PB86-199221

REPORT NO. FHWA-RD-81-154

FLEXIBLE AND COMPOSITE STRUCTURES FOR PREMIUM PAVEMENTS

Volume 1 Development of Design Procedure

H.L. von Quintus, F.N. Finn, W.R. Hudson, F.L. Roberts

NOVEMBER 1980

FINAL REPORT

This document is available to the public through
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Springfield, Virginia 22161

Prepared for:

FEDERAL HIGHWAY ADMINISTRATION
OFFICES OF RESEARCH AND DEVELOPMENT
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Technical Report Documentation Page

1. Report No. FHWA-RD-81-154	2. Government Accession No.	3. Recipient's Catalog No. PB86 199221 /AS
4. Title and Subtitle Flexible and Composite Structures for [redacted] Pavements PREMIUM Vol. 1 - Development of Design Procedure		5. Report Date November 1980
		6. Performing Organization Code
7. Author(s) F.L. Roberts H.L. Von Quintus, F.N. Finn, W.R. Hudson		8. Performing Organization Report No.
9. Performing Organization Name and Address Austin Research Engineers Inc 2600 Dellana Lane Austin, Texas 78746		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. DOT-FH-11-9348
12. Sponsoring Agency Name and Address Federal Highway Administration Office of Research Structures and Applied Mechanics Division Washington, D.C. 20590		13. Type of Report and Period Covered Final Report
		14. Sponsoring Agency Code

15. Supplementary Notes

Mr. William J. Kenis, Sr.
FHWA Contract Manager

16. Abstract

This document provides results of a detailed study to identify flexible and composite structures which will perform as zero-maintenance pavements. To accomplish this, the report identifies the major distress types which have seriously limited the maintenance-free life of flexible and composite pavements, selects analytical models for use in a structural analysis for pavement structures, and develops design criteria for each distress type. A materials review was also completed to establish materials capable of satisfying the zero-maintenance criteria. As a result of this project, a detailed design procedure has been presented which uses fatigue and transverse cracking, rutting, roughness, and environmental factors of temperature and moisture, to establish thickness and material requirements for the design and construction of zero-maintenance pavements.

17. Key Words Zero-Maintenance, flexible pavements, composite pavements, fatigue cracking, rutting, low-temperature cracking, reflection cracking		18. Distribution Statement No Restrictions	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 311	22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

PREFACE

This is the first of two documents describing development of new design and construction procedures for "premium" flexible and composite pavements. This report documents all development work of the design procedure and criteria. It identifies the major distress manifestations by pavement type and presents the selected analytical models for each pavement and distress type. Cost analysis models were also reviewed for the evaluation of initial construction, maintenance, and user costs for flexible and composite pavement structures. This report also identifies the materials and pavement cross-sections which have the greatest potential to meet the zero-maintenance requirements for premium pavements. Volume 2 presents the design procedure and materials and construction specifications to ensure that the pavement configuration will meet the zero-maintenance requirement.

The work presented in this report was accomplished by a research team including Harold L. Von Quintus, Fred N. Finn, W. Ronald Hudson, Freddy L. Roberts, and Lee J. Ream with technical support from B. Frank McCullough, Professor Carl L. Monismith and Thomas W. Kennedy, and many others.

Support for the project was provided by the Federal Highway Administration, Office of Research and Development, Contract No. DOT-FH-11-9348. We are grateful for the technical coordination provided by Mr. William J. Kenis, Sr., Contract Manager.

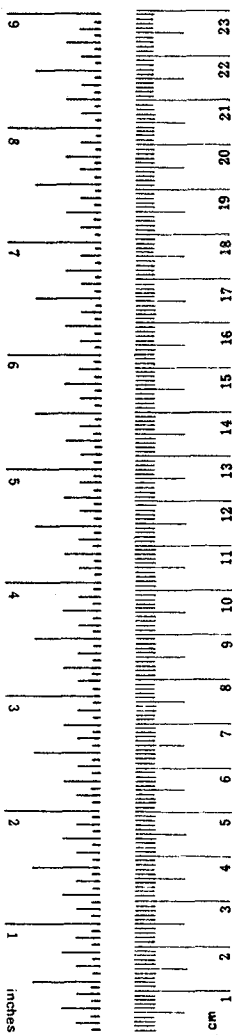
Austin Research Engineers Inc

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

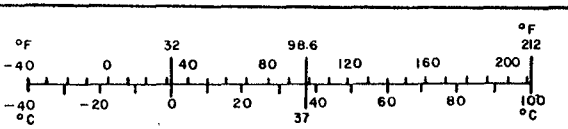
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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CHAPTER 1. INTRODUCTION

This is the first volume of a two volume report emanating from FHWA Research Project 5E, entitled "Premium Pavements for Zero-Maintenance". This volume presents background studies while Volume II, "Design Manual" presents a complete design method in catalogue form.

This document describes the development of design procedures for flexible and composite pavements to withstand heavy traffic volumes without requiring major structural maintenance for 20-30 years. Similar design procedures for rigid pavements have already been presented by Darter et. al., documented in References 1[1], 2[2], and 3[3]. The term structural maintenance refers to such maintenance activities as reconstruction, overlays, recycling, seal coats, patching, grinding and crack sealing. Other activities such as painting or striping, mowing, lane widening and guard rail repair are not considered to be structural maintenance in this report, which is consistent with the definition set forth by the Federal Highway Administration and used in the rigid pavement procedure (Ref. 3).

It should be pointed out here that there is very little observed data on the performance of flexible or composite pavements which have lasted 20 or more years under very heavy traffic. The work presented here is a significant extrapolation of observed performance, but is based on theoretical concepts and modern improvements in material characteristics and construction quality control. The best key to premium pavements is the conscientious application by a knowledgeable pavement engineer of the concepts and criteria developed herein. Ultimate proof and verification of the methodology must come from long-term observations and trial usage.

BACKGROUND

Many highways are subject to large traffic volumes which, when combined with the effects of age, environment, and material variability, cause deterioration and premature failure of pavement structures.

---

[1]Darter, M.F. and E.J. Barenberg, "Zero-Maintenance Pavement: Results of Field Studies on the Performance Requirements and Capabilities of Conventional Pavement Systems", Report No. FHWA-RD-76-105, Federal Highway Administration, April 1976.

[2]Darter, M.I., "Design of Zero-Maintenance Plain Jointed Concrete Pavement, Vol. 1 - Development of Design Procedures", Report ZM-2-77, Federal Highway Administration, June 8, 1977.

[3]Darter, M.J. and E.J. Barenberg, "Design of Zero-Maintenance Plain Jointed Concrete Pavement, Vol. II - Design Manual", Report No. FHWA-RD-77-112, Federal Highway Administration, June 1977.

Extending the life of these pavements sometimes requires considerable maintenance, rehabilitation, or complete reconstruction, which results in delays, increased user and vehicle operating costs and other costs. In view of these concerns, the Federal Highways Administration has undertaken a research program to develop guidelines for higher quality or "zero-maintenance pavements." A "zero-maintenance pavement" is defined as one designed for specific conditions which will perform essentially free of structural maintenance for 20 years and which will require a minimum of maintenance for an additional 10 to 20 years.

In developing criteria for zero-maintenance pavements, it is important that the definitions of other terms, such as distress, response, and other descriptors of pavement performance be compatible with other FHWA research projects to minimize the variation in interpretation and any misunderstandings between engineers. Therefore, the definitions developed under the FHWA project "Models and Significant Material Properties for Predicting Distress in Zero-Maintenance Pavements" (Ref. 4[1]) are accepted in this project, and are presented in Appendix A for ready reference.

OBJECTIVE

The main objective of this report is to provide background and to develop procedures for the design and construction of flexible and composite pavements that satisfy the criteria set forth for zero-maintenance. This requires the development of methodologies, both in design and construction, that will minimize distress manifestations and deterioration from large traffic volumes over long periods of time and under various environments. The methods themselves are presented in Volume II (Ref. 5[2]).

ZERO-MAINTENANCE APPROACH

There are a number of conventional pavement design procedures which have been adequate for designing pavements. An extrapolation of these conventional procedures for large traffic volumes would seem to indicate that an increase in structural thickness would satisfy the zero-maintenance criteria. Experience, on the other hand, indicates that simply

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[1]Rauhut, J.B., F.L. Roberts, and T.W. Kennedy, "Models and Significant Material Properties for Predicting Distress in Zero-Maintenance Pavements", Draft Report No. FHWA-RD-78-84, Federal Highway Administration, June 1978.

[2]Von Quintus, H.L., F.N. Finn, W.R. Hudson, and F.L. Roberts, "Flexible and Composite Structures for Premium Pavements - Vol. 2, Design Manual", Draft Report No. FHWA-RD-79, Federal Highway Administration, December 1979.

increasing thickness will not reliably provide long-term maintenance-free pavements. Other factors not controlled by thickness, such as materials durability, environmental variables, and/or construction variability, can be dominant factors in reducing the maintenance-free life of a pavement structure. Therefore, in order to develop zero-maintenance flexible and composite pavement design and construction procedures, information from field and laboratory studies, analytical evaluations and prior research findings was compiled and subject to a comprehensive examination.

A thorough field survey of flexible and composite pavements has been completed by Darter and Barenberg in developing the rigid pavement design procedure (Ref. 1). This study along with a detailed pavement survey and interview completed by Smith, et al., (Ref. 6[1]) and the findings presented by Hoff, et al., (Ref. 7[2]), indicate that few pavements will meet the zero-maintenance criteria. Therefore, a thorough and extensive search for existing pavements which meet the zero-maintenance criteria was not attempted; rather, available performance information (for example, Refs. 8[3], 9[4], 10[5], 11[6], 12[7], 13[8], 14[9], and 15[10]) was utilized and combined with the most current analytical models and design and construction practices to develop pavement configurations which will function as zero-maintenance pavements. This approach is illustrated in Figure 1.1.

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[1]Smith, W. and F. Finn, et al., "Bayesian Analysis Methodology for Verifying Recommendations to Minimize Asphalt Pavement Distress", NCHRP Project 9-4A, National Cooperative Research Program, January 1977.

[2]Hoff, G.C., L.N. Godwin, K.L. Sancian, A.D. Buck, T.B. Husbands, and K. Mather, "Identification of Candidate Zero-Maintenance Paving Materials", Report No. FHWA-RD-77-109, Federal Highway Administration, May 1977.

[3]Barker, R.F., "New Jersey Composite Pavement Project", HRR No. 434, Highway Research Board, 1973.

[4]Ryell, J. and J.T. Corkill, "Long Term Performance of an Experimental Composite Pavement", HRR No. 434, Highway Research Board, 1973.

[5]Kennedy, T.W., R. Haas, P. Smith, G.A. Kennepohl, and E.T. Hignell, "An Engineering Evaluation of Sulphur Asphalt Mixtures", Paper prepared for presentation at the 56th Annual Meeting of the Transportation Research Board, January 1977.

[6]Morris, G.R. and C.H. McDonald, "Asphalt-Rubber Stress Absorbing Membranes - Field Performance and State-of-the-Art", TRR No. 595, Transportation Research Board, 1976.

