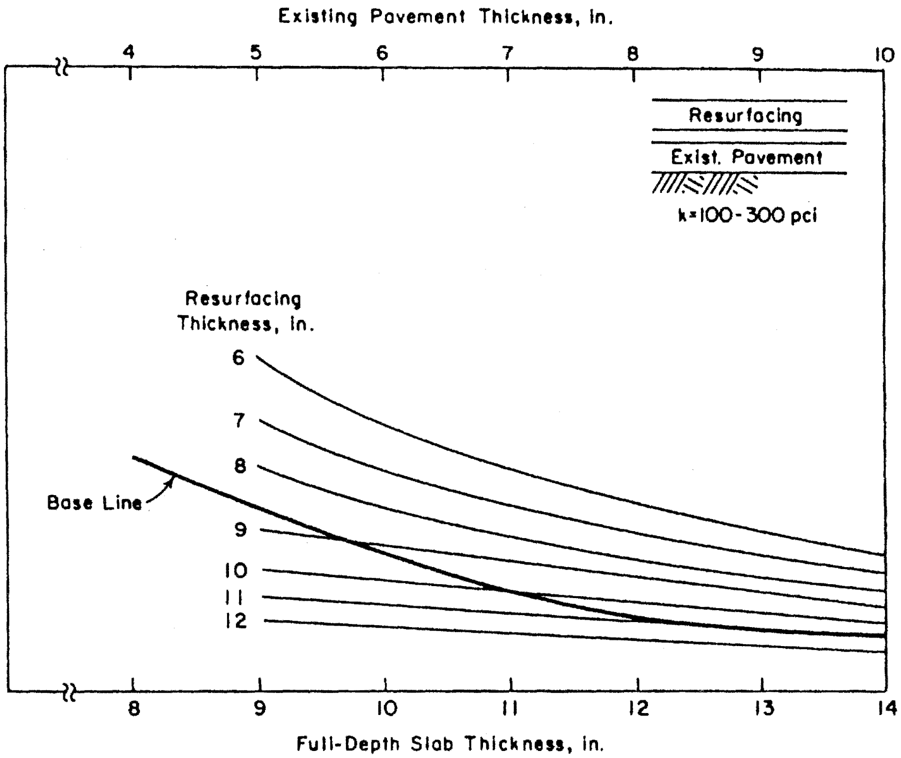


Figure 5-4.14. Design Chart for Case 2 Condition of Existing Pavement (13).

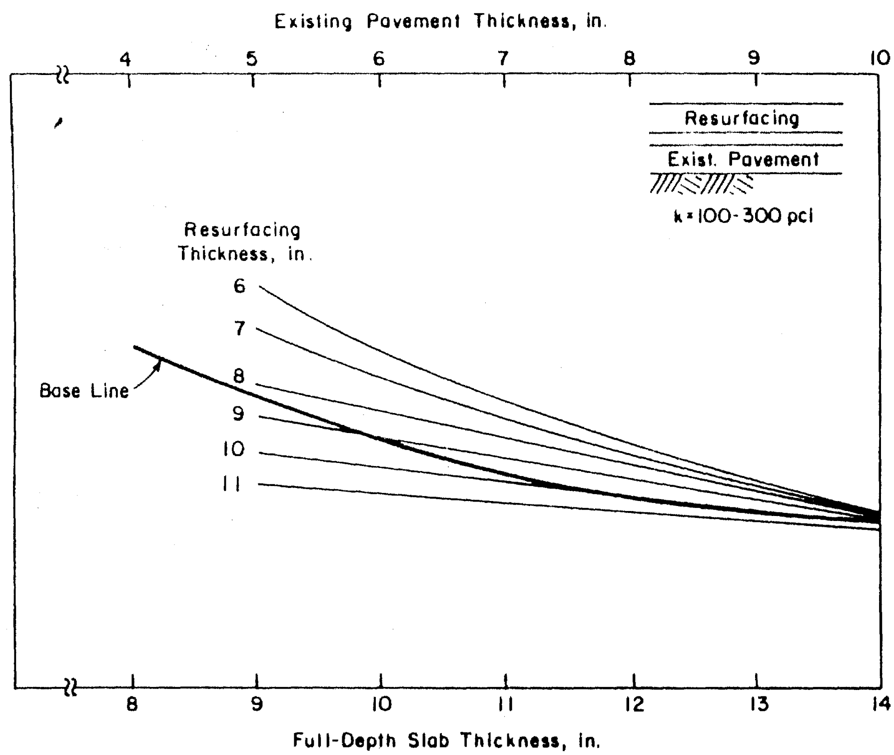


Figure 5-4.15. Design Chart for Case 3 Condition of Existing Pavement (13).

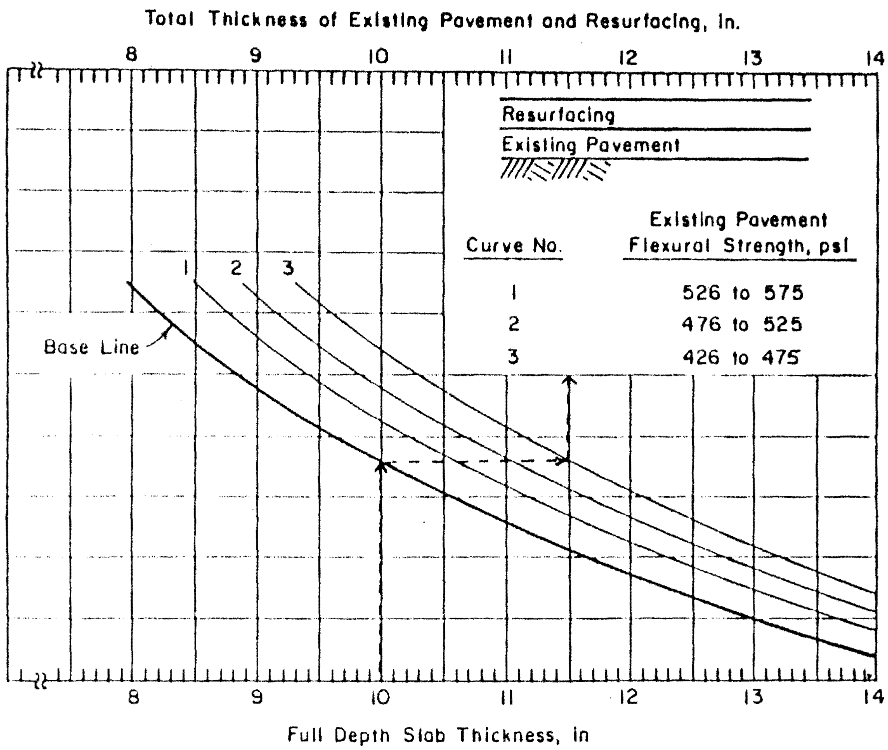


Figure 5-4.16. Design Chart for Bonded Resurfacing (13).

The first step in the design process involves the determination of the thickness of a full-depth concrete pavement for the overlay design traffic. The support condition used to determine this thickness was determined in the subgrade evaluation step. To determine the thickness the following inputs are required:

1. Overlay design traffic.
2. Pavement support condition.
3. Concrete flexural strength.

The full-depth thickness can be determined using the PCA method or AASHTO method of new pavement design (1). Figures 5-4.13, 5-4.14 and 5-4.15 are used to compute the thickness required for resurfacing. Use of the design charts is illustrated in Figure 5-4.13. It should be noted that the minimum thickness on the design charts is 6 inches. Use of a thinner unbonded overlays is not recommended.

Representative values of resurfacing thickness determined from the design charts are summarized in Table 5-4.6. These thicknesses are listed for different values of existing pavement thickness and equivalent full-depth concrete thickness. The resurfacing thickness requirements range from 0.5 to 3 inches less than the full-depth pavement. The determination of the actual resurfacing thickness is influenced by the condition and thickness of the existing pavement.

When a tied shoulder is used in conjunction with resurfacing, then the thickness may be reduced by one inch provided a minimum thickness of 6 inches is used. This is based on the results of a field evaluation of pavement sections with tied shoulder performed in Minnesota (13).

9.1.3 Design Procedures for Bonded Resurfacing

The critical tensile stresses were determined by use of the JSLAB program. The computed tensile stresses were then used to prepare design charts of the determination of bonded resurfacing thickness. These design charts are applicable for the following condition:

1. Modulus of elasticity of full-depth concrete pavement of 4,000,000 to 5,000,000 psi.
2. Flexural strength of new full-depth concrete pavement of 600 psi to 650 psi.
3. Value of constant D of 6000 to 7000 in the following relationship for the existing concrete pavement:

$$E_c = Df_r.$$

Design charts shown in Figure 5-4.16 have been prepared for the following three categories of existing pavement flexural strength:

Table 5-4.6. Representative Values of Resurfacing Thickness (13).

t_n, in.	t_e, in.	Resurfacing Thickness, in		
		Case 1	Case 2	Case 3
8	8	6.8	(6.0)	(6.0)
	7	7.2	6.0	6.0
	6	7.6	6.8	6.0
10	9	9.0	7.0	(6.0)
	8	9.2	7.8	6.0
	7	9.4	8.8	8.0
12	9	11.5	10.4	9.0
	8	11.7	10.8	10.0
	7	11.8	11.2	10.8

Notes: t_n = equivalent full depth new pavement thickness
t_e = existing pavement thickness
Cases 1, 2 and 3 refer to condition of existing pavement described in the text
Values in parentheses indicate minimum thickness requirement of 6 in.

1. 425 to 475 psi

2. 476 to 525 psi

3. 526 to 575 psi

Charts have not been prepared for existing pavement flexural strength of less than 425 psi or greater than 575 psi. For cases when the flexural strength is less than 425 psi, the large thickness requirement may not warrant a bonded resurfacing. For cases where the flexural strength is greater than 575 psi, the overlay thickness should equal the difference between the required full-depth pavement thickness and the existing pavement thickness plus the depth of surface removal.

It should be noted that the maximum bonded surface thickness recommended is 5 inches (13). In addition, use of a bonded resurfacing of less than 2 inches is not recommended for highway pavement strengthening (13).

The first step in the determination of a bonded resurfacing involves the determination of the thickness of a full-depth concrete pavement for the future traffic, as with the unbonded overlay.

The design chart is given in Figure 5-4.16 is used to determine the thickness of the existing pavement plus the bonded resurfacing, to$_{to}$. Use of the design charts is illustrated in Figure 5-4.16. After t_o is determined, the actual as-constructed resurfacing thickness is determined as follows:

$$t_o = t_t - t_e$$

where:

t_o	=	As-constructed bonded overlay thickness.
t_t	=	Total thickness of existing pavement and bonded surface.
t_e	=	Existing pavement thickness after milling.

A detailed overlay design example is provided in Section 11 using the PCA method.

10.0 DESIGN OF CRCP OVERLAYS OVER RIGID PAVEMENTS

At this time there is little published on the design and performance of CRC overlays for concrete pavements. A number of experimental CRC overlays have been constructed. Overall performance has been fairly good (9) with some exceptions. The major problem seems to be the reflection of joints in the existing slab through the CRC overlay where poor load transfer and loss of support exists. The base slab must be stabilized so that low differential deflections exist at the joints. The use of load transfer restoration at the transverse joints may also be recommended. The use of a one- to two-inch AC bondbreaker and level-up is essential.

10.1 Thickness Requirements

Thickness design has largely been based on engineering judgment. The procedures for the design of unbonded jointed concrete overlays could probably be followed with reasonable results. It is not recommended to provide any thinner slab than is required for jointed concrete unbonded overlay. Only the long-term performance of these pavements will determine the validity of this approach.

A comprehensive procedure was published by McCullough and Cawley (6) for the design of CRCP overlays based on theoretical and field performance. The basic thickness design equation is the same equation from the rigid jointed AASHTO Road Test. However, various design charts for subbase and steel design are provided.

10.2 Steel Requirements

The amount of longitudinal steel required for CRC overlays should be that which will keep the transverse cracks tight in the shrinkage and temperature changes in the pavement.

The steel amount is usually expressed as a percentage of the cross-sectional area of the concrete. In general practice, this ranges from a minimum of about 0.5 percent in southern states to a maximum of 0.8 percent in the northern states.

A comprehensive analysis of the quantity of steel for CRC pavements is given in Reference 1. To determine levels of steel reinforcement, limits on acceptable levels of crack spacing, crack width and steel stress are established which minimize distress manifestations. Limiting values are then used to estimate the required percentage of reinforcement which will enable the pavement to satisfactorily perform under anticipated environmental and vehicular loading conditions.

The AASHTO recommendations for reinforcement design may be inadequate due to lack of consideration of the actual friction factor and the use of corrosion protected dowels. The actual friction may be very high between the CRC slab and the interlayer course.

CRCP overlays must be placed on an interlayer course. The steel percentage should be based on the overlay cross-section area only, without regard to the old pavement.

11.0 EXAMPLE PROBLEMS

11.1 AASHTO Design Procedure for the Design of Flexible Overlays Over Rigid Pavements

Step 1. Collect basic information and design criteria

ORIGINAL PAVEMENT

Four lane highway, age = 18 years

Existing pavement: 9.0-inch JRCP
6.0-inch granular subbase
40-foot joint spacing
Load transfer - dowel bars

Roadbed soil: Silty-clay

Major distress: Working transverse cracks
Joint deterioration
Pumping

Drainage: Non-drainable subbase and roadbed soil (i.e., 1 month, and percent time pavement structure is saturated is > 25 percent time). Materials have "poor" drainage characteristics ($C_d = 0.80$).

Existing serviceability index: 2.7

Accumulated 18-kip ESALs: 5×10^6 outer traffic lane

Design period: 20 years

Future overlay design traffic: 10×10^6 outer traffic lane

See Figure 5-4.17 for illustration.

Step 2. Determine the structural number for a new pavement to support the future traffic (SN_y)

Use the AASHTO thickness design nomograph and the following input variables to determine SN_y.

Design reliability: 85%
Overall standard deviation: 0.50

Design 18-kip ESAL over 10 years: 10×10^6

Roadbed soil effective resilient modulus: 8,000 psi

Loss of serviceability:
4.5 (after overlay) - 2.5 (design terminal) = 2.0

SOLUTION: $SN_y = 4.6$

Step 3. Determine the effective SN of the existing pavement (SN_{xeff})

$$SN_{xeff} = C_x * D_{xeff} + SN_{xeff\text{-}rp}$$

where:

SN_{xeff}	=	Total effective structural number of the existing pavement
C_x	=	Structural condition value
D_{xeff}	=	Effective existing slab thickness
$SN_{xeff\text{-}rp}$	=	Effective structural number of the subbase

In order to determine the effective existing slab thickness, the in situ pavement properties are required. Corings and borings through the depth of the pavement and into the roadbed soil were taken along the project. Falling Weight Deflectometer deflection basins were also conducted along the project at 9,000 lbs. load and the deflection basins were analyzed. The moduli of the pavement layers and the roadbed soil were backcalculated using elastic layer concepts. Results of these tests are as follows (all values are averages):

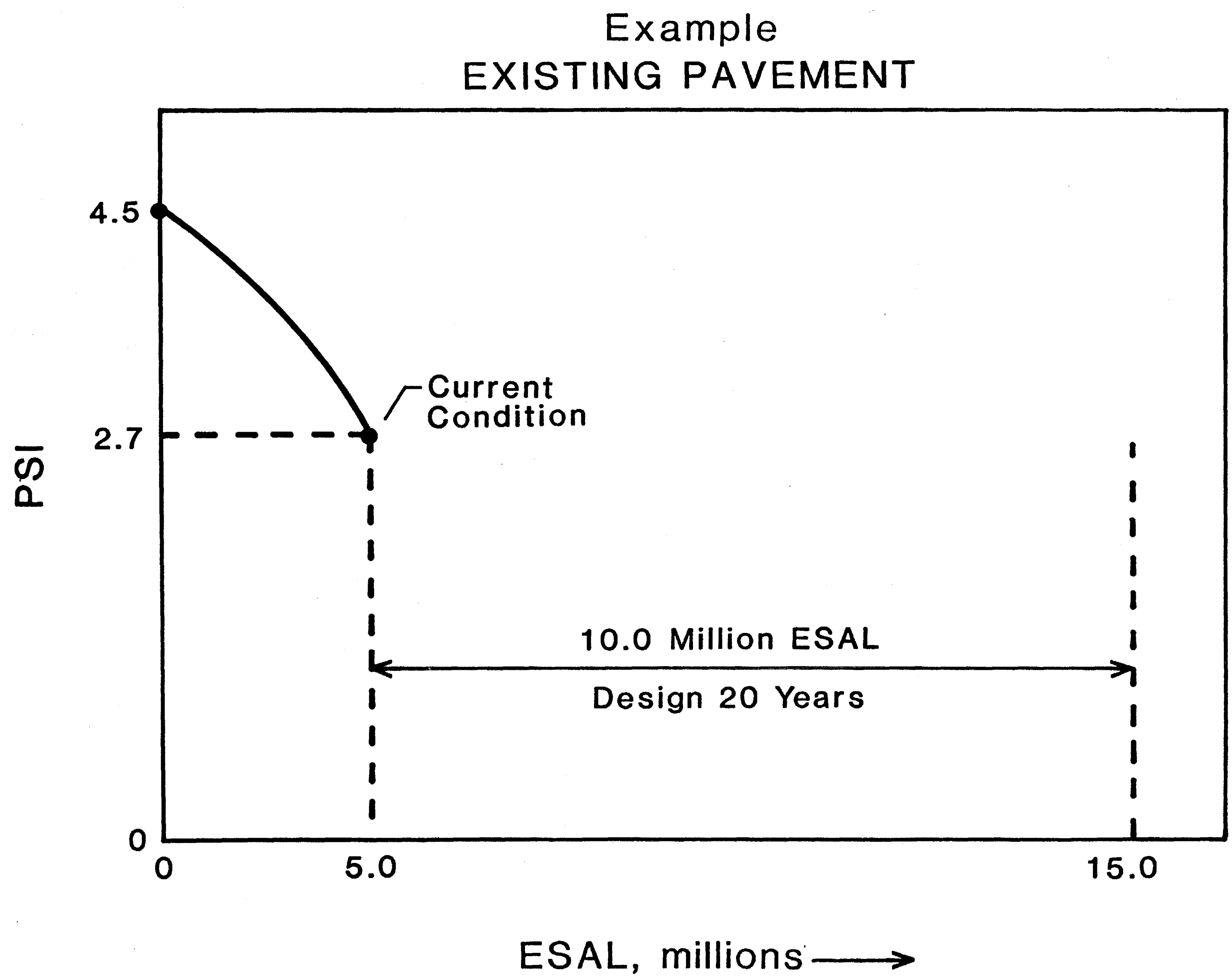


Figure 5-4.17. Existing Condition of Rigid Pavement and Future Design Traffic.

Layer	Core/Bore Thickness	Backcalculated Modulus
PCC	8.8 inches	2.0×10^6 psi *
Granular Subbase	5.8 inches	15,000 psi
Roadbed Soil	--	8,000 psi

* - The E_{pcc} must reflect the amount of cracking in the slabs, thus deflection measurements must be taken near working cracks if they are not to be full depth repaired with doweled joints. This value was backcalculated over several representative working cracks that will not be repaired. The modulus of the concrete was about 5 million psi in non-cracked areas, which indicates a sound concrete with high strength.

The effective existing slab thickness (D_{xeff}) is determined using Figure 5-4.3 with a backcalculated modulus for the PCC surface of 2.0×10^6 and the pavement's existing thickness of 8.8 inches.

SOLUTION: $D_{xeff} = 6.2$ inches

The structural condition value (C_x) is determined using the following equation:

SOLUTION: $C_x = D_{xeff}/D_o = 6.2/8.8 = 0.70$

The structural number of the subbase layer ($SN_{xeff\text{-}rp}$) is calculated using the following equation:

$$SN_{xeff\text{-}rp} = D_{SB} \times a_{SB}$$

where:

D_{SB} = Thickness of the subbase = 6.0 inches
a_{SB} = Structural coefficient for the subbase
= 0.11

SOLUTION: $SN_{xeff\text{-}rp} = 5.8 \times 0.11 = 0.66$

With this information the total effective structural number of the existing pavement (SN_{xeff}) can be calculated:

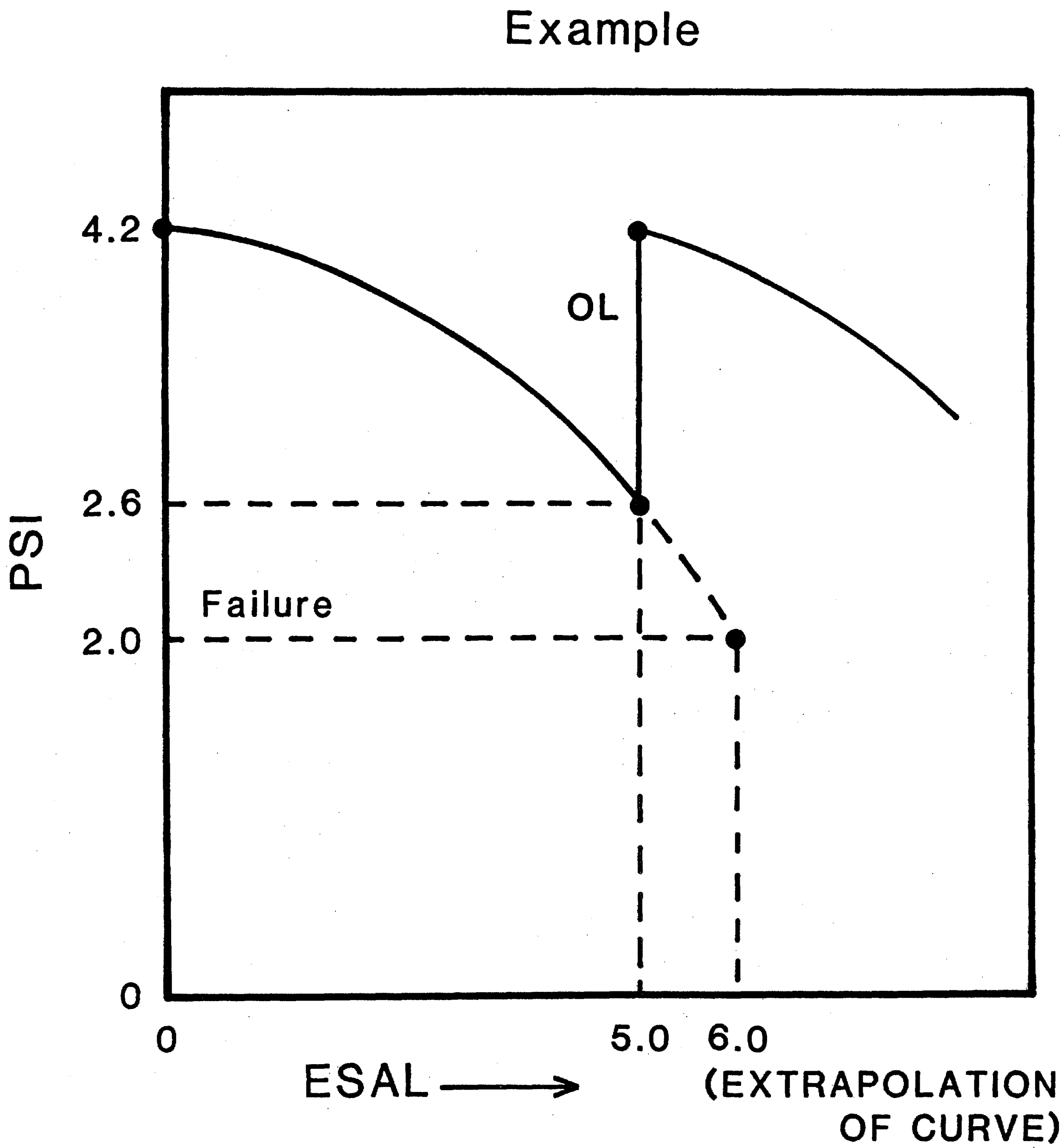
SOLUTION: $SN_{xeff} = 0.70(6.2) + 0.66 = 5.00$

Step 4. Determine the remaining life factor (F_{RL})

The F_{RL} value depends upon:

R_{Lx} = Percent remaining life of existing pavement from a serviceability of 2.7 to 2.0 (0 - 100) = 17

Figure 5-4.18 shows the concepts and calculation of R_{Lx}, percent remaining life of the existing pavement.



$$R_{LX} = \frac{6.0 - 5.0}{6.0} \times 100 = 17$$

Figure 5-4.18. Illustration of Calculation Concepts and Procedures For the Percent Remaining Life of the Existing Pavement (From PSI = 2.6 to 2.0).

R_{Ly} = Percent remaining life of the overlayed pavement from a serviceability of 2.5 to 2.0 (0 - 100) = 17

Figure 5-4.19 shows the concepts and calculation of R_{Ly}, percent remaining life of the overlayed pavement.

The F_{RL} value is then determined using Figure 5-4.9.

SOLUTION: $F_{RL} = 0.58$

Step 5. Computation of final overlay design thickness

$$SN_{OL} = SN_y - F_{RL} \times SN_{xeff}$$

$$= 4.6 - 0.58 \times 5.0 = 1.7$$

$$SN_{OL} = a_{OL} \times D_{OL}$$

The overlay thickness is computed assuming its elastic modulus is 450,000 psi at 68^{o}F. The structural coefficient is 0.44 for asphalt concrete.

SOLUTION: $D_{OL} = 1.7/0.44 = 3.9$ inches

USE: 4 inches AC

Step 6. Reflective Crack Control

The recommended 4 inch AC overlay provides increased structure. It does not prevent reflection cracking which if not controlled, could lead to premature failure of the overlay. A direct method for reflective crack control must be specified. This pavement would be a good candidate for sawing and sealing joints directly over transverse joints and existing working cracks as described in Module 5-1. Longitudinal joints (both centerline and lane shoulder) could be addressed using a fabric-membrane strip placed directly over the joint before the 4 inch AC overlay is placed.

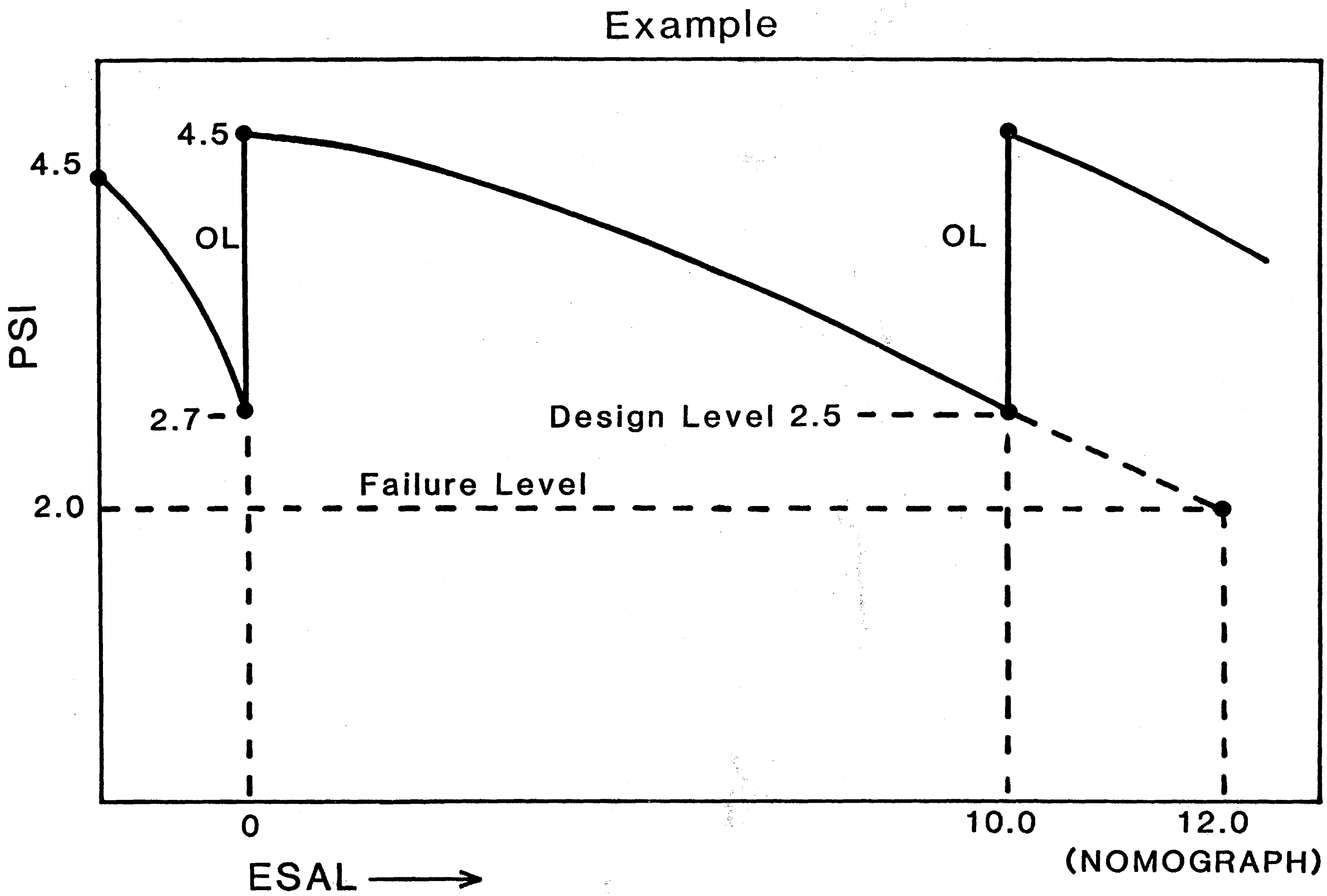
11.2 AASHTO Design Procedure for the Design of Flexible Overlays Over Cracked and Seated Rigid Pavements

As an alternative to the proceeding conventional AC overlay, breaking and seating of the JRCP may be considered. The major problem is to ensure that the steel reinforcement is broken at the cracks, or the cracking and seating will not have much effect.

Step 1. Collect basic information and design criteria

Use the same information that was given in Section 11.1.

The pavement will be broken to a nominal slab fragment size of approximately 30 inches. Construction procedures will ensure that the reinforcement is broken. No post-breaking NDT will be performed.



$$R_{LY} = \frac{12.0 - 10.0}{12.0} \times 100 = 17$$

Figure 5-4.19. Illustration of Calculation and Concepts for the Percent Remaining Life of the Overlayed Pavement (From 2.5 to 2.0).

Step 2. Determine the structural number of the overlay (SN_{OL})

From Table 5-4.2, the appropriate design equation is:

$$SN_{OL} = SN_y - 0.7\,(0.4D_o + SN_{xeff\text{-}rp})$$

where:

SN_{OL} = Structural number of the overlay
SN_y = Structural number of a new pavement to support future traffic = 4.6 (see Section 11.1)
D_o = Existing pavement thickness = 8.8 inches
$SN_{xeff\text{-}rp}$ = Effective thickness of subbase layer = 0.64 (see Section 11.1)

<u>SOLUTION</u>: $SN_{OL} = 4.6 - 0.7\,(0.4 \times 8.8 + 0.64) = 1.7$

Step 3. Required overlay thickness (D_{OL})

$$D_{OL} = SN_{OL}/a_{OL}$$

<u>SOLUTION</u>: $D_{OL} = 1.7/0.44 = 3.9$ inches

<u>USE</u>: 4.0 inches AC

This overlay thickness is the same as that designed over the non-broken and seated JRCP.

11.3 AASHTO Design Procedure for the Design of Rigid Overlays Over Rigid Pavements

Step 1. Collect basic information and design criteria

Use the same information that was given in Section 11.1.

NOTE: Since the existing pavement is in poor condition, the only feasible alternative for a rigid overlay would be an unbonded concrete overlay. To achieve the bond break, a 1.5-inch asphalt leveling course will be placed over the existing rigid pavement.

Step 2. Determine the in situ properties of the pavement layers

The surface layer modulus value and modulus of subgrade reaction were backcalculated using finite element methods. The backcalculated values are shown below:

Surface modulus of elasticity: 2.0×10^6 psi (at cracks)

Effective k value for the subbase/subgrade: 350 pci

Step 3. Determine the effective thickness of the existing concrete slab (D_{xeff})

This value has been previously determined in Step 3 of Section 11.1 as

6.2 inches. This value neglects the influence of the leveling bond breaker course placed on the existing pavement. The leveling course may be accounted for by assuming a 2.5:1 layer substitution value for asphalt to concrete. For the 1.5-inch leveling course, the equivalent PCC thickness is 0.6 inch. Therefore, the total effective PCC thickness (D_{xeff}) is:

$$D_{xeff} = 6.2 + 0.6 = 6.8 \text{ inches}$$

Step 4. Determine the required thickness for new design (D_y)

Using the rigid pavement design nomograph (see Figure 4-2.9) the following input variables, the thickness required for new design is determined:

Design reliability: 85%
Overall standard deviation: 0.50

Design 18-kip ESALs over 10 years: 10×10^6

Loss of Serviceability:
4.5 (after overlay) - 2.5 (terminal) = 2.0

PCC modulus of elasticity (new PCC): 4.0×10^6 psi
Mean PCC modulus of rupture: 700 pci

k value: 350 pci

Load transfer coefficient: 3.2

Drainage coefficient: 0.8 (see Section 11.1)

<u>SOLUTION</u>: $D_y = 10.7$ inches

Step 4. Determine the remaining life factor (F_{RL})

The R_{Lx} value was determined as 17 percent in Section 11.1
The R_{Ly} value is determined as follows:

$$R_{Ly} = (12 \times 10^6 - 10 \times 10^6)/(12 \times 10^6) = 17$$

Note: The 12 million ESAL were determine from Figure 4-2.9 considering a loss of serviceability to 2.0.

<u>SOLUTION</u>: $F_{RL} = 0.58$

Step 5. Computation of final overlay thickness

$$D_{OL}^2 = D^2 - F_{RL}(D_{xeff})^2$$
$$= 10.7^2 - 0.58\,(6.8)^2 = 87.7$$

Therefore,

$$D_{OL} = 9.4 \text{ inches}$$

<u>USE</u>: 9.5 inches unbonded jointed concrete overlay

Step 6. Joint Design

This includes a determination of the need for dowels, joint spacing, and sealant reservoir. These are crucial aspects of the design. Block 4 contains information on this phase of design.

11.4 PCA Method for the Design of Bonded Concrete Overlays

Step 1. Collect basic information and design criteria

<u>ORIGINAL PAVEMENT</u>

JPCP slab thickness: 8.0 inches
Thickness after milling: 7.5 inches

Average core split tensile strength: 430 psi
Standard deviation for split tensile strength: 50 psi

Modulus of subgrade reaction (from backcalculation procedure using 9-kip FWD load): 100 pci (converted from dynamic to static value)

<u>NEW DESIGN</u> (Full-depth concrete parameters)

Thickness: 10 inches.

Design flexural strength: 600 psi.
Design modulus of elasticity of concrete: 4,000,000 psi.

Design modulus of subgrade reaction: 100 pci.

Note: The existing pavement and the properties of the new overlay meet the requirements for a bonded PCC overlay presented in Section 5.2.1.

Step 2. Determine the effective split tensile strength (f_{te})

$$f_{te} = f_t - 1.65 \times s_f$$

where:
f_{te} = Effective split tensile strength
f_t = Average value of split tensile tests
s_f = Standard deviation of tensile tests

<u>SOLUTION</u>: $f_{te} = 430 - 1.65 * 50 = 358$ psi

Step 3. Determine the design flexural strength

$$f_r = A \times B \times f_{te}$$

where:
f_r = Design flexural strength (psi)
A = Regression constant = 1.45
B = Damage factor = 0.90

<u>SOLUTION</u>: $f_r = 1.45 * 0.90 * 358 = 467$ psi

Step 4. Determine the required overlay thickness (t_o)

Using Figure 5-4.16, the thickness of a new pavement to sustain the traffic level (t_t) is:

t_t = 11.5 inches.

The overlay thickness is calculated as follows:

$$t_o = t_t - t_e$$
$$= 11.5 - 7.5$$
$$t_o = 4.0 \text{ inches}$$

SOLUTION: A 4-inch bonded overlay is required.

12.0 AASHTO COMPUTER PROGRAM FOR OVERLAY DESIGN

A computerized version of the AASHTO pavement design procedure has been developed and is available for microcomputers. The computer program (DNPS86) incorporates all of the inputs used in the non-computerized method of pavement design.

The computer program is also capable of determining overlay thicknesses for any overlay type and any existing pavement type. However, the program does not directly consider the existing pavement condition or the remaining life in its analysis. That is, an overlay cannot be designed for a given pavement system under given conditions. Rather, the only means of overlay design is accomplished by considering staged construction.

The staged construction involves choosing an analysis period and dividing this time period into two or more performance periods for each stage of construction. For example, if a 30-year analysis period is selected, and an initial JRCP design is chosen for a 20-year performance period, then a 10-year performance period is required for any subsequent overlays. This is illustrated in Figure 5-4.20, which are actual DNPS86 screens for such a design. Figure 5-4.20 (a) and 5-4.20 (b) are the only additional input screens needed for the overlay design analysis; the other required input screens are the same as those shown in Block 4. Figure 5-4.20 (c) is the output screen providing not only the design of the initial pavement, but also the design of the JPCP overlay.

Thus, when conducting an overlay analysis with DNPS86, the program will determine required thicknesses (given the projected performance periods) for the "new" pavement and the subsequent overlay.

13.0 SUMMARY

AC and PCC overlays on rigid pavements were discussed and several design methods were presented. The AASHTO method was emphasized for both flexible overlays and rigid overlays, however; an alternate method was presented for each overlay type. The Asphalt Institute method was outlined for flexible overlay design and the PCA method was outlined for rigid overlay design. Several mechanistic procedures were mentioned that are available to the design engineer.