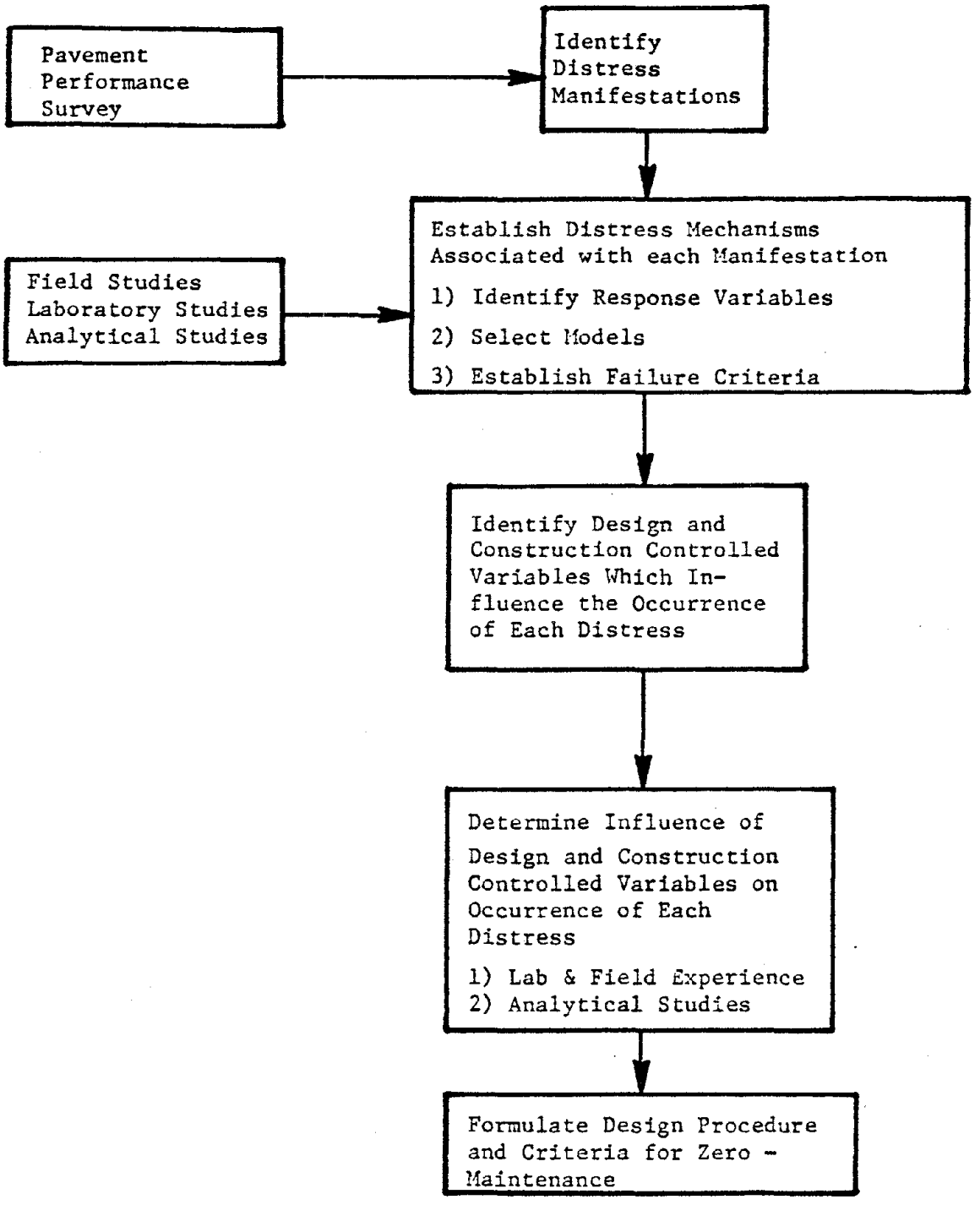

Figure 1.1. Basic approach used for formulating the design procedure for premium flexible and composite pavements for zero-maintenance.

Results of this study include: (1) identifying the types and causes of distress manifestations and deterioration, (2) selecting materials which maximize pavement performance in relation to each distress manifestation, (3) selecting analytical and distress models capable of predicting pavement performance as a function of material and environmental variables, (4) establishing design criteria for a twenty year maintenance-free life and (5) selecting environmental factors which can limit the maintenance-free life of a pavement structure. The report incorporates the above into a detailed design procedure which considers fatigue and transverse cracking, rutting and roughness, as well as the environmental factors of frost, temperature and moisture. The methodology is presented in this volume but the results of the method are presented in the form of a design cost analysis, a user oriented volume (Volume II).

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[7]Coetzee, N.F. and C.C. Monismith, "An Analytical Study of the Applicability of a Rubber Asphalt Membrane to Minimize Reflection Cracking in Asphalt Concrete Overlay Pavements", Department of Civil Engineering, University of Califronia, Berkeley, June 1978.

[8]Barksdale, R. and J. Miller, "Development of Equipment and Techniques for Evaluating Fatigue and Rutting Characteristics of Asphalt Concrete Mixes", GDOT Research Project No. 7305, Georgia Institute of Technology, June 1977.

[9]"Performance Study Report on Insulation Board (Polystyrene)", Subcommittee on Development Evaluation and Recommendations of New Highway Materials, AASHO-ARBA, 1970.

[10]"Bituminous Aggregate Base Course: Survey of State Practices", HRB Special Report No. 177, Highway Research Board, 1971.

CHAPTER 2. PAVEMENT DISTRESS

In order to develop premium pavements, the distresses which limit the maintenance-free life of flexible and composite pavements must first be identified. Once these are identified and the mechanisms causing the distress are established, materials can be selected and designed to produce the optimum performance required of premium pavements.

The identification of distresses was based on information gathered from field observations, literature review, maintenance activities, and experience of the staff and consultants. For example, this includes such information as documented in References 16[1], 17[2], 18[3], 19[4], 20[5], 21[6], 22[7], 23[8], 24[9], 25[10], and 26[11].

The major distress manifestations can be divided into three basic categories: 1) fracture, 2) distortion, and 3) disintegration, all defined in Appendix A and summarized in Table 2.1. Each distress manifestation is a result of two kinds of pavement damage: traffic and nontraffic associated damages. Traffic associated damage to the pavement is cumulative. The rate of its accumulation can vary according to material properties, and changes in distribution and intensity of traffic and environmental loads. Since both environmental effects and material properties vary with seasonal temperature and moisture changes, the dominant damaging effect depends on the moisture and temperature sensitivity of the material. Other distress manifestations not associated with traffic occur under the direct influence of temperature and moisture.

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[1]Darter, M.I., E.J. Barenberg, and J.S. Sawan, "Maintenance-Free Life of Heavily Trafficked Flexible Pavements", TRB No. 602, Transportation Research Board, 1976.

[2]Butler, B.C., Jr., "Economic Analysis of Roadway Occupancy for Freeway Pavement Maintenance and Rehabilitation", Report No. FHWA-RD-76-14, Federal Highway Administration, 1974.

[3]MCKeen, R.G., "Design and Construction of Airport Pavements on Expansive Soils", Report No. FAA-RD-76-66, Federal Aviation Administration, U.S. Department of Transportation, June 1976.

[4]Hudson, W.R., F.N. Finn, B.F. McCullough, K. Nair, and B.A. Vallerga, "Systems Approach to Pavement Design, System Formulation, Report NCHRP 1-10, Materials Research and Development, Inc., Highway Research Board, March 1968.

[5]Eaton, R.A., "CRREL Full-Depth Test Sections: Performance During First Three Winters", HRB Record No. 572, Highway Research Board, 1976.

TABLE 2.1. PAVEMENT DISTRESS MANIFESTATION FOR FLEXIBLE AND COMPOSITE PAVEMENTS

Distress Category		
Fracture	Distortion	Disintegration
Fatigue Cracking	Rutting	Raveling
Thermal Cracking	Differential Frost Heave	Reduced Skid Resistance
Reflection Cracking	Differential Compaction-Swell	Freeze/Thaw Degradation
Slippage Cracking	Settlement or Consolidation	Spalling at Shoulders
	Shoving	
	Corrugations	
	Blow-ups	

FRACTURE

The fracture distress category represents the most common types of distress in all pavements and has received the most attention in terms of laboratory and research studies. Fatigue and low-temperature cracking are the two distress types which are the most common in flexible and composite pavements.

Fatigue cracking, defined in Appendix A, is considered to be a major structural distress limiting the maintenance-free life of a pavement structure. This type of cracking is a progressive localized permanent structural change occurring in bound materials subjected to fluctuating stresses and strains. Although fatigue cracking can occur in any bound material in the pavement structure, these cracks normally initiate at the bottom of the asphalt concrete or portland cement concrete layer and propagate to the surface with continued traffic applications, as shown in Figures 2.1 and 2.2. The primary pavement response or significant indicator of fatigue cracking is the maximum tensile strain at the bottom of the asphalt concrete layer for flexible pavements and maximum tensile stress at the bottom of the portland cement concrete layer for composite pavements. The magnitude of these pavement responses or the time and rate at which fatigue cracking occurs are dependent on environmental factors (temperature, temperature gradient, moisture, etc.), wheel load variables (load magnitude, tire pressure, rate of wheel load applications, etc.), material properties (stiffness,

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[6]"Pavement Rehabilitation, Materials, and Techniques", NCHRP Synthesis No. 9, National Cooperative Highway Research Program, Highway Research Board, 1972.

[7]Scott, J.A.N., "Adhesion and Disbonding Mechanisms of Asphalt Used in Highway Construction and Maintenance", Paper presented at the 1978 Annual Meeting of the Association of Asphalt Paving Technologists, February 1978.

[8]"Skid Resistance", NCHRP Synthesis No. 14, National Cooperative Highway Research Program, Highway Research Board, 1972.

[9]McLaughlin, A.L., "Reflection Cracking of Bituminous Overlays for Airport Pavements, A State of the Art", FAA-RD-79-57, Federal Aviation Administration, May 1979.

[10]Carpenter, S.H. and R.L. Lytton, "Thermal Pavement Cracking in West Texas", FHWATX77-18-4F, Texas Transportation Institute and State Department of Highways and Public Transportation, October 1977.

[11]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 No. 128, National Cooperative Highway Research Program, Highway Research Board, 1972.

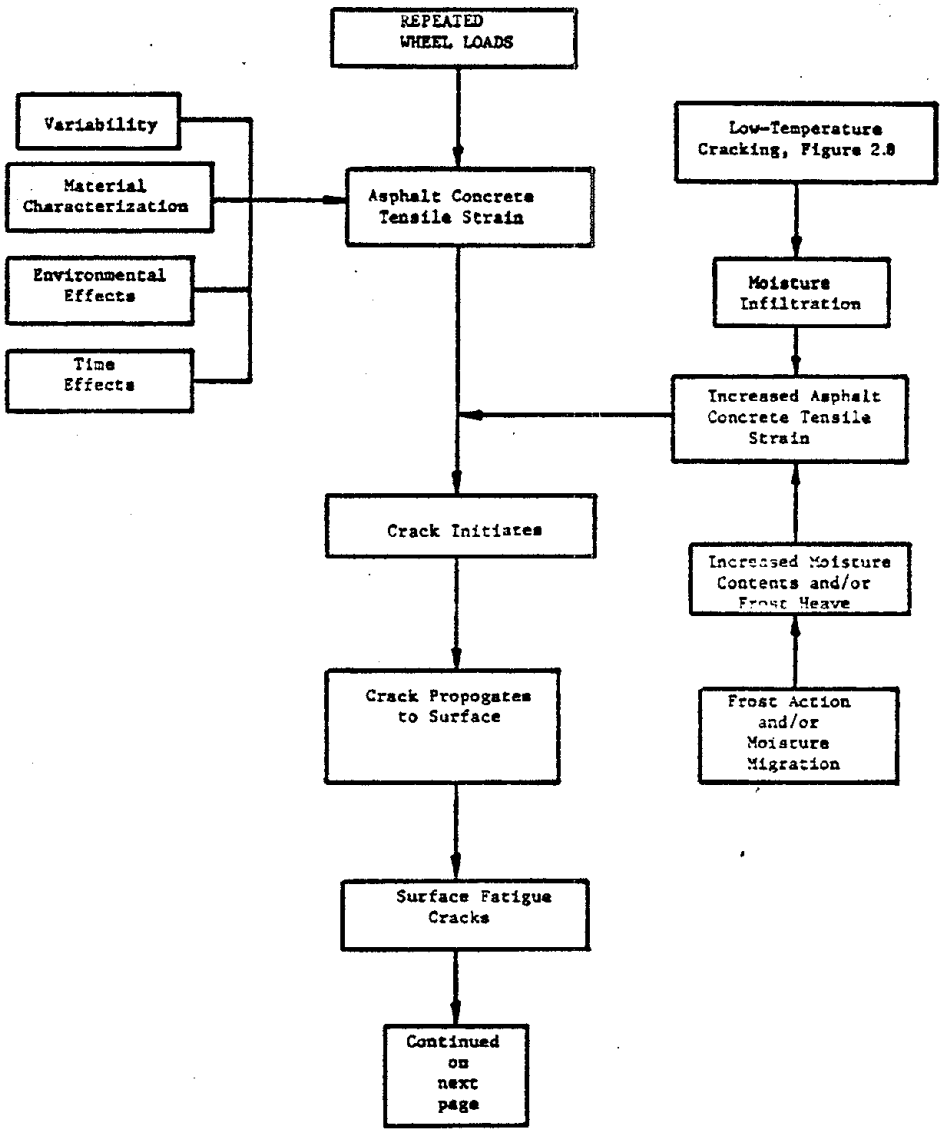

Figure 2.1. Fatigue cracking development in an asphalt concrete pavement structure.