*** RIGID OVERLAY DESIGN INPUTS ***

SERVICEABILITY INDEX AFTER OVERLAY CONSTRUCTION .	4.50
OVERLAY STANDARD DEVIATION (LOG REPETITIONS) . .	0.390
STRUCTURAL CHARACTERISTICS & MATERIAL PROPERTIES	
Rigid Overlay Type	JPCP
Minimum Thickness (inches)	3.00
PCC Elastic Modulus (psi)	4200000
Average PCC Modulus of Rupture (psi)	690
Load Transfer Coefficient	3.20
Bond Coefficient	1.00
Drainage Coefficient	1.00
Loss of Support Factor	0.00

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

a

*** RIGID OVERLAY COST INPUTS ***

OVERLAY CONSTRUCTION COSTS AND SALVAGE VALUE	
Unit Cost of Overlay Material ($/CY)	0.00
Salvage Value (percent)	0
Shoulders, If Different Than Overlay ($/lin ft)	0.00
Mobilization and Other Fixed Costs ($/lin ft) .	0.00
OVERLAY MAINTENANCE COST	
Initial Year Costs Begin to Accrue	0
Yearly Increase ($/lane mile/year)	0.00

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

b

*** SOLUTION FOR INPUT DATA FILE: pccol.dat ***

RIGID PAVEMENT STRUCTURAL DESIGN	
Pavement Type	JRCP
Required Thickness (in)	9.65
Performance Life (yrs)	20.0
18-kip ESAL Repetitions	7444510.
DESIGN FOR PROJECTED FUTURE OVERLAY	
Overlay Type	JPCP
Required Thickness (in)	5.09
Performance Life (yrs)	10.0
18-kip ESAL Repetitions	6576702.

LIFE CYCLE COSTS ($/SY)	
Initial Pavement	
Construction	.00
Maintenance	.00
Salvage Value	.00
Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00
Net Present Value	.00

Press Any Key to Continue ...

c

Figure 5-4.20. Example DNPS86 Rigid Overlay Design Input Screen, Rigid Overlay Cost Input Screen, and Pavement Design Output Screen (14).

It is very important that the design engineer utilize another design procedure to check his/her design, regardless of the original procedure utilized.

There have been many overlays that have failed prematurely due to the lack of proper evaluation and preoverlay repair. This aspect of overlay design cannot be overemphasized. The direct consideration of reflection cracking is also absolutely essential.

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BLOCK 6

Development and Evaluation of Pavement Design Alternatives

BLOCK 6

DEVELOPMENT AND EVALUATION OF NEW PAVEMENT AND REHABILITATION DESIGN ALTERNATIVES

1.0 INSTRUCTIONAL OBJECTIVES

This module familiarizes course participants with the development and evaluation of new pavement design and rehabilitation alternatives. This includes the development of design alternatives, engineering analysis of alternatives, and economic analysis of different alternatives. Upon completion of this module participants will be able to accomplish the following:

1. List the major steps involved in the development of design alternatives.

2. List the major factors to be considered in the evaluation of alternative designs.

3. Identify the major costs involved in constructing, operating, and maintaining a pavement over an analysis period. Describe procedures for estimating those costs.

4. Determine the effects of interest and inflation on the economic analysis.

5. Conduct a life-cycle cost analysis for a pavement design alternative.

6. Conduct an engineering and economic analysis of two or more pavement design alternatives using principles contained in this module.

2.0 INTRODUCTION

Decreasing highway revenues and increasing construction costs have caused many highway agencies to place greater emphasis on improved pavement management over the past few years. Highway administrators have been trying to get the best pavement performance for the highway dollar. With this emphasis, they have taken a hard look at pavement design alternative selection. This emphasis has initiated great interest in generating and evaluating alternative designs.

There is no one universally accepted method for developing feasible design alternatives for a highway improvement. A considerable amount of professional engineering judgment must be applied to each project. Also, design alternatives must be selected within the framework of the overall management of the pavement network. The alternative that may be the best for a given project may not be best for the network as a whole.

Several design alternatives exist for new pavement construction and for rehabilitation. A new pavement design project may have, for example, the following design alternatives:

1. An asphalt concrete surface course over a granular or stabilized base course.

2. Full-depth asphalt concrete.

3. A jointed plain concrete surface course over a granular or stabilized subbase course.

4. A jointed reinforced concrete surface cause over a granular stabilized subbase course.

5. A continuously reinforced concrete surface course over a granular or stabilized subbase course.

Rehabilitation of the original pavement can take several forms, which may impact the initial selection and design of the pavement:

1. Overlay with asphalt or concrete after minor repair of the existing pavement.

2. Recycle one or more layers of the existing pavement.

3. Restore the existing pavement (without an overlay) through patching, grinding, etc.

4. Reconstruct the pavement.

Each rehabilitation alternative must be considered as an integral part of the initial design. This is the main purpose for including the concept of design period and analysis period in the design strategy. The stage construction option discussed in Blocks 3 and 4, is an example of a combined new design/rehabilitation alternative which may be considered if permitted by agency policy.

The policy of the Federal Highway Administration on pavement design alternative selection is to provide the public with acceptable highway service at a minimum cost, while permitting the opportunity for the use of competing materials and different design details (26). Pavements are to be designed in accordance with procedures which experience has proven provide an economical, durable, and satisfactory roadway structure for the conditions which will prevail at the project site over the life of the pavement. Designs are to be based on expected traffic volume and axle loads, and selected on the basis of an engineering and economic analysis of all governing factors.

A policy statement on pavement design alternative selection released by the FHWA on October 8, 1981 contained four key points (26):

1. Pavement design alternative selection should be based upon an engineering evaluation considering the factors contained in the 1960 AASHTO publication, "An Informational Guide on Project Procedures." Comparison of costs is included among the primary decision factors listed in this publication.

2. The economic analysis of design alternatives should be made on the basis of life-cycle costs, which encompass all the costs associated with constructing, maintaining, and rehabilitating the pavement over the analysis period being used.

3. Agencies often make decisions about new design and rehabilitation alternatives for particular projects years in advance of the actual performance of the work. The engineering and economic analysis of design alternatives should be repeated a short time before the project is advertised, to accurately reflect fluctuations in market prices, traffic levels, and other factors influencing alternative selection. This is particularly important for rehabilitation projects, since the existing pavement may deteriorate significantly during the time that rehabilitation is delayed. Failure to conduct a pavement evaluation and accurately compute needed material quantities just prior to performing the rehabilitation work has resulted in substantial cost overruns on many projects.

4. When the engineering and economic analysis reveals that two or more initial designs are comparable or equivalent in performance and cost, alternate bids may be permitted by the contracting agency. The following requirement must be met for this to apply:

    a. Initial designs must be comparable or equivalent. All designs must be based on the same traffic over the same analysis period. The AASHTO "Guide for Design of Pavement Structures" should be used to evaluate the relative adequacy of designs. Stage construction designs do not qualify as initial designs. Given the uncertainty associated with future interest and inflation rates and material prices, it is felt to be unfair to contractors to include consideration of future improvements in the selection of the low bidder.

    b. Predicted performance of the designs must be comparable or equivalent. The agency must have adequate data to document the performance of each design alternative in the state. This must include current performance and life-cycle cost data that reflect comparable or equivalent service life.

Although this policy does not apply to all agencies and all projects, the principles are valid for any new construction or rehabilitation project. The engineer has the responsibility for developing alternatives of equivalent adequacy and predicted performance, and evaluating these alternatives on the basis of life-cycle cost and all other significant engineering factors. Alternate bids are an acceptable means of alternative selection when the engineer's analysis reveals no clear-cut choice among the design alternatives.

3.0 ENGINEERING ANALYSIS

The AASHTO "Informational Guide on Project Procedures" identifies the following as primary factors to be considered in development of design alternatives for both new construction and rehabilitation:

1. Traffic: Although total traffic volume influences geometric design of the roadway, design of the pavement structural section is based on cumulative heavy axle loadings expected over the life of the pavement.

2. Soil characteristics: Strength, deformation, gradation, and permeability properties of the subgrade soil influence the design of the pavement structural section and the need for positive drainage. The susceptibility of certain soils to volume change (e.g., swelling or frost heaving) may also influence pavement design selection.

3. Weather: Rainfall, snow, ice, frost penetration, cyclic freezing and thawing, and daily and seasonal temperature cycling all influence the subgrade soil and the pavement layers. The performance of similar pavement designs in the same region provide valuable information about the probable influence of climatic factors on the performance of the various design alternatives.

4. Construction considerations: Time required for initial construction, time when major rehabilitation is required, and frequency of future maintenance are particularly important for urban roadways and other high-volume routes, where traffic control is costly and lane closure time must be minimized. Designs with long initial performance periods and low maintenance needs may be favored in these situations.

5. Recycling: Rising costs of aggregate, asphalt cement, and other paving materials has heightened the interest in recycling existing pavements. Some thought should be given in new design to the value of the pavement as a recycled material at the end of its service life.

6. Cost comparisons: Federal and state government agencies have recognized that assessing all of the costs of a highway improvement over a certain design analysis period is more desirable than comparing only the initial cost of construction of the different alternatives. An economic analysis that compares major costs of a highway improvement over a chosen analysis period must consider initial construction costs, maintenance costs, rehabilitation costs, and road-user costs (26).

Secondary factors which may also be pertinent to the development of design alternatives are:

1. Performance of similar pavements in the area.

2. Adjacent existing pavements.

3. Conservation of materials and energy.

4. Availability of local materials or contractor capabilities.

5. Traffic safety.

6. Incorporation of experimental factors.

7. Stimulation of competition.

8. Municipal preference, participating local government preference, and recognition of local industry.

There are other factors not directly related to the structural design of the mainline pavement that may influence the costs of different design alternatives which should also be considered. These include the presence of grade controls, drainage facilities, lateral and overhead clearances, and existing structures.

4.0 ECONOMIC ANALYSIS

Once the alternative designs have been developed, an economic analysis must be conducted. Recent publications on life-cycle cost analysis have added greatly to the available information on this important topic (19, 20, 21, 22). The NCHRP synthesis on "Life-Cycle Cost Analysis of Pavements" by D. E. Peterson is the most comprehensive document available and is recommended for further information (22).

The following costs must be considered for both new construction and rehabilitation:

1. Initial construction (of new pavement or rehabilitation).

2. Future maintenance and rehabilitation.

3. Future salvage value.

Life-cycle costs can be expressed in terms of their "present worth (PW)" or their "equivalent uniform annual cost (EUAC)" (14, 15). The procedures to calculate both PW and EUAC are given in Figure 6-1.1. The present worth method converts all future costs to their equivalent present costs using a selected discount rate. The converted future costs can be combined with the initial construction cost to give a total present worth cost over the analysis period. The equivalent uniform annual cost method converts this present worth to an equivalent annual cost over the analysis period.

4.1 Analysis Period

The analysis period refers to the time over which the economic analysis is to be conducted. Analysis periods for new pavement design are typically twenty to forty years. For rehabilitation work, the analysis period will usually be shorter, such as ten to twenty or more years. An analysis period of at least ten years is recommended for rehabilitation so that future costs are reasonably considered.

In order to use the present worth method, the analysis periods of all alternatives being considered must be equal. New construction design alternatives which have equal design lives (i.e., are designed for the same traffic over the same number of years) can easily be evaluated over the same

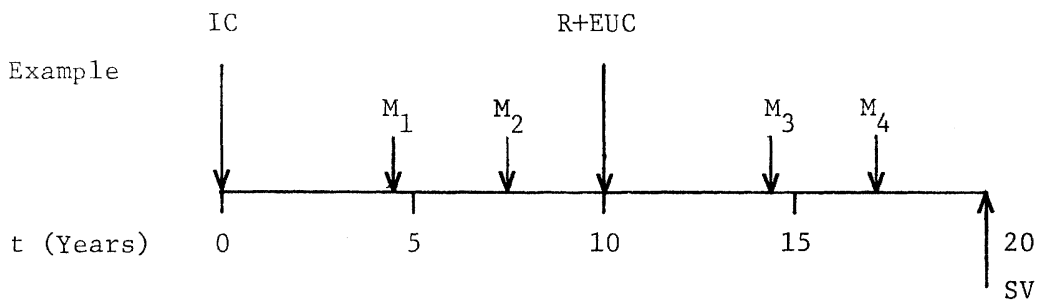
Figure 6-1.1. Life-Cycle Cost Computation Example.

Information Required

Analysis Period -- Life span over which alternatives will be compared. (Note: Analysis period must be the same for each alternative).

Discount Rate -- Average annual discount rate appropriate for the alternatives being analyzed (difference between market interest rate and construction inflation rate). (Note: Rate is average anticipated over the analysis period.)

Cash Flows:
- IC - Initial Construction Cost
- Mi - Maintenance Costs
- R - Rehabilitation Costs
- EUC- Extra User Costs due to major rehabilitation activity
- SV - Salvage Value (pavements value at the end of the analysis period)



IC = \$120,000	M_1 = \$10,000
R = \$ 90,000	M_2 = \$12,000
EUC = \$ 30,000	M_3 = \$25,000
SV = \$ 50,000	M_4 = \$20,000

Note: Costs are in todays dollars and have not been inflated.

Figure 6-1.1. Continued

Analysis

All cashflows must first be brought back to time t=0. The present worth (PW) of the cashflows can be determined as follows:

$$PW = CASHFLOW \frac{1}{(1+i)^n}$$

where:

n = number of years to be discounted over
i = discount rate (assumed 5%)

CASHFLOW	n	$\frac{1}{(1+i)^n}$	PW of CASHFLOW
IC = $120,000	0	1	120,000
M_1 = $ 10,000	4	0.8227	8,227
M_2 = $ 12,000	7	0.7107	8,528
R_2 = $ 90,000	10	0.6139	55,252
EUC= $ 30,000	10	0.6139	18,417
M_3 = $ 25,000	14	0.5051	12,627
M_4 = $ 20,000	17	0.4363	8,726
SV = $ 50,000	20	0.3769	(18,844)

Total Present Worth = 212,933

It is then possible to determine what series of equivalent uniform annual costs (EUAC) would be equivalent to this present worth lump sum. This is calculated as follows:

$$PW = EUAC \left(\frac{1-(1+i)^n}{i}\right)$$

$$EUAC = 212,933 \left(\frac{0.05}{1-(1.05)^{-20}}\right) = \$17,086/\text{Year}$$

analysis period. However, life-cycle cost comparisons must sometimes be made among alternatives with unequal lives. There are three ways to adapt design alternatives so that they may be compared over equal analysis periods:

1. Assume that each alternative is repeated by itself until the ends of the design lives of all the alternatives coincide. Use the least common multiple of the design lives as the analysis period. For example, suppose a bonded concrete overlay for a particular pavement has a predicted life of 15 years while an asphalt concrete overlay for the same pavement has a predicted life of 10 years. The two alternatives could be compared over a 30-year analysis period, during which the concrete overlay is repeated once and the asphalt overlay is repeated twice. This permits comparison of the relative magnitudes of the present worths of the alternatives. However, the dollar amounts used in the analysis will not be what is actually intended to be spent on the project.

2. Add future rehabilitation work to one or more of the alternatives to equalize their lives. For the example described above, the asphalt overlay with the 10-year life could be followed by a thinner overlay which will last 5 years. This combination could be compared with the bonded concrete overlay alternative over an analysis period of 15 years. This is perhaps a more realistic adaptation of the alternatives than least common multiple approach.

3. The design life of one of the alternatives (preferably the shortest one) could be selected as the analysis period, and the "remaining life" of each of the other alternatives can be expressed monetarily as its salvage value. One difficulty with this approach lies in determining the remaining life in terms of fatigue, serviceability, etc. of a pavement at any point in time. The other difficulty lies in expressing this remaining life in dollars. This may be the true salvage value of the pavement, i.e., its value as a recycled material. However, even the pavement with the shortest design life has some value as a recycled material, and thus has a salvage value.

A considerable amount of engineering judgement is required to adapt pavement design alternatives so that they may be compared over equal analysis periods using the present worth method. The alternative is to compare equivalent uniform annual costs, which does not require equal analysis periods.

4.2 Performance Period

This is the time period between the beginning of the life of an alternative and the time when major rehabilitation is next required. The performance period may or may not be equal to the design period. The design period is how long the pavement is supposed to last, while the performance period is how long it actually lasts. For example, a pavement may be designed for 20 years, but due to factors not adequately considered in the design such as an unexpectedly high rate of increase in truck traffic, it may actually last only 14 years. It is essential that the engineer review any information available on how the various pavement design or rehabilitation alternatives being considered have performed under similar conditions of climate and traffic so that he or she selects realistic performance periods for the alternatives for use in the cost analysis. Of the two, rehabilitation performance is significantly harder to predict than new construction performance. Some information is available on the performance of various 4R techniques in References 20, 21, and 23.

Each agency should make a concerted effort to develop a pavement performance data bank of new construction and rehabilitation projects. Eventually this data bank will be a valuable source of information on performance of design alternatives which is valid for local conditions. Several states already have performance data banks that are providing this information. Until this information becomes widely available, professional engineering judgement is required to obtain estimates of alternative life.

4.3 Discount Rate

The discount rate (commonly called an interest rate in business investments) represents the time value of money. It is usually expressed as an annual compounded rate that represents the rate of interest money will earn over a future period. The AASHTO Design Guide explains the discount rate in the following way:

> "A governmental unit that decides to spend money improving a highway, for example, loses the opportunity to "invest" this money elsewhere. That rate at which money could be invested elsewhere is sometimes known as the "Opportunity Cost Of Capital" and is the appropriate discount rate for use in economic studies ... The discount rate for performing present value calculations on public projects should represent the opportunity cost of capital to the taxpayer as reflected by the average market rate of return. However, the market ... rate of interest includes an allowance for expected inflation as well as a return that represents the real cost of capital. For example, a current market rate of interest of 12 percent may well represent a 7 percent opportunity cost component and a 5 percent inflation component." (18).

The NCHRP synthesis on life-cycle cost analysis states:

> "There is general agreement that the discount rate or real discount rate should be the difference between the market interest rate and inflation using constant dollars (22)."

The proper discount rate to use for pavement projects, then, is the difference between the the market rate of interest and the rate of inflation (18, 22, 24). Future costs should be estimated in present dollars and kept constant in the future. The proper discount rate for the above example would be the difference between 12 and 5, or 7 percent.

Epps and Wootan recommended a real discount rate of 4 percent based on their determination that the real long-term rate of return on capital had been between 3.7 and 4.4 percent since 1966 (23). Oglesby and Hicks stated that "the minimum rate for governmental investment should reflect the real cost of capital, which some have estimated as being in the range of 4 percent" (24). More detailed discussion of the selection of the discount rate is given in References 16, 18, 22, 24 and 25.

An illustration of inflation and interest rates (corporate bond rate and Treasury bill rate) variation over time is given in Figure 6-1.2. The two rates typically follow each other as shown, with a mean difference over many years typically less than 4 percent.