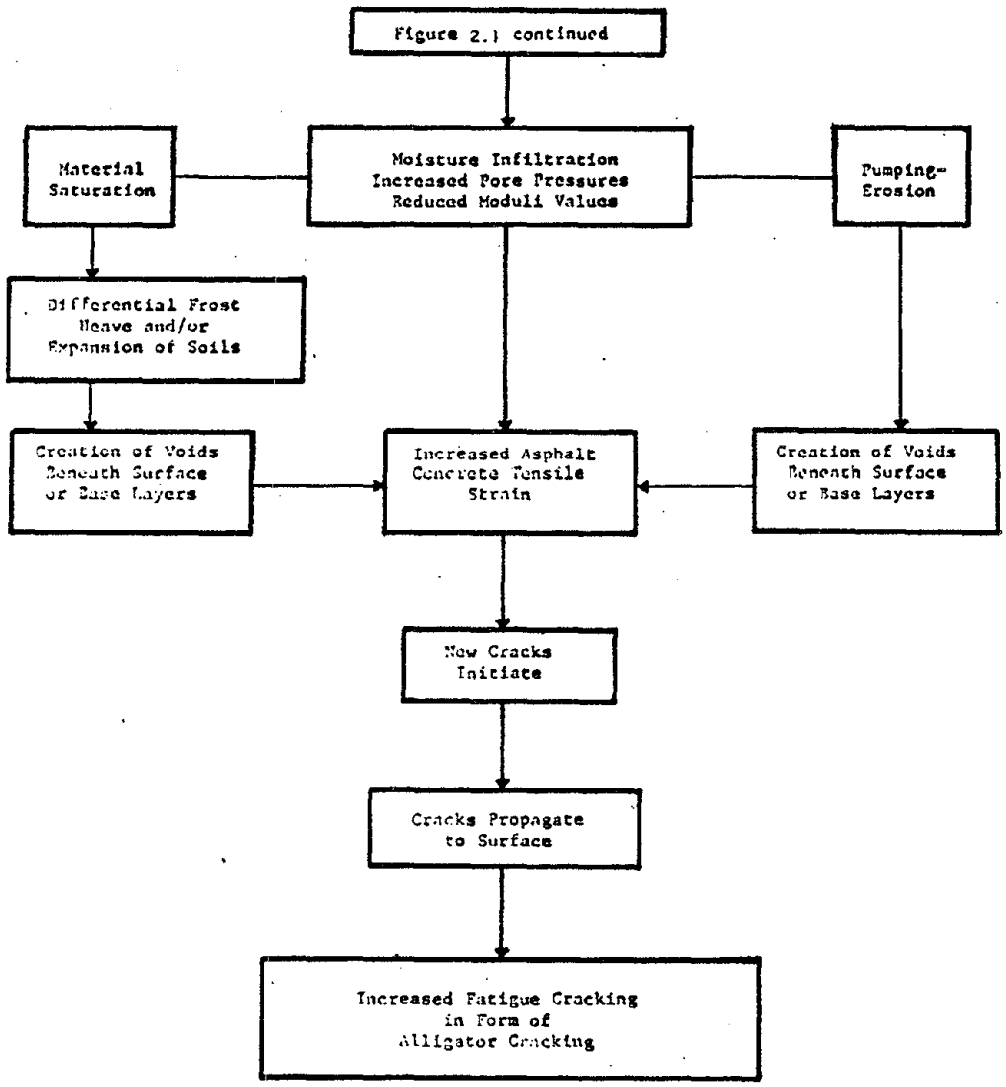

Figure 2.1. Fatigue cracking development in an asphalt concrete pavement structure (continued).

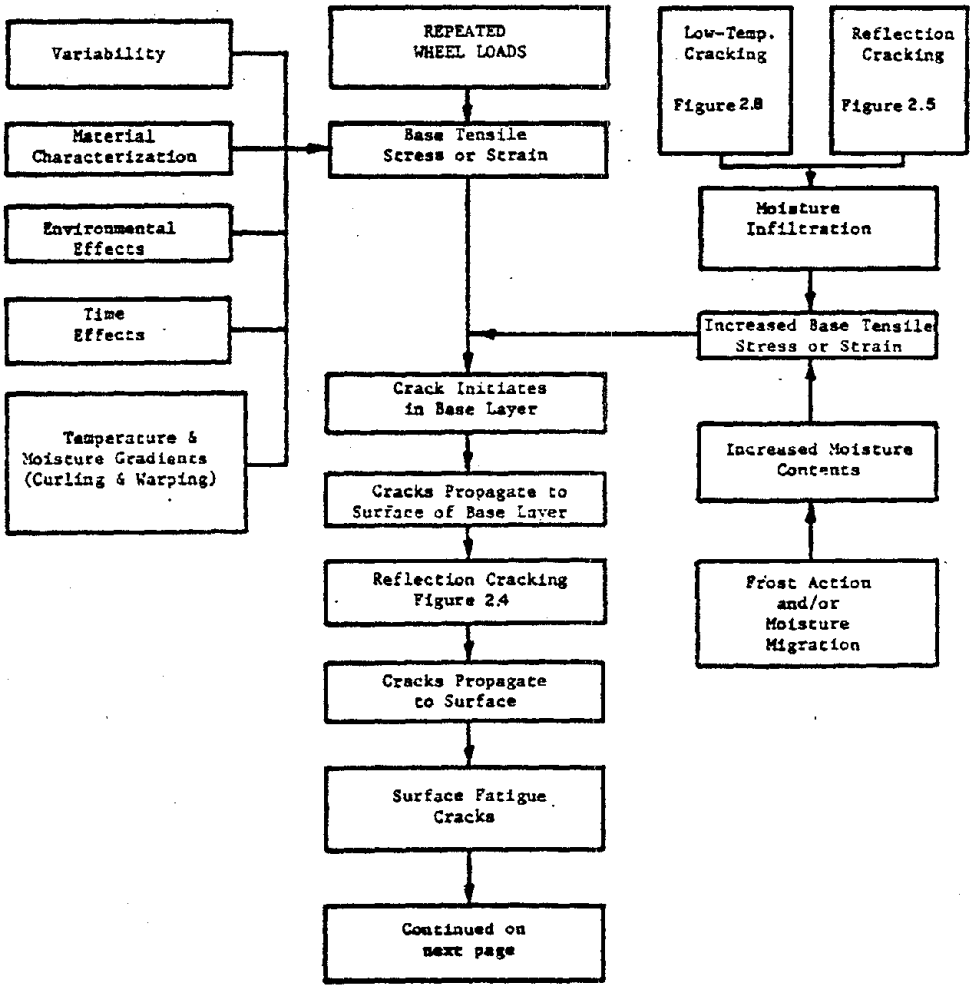

Figure 2.2. Fatigue cracking development in a composite pavement structure.

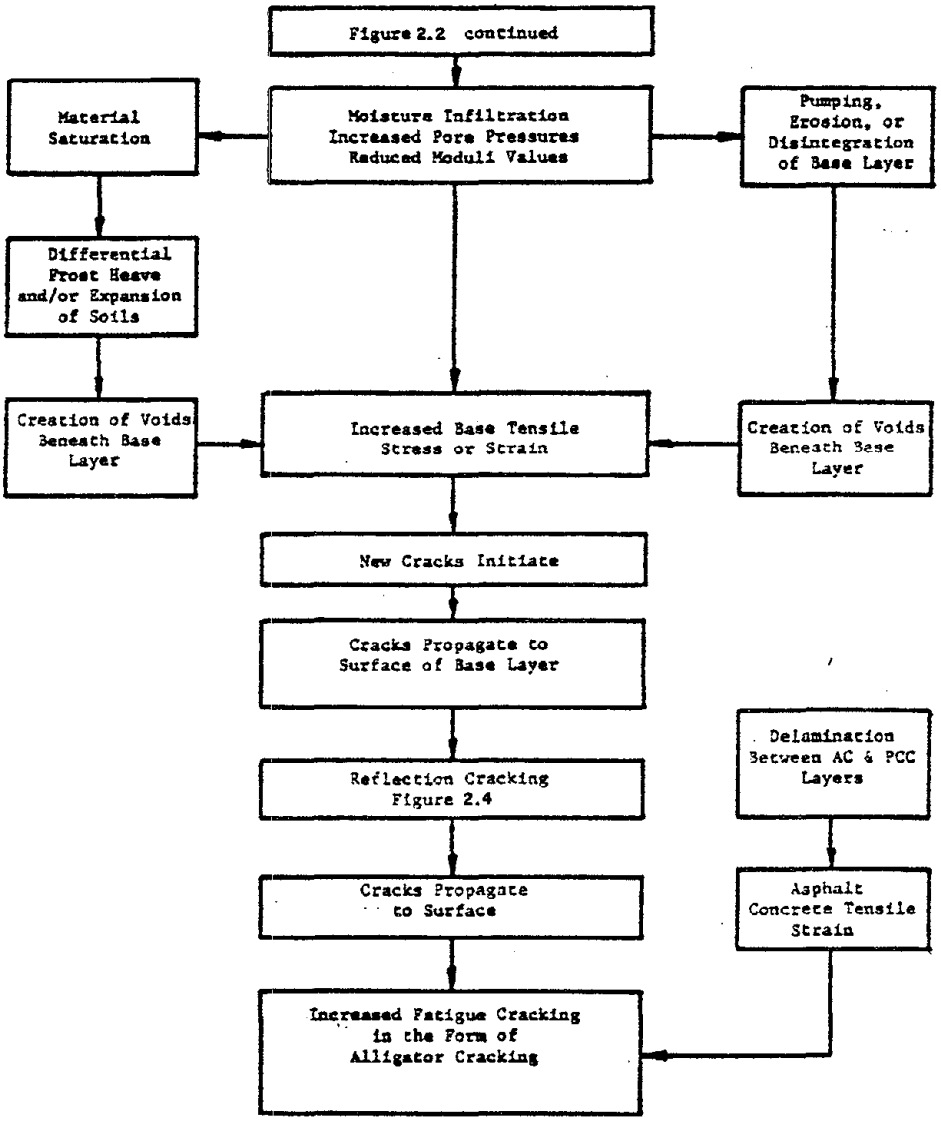

Figure 2.2. Fatigue cracking development in a composite pavement structure (continued).

aging and fatigue characteristics, Poisson's ratio, etc.) and boundary conditions (protected corner, free edge, interior loads, etc.). This type of cracking results in the structural disintegration of the pavement structure, and usually requires extensive structural maintenance.

The non-traffic associated fracture distress that is of special concern is contraction or low-temperature cracks. This type of cracking is very common to the northern United States and presents a serious problem, in that these cracks indirectly affect the serviceability of the pavement structure. Low-temperature cracking permits the intrusion of water which causes stripping, ravelling and pumping of the fine fraction of base material. Also, during the winter when the cracks are wide, solution can cause localized thawing resulting in a depression adjacent to the cracks or, in other cases, the water can freeze to form ice lenses producing upward lipping of crack edges. In either case, the final result is a decrease in pavement serviceability or increase in maintenance cost. Low-temperature cracking results when tensile stresses, caused by temperature variations or low-temperatures, exceed the materials fracture strength, as illustrated in Figure 2.3. The rate at which this type of cracking occurs depends on the material properties (fracture strength, stiffness, thermal coefficient, etc.) and environmental factors (temperature, rate of temperature change, etc.).

The other distress manifestation, reflection cracking, is common to all parts of the U.S. and presents a particular problem, since it is a result of a combination of fatigue, thermal and moisture effects. The basic mechanisms leading to the development of reflection cracks are horizontal and differential vertical movements between different layers at a discontinuity (joint or crack) in the pavement structure (Figure 2.4). Some of the distress mechanisms which cause reflection cracks are summarized in Table 2.2. Horizontal induced reflection cracks are generally a result of the expansion and contraction of two layers due to a temperature differential and are caused when the tensile stress exceeds the fracture strength. Differential vertical induced cracks are primarily load associated, and are a result of a shear-fatigue phenomenon. In most cases, it is very difficult to determine which cracks have primarily resulted from horizontal or differential vertical movements. Treybig, et al., (Ref. 27[1]) have studied this distress manifestation in great detail and have presented mechanisms and material properties which are significantly related to reflection cracking.

Initially, neither reflection and low-temperature cracks are not very obvious, nor do they require maintenance. But, as the pavement experiences more traffic applications and lower temperatures, these

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[1]Treybig, H.J., B.F. McCullough, P. Smith, and H. Von Quintus, "Overlay Design and Reflections Cracking Analysis for Rigid Pavements, Volume 1, Development of New Design Criteria", Final Report No. FHWA-RD-77-66, Federal Highway Administration, January 1978.