Sometimes certain material costs may escalate more rapidly than others, and the effects of inflation on each item will have to be evaluated separately. Future inflated costs of each material or rehabilitation process can be estimated and used directly.

The average annual increase in highway construction costs was 9.4 percent from 1970 to 1979, and 7.4 percent for highway maintenance costs (16). Reference 16 provides an excellent discussion of inflation considerations, and concludes that "the effects of inflation cannot be ignored in engineering economic evaluation of alternatives ... failure to account for the effects of inflation in comparing the cash flows of highway construction on maintenance alternatives will significantly understate real costs."

4.4 Life-Cycle Cost Example

An example of a life-cycle cost analysis is provided in Figure 6-1.3. It should be recognized that such procedures are not precise since reliable data for maintenance, subsequent stages of construction, salvage value, and pavement life are not always available and it is usually necessary to apply engineering judgement to make reasonable estimates. Despite these difficulties, life-cycle cost analysis is believed to provide the best potential to obtain the greatest service from a pavement construction or rehabilitation project at the lowest possible cost.

The computations required to perform life-cycle cost analysis are easily adaptable to a spreadsheet or interactive program for use on a personal computer. One such program available for life-cycle cost analysis is ECON, written in BASIC. Figure 6-1.4 illustrates the inputs required to use ECON to solve the example problem, and the outputs produced. Note that the ECON program cannot consider "negative costs" (i.e., salvage values), so the results produced are not exactly the same as shown in Figure 6-1.3. The results in Figure 6-1.3 and Figure 6-1.4 can be compared to observe the impact of neglecting salvage value in life-cycle cost analysis.

5.0 EVALUATION OF DECISION FACTORS

Life-cycle cost should be the dominant factor in selecting different pavement designs. There may be several other decision factors, both technical and non-technical, which influence the selection of a pavement design alternative. In some cases, these factors may weigh very heavily in the decision process and even outweigh cost considerations, geven agency policy. Some of these other decision factors include:

1. Overall pavement management of network (policies, funds available for projects).

2. Future rehabilitation options and needs.

3. Auto and truck traffic volume.

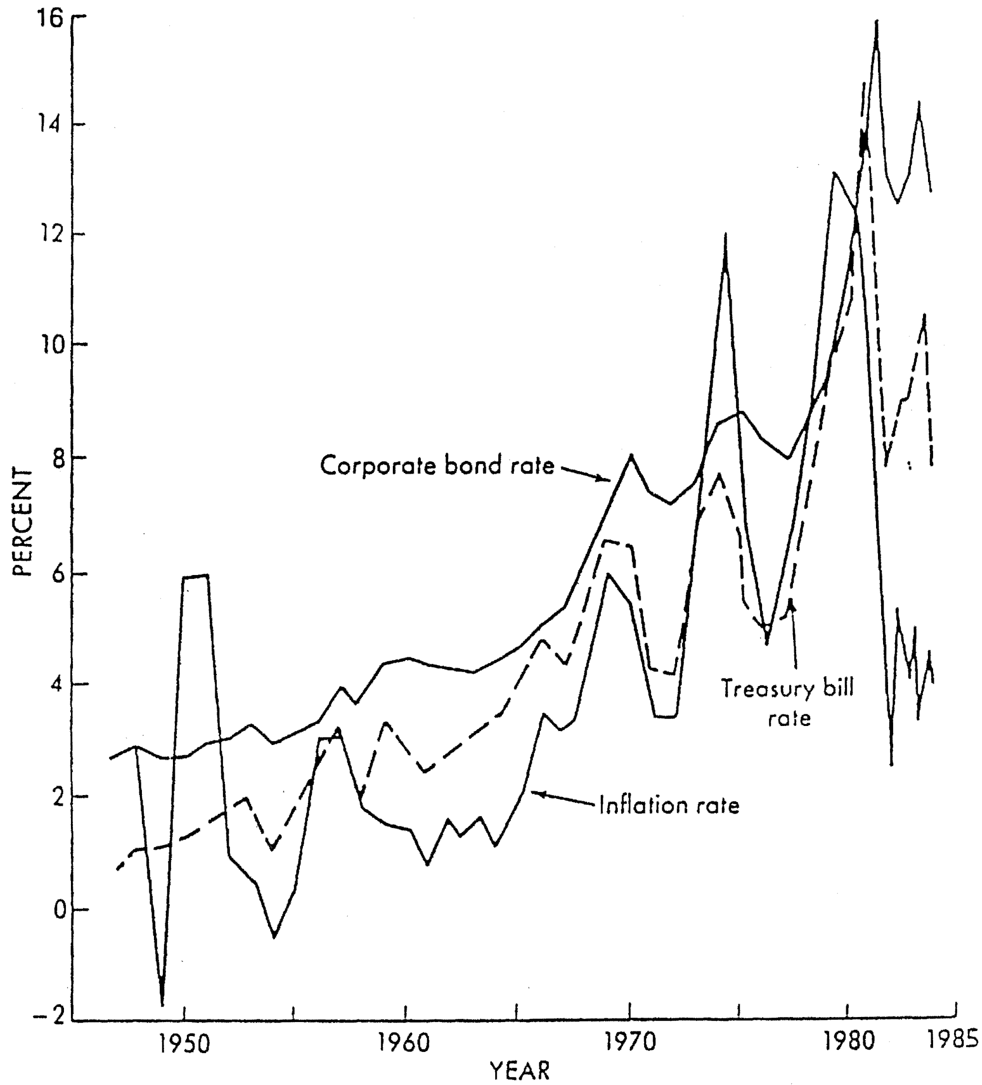


Figure 6-1.2. Three-Month Treasury Bill Rate, Aaa Corporate Bond Rate, and Inflation (Consumer Price Index), 1947-1984.

Figure 6-1.3. Example of a Preliminary Life-Cycle Cost Analysis of Pavement Rehabilitation Alternatives.

Note: This cost analysis is an example of the procedures and concepts only, and nothing should be concluded as to the economic benefits of one alternative over the others. Construction costs and pavement conditions vary widely, and completely different results may be obtained in other analyses with different pavement conditions and costs.

Step 1. Existing Pavement Design:

Jointed plain concrete pavement, controlled-access rural four-lane divided highway. Design is 9-inch slab, 4-inch cement-treated base over a silty clay subgrade. Joints are randomly spaced (12 to 19 feet), skewed with no dowels. Pavement age is 15 years.

Step 2. Existing Pavement Condition:

a. 7 percent shattered slabs to be replaced in outer lane.
b. 50 square yards partial-depth patching for spall repair.
c. Pumping along approximately 3/4 of the project (outer lane only).
d. Faulting in outer lane is serious, averaging 0.15 inches. Inner lane is only 0.05 inches.
e. Serviceability Index averages 2.8 in the outer lane and 3.8 in the inner lane.
f. Asphalt concrete outer shoulder is in fair condition.

Step 3. Select Feasible Rehabilitation Alternatives:

WORK ITEMS	Alternatives No. 1	No. 2	No. 3
Replace Shattered Slabs	X	X	
Patch Spalls	X		X (inner lane)
Subseal Slabs	X	X	
Subdrains	X	X	
Grind Surface Outer Lane	X		
AC Overlay		X	
Replace Outer Lane (recycle existing concrete)			X
Place Tied PCC Shoulder			X
Initial Service Life (years)	7	12	20

Figure 6-1.3. continued.

Step 4. Select Analysis Period:

The three alternatives have different initial service lives. All alternatives must be analyzed over the same analysis period (e.g., 20 years) to conduct a life-cycle cost analysis.

Step 5. Select Discount Rate:

A discount rate of 4 percent is selected.

Step 6. Estimate Life-Cycle Costs and Compute Present Worth:

The unit costs to perform each of the work tasks are estimated using previous bid estimates and other information. The unit costs are estimated for the first year of the analysis period.

Preventative maintenance is planned to be applied about every 5 to 7 years for each of the alternatives. This includes resealing joints, sealing cracks and stabilizing slabs, if necessary.

The salvage value is the expected worth of the existing pavement at the end of its service life. It was estimated as a percentage of the original construction and subsequent rehabilitation costs.

All costs are computed per two-lane mile including shoulders.

No. 1. Restoration

Year	Work Type	Cost	Present Worth
0	Restoration	90,580	90,580
7	Restoration	45,760	34,774
14	Resurfacing	144,322	83,342
20	End of Service Life Salvage Value	-67,500	-30,806

Total Present Worth = $177,890

Total life of alternative = 20 years

Figure 6-1.3. continued.

No. 2. Resurfacing

Year	Work Type	Cost	Present Worth
0	Resurfacing	180,162	180,162
6	Maintenance	10,000	7,903
12	Resurfacing	96,215	60,096
20	End of Service Life		
	Salvage Value	-85,000	-38,793

Total Present Worth = $209,368

Total life of alternative = 20 years

No. 3 Reconstruct Outer Lane And Shoulder

Year	Work Type	Cost	Present Worth
0	Replace Lane/Shoulder	255,700	255,700
10	Maintenance	10,000	6,756
15	Maintenance	10,000	5,553
20	End of Service Life		
	Salvage Value	-60,000	-27,383

Total Present Worth = $240,626

Total life of alternative = 20 years

Present worths were calculated using the following expression:

$$\text{Present Worth} = \text{Cost}\ [\ 1 / (1 + i)^n\ ]$$

where $i = 0.04$
n = time in years

Step 7. Compute Equivalent Uniform Annual Costs:

Alternative	Analysis Period (years)	Present Worth	CRF*	EUAC**
No. 1 Restoration	20	177,890	0.0736	$13,093
No. 2 Resurfacing	20	209,368	0.0736	$15,409
No. 3 Replace Lane/Sh.	20	240,626	0.0736	$17,710

* CRF = Capital Recovery Factor. Present Worth * CRF = EUAC.

**EUACs were calculated using the following expression:

$$\text{EUAC} = (\text{Present Worth})\ \{\ i / [\ 1 - 1 / (1 + i)^n\ ]\ \}$$

Figure 6-1.3. continued.

Step 8. Summary of Results

The life-cycle cost analysis shows that Alternative 1 (Restoration) has the lowest equivalent uniform annual cost. This alternative would cost the agency the equivalent of $13,093 a year over a 20-year period at the assumed discount rate. Alternative 2 (Resurfacing) and Alternative 3 (Reconstruct Outer Lane) are 18 percent and 35 percent more costly than Alternative 1, respectively.

User costs due to lane closures or extra user costs due to increased roughness have not been included. If traffic was not heavy, the user costs would probably not effect the results of the analysis much. However, if traffic volume was very high, the alternative that had the most lane closure time would have a much higher user cost and the consideration of user costs could change the cost analysis significantly.

Step 9. Effect of Error in Life or Cost Estimates

Given the preceding results, an important question that could be asked is: how much would an error in the estimate of pavement life affect the computed life-cycle costs? To answer this question, assume that the restoration alternative only lasts 5 years instead of the 7 assumed. This would be an error of 40 percent. The average annual cost for this service life (using the same unit costs as before and placing an overlay at 10 years instead of 14 years is $14,343, or 9.5 percent greater than the cost coumputed before. Thus, an error in life prediction results in a smaller error in the average annual cost, because of the effect of discounted future costs.

ECONOMIC ANALYSIS OF ALTERNATIVES

PAVEMENT ID: EXAMPLE
ALTERNATIVE ID.: 1
ALT. DESC.: RESTORATION
YEAR TO START ANALYSIS: 1987

ALTERNATIVE LIFE: 20
INTEREST RATE: 4
INFLATION RATE: 0

YEAR	DESCRIPTION	PRES.COST	PRES.VALUE	EUAC	EUAC/SY
1987	RESTORATION	90580.00	90580.00	6665.04	0.05
1994	RESTORATION	45760.00	34773.84	2558.72	0.02
2001	RESURFACING	144322.00	83342.38	6132.48	0.05

INITIAL COST : 90580.00
PRESENT VALUE : 208696.22
EUAC : 15356.24
EUAC/SY : 0.12

ECONOMIC ANALYSIS OF ALTERNATIVES

PAVEMENT ID: EXAMPLE
ALTERNATIVE ID.: 2
ALT. DESC.: OVERLAY
YEAR TO START ANALYSIS: 1987

ALTERNATIVE LIFE: 20
INTEREST RATE: 4
INFLATION RATE: 0

YEAR	DESCRIPTION	PRES.COST	PRES.VALUE	EUAC	EUAC/SY
1987	RESURFACING	180162.00	180162.00	13256.64	0.10
1993	MAINTENANCE	10000.00	7903.15	581.53	0.00
1993	MAINTENANCE	10000.00	7903.15	581.53	0.00

INITIAL COST : 180162.00
PRESENT VALUE : 195968.28
EUAC : 14419.70
EUAC/SY : 0.11

Figure 6-1.4. Life-Cycle Cost Example Using ECON Program.

ECONOMIC ANALYSIS OF ALTERNATIVES

PAVEMENT ID: EXAMPLE
ALTERNATIVE ID.: 3
ALT. DESC.: RECONSTRUCT OUTER LANE
YEAR TO START ANALYSIS: 1987

ALTERNATIVE LIFE: 20
INTEREST RATE: 4
INFLATION RATE: 0

YEAR	DESCRIPTION	PRES.COST	PRES.VALUE	EUAC	EUAC/SY
1987	REPLACE LANE/SHOULDER	255700.00	255700.00	18814.86	0.15
1997	MAINTENANCE	10000.00	6755.64	497.09	0.00
2002	MAINTENANCE	10000.00	5552.65	408.57	0.00

INITIAL COST	:	255700.00
PRESENT VALUE	:	268008.31
EUAC	:	19720.53
EUAC/SY	:	0.16

Figure 6-1.4. continued.

4. Initial construction cost.

5. Future maintenance requirements.

6. Traffic control during construction (safety and congestion).

7. Construction considerations (duration of construction).

8. Conservation of materials and energy.

9. Potential foundation problems.

10. Potential climatic problems.

11. Performance of similar pavements in the area.

12. Availability of local materials and contractor capabilities.

13. Worker safety during construction.

14. Incorporation of experimental features.

15. Stimulation of competition.

16. Municipal preference, participating local government preference and recognition of local industry.

The difficulty in considering these factors in the selection process is that most of them are difficult to quantify. Figure 6-1.5 illustrates a suggested procedure that considers any number of decision factors which do not need to be experienced in monetary units. This procedure has been used successfully in value engineering studies to select the "preferred" alternative. The following general procedure can be used (the example shown was actually developed by a state department of transportation):

1. Alternative design strategies are developed over a selected analysis period. In the example given in Figure 6-1.5, ten alternatives are considered.

2. Decision factors considered important in selecting the preferred alternatives are selected. In the example, these are initial cost, duration of construction, service life, repairability and maintenance effort, rideability and traffic orientation, and proven design in state climate. Others can be used as deemed necessary by the agency.

3. The decision factors must be weighted. Individual decision factors are typically more influential on the final decision than others. This can be taken into account through weighting factors using the form in Figure 6-1.5. The alternatives and decision factors are listed as shown on the form. A percentage weighting scheme can also be used, in which each decision factor is assigned a numerical percentage depending on its relative importance, and the total adds to 100. This weighting must be done by a group representative of the agency who are involved in decision making (managers as well as designers).

EVALUATION OF DESIGN ALTERNATIVES

	CRITERIA								
	INITIAL COST	DURATION OF CONSTRUCTION	SERVICE LIFE	REPAIRABILITY & MAINTENANCE EFFORT	RIDEABILITY & TRAFFIC ORIENTATION	PROVEN DESIGN IN STATE CLIMATE		TOTAL SCORE	RANK
RELATIVE IMPORTANCE	20%	20%	25%	15%	5%	15%		100	
ALTERNATE 1	60 / 12	60 / 12	100 / 25	80 / 12	90 / 4.5	100 / 15		80.5	1
ALTERNATE 1A	60 / 12	60 / 12	100 / 25	80 / 12	90 / 4.5	100 / 15		80.5	1
ALTERNATE 2	60 / 12	60 / 12	70 / 17.5	50 / 7.5	60 / 3	40 / 6		58	5
ALTERNATE 2A	60 / 12	60 / 12	70 / 17.5	50 / 7.5	60 / 3	40 / 6		58	5
ALTERNATE 3	60 / 12	40 / 8	100 / 25	80 / 12	100 / 5	90 / 13.5		75.5	2
ALTERNATE 4	60 / 12	80 / 6	40 / 10	20 / 3	40 / 2	20 / 3		44	8
ALTERNATE 5	40 / 8	60 / 12	40 / 10	50 / 7.5	50 / 2.5	30 / 4.5		44.5	7
ALTERNATE 6	70 / 14	80 / 16	60 / 12.5	50 / 7.5	80 / 4	40 / 6		60	4
ALTERNATE 7	100 / 20	100 / 20	20 / 5	20 / 3	40 / 2	40 / 6		56	6
ALTERNATE 8	30 / 6	60 / 12	100 / 25	100 / 15	100 / 5	30 / 4.5		67.5	3

Figure 6-1.5. Procedure for Determining the "Preferred" Alternative (Actual Data for an Urban Freeway 4R Alternative Evaluation).

4. Any needed analyses are performed to supply information about each of the alternatives (e.g., construction costs, future maintenance and rehabilitation costs. etc.)

5. Next, each alternative is rated independently against the decision factors using a selected scale, such as the 0-to-100 scale used in the example. It is recommended that the rating be performed from top to bottom rather than from right to left. The ratings are placed in the upper left-hand triangle as shown in Figure 6-1.5.

6. The overall rating of each alternative is then computed by multiplying the weights by the ratings and inserting the results in the lower right-hand triangles. The total rating score is obtained by summing across for each alternative.

The alternatives are now ranked in order of highest rating score. The following total scores were obtained from Figure 6-1.5 for the top four alternatives.

Alternative 1	80.5
Alternative 1A	80.5
Alternative 3	75.5
Alternative 8	67.5

In this example, Alternatives 1 and 1A are the most-preferred designs. Thus, either 1 or 1A or both could be recommended to management for final selection.

6.0 DETAILED DESIGN OF SELECTED ALTERNATIVE

After the preferred alternative has been selected, a more detailed design and cost estimate should be made. This may require further field and laboratory testing and evaluation. The final design should be reasonably close to the design used in the preliminary analysis. The plans and specifications must be carefully prepared to reflect the final design.

7.0 SUMMARY

This block describes the steps required in the development and selection of pavement design and rehabilitation alternatives. The purpose of alternative development and selection is to identify the design which will provide the greatest service to the public at the lowest possible cost. Key items include:

1. Carefully evaluating existing pavement conditions to help with rehabilitation as well as with initial design considerations.

2 For both new construction and rehabilitation be creative in developing feasible alternatives which may be applicable.

3. Select the "preferred" alternative on the basis of life-cycle cost and other important decision factors.

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BLOCK 7

Workshops

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MODULE 7-1

RIGID PAVEMENT DESIGN WORKSHOP PROBLEMS

1.0 INSTRUCTIONAL OBJECTIVES

This module provides participants the opportunity to apply the concepts and procedures presented throughout the course to a realistic rigid pavement design problem. Upon completion of this workshop the participant will have demonstrated his or her ability to apply the design procedures presented throughout the course. The problems can be solved by hand or through use of the AASHTO (by FHWA or AASHTO) or PCA computer programs.

2.0 INTRODUCTION

The first step in the design of a rigid pavement system is to select the general type of rigid pavement to be considered (JPCP, JRCP, CRCP). The steps which follow should be performed for each pavement design type:

1. Select general cross-section of pavement (materials, drainage, shoulders) and consider overall performance period and rehabilitation strategy
2. Obtain design inputs
3. Determine required layer thicknesses
4. Develop joint design
5. Reinforcement design, if used
6. Shoulder design
7. Evaluate overall pavement design
8. Estimate life cycle costs
9. Conduct design checks using other procedures
10. Revise design, if necessary

All needed design inputs are provided, however, some are in raw data form and must be reduced somewhat to input directly into the design procedures.

It is recommended that the AASHTO Design Guide be used to develop the initial design, and then if time remains, the PCA design procedure be utilized to compare with and check the AASHTO design results. The PREDICT computer program can also be used to evaluate and check the resulting design to predict the future faulting, cracking, joint deterioration, pumping and serviceability loss.

3.0 RIGID PAVEMENT DESIGN INPUTS

Inputs for traffic, materials, subgrade and climate are either provided or listed for the class participant "designer" to determine (as indicated by ???) for the given pavement site under design. Information can be found on the page or figure number listed next to the input variable. The new pavement to be designed is an existing Interstate highway where the existing pavement will be removed and a new pavement reconstructed.

3.1 Overall Design Criteria

1. Type of highway: Interstate highway
2. Number of traffic lanes one direction: Two
3. Lane width: 12 feet
4. Width of shoulders: OUTER - 10 feet; INNER - 6 feet
5. Performance period of initial design: ??? years
6. Analysis period: 40 years
7. Design reliability = ??? (See pg. 511)
8. Overall standard deviation = ??? (See pg. 276)
9. Performance criteria:
   initial serviceability = ??? (See pg. 513)
   terminal serviceability = ??? (See pg. 513)

3.2 Traffic Level

1. Two-Way ADT and ADTT history: (See Figure 7-1.1 (Future volume for ADT and ADTT are expected to increase as in the past ten years.))
2. Mean 18-kip ESAL per commercial truck history: (See Figure 7-1.1)
3. Truck lane distribution = ??? percent (See pg. 226)
4. Direction distribution of trucks = ??? percent (See pg. 223)
5. Truck traffic growth rate = ??? percent (See Figure 7-1.1)
6. Initial year 18-kip ESAL = ??? (See Table 7-1.1)

3.3 Materials

1. Portland cement concrete:
   Mean modulus of rupture at 28 days = ??? psi (See pg. 101, 518)
   Mean modulus of elasticity = ??? psi (See pg. 101, 518)

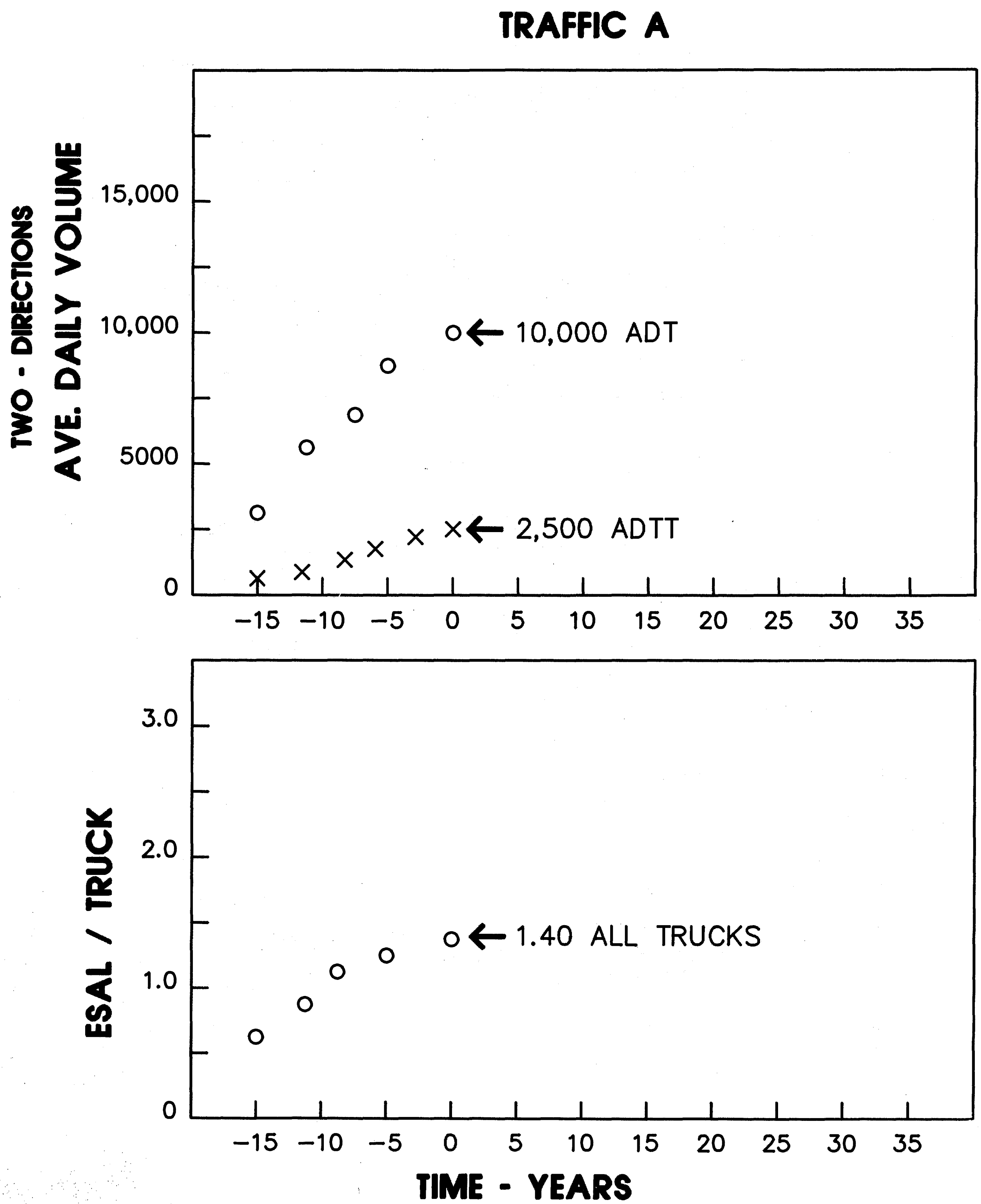

Figure 7-1.1. Traffic Level A For Rigid Workshop.

Table 7-1.1. Traffic Information For the Rigid Workshop.

Calculation of Initial Year 18-kip ESAL in Design lane for Traffic Level A:

18-kip ESAL = (2500 trucks/day * 365 days/year)
 * Directional Distribution (0.5)
 * Lane Distribution (0.87) * ESAL/truck (1.4) = 555,712*

Growth Rate of ESAL Estimate:

	TODAY	10 YEARS
ADTT	2500	4200
ESAL/Truck	1.4	1.7
ESAL/Day	3500	7140

SIMPLE GROWTH: $7140 = 3500(1 + 10*i)$
$i = 10.4$ %/year

COMPOUND GROWTH: $7140 = 3500(1 + i)^{10}$
$i = 7.4$ %/year

---

* NOTE: For input into the DNPS86 program, this is the traffic in the design lane in one direction. If this number is input, the directional distribution and lane distribution should be input as 1.0.

2. Alternative base layers:

Modulus of dense graded crushed stone: 20,000 psi
Modulus of permeable asphalt treated crushed stone: 75,000 psi
Modulus of lean concrete: 1,500,000 psi

3.4 Subgrade and Climate

1. Subgrade soil: Fine grained silty clay soil (A-6)

2. Swelling soil: None

3. Frost heave: Yes or no depending upon climate of the design location

4. Resilient modulus values: (See Table 7-1.2)

3.5 Miscellaneous Factors

1. Load Transfer Coefficient = ??? (See pg. 522 NOTE: Depends on the use of dowels and tied PCC shoulders)

2. Drainage Coefficient = ??? (See pg. 520)

3. Loss of Support Factor = ??? (See pg. 523)

4.0 RIGID PAVEMENT DESIGN SLAB THICKNESS

Determine the required slab thickness for various design alternatives including the following:

1. Different pavement types (JPCP, JRCP, CRCP)

2. With and without tied PCC shoulders

3. Different base types

5.0 JOINT DESIGN

Develop joint designs for the following joint systems:

1. Transverse joints
   - load transfer
   - sealant reservoir
   - spacing
   - skew, if any

2. Longitudinal lane to shoulder joint

3. Longitudinal lane to lane joint

6.0 REINFORCEMENT DESIGN

If designing a JRCP or CRCP, determine required steel reinforcement.

Table 7-1.2. Resilient Modulus Values For Rigid Workshop.

MONTH	RESILIENT MODULUS (psi) Frost Area	Nonfrost Area
January	50,000	5,000
February	50,000	5,000
March	5,000	5,000
April	6,000	5,000
May	10,000	8,000
June	12,000	8,000
July	12,000	10,000
August	12,000	10,000
September	12,000	10,000
October	12,000	10,000
November	12,000	10,000
December	12,000	10,000

7.0 OVERLAYS OR REHABILITATION

Develop plan for rehabilitation of the pavement once terminal serviceability is reached (overlays, CPR) if the performance period is less than the analysis period (which is set at 40 years for this project). Do not develop detailed overlay design.

8.0 LIFE CYCLE COSTS AND SALVAGE VALUES

1. Unit costs and salvage value: (See Table 7-1.3)

2. Discount rate: 4 percent

Use these values to determine the life cycle costs over the 40 year analysis period for each alternative:

1. Initial pavement construction, maintenance and salvage value
2. Overlay pavement construction, maintenance and salvage value
3. Net present value

9.0 PAVEMENT PERFORMANCE

Plot a simple graph of serviceability index versus age to illustrate the serviceability performance curve for each design alternative over the 40 year analysis period. Label all rehabilitations and place cost figures on graph.

Table 7-1.3. Unit Costs and Salvage Values for Rigid Workshop Example.

MATERIAL TYPE	UNIT COSTS*	SALVAGE, PERCENT
PCC SLAB OR OVERLAY		
JPCP	70.00 $/cy	30
JRCP	70.00 $/cy	30
AC OVERLAY		
AC overlay	58.00 $/cy	40
BASE COURSE		
Dense crushed stone base	18.00 $/cy	30
Permeable base	24.00 $/cy	5
Cement treated stone	30.00 $/cy	10
Lean PCC base	35.00 $/cy	20
SHOULDERS		
AC Shoulders	30.00 $/feet	30
PCC Shoulders	40.00 $/feet	30
MOBILIZATION		
Mobilization and other fixed costs	1.00 $/feet	0
MAINTENANCE		
Yearly increase	100 $/lane mile/year	0
Year costs begin to accrue - Year 3		

* Includes all inplace costs including joints, dowels, etc.

MODULE 7-2

FLEXIBLE PAVEMENT DESIGN WORKSHOP

1.0 INSTRUCTIONAL OBJECTIVES

This module provides participants the opportunity to apply the concept and procedures developed throughout the course. Each module of instruction provided example problems which developed data useful in a pavement design. Upon completion of this workshop the participant will have demonstrated the ability to apply the design procedures presented. The solution procedures using micro computer based programs will be emphasized throughout the workshop. Avaliable programs include the AASHTO design program, DNPS86, ESALCALC, DRAINIT, the Federal Highway Administration Symphony spreadsheet program, the economic analysis program ECON, and the elastic layer anallysis program ELSYM5, as well as any others available.

The Federal Highway Administration Symphony spreadhseet design program will be presented and discussed in this workshop as an alternative procedure to investigate multiple options in an AASHTO design.

2.0 INTRODUCTION

The flexible pavement design problem is developed in distinct stages in this module. Each step requires the participant to calculate a pavement design, and evaluate the design. This evaluation may require an elastic layer analysis to examine criticl stresses, an economic analysis to determine EUAC or Present Worth, or use of an alternate design procedure to compare with the AASHTO design. The basic design properties have been developed in the individual instructional modules and will be used as the input for the design problem here.

3.0 FLEXIBLE PAVEMENT DESIGN PARAMETERS

Inputs for traffic, materials, subgrade and climate are either provided or listed for the designer to determine (indicated by ??? in the list) for the given pavement site under design. Page references are provided to assist in selecting appropriate input parameters without excessive searching through the manual. The new pavement to be designed is an existing highway alignment where the existing pavement will be removed and a new pavement reconstructed.

3.1 Overall Design Criteria

1.	Type of Highway	-Interstate
2.	Number of traffic lanes in one direction	-2
3.	Lane width	-12 ft.
4.	Combined width of shoulders	-16 ft.
5.	Analysis period	-20 years.
6.	Performance period of initial design	-??? years.
7.	Design Reliability	-???
8.	Standard Deviation	-???
9.	Performance criteria:	
a.	Initial serviceability	-???
b.	Terminal serviceability	-???

3.2 Traffic Levels

The traffic levels were developed in the example problems for Module 2-5. They represented three metnods of calculating the ESAL values depending on the asignment of percent growth by ADT or by truck classification. They were:

1. Percent growth by ADT - 43,800,000 18-k ESAL.
2. Three classes of truck growth - 53,700,000 18-k ESAL.
3. Two percent error in estimating traffic in 2 - 66,400,000 18-k ESAL.

These were developed using equivalency factors for rigid pavements, and the resulting 18-k ESAL values are not exactly correct for flexible pavements. While ESALCALC should be used to develop more precise design traffic estimates for a flexible pavement, a general reduction factor shows that flexible ESALs are 0.67 of the Rigid ESALs. What is the magnitude of the difference?

Determine the design ESAL for each of the three traffic levels for the design lane assuming a directional distribution factor of 0.5 and a lane distribution factor of 0.9:

1. Situation 1 -
2. Situation 2 -
3. Situation 3 -

3.3 Available Materials

1. Asphalt Concrete:

   a. Marshall Stability - 1700 Pounds.
   b. Hveem R value - 68.

2. Base materials may be chosen from the following materials.

	Crushed Stone Base	Gravel Base
Top Size	1.5 inches	1.5 inches
-#200 material	11 percent	9 percent
Plasticity Index	NP	3.5
Compacted Density	139 pcf.	140 pcf.
California Bearing Ratio	100	30
Gradation		
Sieve Size	Percent Passing	
1.5	100	100
1	90	98
3/4	80	-
1/2	68	74
#4	50	49
#40	21	23
#200	11	9

3.4 Subgrade and Climate

1. A fine grained silty clay soil - A-6 (19).
2. Swelling Soil - None.
3. Frost Heave -Dependent on Course location
4. Resilient modulus values have been established through field NDT testing and/or laboratory testing for three conditions and were presented in Module 2-2. The effective resilient modulus used in this design should be the value you determined in the example problem for Module 2.2 (see Table on page 93).
5. Climate will be determined by the geographic area of the course.

3.5 Costs and Salvage Values

Use the following values for all costs in this problem

MATERIAL TYPE	UNIT COSTS *	SALVAGE, PERCENT
AC Surface	58.00 $/cy	45
Dense crushed stone base	18.00 $/cy	30
Gravel-Sand Base	14.00 $/cy	10
Permeable base	24.00 $/cy	5
Cement treated Stone	30.00 $/cy	10
AC overlay	58.00 $/cy	40
AC Shoulders	30.00 $/ft	30
PCC Shoulders	40.00 $/ft	30
Mobilization and other fixed costs	1.00 $/ft	0
Maintenance Costs		
(first year	??? $/ln.mile	0
yearly increase	??? ?/ln.mile/year	0

*-This includes all costs.

4.0 FLEXIBLE PAVEMENT THICKNESS DESIGN

4.1 Problem 1.

1. Determine the layer coefficients for the surface and granular materials from charts on pages indicated:

   a. a_1 (page 130 and 354)
   b. a_2 (page 355)
   c. a_3 (page 356)

2. Determine the drainage coefficients for the base materials given above:

   a. m_2 (use DRAINIT calculation for materials which
   b. m_3 are the same as those on Page 200)

3. Use the three traffic situations described above to design three pavement structures using the AASHTO DNPS86 program. Use a three layer pavement structure for this design. Use the following values to supplement those given already:

   a. Performance period of initial design -20 years.
   b. Design Reliability -.50
   c. Standard Deviation -0.40
   d. Performance criteria:
      Initial serviceability -4.2
      Terminal serviceability -2.5

4. If you added a positive drainage element to the pavment design, what would the impact be on the pavement thickness design for each traffic level?

4.2 PROBLEM 2.

1. Change the design reliability to a more reasonable 85 percent and repeat the calculation for step 3 above. How do the thicknesses compare?

2. Compare the effect on thickness when the reliability is at 50 percent and when it is at 85 percent. What should happen if the reliability were set at 99.9 percent, a recommended upper level in the AASHTO Design Procedure? Note any difference in the changes that are produced at the three different traffic levels.

4.3 PROBLEM 3.

Using the three pavement structures from Problem 2, No. 1 above, perform an elastic layer analysis on each of the structures. Calculate the radial tensile stress at the bottom of the asphalt layer for each pavement structure, and the vertical compressive stress on top of the subgrade.

Compare these stresses for all three pavements. Each pavement is predicted to have the same life by AASHTO. Do the stresses indicate that the life for each pavement should be the same? Be prepared to discuss any differences.

4.4 PROBLEM 4.

If time permits, a parameter study can be performed to show the interaction of drainage, standard deviation, and design reliability and traffic level. Plots showing the interaction of these parameters provide a useful tool for the designer to use in evaluating which parameters are most influential on the pavement design being worked on.

5.0 FHWA AASHTO PROGRAM

The Federal Highway Administration AASHTO design program developed by Hallin and Owens (1) provides an excellent means of performing the parametric study called for in Problem 3. This program is a series of Symphony spreadsheets which has the capability to perform the "what if" type of calculations spreadsheets have become famous for. This program has the added capability of combining the "what if" calculations into a graphical representation of the calculated values. This makes it a particularly valuable tool to the engineer to show the graphical trends in thicknesses as design parameters change. It does not have the cost capabilities of the AASHTO DNPS86 program, but it has more capability in overlay design.

This program will be demonstrated breifly by the instructor, and the participants will be able to work with the program to study different design situations as time permits. You are encouraged to use any program at your disposal.

SOLUTION SET FOR RIGID PAVEMENT WORKSHOP

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 1 Page 1
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

GENERAL DESIGN INPUT REQUIREMENTS

Analysis Period (years)	40.0
Discount Rate (percent per year)	4.00
Number of Traffic Lanes (one direction)	2
Lane Width (feet)	12.0
Combined Width of Shoulders (feet, one direction)	16.