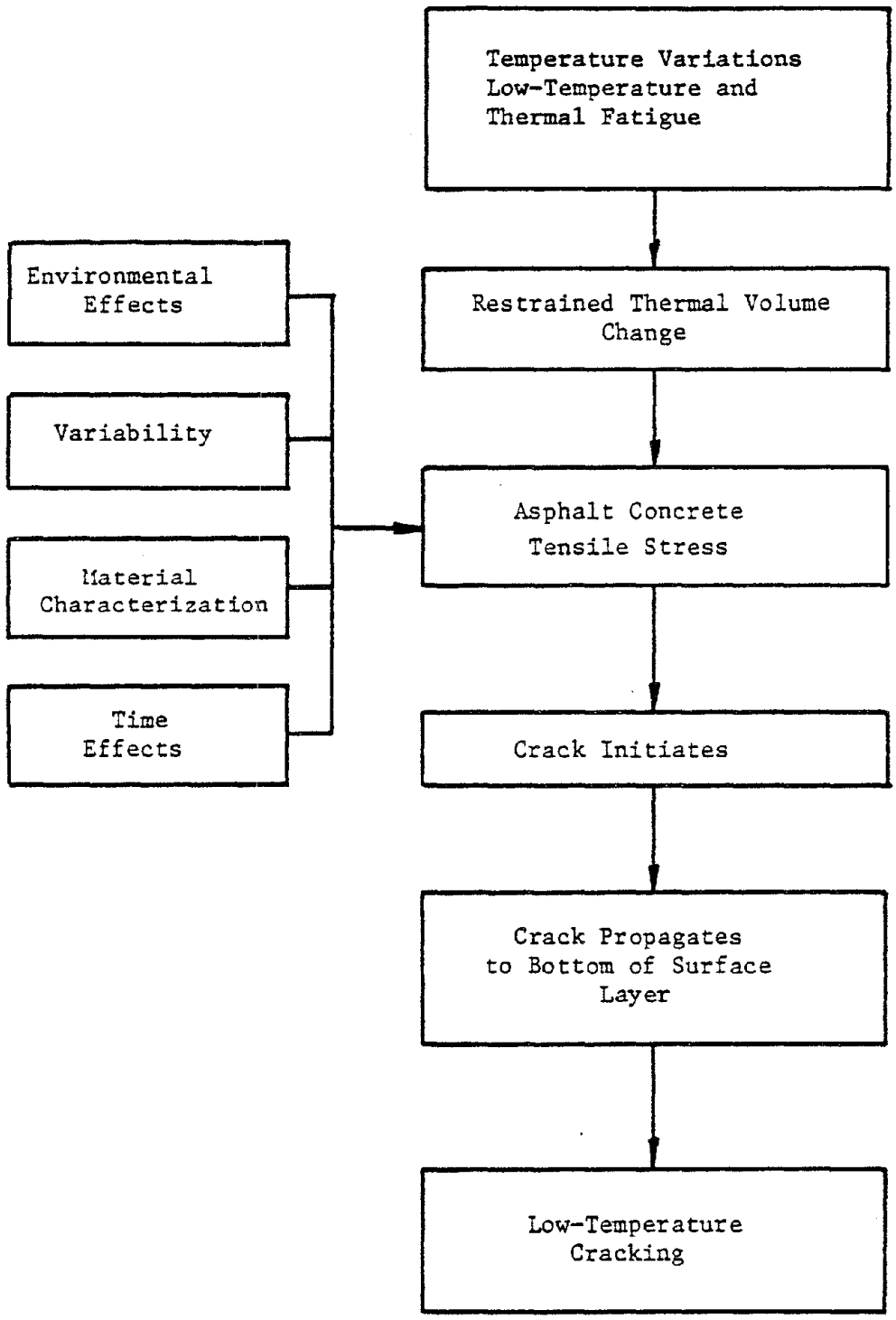

Figure 2.3. Low temperature cracking development in asphalt concrete.

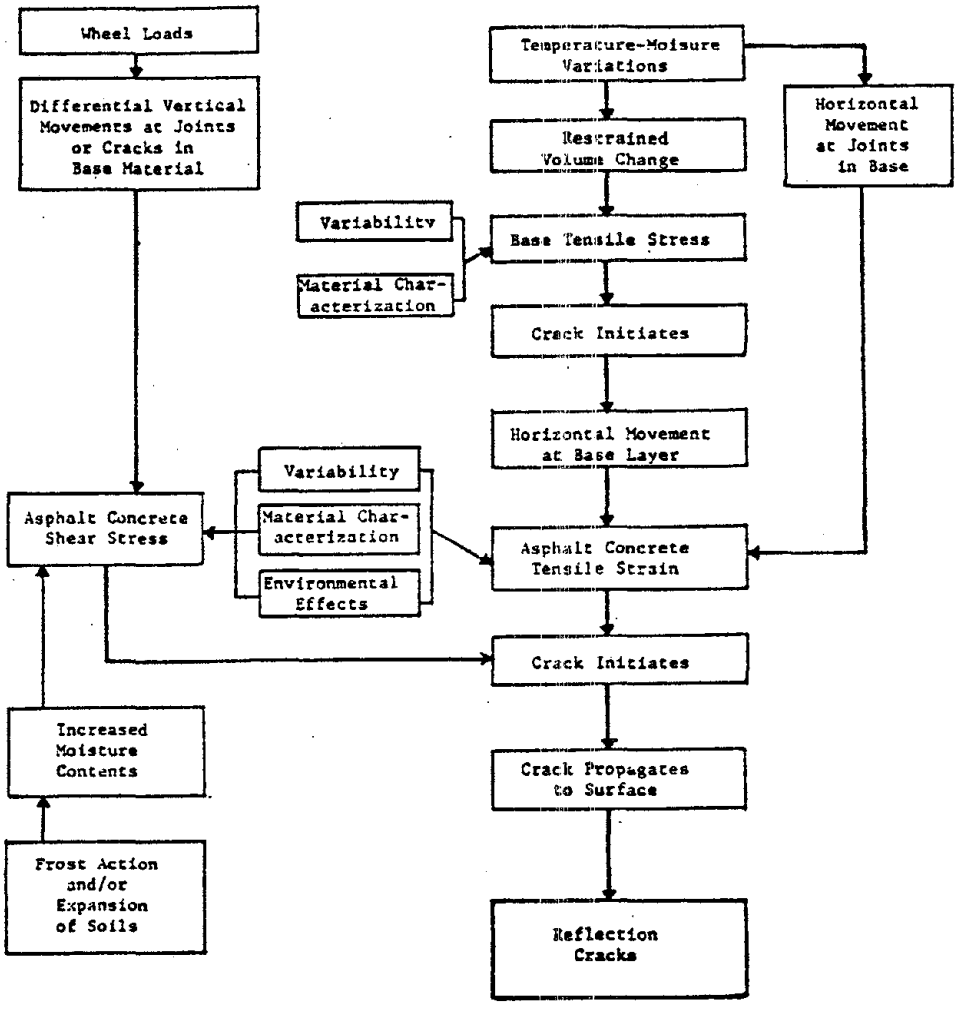

Figure 2.4. Reflection cracking development in a composite pavement structure.

TABLE 2.2. SIGNIFICANT DISTRESS OF THE RIGID LAYER IN COMPOSITE PAVEMENTS WHICH MAY LEAD TO REFLECTION CRACKING IN THE ASPHALT CONCRETE SURFACE LAYER

Distress Category	Rigid Layer Type	
	Jointed Plain or Jointed Reinforced Concrete	Continuous Reinforced Concrete
Fracture	Low Temperature Cracking Shrinkage Cracking Fatigue Cracking	Low Temperature Cracking Shrinkage Cracking Fatigue Cracking Steel Rupture
Distortion	Joint or Crack Faulting	Punchouts
Disentigration	D-Cracking Joint or Crack Spalling	Crack Spalling

cracks will widen. As this happens, water and foreign materials will enter the pavement structure causing raveling and spalling to occur, thus decreasing the structural capacity. As this erosion continues, these cracks produce an unsightly appearance and eventually will become sufficiently wide and rough enough to require structural maintenance.

Slippage cracking, unlike fatigue, low-temperature and reflection cracking, is a result of large horizontal forces applied to the pavement structure and/or delamination between two adjacent layers. This type of cracking is more common on airfield pavements and at intersections where braking or acceleration forces are applied to the pavement than on highway type pavements.

DISTORTION

Distortion is related to surface elevation changes and can have a most definite effect on pavement roughness. Rutting is the distress which is the most common in this category and is generally considered to be one of the most important distress manifestations, along with fatigue cracking, which leads to the deterioration of a pavement structure. However, frost heave, expansive soils, and consolidation present difficult obstacles to alter or minimize their occurrences over a 20-year period.

Rutting is a surface depression which results from plastic flow or creep caused by repeated traffic loadings or by few excessive loads, as illustrated in Figures 2.5 and 2.6. The magnitude or rate at which rutting occurs is dependent on environmental factors (temperature, moisture, etc.), wheel load variables (load magnitude, tire pressure, rate of wheel load applications, etc.), and material properties (stiffness, creep, aging characteristics, etc.). The capacity of the pavement to resist rutting by consolidation under repeated loads or by excessive flow or creep is related to several factors: (1) the vertical compressive strain in the top of the subgrade, (2) the radial stress at the bottom of an unbound base course (Ref. 28[1]), and (3) the vertical compressive stress in each pavement layer. Significant amounts of rutting can lead to major structural failures and hydroplaning potential, which require structural maintenance.

Non-traffic associated distress modes of special concern are distortions from differential heave caused by frost action or expansive soils. These types of distress are primarily caused by moisture changes, shrinkage, and differential settlement caused by consolidation in the subgrade, each of which are very important for premium pavements (Figures 2.7 and 2.8).

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[1]Brown, S.F. and P.S. Pell, "A Fundamental Structural Design Procedure for Flexible Pavements", Proceedings, Third International Conference on Structural Design of Asphalt Pavements, 1972.

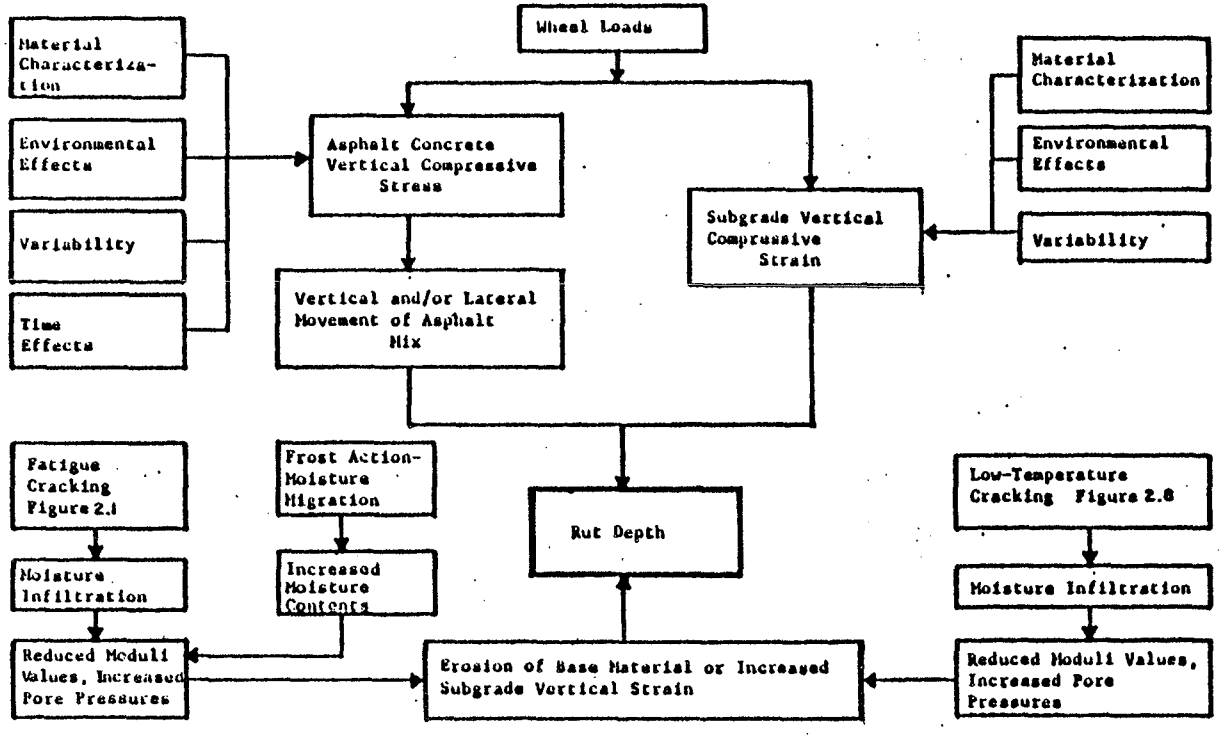

Figure 2.5. Rutting development in an asphalt concrete pavement structure.