ROADBED SOIL RESILIENT MODULI

Season:	1	2	3	4	5	6
Modulus (psi):	50000.	50000.	5000.	6000.	10000.	12000.

Season:	7	8	9	10	11	12
Modulus (psi):	12000.	12000.	12000.	12000.	12000.	12000.

DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS

Desired Level of Reliability (percent)	95.00
Design Terminal Serviceability	3.00
Roadbed Soil Swelling	(Not Considered)
Frost Heave	(Not Considered)

Solution #1. Dowelled JPCP with Crushed Stone Base and AC Shoulders.

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PROBLEM NO. 1 Page 2
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

RIGID PAVEMENT DESIGN INPUTS

Performance Period for Initial Pavement (years)	40.0
Serviceability Index After Initial Construction	4.50
Traffic	
Growth Rate (percent)	7.40
Type of Growth	COMPOUND
Initial Yearly 18-kip ESAL (both directions)	517388.
Directional Distribution Factor (percent)	50.
Lane Distribution Factor (percent)	87.
Overall Standard Deviation (log repetitions)	.390
Subbase	
Subbase Type	CS
Thickness (inches)	6.00
Elastic Modulus (psi)	20000.
Unit Cost ($/CY)	18.00
Salvage Value (percent)	30.
Portland Cement Concrete Slab	
Type of Construction	JPCP
PCC Elastic Modulus (psi)	4200000.
Average PCC Modulus of Rupture (psi)	700.
Unit Cost of PCC ($/CY)	70.00
Salvage Value (percent)	30.
Structural Characteristics	
Load Transfer Coefficient	3.20
Drainage Coefficient	1.00
Loss of Support Factor	.00
Other Construction Related Costs	
Shoulders, If Not Full Strength ($/linear foot)	30.00
Drainage ($/linear foot)	6.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #1. (Continued) Dowelled JPCP with Crushed Stone Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
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PROBLEM NO. 1 — Page 3

Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

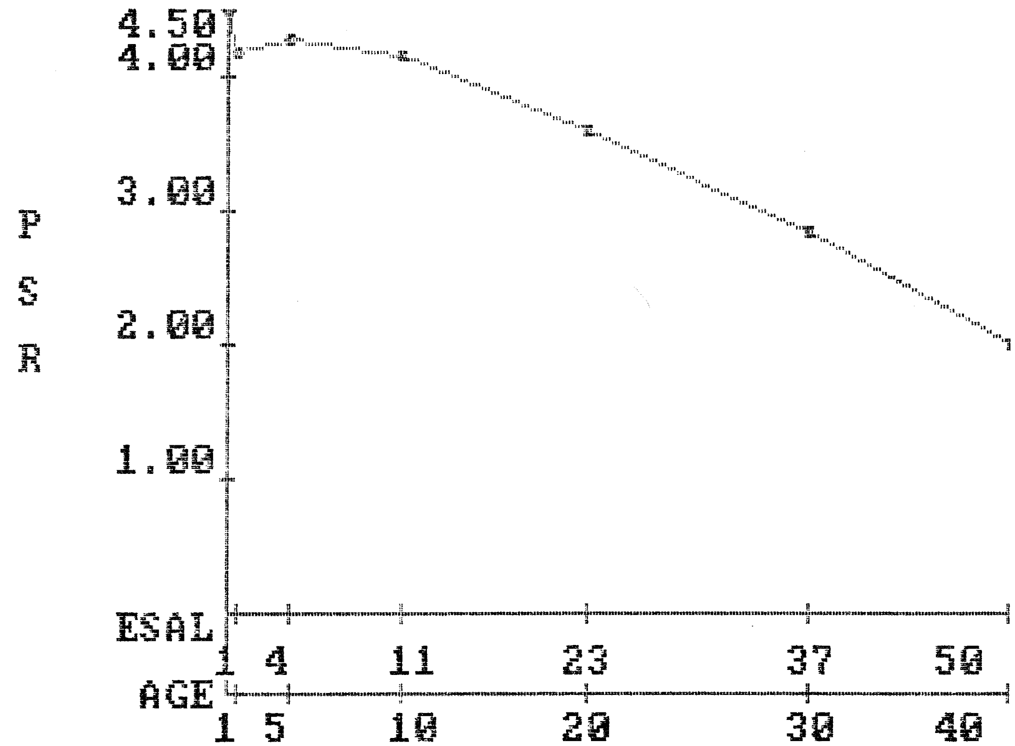
RIGID PAVEMENT STRUCTURAL DESIGN

Effective Modulus of Subgrade Reaction (pci)	597.
Subbase Type	CS
Subbase Thickness (inches)	6.00
Pavement Type	JPCP
Required Slab Thickness (inches)	13.09
Performance Life (years)	40.0
Allowable 18-kip ESAL Repetitions	49832960.

LIFE-CYCLE COSTS ($/SY)

Initial Construction	42.32
Maintenance	3.80
Salvage Value	-1.78
First Overlay Construction	.00
First Overlay Maintenance	.00
First Overlay Salvage Value	.00
Second Overlay Construction	.00
Second Overlay Maintenance	.00
Second Overlay Salvage Value	.00
Total Net Present Value	44.35

Solution #1. (Continued) Dowelled JPCP with Crushed Stone Base and AC Shoulders.



Solution #1. (Continued) Serviceability Curve for the Dowelled JPCP with Crushed Stone Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 3 Page 1
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Permeable AC - Dowels - AC Shoulder

GENERAL DESIGN INPUT REQUIREMENTS

Analysis Period (years)	40.0
Discount Rate (percent per year)	4.00
Number of Traffic Lanes (one direction)	2
Lane Width (feet)	12.0
Combined Width of Shoulders (feet, one direction)	16.

ROADBED SOIL RESILIENT MODULI

Season:	1	2	3	4	5	6
Modulus (psi):	50000.	50000.	5000.	6000.	10000.	12000.

Season:	7	8	9	10	11	12
Modulus (psi):	12000.	12000.	12000.	12000.	12000.	12000.

DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS

Desired Level of Reliability (percent)	95.00
Design Terminal Serviceability	3.00
Roadbed Soil Swelling	(Not Considered)
Frost Heave	(Not Considered)

Solution #2. Dowelled JPCP with Permeable AC Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 3 Page 2
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Permeable AC - Dowels - AC Shoulder

RIGID PAVEMENT DESIGN INPUTS

Performance Period for Initial Pavement (years)	40.0
Serviceability Index After Initial Construction	4.50
Traffic	
Growth Rate (percent)	7.40
Type of Growth	COMPOUND
Initial Yearly 18-kip ESAL (both directions)	517388.
Directional Distribution Factor (percent)	50.
Lane Distribution Factor (percent)	87.
Overall Standard Deviation (log repetitions)	.390
Subbase	
Subbase Type	PERM AC
Thickness (inches)	6.00
Elastic Modulus (psi)	75000.
Unit Cost ($/CY)	21.00
Salvage Value (percent)	5.
Portland Cement Concrete Slab	
Type of Construction	JPCP
PCC Elastic Modulus (psi)	4200000.
Average PCC Modulus of Rupture (psi)	700.
Unit Cost of PCC ($/CY)	70.00
Salvage Value (percent)	30.
Structural Characteristics	
Load Transfer Coefficient	3.20
Drainage Coefficient	1.00
Loss of Support Factor	.00
Other Construction Related Costs	
Shoulders, If Not Full Strength ($/linear foot)	30.00
Drainage ($/linear foot)	6.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #2. (Continued) Dowelled JPCP with Permeable AC Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 3 Page 3
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Permeable AC - Dowels - AC Shoulder

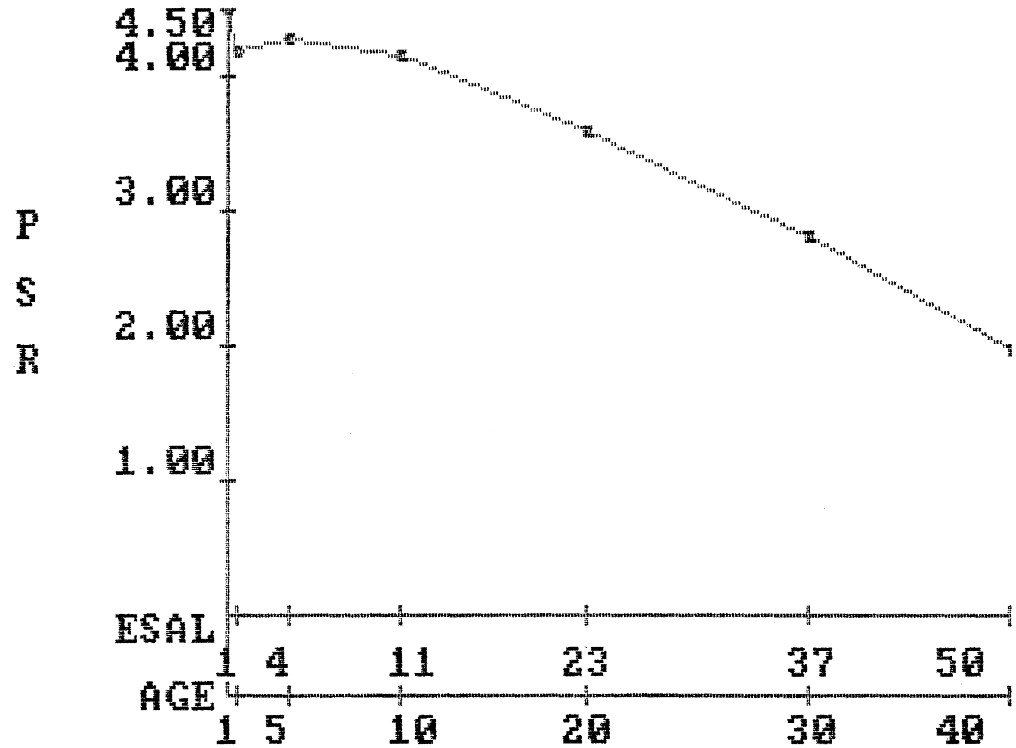
RIGID PAVEMENT STRUCTURAL DESIGN

Effective Modulus of Subgrade Reaction (pci)	746.
Subbase Type	PERM AC
Subbase Thickness (inches)	6.00
Pavement Type	JPCP
Required Slab Thickness (inches)	12.92
Performance Life (years)	40.0
Allowable 18-kip ESAL Repetitions	49832960.

LIFE-CYCLE COSTS ($/SY)

Initial Construction	42.51
Maintenance	3.80
Salvage Value	-1.61
First Overlay Construction	.00
First Overlay Maintenance	.00
First Overlay Salvage Value	.00
Second Overlay Construction	.00
Second Overlay Maintenance	.00
Second Overlay Salvage Value	.00
Total Net Present Value	44.70

Solution #2. (Continued) Dowelled JPCP with Permeable AC Base and AC Shoulders.



Solution #2. (Continued) Serviceability Curve for Dowelled JPCP with Permeable AC Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
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PROBLEM NO. 3 Page 1
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - CTB - Dowels - AC Shoulder

GENERAL DESIGN INPUT REQUIREMENTS

Analysis Period (years)	40.0
Discount Rate (percent per year)	4.00
Number of Traffic Lanes (one direction)	2
Lane Width (feet)	12.0
Combined Width of Shoulders (feet, one direction)	16.

ROADBED SOIL RESILIENT MODULI

Season:	1	2	3	4	5	6
Modulus (psi):	50000.	50000.	5000.	6000.	10000.	12000.

Season:	7	8	9	10	11	12
Modulus (psi):	12000.	12000.	12000.	12000.	12000.	12000.

DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS

Desired Level of Reliability (percent)	95.00
Design Terminal Serviceability	3.00
Roadbed Soil Swelling	(Not Considered)
Frost Heave	(Not Considered)

Solution #3. Dowelled JPCP with Cement-Treated Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 3 Page 2
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - CTB - Dowels - AC Shoulder

RIGID PAVEMENT DESIGN INPUTS

Performance Period for Initial Pavement (years)	40.0
Serviceability Index After Initial Construction	4.50
Traffic	
Growth Rate (percent)	7.40
Type of Growth	COMPOUND
Initial Yearly 18-kip ESAL (both directions)	517388.
Directional Distribution Factor (percent)	50.
Lane Distribution Factor (percent)	87.
Overall Standard Deviation (log repetitions)	.390
Subbase	
Subbase Type	CTB
Thickness (inches)	6.00
Elastic Modulus (psi)	1500000.
Unit Cost ($/CY)	30.00
Salvage Value (percent)	10.
Portland Cement Concrete Slab	
Type of Construction	JPCP
PCC Elastic Modulus (psi)	4200000.
Average PCC Modulus of Rupture (psi)	700.
Unit Cost of PCC ($/CY)	70.00
Salvage Value (percent)	30.
Structural Characteristics	
Load Transfer Coefficient	3.20
Drainage Coefficient	1.00
Loss of Support Factor	.00
Other Construction Related Costs	
Shoulders, If Not Full Strength ($/linear foot)	30.00
Drainage ($/linear foot)	6.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #3. (Continued) Dowelled JPCP with Cement-Treated Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 3 Page 3
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - CTB - Dowels - AC Shoulder

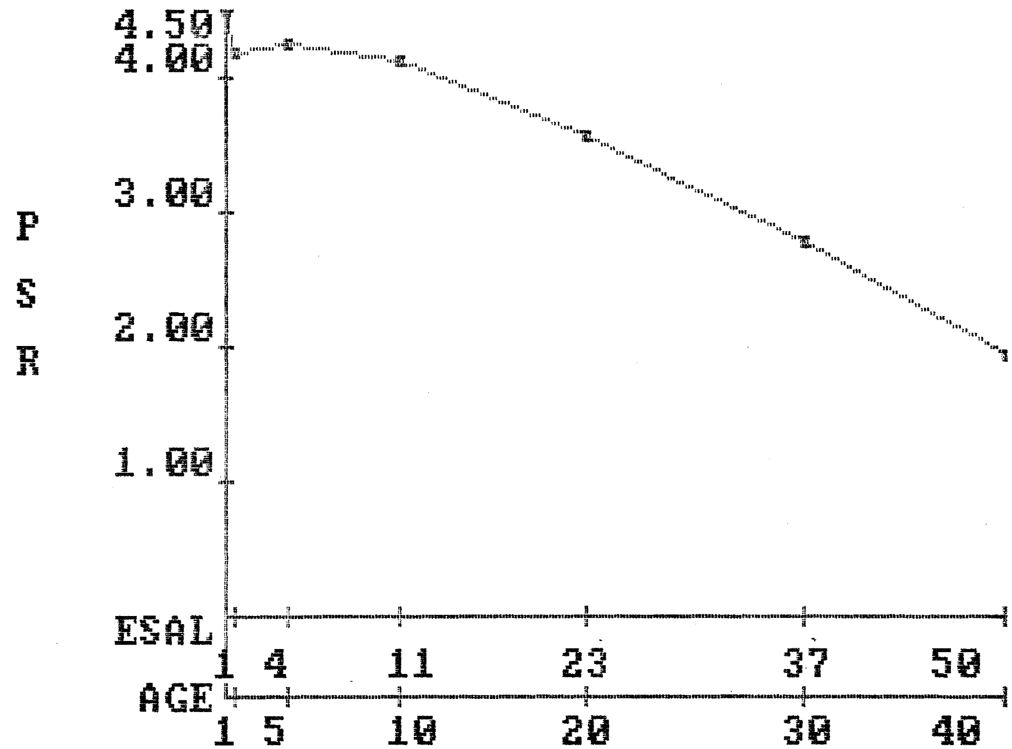
RIGID PAVEMENT STRUCTURAL DESIGN

Effective Modulus of Subgrade Reaction (pci)	1234.
Subbase Type	CTB
Subbase Thickness (inches)	6.00
Pavement Type	JPCP
Required Slab Thickness (inches)	12.48
Performance Life (years)	40.0
Allowable 18-kip ESAL Repetitions	49832960.

LIFE-CYCLE COSTS ($/SY)

Initial Construction	43.15
Maintenance	3.80
Salvage Value	-1.62
First Overlay Construction	.00
First Overlay Maintenance	.00
First Overlay Salvage Value	.00
Second Overlay Construction	.00
Second Overlay Maintenance	.00
Second Overlay Salvage Value	.00
Total Net Present Value	45.33

Solution #3. (Continued) Dowelled JPCP with Cement-Treated Base and AC Shoulders.



Solution #3. (Continued) Serviceability Curve for Dowelled JPCP with Cement-Treated Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 6 Page 1
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - No Dowels - AC Shoulder

GENERAL DESIGN INPUT REQUIREMENTS

Analysis Period (years)	40.0
Discount Rate (percent per year)	4.00
Number of Traffic Lanes (one direction)	2
Lane Width (feet)	12.0
Combined Width of Shoulders (feet, one direction)	16.

ROADBED SOIL RESILIENT MODULI

Season:	1	2	3	4	5	6
Modulus (psi):	50000.	50000.	5000.	6000.	10000.	12000.

Season:	7	8	9	10	11	12
Modulus (psi):	12000.	12000.	12000.	12000	12000.	12000.

DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS

Desired Level of Reliability (percent)	95.00
Design Terminal Serviceability	3.00
Roadbed Soil Swelling	(Not Considered)
Frost Heave	(Not Considered)

Solution #4. Non-Dowelled JPCP with Crushed Stone Base and AC Shoulders.