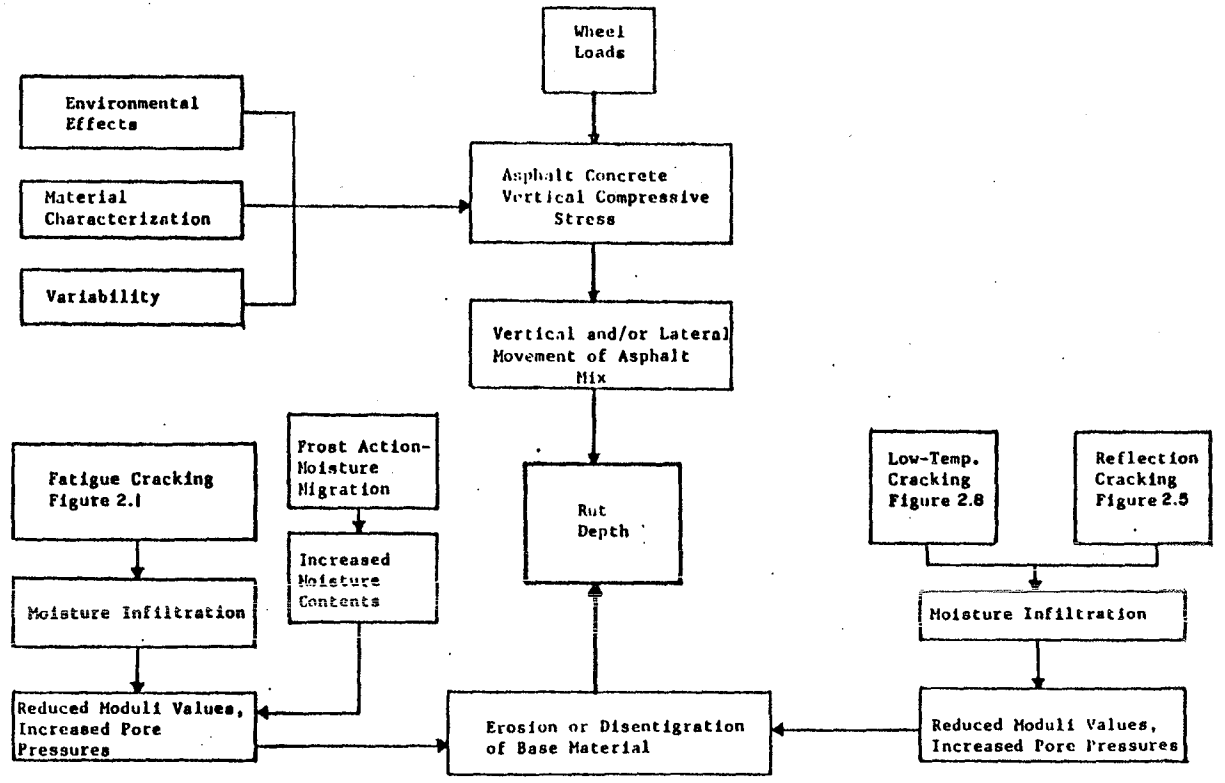

Figure 2.6. Rutting development in a composite pavement structure.

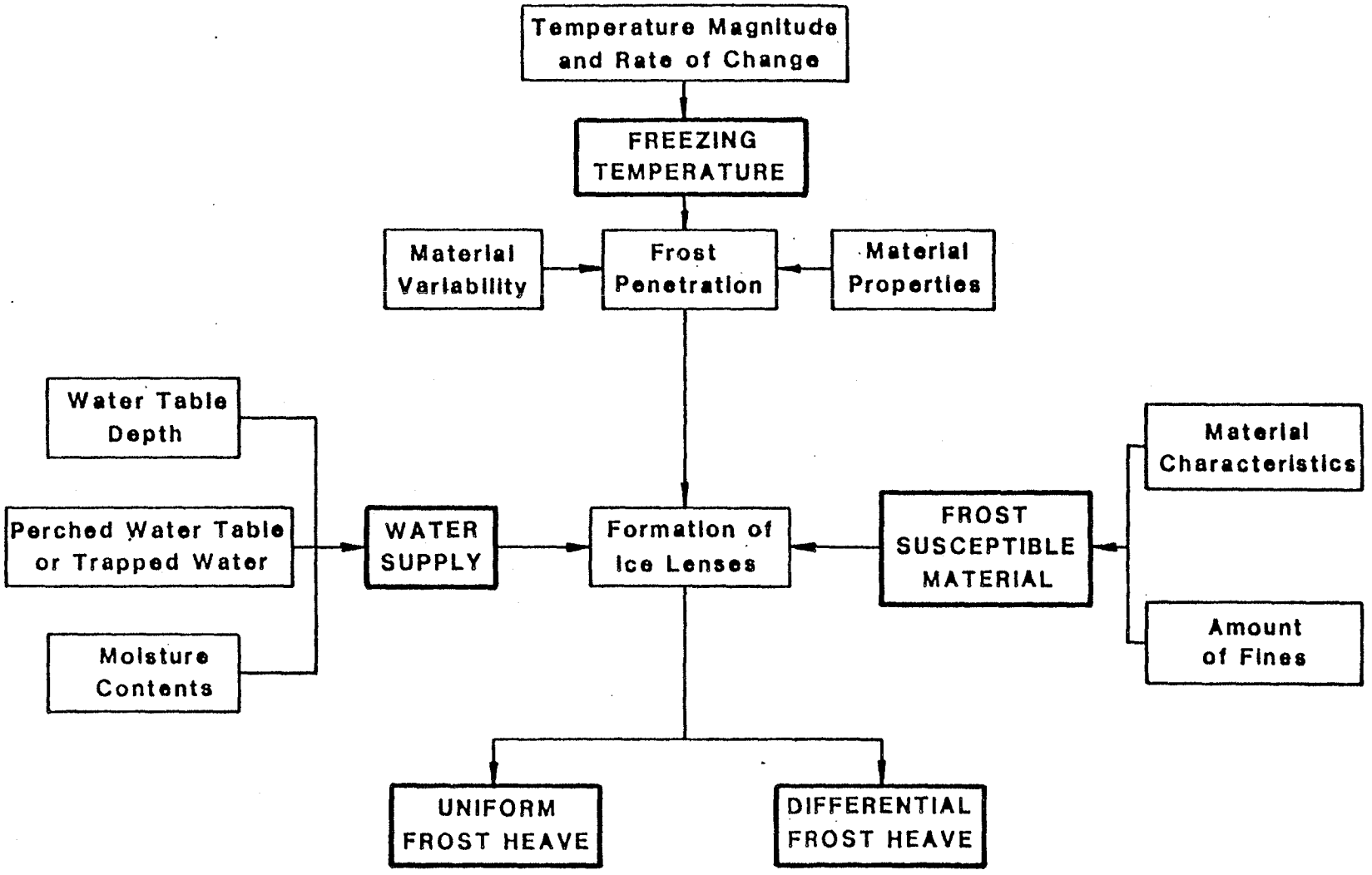

Figure 2.7. Frost heave in a flexible or composite pavement structure.

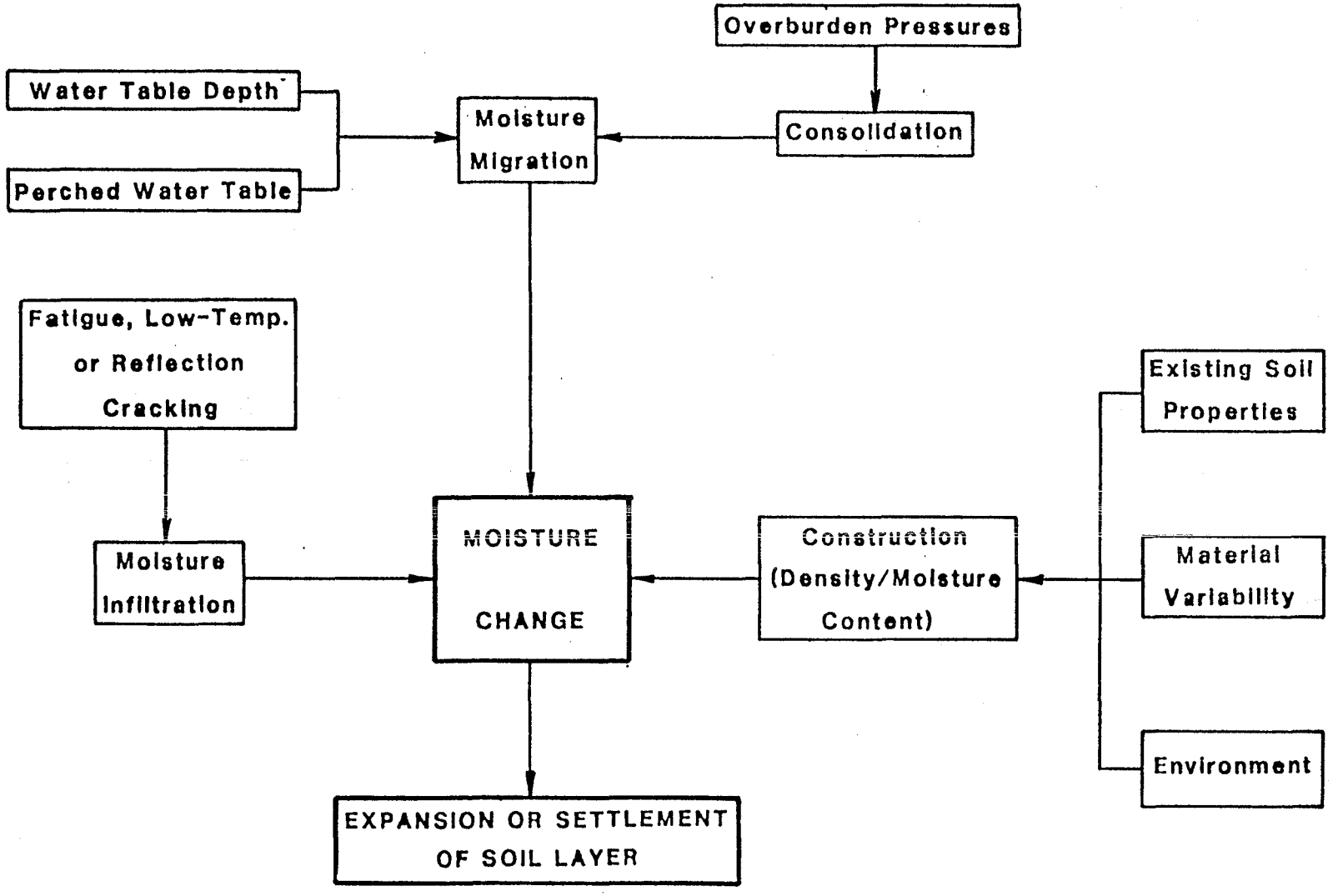

Figure 2.8. Soil expansion or settlement in flexible and composite pavement structures.

Distortion caused by frost heave, as shown in Figure 2.9, (Ref. 29[1]) often is considered a transient manifestation of distress that produces roughness only in winter, sometimes leaving residual irregularities after thawing is complete which become more severe after several years of seasonal freezing. The damage caused from frost action consists of bumps or waves and random cracking in flexible pavements and tilted or broken slabs in rigid pavements. These detrimental effects are a result of differential frost heave (ice segregation), differential loss of strength and load-carrying capacity of the pavement structure, and restriction of drainage by the frozen soil as illustrated in Figure 2.10 (Ref. 30[2]). These conditions occur because the growth of ice lenses partially remolds the soil, decreasing densities and increasing moisture contents. It has also been found that repeated freeze-thaw cycles can be very damaging to fine-grained soils (Ref. 31[3]). The degree to which a soil loses strength during a frost-melting period and the length of the period during which the strength of the soil is reduced, as shown in Figure 2.11 (Ref. 32[4]) depends on the soil type, temperature conditions during freezing-and-thawing periods, the amount and type of traffic during the frost-melting periods, seasonal moisture supply, and drainage conditions. Normally, the greatest amount of pavement distress will occur when cold winters are associated with high levels of precipitation.

Moisture changes can occur independently of temperatures, although to a large extent moisture migrations and detrimental changes in moisture content can take place under the influence of both seasonal and short-term temperature gradients and freeze/thaw cycles. Shrinkage or expansion of supporting layers (Figure 2.8) can cause reflection cracking that is induced by falling temperatures as well as desiccation or swelling soils. Differential frost heave and subsequent settlements can cause severe pavement distortion resulting in premature load associated cracking. Generally, the rate of deterioration fluctuates throughout the year, especially during the spring thaw period when the damage increases to much higher rates.