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VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 6 Page 2
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - No Dowels - AC Shoulder

RIGID PAVEMENT DESIGN INPUTS

Performance Period for Initial Pavement (years)	40.0
Serviceability Index After Initial Construction	4.50
Traffic	
Growth Rate (percent)	7.40
Type of Growth	COMPOUND
Initial Yearly 18-kip ESAL (both directions)	517388.
Directional Distribution Factor (percent)	50.
Lane Distribution Factor (percent)	87.
Overall Standard Deviation (log repetitions)	.390
Subbase	
Subbase Type	CS
Thickness (inches)	6.00
Elastic Modulus (psi)	20000.
Unit Cost ($/CY)	18.00
Salvage Value (percent)	30.
Portland Cement Concrete Slab	
Type of Construction	JPCP
PCC Elastic Modulus (psi)	4200000.
Average PCC Modulus of Rupture (psi)	700.
Unit Cost of PCC ($/CY)	70.00
Salvage Value (percent)	30.
Structural Characteristics	
Load Transfer Coefficient	4.10
Drainage Coefficient	1.00
Loss of Support Factor	.00
Other Construction Related Costs	
Shoulders, If Not Full Strength ($/linear foot)	30.00
Drainage ($/linear foot)	6.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #4. Non-Dowelled JPCP with Crushed Stone Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 6 — Page 3
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - No Dowels - AC Shoulder

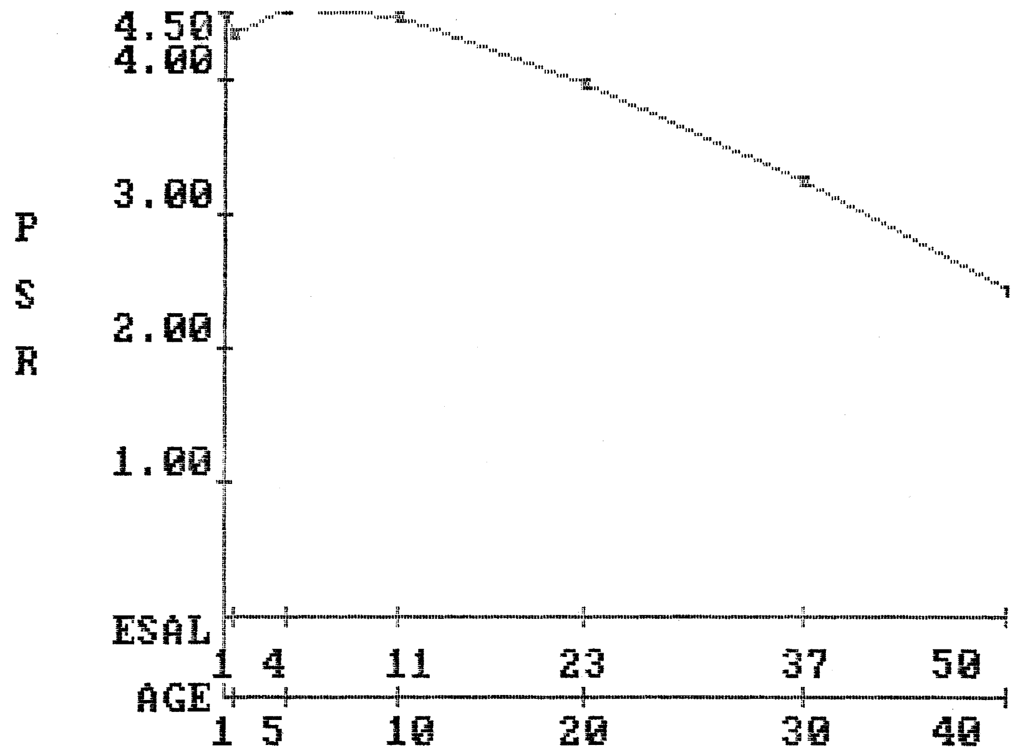
RIGID PAVEMENT STRUCTURAL DESIGN

Effective Modulus of Subgrade Reaction (pci)	600.
Subbase Type	CS
Subbase Thickness (inches)	6.00
Pavement Type	JPCP
Required Slab Thickness (inches)	14.86
Performance Life (years)	40.0
Allowable 18-kip ESAL Repetitions	49832960.

LIFE-CYCLE COSTS ($/SY)

Initial Construction	45.77
Maintenance	3.80
Salvage Value	-1.99
First Overlay Construction	.00
First Overlay Maintenance	.00
First Overlay Salvage Value	.00
Second Overlay Construction	.00
Second Overlay Maintenance	.00
Second Overlay Salvage Value	.00
Total Net Present Value	47.58

Solution #4. Non-Dowelled JPCP with Crushed Stone Base and AC Shoulders.



Solution #4. (Continued) Serviceability Curve for the Dowelled JPCP with Crushed Stone Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 1
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - Dowels - Tied PCC Shoulder

GENERAL DESIGN INPUT REQUIREMENTS

Analysis Period (years)	40.0
Discount Rate (percent per year)	4.00
Number of Traffic Lanes (one direction)	2
Lane Width (feet)	12.0
Combined Width of Shoulders (feet, one direction)	16.

ROADBED SOIL RESILIENT MODULI

Season:	1	2	3	4	5	6
Modulus (psi):	50000.	50000.	5000.	6000.	10000.	12000.

Season:	7	8	9	10	11	12
Modulus (psi):	12000.	12000.	12000.	12000.	12000.	12000.

DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS

Desired Level of Reliability (percent)	95.00
Design Terminal Serviceability	3.00
Roadbed Soil Swelling	(Not Considered)
Frost Heave	(Not Considered)

Solution #5. Dowelled JPCP with Crushed Stone Base and Tied PCC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 1
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - Dowels - Tied PCC Shoulder

RIGID PAVEMENT DESIGN INPUTS

Performance Period for Initial Pavement (years)	40.0
Serviceability Index After Initial Construction	4.50
Traffic	
Growth Rate (percent)	7.40
Type of Growth	COMPOUND
Initial Yearly 18-kip ESAL (both directions)	517388.
Directional Distribution Factor (percent)	50.
Lane Distribution Factor (percent)	87.
Overall Standard Deviation (log repetitions)	.390
Subbase	
Subbase Type	CS
Thickness (inches)	6.00
Elastic Modulus (psi)	20000.
Unit Cost ($/CY)	18.00
Salvage Value (percent)	30.
Portland Cement Concrete Slab	
Type of Construction	JPCP
PCC Elastic Modulus (psi)	4200000.
Average PCC Modulus of Rupture (psi)	700.
Unit Cost of PCC ($/CY)	70.00
Salvage Value (percent)	30.
Structural Characteristics	
Load Transfer Coefficient	2.50
Drainage Coefficient	1.00
Loss of Support Factor	.00
Other Construction Related Costs	
Shoulders, If Not Full Strength ($/linear foot)	40.00
Drainage ($/linear foot)	6.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #5. (Continued) Dowelled JPCP with Crushed Stone Base and Tied PCC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 1 Page 3
Pavement Design Rigid Workshop - NO OVERLAY
JPCP - Crushed Stone - Dowels - Tied PCC Shoulder

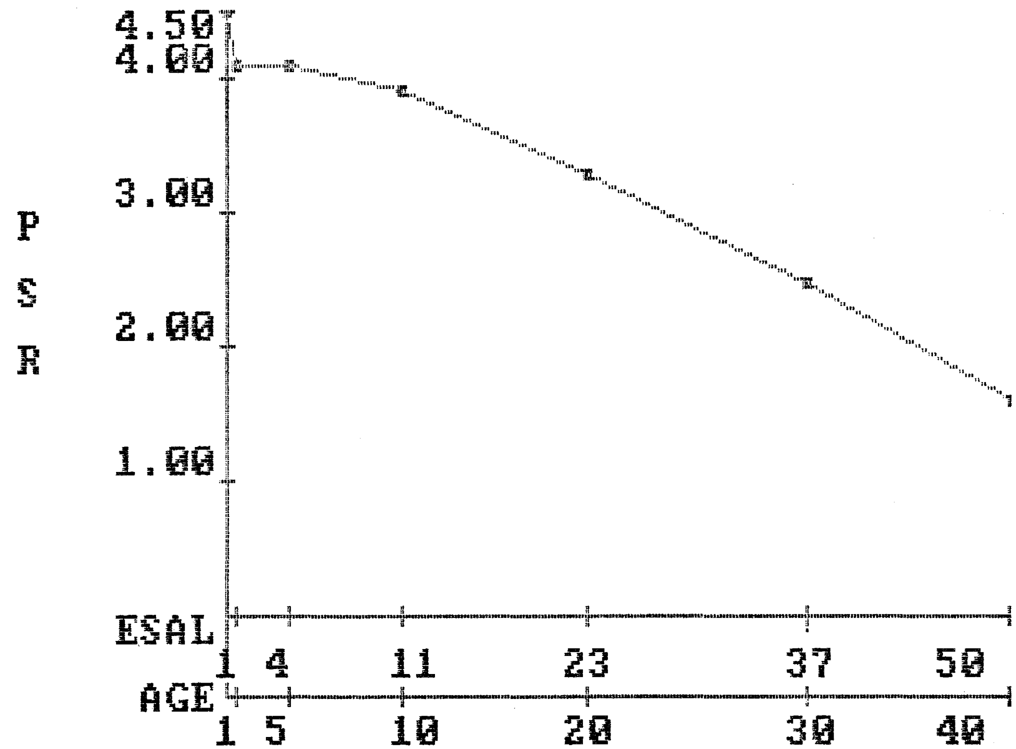
RIGID PAVEMENT STRUCTURAL DESIGN

Effective Modulus of Subgrade Reaction (pci)	594.
Subbase Type	CS
Subbase Thickness (inches)	6.00
Pavement Type	JPCP
Required Slab Thickness (inches)	11.50
Performance Life (years)	40.0
Allowable 18-kip ESAL Repetitions	49832960.

LIFE-CYCLE COSTS ($/SY)

Initial Construction	42.98
Maintenance	3.80
Salvage Value	-1.58
First Overlay Construction	.00
First Overlay Maintenance	.00
First Overlay Salvage Value	.00
Second Overlay Construction	.00
Second Overlay Maintenance	.00
Second Overlay Salvage Value	.00
Total Net Present Value	45.20

Solution #5. (Continued) Dowelled JPCP with Crushed Stone Base and Tied PCC Shoulders.



Solution #5. (Continued) Serviceability Curve for the Dowelled JPCP with Crushed Stone Base and AC Shoulders.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 2 Page 1
Pavement Design Rigid Workshop - AC OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

GENERAL DESIGN INPUT REQUIREMENTS

Analysis Period (years)	40.0
Discount Rate (percent per year)	4.00
Number of Traffic Lanes (one direction)	2
Lane Width (feet)	12.0
Combined Width of Shoulders (feet, one direction)	16.

ROADBED SOIL RESILIENT MODULI

Season:	1	2	3	4	5	6
Modulus (psi):	50000.	50000.	5000.	6000.	10000.	12000.

Season:	7	8	9	10	11	12
Modulus (psi):	12000.	12000.	12000.	12000.	12000.	12000.

DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS

Desired Level of Reliability (percent)	95.00
Design Terminal Serviceability	3.00
Roadbed Soil Swelling	(Not Considered)
Frost Heave	(Not Considered)

Solution #6. Dowelled JPCP with Crushed Stone Base, AC Shoulders, and AC Overlay.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 2 Page 2
Pavement Design Rigid Workshop - AC OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

RIGID PAVEMENT DESIGN INPUTS

Performance Period for Initial Pavement (years)	20.0
Serviceability Index After Initial Construction	4.50
Traffic	
Growth Rate (percent)	7.40
Type of Growth	COMPOUND
Initial Yearly 18-kip ESAL (both directions)	517388.
Directional Distribution Factor (percent)	50.
Lane Distribution Factor (percent)	87.
Overall Standard Deviation (log repetitions)	.390
Subbase	
Subbase Type	CS
Thickness (inches)	6.00
Elastic Modulus (psi)	20000.
Unit Cost ($/CY)	18.00
Salvage Value (percent)	30.
Portland Cement Concrete Slab	
Type of Construction	JPCP
PCC Elastic Modulus (psi)	4200000.
Average PCC Modulus of Rupture (psi)	700.
Unit Cost of PCC ($/CY)	70.00
Salvage Value (percent)	30.
Structural Characteristics	
Load Transfer Coefficient	3.20
Drainage Coefficient	1.00
Loss of Support Factor	.00
Other Construction Related Costs	
Shoulders, If Not Full Strength ($/linear foot)	30.00
Drainage ($/linear foot)	6.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #6. (Continued) Dowelled JPCP with Crushed Stone Base, AC Shoulders, and AC Overlay.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 2 Page 3
Pavement Design Rigid Workshop - AC OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

FLEXIBLE OVERLAY INPUTS

Performance Period for First Overlay (years)	20.0
Serviceability Index After Overlay Construction	4.50
Overlay Standard Deviation (log repetitions)	.490
Overlay Characteristics	
Material Type	AC
Layer Coefficient	.44
Minimum Thickness (inches)	1.00
Overlay Construction Costs and Salvage Value	
Unit Cost ($/CY)	58.00
Salvage Value (percent)	40.
Shoulders, If Different Than Overlay ($/linear ft)	.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Overlay Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #6. (Continued) Dowelled JPCP with Crushed Stone Base, AC Shoulders, and AC Overlay.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 2 Page 4
Pavement Design Rigid Workshop - AC OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

RIGID PAVEMENT STRUCTURAL DESIGN

Effective Modulus of Subgrade Reaction (pci)	589.
Subbase Type	CS
Subbase Thickness (inches)	6.00
Pavement Type	JPCP
Required Slab Thickness (inches)	10.48
Performance Life (years)	20.0
Allowable 18-kip ESAL Repetitions	9639774.

DESIGN FOR PROJECTED FUTURE OVERLAY(S)

First Overlay Type	AC
Required Thickness (inches)	1.00
Performance Life (years)	20.0
Allowable 18-kip ESAL Repetitions	40193180.

LIFE-CYCLE COSTS ($/SY)

Initial Construction	37.25
Maintenance	1.41
Salvage Value	-1.46
First Overlay Construction	1.40
First Overlay Maintenance	.64
First Overlay Salvage Value	-.13
Second Overlay Construction	.00
Second Overlay Maintenance	.00
Second Overlay Salvage Value	.00
Total Net Present Value	39.10

Solution #6. (Continued) Dowelled JPCP with Crushed Stone Base, AC Shoulders, and AC Overlay.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 1 Page 1
Pavement Design Rigid Workshop - PCC OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

GENERAL DESIGN INPUT REQUIREMENTS

Analysis Period (years)	40.0
Discount Rate (percent per year)	4.00
Number of Traffic Lanes (one direction)	2
Lane Width (feet)	12.0
Combined Width of Shoulders (feet, one direction)	16.

ROADBED SOIL RESILIENT MODULI

Season:	1	2	3	4	5	6
Modulus (psi):	50000.	50000.	5000.	6000.	10000.	12000.
Season:	7	8	9	10	11	12
Modulus (psi):	12000.	12000.	12000.	12000.	12000.	12000.

DESIGN INPUTS FOR FLEXIBLE AND RIGID PAVEMENTS

Desired Level of Reliability (percent)	95.00
Design Terminal Serviceability	3.00
Roadbed Soil Swelling	(Not Considered)
Frost Heave	(Not Considered)

Solution #7. Dowelled JPCP with Crushed Stone Base, AC Shoulders, and PCC Overlay.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 1 Page 2
Pavement Design Rigid Workshop - PCC OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

RIGID PAVEMENT DESIGN INPUTS

Performance Period for Initial Pavement (years)	20.0
Serviceability Index After Initial Construction	4.50
Traffic	
Growth Rate (percent)	7.40
Type of Growth	COMPOUND
Initial Yearly 18-kip ESAL (both directions)	517388.
Directional Distribution Factor (percent)	50.
Lane Distribution Factor (percent)	87.
Overall Standard Deviation (log repetitions)	.390
Subbase	
Subbase Type	CS
Thickness (inches)	6.00
Elastic Modulus (psi)	20000.
Unit Cost ($/CY)	18.00
Salvage Value (percent)	30.
Portland Cement Concrete Slab	
Type of Construction	JPCP
PCC Elastic Modulus (psi)	4200000.
Average PCC Modulus of Rupture (psi)	700.
Unit Cost of PCC ($/CY)	70.00
Salvage Value (percent)	30.
Structural Characteristics	
Load Transfer Coefficient	3.20
Drainage Coefficient	1.00
Loss of Support Factor	.00
Other Construction Related Costs	
Shoulders, If Not Full Strength ($/linear foot)	30.00
Drainage ($/linear foot)	6.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #7. (Continued) Dowelled JPCP with Crushed Stone Base, AC Shoulders, and PCC Overlay.

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 1 Page 3

Pavement Design Rigid Workshop - PCC OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

RIGID OVERLAY INPUTS

Serviceability Index After Overlay Construction	4.50
Overlay Standard Deviation (log repetitions)	.390
Structural Characteristics and Material Properties	
Rigid Overlay Type	BONDED
Minimum Thickness (inches)	2.00
PCC Elastic Modulus (psi)	4200000.
Average PCC Modulus of Rupture (psi)	690.
Load Transfer Coefficient	3.20
Bond Coefficient	1.00
Drainage Coefficient	1.00
Loss of Support Factor	.00
Overlay Construction Costs and Salvage Value	
Unit Cost of Overlay Material ($/CY)	70.00
Salvage Value (percent)	30.
Shoulders, If Different Than Overlay ($/linear ft)	30.00
Mobilization and Other Fixed Costs ($/linear foot)	1.00
Overlay Maintenance Cost	
Initial Year Costs Begin to Accrue	3.0
Yearly Increase ($/lane mile/year)	100.00

Solution #7. **(Continued) Dowelled JPCP with Crushed Stone Base, AC Shoulders, and PCC Overlay.**

DNPS86 (1) - AASHTO DESIGN OF NEW PAVEMENT STRUCTURES PROGRAM
VERSION 1 - SEPTEMBER 1986

PROBLEM NO. 1 Page 4

Pavement Design Rigid Workshop - PCC OVERLAY
JPCP - Crushed Stone - Dowels - AC shoulder

RIGID PAVEMENT STRUCTURAL DESIGN

Effective Modulus of Subgrade Reaction (pci)	589.
Subbase Type	CS
Subbase Thickness (inches)	6.00
Pavement Type	JPCP
Required Slab Thickness (inches)	10.48
Performance Life (years)	20.0
Allowable 18-kip ESAL Repetitions	9639774.

DESIGN FOR PROJECTED FUTURE OVERLAY(S)

First Overlay Type	BONDED
Required Thickness (inches)	6.29
Performance Life (years)	20.0
Allowable 18-kip ESAL Repetitions	40193180.

LIFE-CYCLE COSTS ($/SY)

Initial Construction	37.25
Maintenance	1.41
Salvage Value	-1.46
First Overlay Construction	10.89
First Overlay Maintenance	.64
First Overlay Salvage Value	-.76
Second Overlay Construction	.00
Second Overlay Maintenance	.00
Second Overlay Salvage Value	.00
Total Net Present Value	47.96

Solution #7. **(Continued) Dowelled JPCP with Crushed Stone Base, AC Shoulders, and PCC Overlay.**

SOLUTION SET FOR FLEXIBLE PAVEMENT WORKSHOP

DNPS86 (1) No. 1

* * * FLEXIBLE PAVEMENT DESIGN INPUTS * * *

PERFORMANCE PERIOD FOR INITIAL PAVEMENT (YEARS) .	20.0
SERVICEABILITY INDEX AFTER INITIAL CONSTRUCTION .	4.50
TRAFFIC	
Growth Rate (percent per year)	4.00
(S)imple or (C)ompound Growth	C
Initial Yearly 18-kip ESAL (both directions) .	985500
Directional Distribution Factor (percent) . . .	50
Lane Distribution Factor (percent)	90
Calculated Total 18-kip ESAL During the Analysis Period (in the design lane)	13205833
OVERALL STANDARD DEVIATION (LOG REPETITIONS) . .	0.440

DNPS86 (1) No. 1

* * * ADDITIONAL FLEXIBLE PAVEMENT DESIGN INPUTS * * *
AND ASSOCIATED COSTS

PAVEMENT LAYER CHARACTERISTICS, MATERIAL PROPERTIES & COSTS

No.	Description	Spcfd Thick (in.)	Layer Coef	Elastic Modulus (psi)	Drain Coef	Unit Cost ($/CY)	Salv Value (%)
1	Asphalt	0.00	0.42	350000	1.00	0.00	0
2	crsh stone	0.00	0.14	40000	0.80	0.00	0
3	gravel	0.00	0.11	20000	1.00	0.00	0
4		0.00	0.00	0	1.00	0.00	0
5		0.00	0.00	0	1.00	0.00	0

OTHER CONSTRUCTION RELATED COSTS	
Shoulders, If Not Full Strength ($/linear ft) .	0.00
Drainage ($/linear foot)	0.00
Mobilization and Other Fixed Costs ($/lin ft) .	0.00
MAINTENANCE COST	
Initial Year Costs Begin to Accrue	0
Yearly Increase ($/lane mile/year)	0.00

F1: HELP F2: IMPORT/STORE F3: ANALYZE/PRINT/EXIT F4: DISPLAY RESULTS

Traffic and Material Inputs for the Low Traffic Example, R = 50

DNPS86 (1) No. 1

* * * SOLUTION FOR INPUT DATA FILE: pdes * * *

FLEXIBLE PAVEMENT STRUCTURAL DESIGN

Performance Life (yrs)	20.0
18-kip ESAL Repetitions	13205820.