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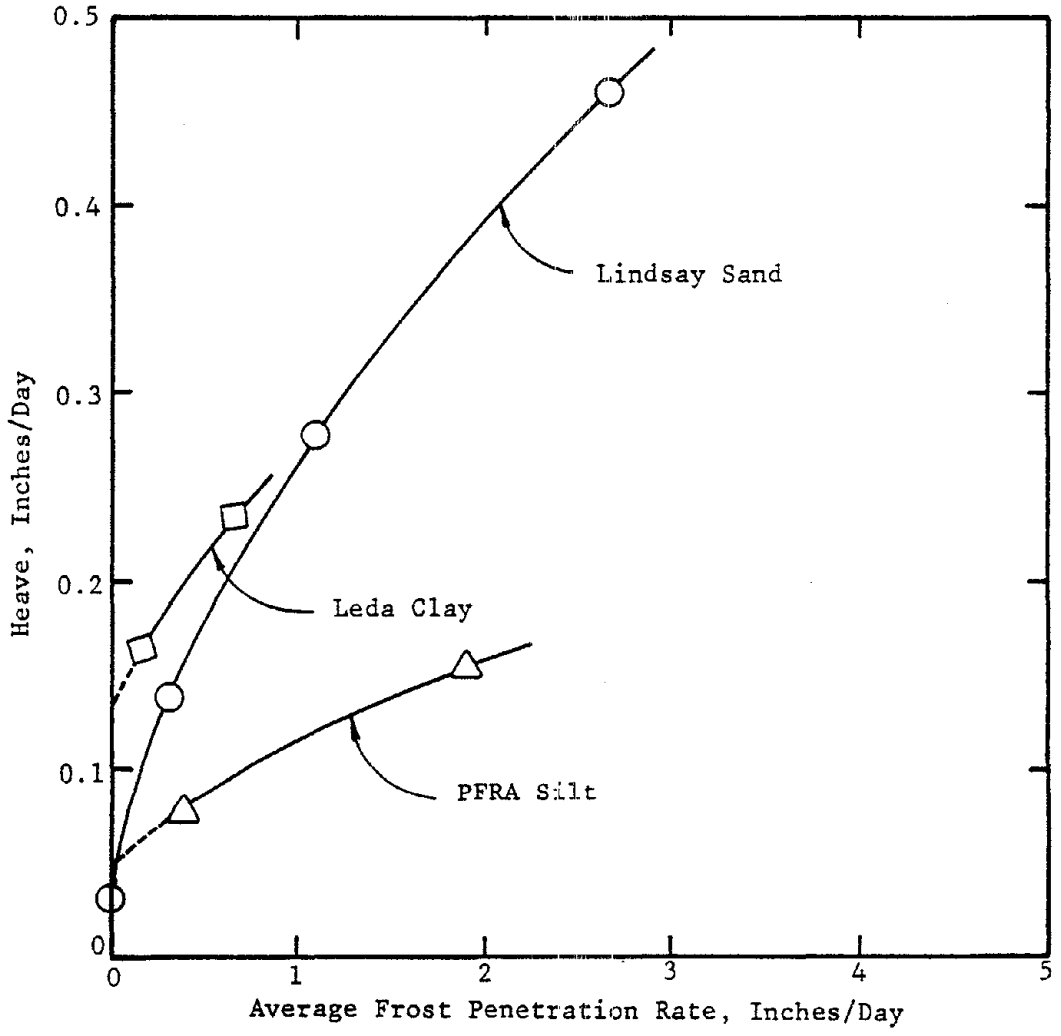
[1]Wissa, A.E.Z. and R.T. Martin, "Frost Susceptibility of Massachusetts Soils - Evaluation of Rapid Frost Susceptibility Tests", Soils Publication 320, Massachusetts, Institute of Technology, 1973.

[2]Soils and Geology - Pavement Design for Frost Conditions, TM 5-818-2, Department of Army Technical Manual, Headquarters, Department of the Army, July 1965.

[3]Krebs, R.D. and R.D. Walker, Highway Materials, McGraw-Hill Book Company, 1971.

[4]Sayman, W.C., "Plate-Bearing Study of Loss of Pavement Supporting Capacity Due to Frost", HRB Bulletin 111, Highway Research Board, 1955.



1 inch = 25.4 mm

Figure 2.9. Average rate of frost heave for various rates of frost penetration (Ref.29).

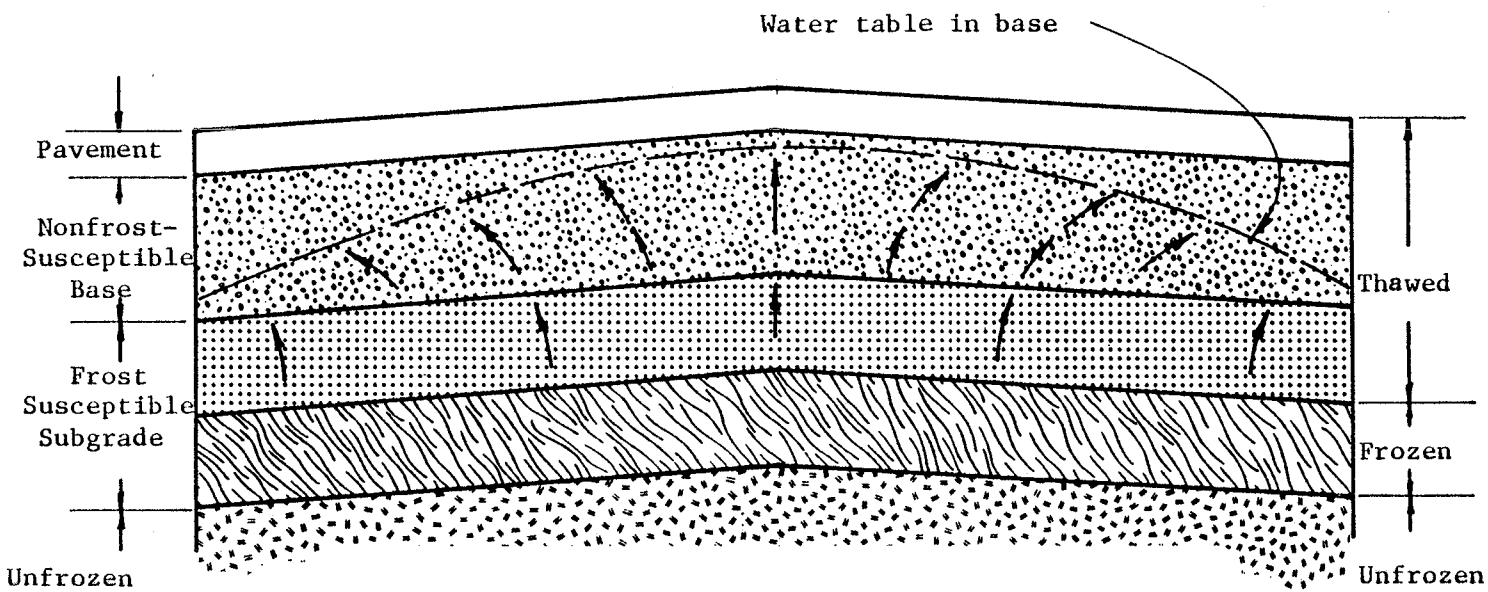


Figure 2.10. Illustration of the moisture movement upward into the base course during thaw (Ref. 30).

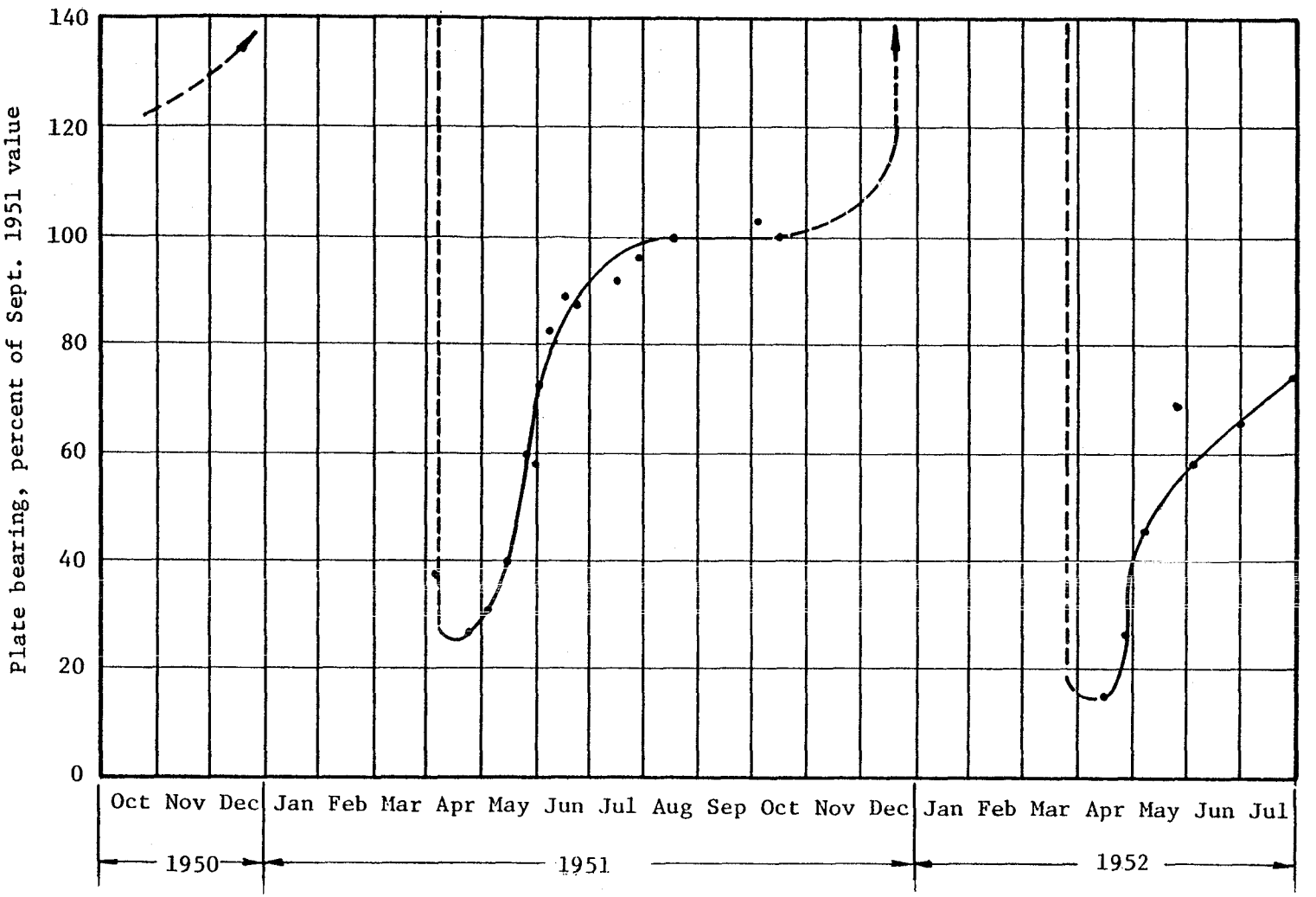


Figure 2.11. Seasonal reduction in bearing capacity of a pavement with a granular base, frost-susceptible soil, and high water table (Ref. 32).

DISINTEGRATION

Disintegration is generally related to environmental effects and the individual components and material properties that comprise the pavement structure. Ravelling and reduced skid resistance are two disintegration distresses, of particular concern, which are dependent upon material properties and wheel load applications. Although, reduced skid resistance is not considered to need structural maintenance.

Ravelling can significantly reduce the maintenance-free life of a pavement structure. This distress manifestation is primarily related to mix design properties, such as asphalt consistency and aggregate characteristics, as illustrated in Figure 2.12. Although ravelling is not as common nor significant a distress manifestation as fatigue cracking and rutting, it can require a large amount of structural maintenance within the first few years of service.

Another principle form of nontraffic associated mode of distress, which depends on cold region environmental variables, is disintegration of stabilized layers caused by freeze/thaw degradation. In most cases damage takes place under repeated applications of wheel loads during cycles of freeze/thaw: each repetition causing damage which in its cumulative effects eventually leads to a level of distress such that a pavement is no longer serviceable and must be rehabilitated. The rate of damage accumulation is highly dependent on seasonal and environmental changes. Freeze/thaw degradation of stabilized layers clearly is progressive as the number of freezing cycles increases. Figure 2.13 (Ref. 33[1]) illustrates the effect freeze/thaw cycles can have on the tensile strength of some asphalt concrete mixtures as a function of percent air voids.

SUMMARY

In summary, pavement roughness and deterioration result from the cumulative effects of traffic loads and from various non-traffic associated causes. For example, differential frost heave causes a distortion of the surface which is only temporary, although some residual irregularity of the surface will remain. This type of pavement roughness would increase gradually, but when combined with the effects of traffic it will lead to the acceleration of rutting and cracking in flexible pavements and to cracking and faulting composite pavements, as a result of the weakened subgrade during spring-thaw. Hence, failure of a pavement system occurs due to wheel loads influenced by thermal and moisture effects and is strongly influenced by the stress/strain properties of the subgrade, base, and surface layers. Tests which characterize the pertinent material properties should be dependent on levels

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[1]Schmidt, R.J., "The Effect of Temperature, Freeze/Thaw, and Various Moisture Conditions on the Resilient Modulus of Asphalt-Treated Mixes", TRB Record No. 515, Transportation Research Board, 1974.

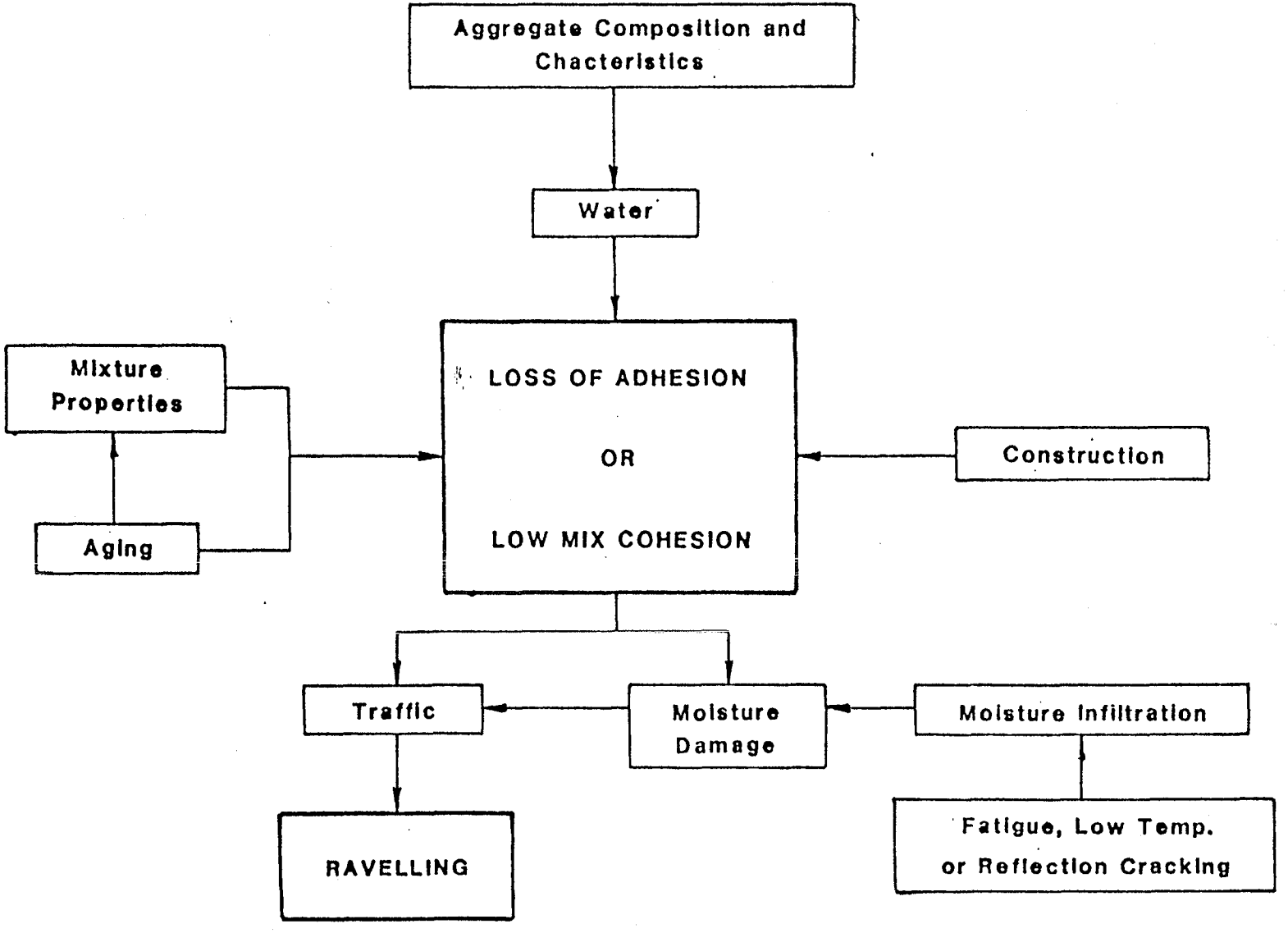

Figure 2.12. Development of ravelling in asphalt concrete layers.

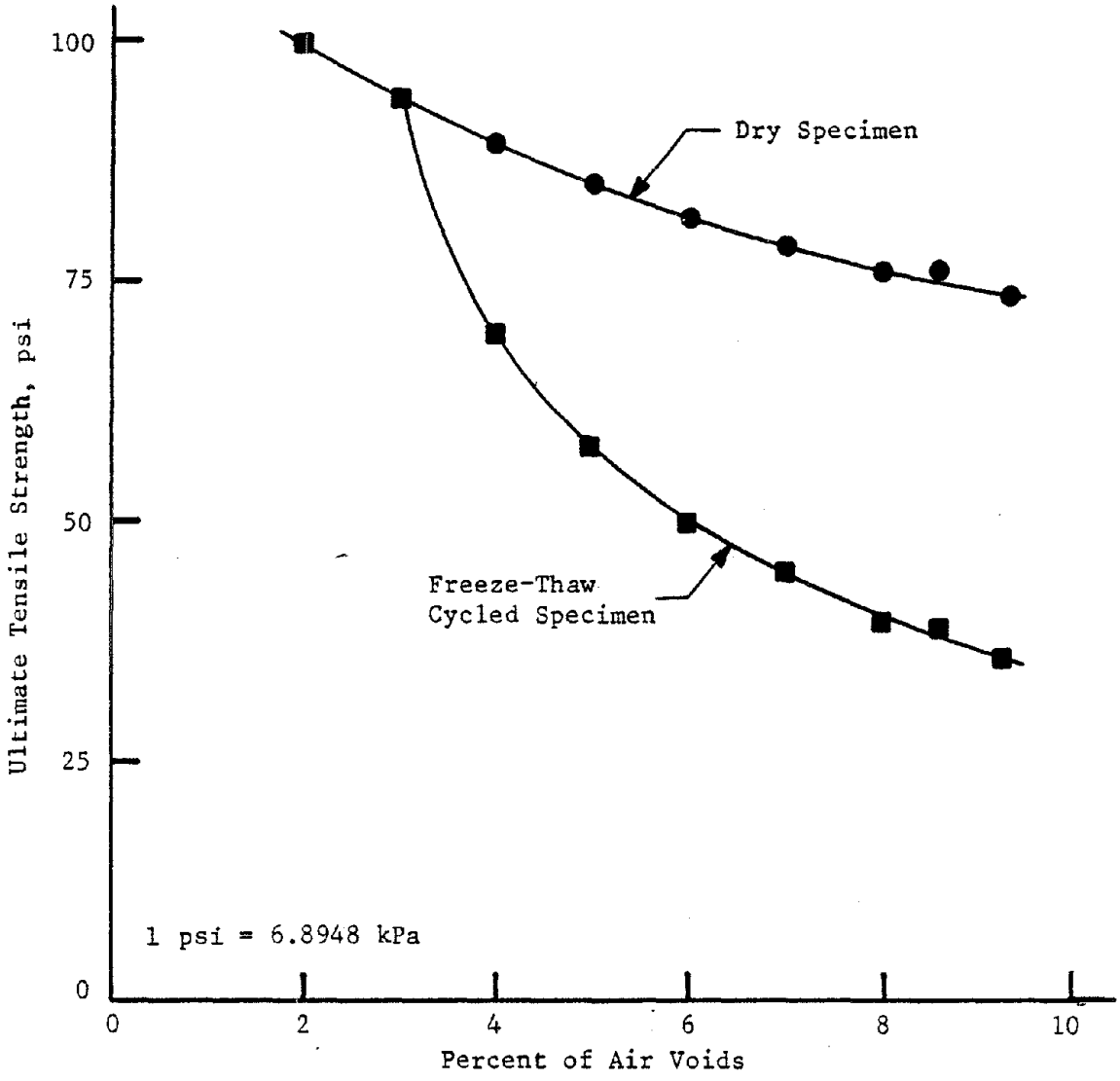

Figure 2.13. Ultimate tensile strength for various mixture voids when subjected to freeze-thaw cycles (Ref. 33).

of stress, loading time, temperature, loading and freeze/thaw history, moisture conditions; or all of these depending on the type of material to adequately predict the materials behavior under various conditions. Figures 2.2, 2.3, 2.4, 2.5, 2.6, 2.7, 2.8, 2.9 and 2.13 are used as guidelines to determine materials or a combination of materials and material properties that are required to minimize each distress manifestation limiting the zero-maintenance life of the pavement structure.

CHAPTER 3. CANDIDATE MATERIALS FOR ZERO-MAINTENANCE

Once the distress manifestations and mechanisms have been established, the materials or combination of materials can be selected to provide optimum performance. For example, while some low-temperature cracking of asphalt concrete is not detrimental to the pavement structure, damage occurs when water enters the structure and penetrates a moisture sensitive (strength decreases with an increase in moisture content) or erodable material. This process can result in increased fatigue cracking or pumping of the pavement structure as illustrated in Figure 3.1. Whereas, if a moisture barrier or impervious fabric is placed above the moisture sensitive material or if the water is quickly removed by a subsurface drainage layer, then the pavement structure may function as zero-maintenance, even with some low-temperature cracking.

Various field surveys and other research work (Refs. 1, 6, 7, 34[1]) were used to study the nature of existing flexible and composite pavements which have acted essentially as zero-maintenance pavements. But, as stated previously, most of the information reviewed indicated that flexible and composite pavements cannot meet the zero-maintenance requirements as summarized by Darter, et al., (Ref. 16) and Hoff, et al., (Ref. 7). However, some engineers, as stated by Smith, et al., in Reference 6, indicated that a small percentage (approximately 10 percent) of flexible pavements do perform for 50+ years. Therefore, the direction of this research project, as stated previously, was not to collect a large amount of field data in search of flexible and composite structures which meet the premium pavement requirements, but instead to develop design and construction procedures for conventional materials based on analytical and laboratory observations.

MATERIALS

Under this investigation five basic materials plus the combination of these materials were considered for conventional premium pavements. These included asphalt, portland cement, granular or crushed aggregate, lime and pozzolanic materials. To evaluate a material's potential to function within a zero-maintenance pavement configuration, the identification of the material properties and factors which affect those properties of each material or combination of materials, must be established. Factors that have an important effect on zero-maintenance and distress development can be categorized into four groups: (1) materials, (2) environment, (3) traffic, and (4) construction. These are summarized in Appendix B for each pavement type.