Layer No.	Layer Description	Required Thickness (inches)
1	Asphalt	5.30
2	crsh stone	4.66
3	gravel	16.30
4		
5		

DESIGN FOR PROJECTED FUTURE OVERLAY(S)

	First	Second
Overlay Type	(none)	(none)
Req'd Thick (in)		
Perf Life (yrs)		
18-kip ESAL Reps		

LIFE CYCLE COSTS ($/SY)

Initial Pavement	
Construction	.00
Maintenance	.00
Salvage Value	.00
First Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00
Second Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00

Net Present Value	.00

Press Any Key to Continue ...

Thicknesses for the Low Traffic Example, R = 50

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DNPS86 (1)                                                No.       1

            * * *  FLEXIBLE PAVEMENT DESIGN INPUTS  * * *

     PERFORMANCE PERIOD FOR INITIAL PAVEMENT (YEARS) .        20.0

     SERVICEABILITY INDEX AFTER INITIAL CONSTRUCTION .        4.50

     TRAFFIC
       Growth Rate (percent per year)  . . . . . . . .        4.00
       (S)imple or (C)ompound Growth . . . . . . . . .           C
       Initial Yearly 18-kip ESAL (both directions)  .     1208200
       Directional Distribution Factor (percent) . . .          50
       Lane Distribution Factor (percent)  . . . . . .          90
       Calculated Total 18-kip ESAL During the
         Analysis Period (in the design lane)  . . . .    16190044

     OVERALL STANDARD DEVIATION (LOG REPETITIONS)  . .       0.400
```

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DNPS86 (1)                                                No.       1
       * * *  SOLUTION FOR INPUT DATA FILE: pdes                * * *

  FLEXIBLE PAVEMENT STRUCTURAL DESIGN         LIFE CYCLE COSTS ($/SY)

    Performance Life (yrs)      20.0            Initial Pavement
    18-kip ESAL Repetitions  16190020.            Construction          .00
                                                  Maintenance           .00
    Layer      Layer         Required             Salvage Value         .00
     No.    Description      Thickness
                             (inches)           First Overlay
      1     Asphalt            5.47               Construction          .00
      2     crsh stone         4.79               Maintenance           .00
      3     gravel            16.68               Salvage Value         .00
      4
      5                                         Second Overlay
                                                  Construction          .00
  DESIGN FOR PROJECTED FUTURE OVERLAY(S)          Maintenance           .00
                                                  Salvage Value         .00
                          First     Second                        ----------
    Overlay Type          (none)    (none)      Net Present Value       .00
    Req'd Thick (in)
    Perf Life (yrs)
    18-kip ESAL Reps                            Press Any Key to Continue ...
```

Medium Traffic Example, R = 50

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DNPS86 (1)                                                No.       1

            * * *  FLEXIBLE PAVEMENT DESIGN INPUTS  * * *

     PERFORMANCE PERIOD FOR INITIAL PAVEMENT (YEARS) .        20.0

     SERVICEABILITY INDEX AFTER INITIAL CONSTRUCTION .        4.50

     TRAFFIC
       Growth Rate (percent per year)  . . . . . . . .        4.00
       (S)imple or (C)ompound Growth . . . . . . . . .           C
       Initial Yearly 18-kip ESAL (both directions)  .     1494000
       Directional Distribution Factor (percent) . . .          50
       Lane Distribution Factor (percent)  . . . . . .          90
       Calculated Total 18-kip ESAL During the
         Analysis Period (in the design lane)  . . . .    20019802

     OVERALL STANDARD DEVIATION (LOG REPETITIONS)  . .       0.400
```

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DNPS86 (1)                                                No.       1
       * * *  SOLUTION FOR INPUT DATA FILE: pdes                * * *

  FLEXIBLE PAVEMENT STRUCTURAL DESIGN         LIFE CYCLE COSTS ($/SY)

    Performance Life (yrs)      20.0            Initial Pavement
    18-kip ESAL Repetitions  20019780.            Construction          .00
                                                  Maintenance           .00
    Layer      Layer         Required             Salvage Value         .00
     No.    Description      Thickness
                             (inches)           First Overlay
      1     Asphalt            5.66               Construction          .00
      2     crsh stone         4.94               Maintenance           .00
      3     gravel            17.07               Salvage Value         .00
      4
      5                                         Second Overlay
                                                  Construction          .00
  DESIGN FOR PROJECTED FUTURE OVERLAY(S)          Maintenance           .00
                                                  Salvage Value         .00
                          First     Second                        ----------
    Overlay Type          (none)    (none)      Net Present Value       .00
    Req'd Thick (in)
    Perf Life (yrs)
    18-kip ESAL Reps                            Press Any Key to Continue . .
```

High Traffic Example, R = 50

DNPS86 (1) No. 1

* * * SOLUTION FOR INPUT DATA FILE: pdes * * *

FLEXIBLE PAVEMENT STRUCTURAL DESIGN

Performance Life (yrs) 20.0
18-kip ESAL Repetitions 20019780.

Layer No.	Layer Description	Required Thickness (inches)
1	Asphalt	6.60
2	crsh stone	5.63
3	gravel	18.80
4		
5		

DESIGN FOR PROJECTED FUTURE OVERLAY(S)

	First	Second
Overlay Type	(none)	(none)
Req'd Thick (in)		
Perf Life (yrs)		
18-kip ESAL Reps		

LIFE CYCLE COSTS ($/SY)

Initial Pavement	
Construction	.00
Maintenance	.00
Salvage Value	.00
First Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00
Second Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00

Net Present Value	.00

Press Any Key to Continue ...

High Traffic Example, R = 85

DNPS86 (1) No. 1

* * * SOLUTION FOR INPUT DATA FILE: pdes * * *

FLEXIBLE PAVEMENT STRUCTURAL DESIGN

Performance Life (yrs) 20.0
18-kip ESAL Repetitions 20019780.

Layer No.	Layer Description	Required Thickness (inches)
1	Asphalt	7.21
2	crsh stone	6.06
3	gravel	19.82
4		
5		

DESIGN FOR PROJECTED FUTURE OVERLAY(S)

	First	Second
Overlay Type	(none)	(none)
Req'd Thick (in)		
Perf Life (yrs)		
18-kip ESAL Reps		

LIFE CYCLE COSTS ($/SY)

Initial Pavement	
Construction	.00
Maintenance	.00
Salvage Value	.00
First Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00
Second Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00

Net Present Value	.00

Press Any Key to Continue ...

High Traffic, R = 90

DNPS86 (1) No. 1

* * * SOLUTION FOR INPUT DATA FILE: pdes * * *

FLEXIBLE PAVEMENT STRUCTURAL DESIGN

Performance Life (yrs) 20.0
18-kip ESAL Repetitions 20019780.

Layer No.	Layer Description	Required Thickness (inches)
1	Asphalt	7.95
2	crsh stone	6.53
3	gravel	21.00
4		
5		

DESIGN FOR PROJECTED FUTURE OVERLAY(S)

	First	Second
Overlay Type	(none)	(none)
Req'd Thick (in)		
Perf Life (yrs)		
18-kip ESAL Reps		

LIFE CYCLE COSTS ($/SY)

Initial Pavement	
Construction	.00
Maintenance	.00
Salvage Value	.00
First Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00
Second Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00

Net Present Value	.00

Press Any Key to Continue ...

High Traffic Example, R = 99

DNPS86 (1) No. 1

* * * SOLUTION FOR INPUT DATA FILE: pdes * * *

FLEXIBLE PAVEMENT STRUCTURAL DESIGN

Performance Life (yrs) 20.0
18-kip ESAL Repetitions 20019780.

Layer No.	Layer Description	Required Thickness (inches)
1	Asphalt	8.85
2	crsh stone	7.02
3	gravel	22.41
4		
5		

DESIGN FOR PROJECTED FUTURE OVERLAY(S)

	First	Second
Overlay Type	(none)	(none)
Req'd Thick (in)		
Perf Life (yrs)		
18-kip ESAL Reps		

LIFE CYCLE COSTS ($/SY)

Initial Pavement	
Construction	.00
Maintenance	.00
Salvage Value	.00
First Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00
Second Overlay	
Construction	.00
Maintenance	.00
Salvage Value	.00

Net Present Value	.00

Press Any Key to Continue ...

High Traffic Example, R = 99.9

Displacements

XP	YP	UX	UY	UZ
.00	.00	.000E+00	.000E+00	.266E-01

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = .05

Normal Strains / Shear Strains

XP	YP	EXX	EYY	EZZ	EXY	EXZ	EYZ
.00	.00	.242E-03	.242E-03	-.305E-03	.000E+00	.000E+00	.000E+00

Principal -- Strains / Shear Strains

XP	YP	PE 1	PE 2	PE 3	PSE1	PSE2	PSE3
.00	.00	.242E-03	.242E-03	-.305E-03	.547E-03	.000E+00	.547E-03

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = 5.47

Normal Stresses / Shear Stresses

XP	YP	SXX	SYY	SZZ	SXY	SXZ	SYZ
.00	.00	.194E+00	.194E+00	-.193E+01	.000E+00	.000E+00	.000E+00

Principal -- Stresses / Shear Stresses

XP	YP	PS1	PS2	PS3	PSS1	PSS2	PSS3
.00	.00	.194E+00	.194E+00	-.193E+01	.106E+01	.000E+00	.106E+01

RESULTS MENU FOR ELSYM5

LAYER = 4 Z = 27.00

ELSYM5 Solution, Medium Traffic, R = 50

Displacements

XP	YP	UX	UY	UZ
.00	.00	.000E+00	.000E+00	.264E-01

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = .05

Normal Strains / Shear Strains

XP	YP	EXX	EYY	EZZ	EXY	EXZ	EYZ
.00	.00	.242E-03	.242E-03	-.308E-03	.000E+00	.000E+00	.000E+00

Principal -- Strains / Shear Strains

XP	YP	PE 1	PE 2	PE 3	PSE1	PSE2	PSE3
.00	.00	.242E-03	.242E-03	-.308E-03	.550E-03	.000E+00	.550E-03

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = 5.30

Normal Stresses / Shear Stresses

XP	YP	SXX	SYY	SZZ	SXY	SXZ	SYZ
.00	.00	.193E+00	.193E+00	-.187E+01	.000E+00	.000E+00	.000E+00

Principal -- Stresses / Shear Stresses

XP	YP	PS1	PS2	PS3	PSS1	PSS2	PSS3
.00	.00	.193E+00	.193E+00	-.187E+01	.103E+01	.000E+00	.103E+01

RESULTS MENU FOR ELSYM5

LAYER = 4 Z = 27.50

R = 50% - High Traffic

Displacements

XP	YP	UX	UY	UZ
.00	.00	.000E+00	.000E+00	.259E-01

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = .05

		Normal Strains			Shear Strains		
XP	YP	EXX	EYY	EZZ	EXY	EXZ	EYZ
.00	.00	.235E-03	.235E-03	-.295E-03	.000E+00	.000E+00	.000E+00

		Principal -- Strains			Shear Strains		
XP	YP	PE 1	PE 2	PE 3	PSE1	PSE2	PSE3
.00	.00	.235E-03	.235E-03	-.295E-03	.530E-03	.000E+00	.530E-03

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = 5.66

		Normal Stresses			Shear Stresses		
XP	YP	SXX	SYY	SZZ	SXY	SXZ	SYZ
.00	.00	.182E+00	.182E+00	-.183E+01	.000E+00	.000E+00	.000E+00

		Principal -- Stresses			Shear Stresses		
XP	YP	PS1	PS2	PS3	PSS1	PSS2	PSS3
.00	.00	.182E+00	.182E+00	-.183E+01	.100E+01	.000E+00	.100E+01

RESULTS MENU FOR ELSYM5

LAYER = 4 Z = 27.70

R = 85%, High Traffic

Displacements

XP	YP	UX	UY	UZ
.00	.00	.000E+00	.000E+00	.233E-01

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = .05

		Normal Strains			Shear Strains		
XP	YP	EXX	EYY	EZZ	EXY	EXZ	EYZ
.00	.00	.201E-03	.201E-03	-.252E-03	.000E+00	.000E+00	.000E+00

		Principal -- Strains			Shear Strains		
XP	YP	PE 1	PE 2	PE 3	PSE1	PSE2	PSE3
.00	.00	.201E-03	.201E-03	-.252E-03	.453E-03	.000E+00	.453E-03

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = 6.60

		Normal Stresses			Shear Stresses		
XP	YP	SXX	SYY	SZZ	SXY	SXZ	SYZ
.00	.00	.133E+00	.133E+00	-.144E+01	.000E+00	.000E+00	.000E+00

		Principal -- Stresses			Shear Stresses		
XP	YP	PS1	PS2	PS3	PSS1	PSS2	PSS3
.00	.00	.133E+00	.133E+00	-.144E+01	.785E+00	.000E+00	.785E+00

RESULTS MENU FOR ELSYM5

LAYER = 4 Z = 31.10

R = 95 %, High Traffic

Displacements

XP	YP	UX	UY	UZ
.00	.00	.000E+00	.000E+00	.205E-01

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = .05

		Normal Strains			Shear Strains		
XP	YP	EXX	EYY	EZZ	EXY	EXZ	EYZ
.00	.00	.163E-03	.163E-03	-.203E-03	.000E+00	.000E+00	.000E+00
		Principal -- Strains			Shear Strains		
XP	YP	PE 1	PE 2	PE 3	PSE1	PSE2	PSE3
.00	.00	.163E-03	.163E-03	-.203E-03	.366E-03	.000E+00	.366E-03

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = 7.95

		Normal Stresses			Shear Stresses		
XP	YP	SXX	SYY	SZZ	SXY	SXZ	SYZ
.00	.00	.890E-01	.890E-01	-.109E+01	.000E+00	.000E+00	.000E+00
		Principal -- Stresses			Shear Stresses		
XP	YP	PS1	PS2	PS3	PSS1	PSS2	PSS3
.00	.00	.890E-01	.890E-01	-.109E+01	.591E+00	.000E+00	.591E+00

RESULTS MENU FOR ELSYM5

LAYER = 4 Z = 35.50

R = 99%, High Traffic

Displacements

XP	YP	UX	UY	UZ
.00	.00	.000E+00	.000E+00	.191E-01

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = .05

		Normal Strains			Shear Strains		
XP	YP	EXX	EYY	EZZ	EXY	EXZ	EYZ
.00	.00	.142E-03	.142E-03	-.177E-03	.000E+00	.000E+00	.000E+00
		Principal -- Strains			Shear Strains		
XP	YP	PE 1	PE 2	PE 3	PSE1	PSE2	PSE3
.00	.00	.142E-03	.142E-03	-.177E-03	.319E-03	.000E+00	.319E-03

RESULTS MENU FOR ELSYM5

LAYER = 1 Z = 8.85

		Normal Stresses			Shear Stresses		
XP	YP	SXX	SYY	SZZ	SXY	SXZ	SYZ
.00	.00	.696E-01	.696E-01	-.938E+00	.000E+00	.000E+00	.000E+00
		Principal -- Stresses			Shear Stresses		
XP	YP	PS1	PS2	PS3	PSS1	PSS2	PSS3
.00	.00	.696E-01	.696E-01	-.938E+00	.504E+00	.000E+00	.504E+00

RESULTS MENU FOR ELSYM5

LAYER = 4 Z = 38.30

R = 99.9%, High Traffic

NHI Course No. 13111

NATIONAL HIGHWAY INSTITUTE
FEDERAL HIGHWAY ADMINISTRATION
COURSE EVALUATION FORM

We would appreciate your thoughtful completion of all items on this evaluation form. Your comments and constructive suggestions will be carefully studied and will be of considerable value in our continuing efforts to improve our course presentations.

Course Title: Pavement Design, Principles and Practice
Location:
Date:
Your Name: optional)
Job Title:
Employer:

Please circle the number that you feel best indicates the quality or effectiveness of the item being evaluated. Circle 1 to indicate the highest rating and circle circle 5 for the lowest. Circle 0 if the item does not apply.

			vg	g	avg	p	vp
A.	Course Content						
1.	Were the goals and objectives of the course and each class session clearly defined? COMMENTS:	0	1	2	3	4	5
2.	Was the course subject matter consistent with the stated objectives: COMMENTS:	0	1	2	3	4	5
3.	Were the course objectives accomplished? COMMENTS:	0	1	2	3	4	5
4.	Has the presentation provided an adequate balance between theory and application? COMMENTS:	0	1	2	3	4	5

5. Please indicate the topics which should be added or discussed in more detail
COMMENTS:

6. Please indicate the topics which should be reduced or omitted.
COMMENTS:

B. Instructional Material and Facilities

			vg	g	avg	p	vp
1.	Were the course text and handouts used effectively in the classroom? COMMENTS:	0	1	2	3	4	5
2.	Will the course material serve as a useful reference: COMMENTS:	0	1	2	3	4	5
3.	What is your evaluation of class exercises and/or workshop problems? COMMENTS:	0	1	2	3	4	5

C. Course Instructors
A. (Name)
B. (Name)

1.	How would you evaluate the instructor's apparent familiarity with the subject matter? COMMENTS:	Inst. A.	0	1	2	3	4	5
		Inst. B.	0	1	2	3	4	5
2.	How would you evaluate the instructor's ability to convey his/her knowledge of the subject: COMMENTS:	Inst. A.	0	1	2	3	4	5
		Inst. B.	0	1	2	3	4	5
3.	Was the instructor able to stimulate interest in the subject? COMMENTS:	Inst. A.	0	1	2	3	4	5
		Inst. B.	0	1	2	3	4	5
4.	Were participants given adequate opportunity to ask questions and get satisfactory answers? COMMENTS:	Inst. A.	0	1	2	3	4	5
		Inst. B.	0	1	2	3	4	5

D. General Observations

1. What is your overall assessment of the value and significance of this course to you?

2. Do you believe there is a continuing need for this course?

3. Do you have any comments or suggestions not already covered?