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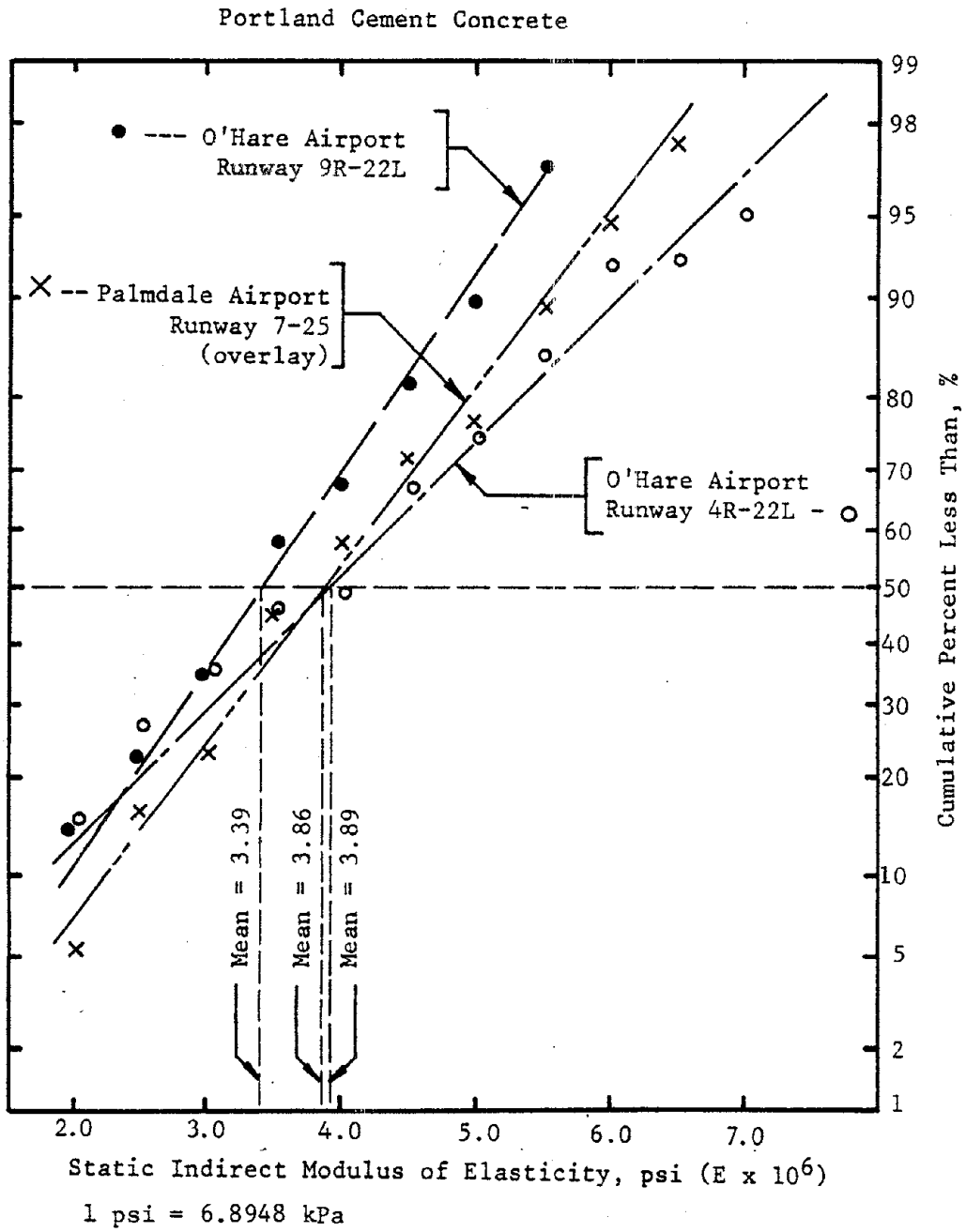
[1]Saxena, S.K., J.W.H. Wang, J.J. Udwari and W.J. RosenKranz, "Unique Concepts and Systems for Zero-Maintenance Pavements State-of-the-Art Report", Report No. FHWA-RD-77-76, Federal Highway Administration, July 15, 1977.



1 psi = 6.8948 kPa

Figure 3.1. Normal probability graph showing a relative cumulative frequency of a normal distribution of Static Indirect Modulus of Elasticity for airports (Ref. 35).

In selecting a candidate material or combination of materials for premium pavements, special consideration was given to the environmental and construction variables; especially since these are not always considered in the design process, but may represent the dominant factors which limit the maintenance-free life of a pavement structure. For example, frost susceptible soils in a wet-freeze region can cause considerable damage to a pavement. Other environmental factors besides frost susceptible soils, include freeze/thaw cycles, low temperatures, thermal fatigue, moisture, and expansive soils. Construction variables are more difficult to explain and quantify, but include factors such as curing temperature and length, water content during compaction, and degree of compaction. Also included under construction is the variability of the in-place properties of materials. Examples of material variability of in-place materials are shown in Figures 3.1 and 3.2 (Ref. 35[1]). Due to the many uncertainties and material property variations within a project (strength, stiffness, moisture, content, voids, permeability, etc.) localized failures may result requiring structural maintenance, thus limiting the maintenance-free life of the pavement structure. Figures 3.3 and 3.4 (Ref. 36[2]) illustrate the variation in fatigue life for two types of materials.

In establishing these primary material properties and factors, previous research and analytical studies were reviewed along with the staff's experience. Some of these studies included Darter's (Refs. 1-3), Rauhut's (Refs. 4, 37[3]), and Saxena's (Ref. 34) zero-maintenance work, and Shahin's (Ref. 38[4]), Finn's (Refs. 39[5], 40[6]), Lottman's (Ref. 41[7]), Schmidt's (Ref. 42[8]), Barksdale's (Ref. 13), Hadley's (Ref. 43[9]) and Treybig's (Ref. 44[10]) studies to name a few. Tables 3.1 and 3.2

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[1]Kennedy, T.W., W.R. Hudson and B.F. McCullough, "Variability of Material Properties for Airport Pavement Systems", Report No. CE-5.1, Austin Research Engineers Inc, December 1974.

[2]Moore, R.K. and T.W. Kennedy, "Tensile Behavior of Subbase Materials under Repetitive Loading", Research Report 98-12, Center for Highway Research, The University of Texas at Austin, October 1971.

[3]Rauhut, J.B., J.C. O'Quin and W.R. Hudson, "Sensitivity Analysis of FHWA Structural Models VESYS II, Vol. 2", Report No. FHWA-RD-76-24, Federal Highway Administration, March 1976.

[4]Shahin, M.Y., "Prediction of Low-Temperature and Thermal-Fatigue Cracking of Bituminous Pavements", University of Texas, Ph.D. Dissertation, August 1972.

[5]Finn, F.N., K. Nair and J. Hilliard, "Minimizing Premature Cracking in Asphalt Concrete Pavement", Final Report NCHRP 9-4, September 1973.

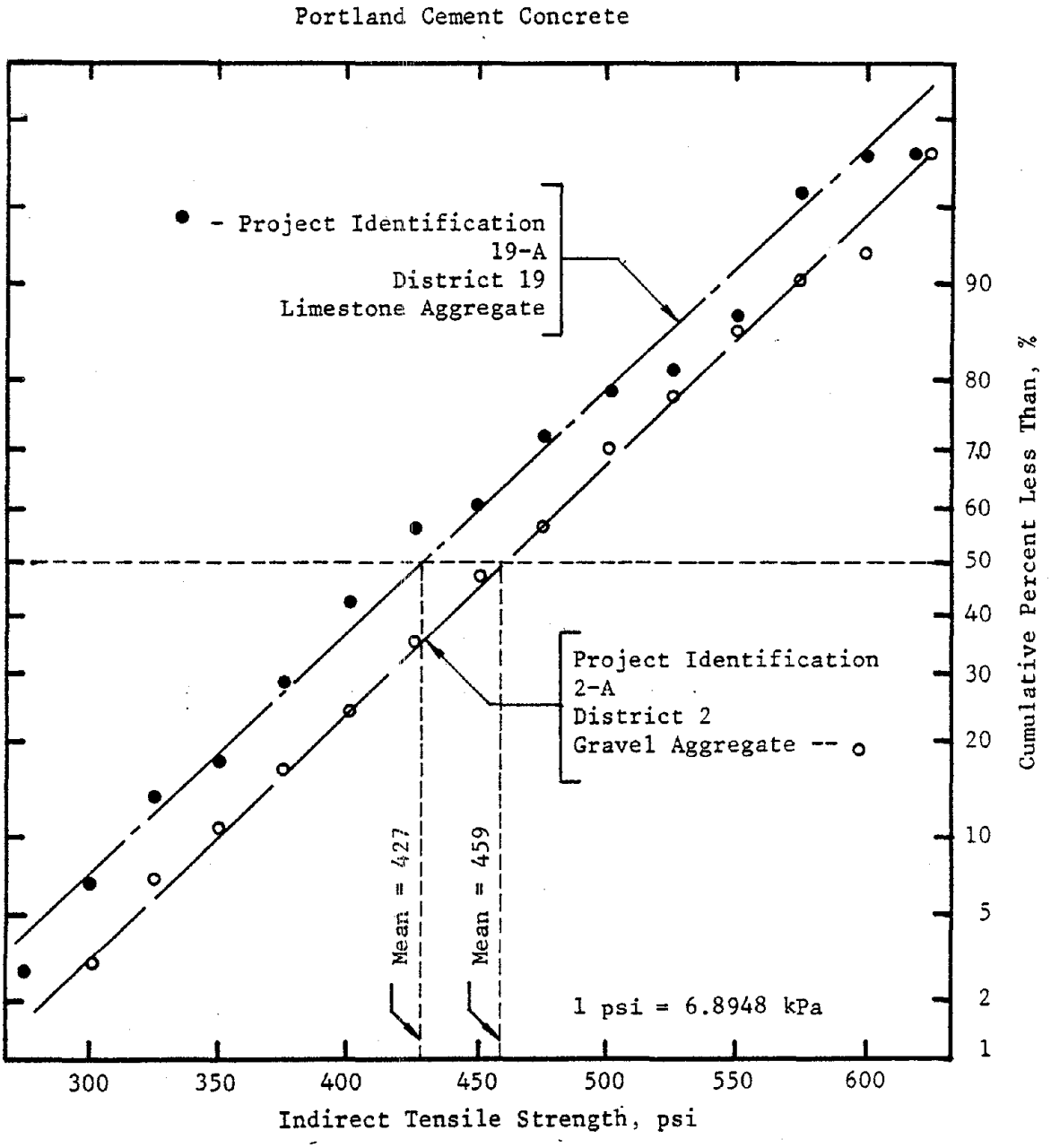

Figure 3.2. Normal probability graph showing a relative cumulative frequency of a normal distribution of indirect tensile strength for highways (Ref.35).

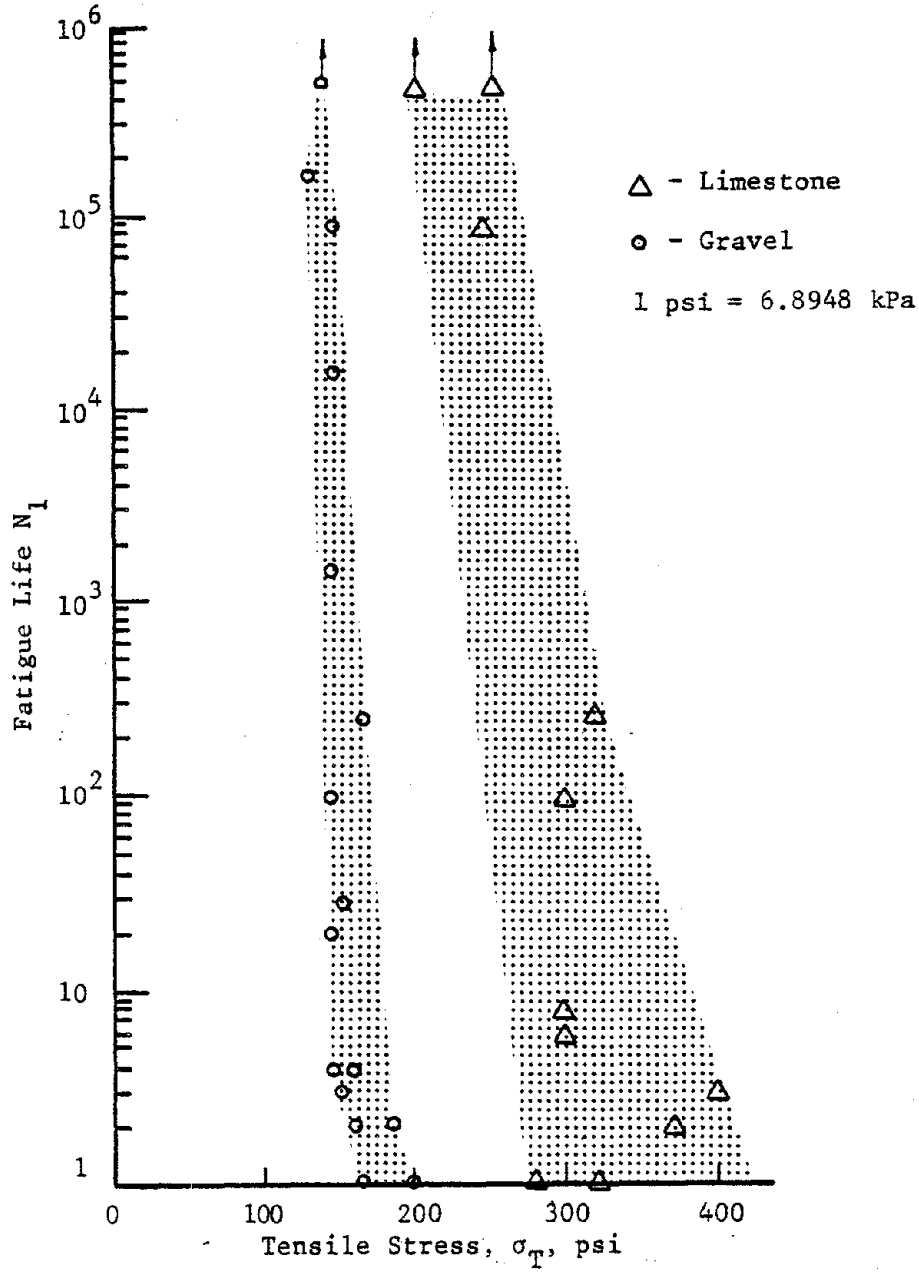

Figure 3.3. Relationships between logarithm of fatigue life and tensile stress for cement-treated materials (Ref. 36).

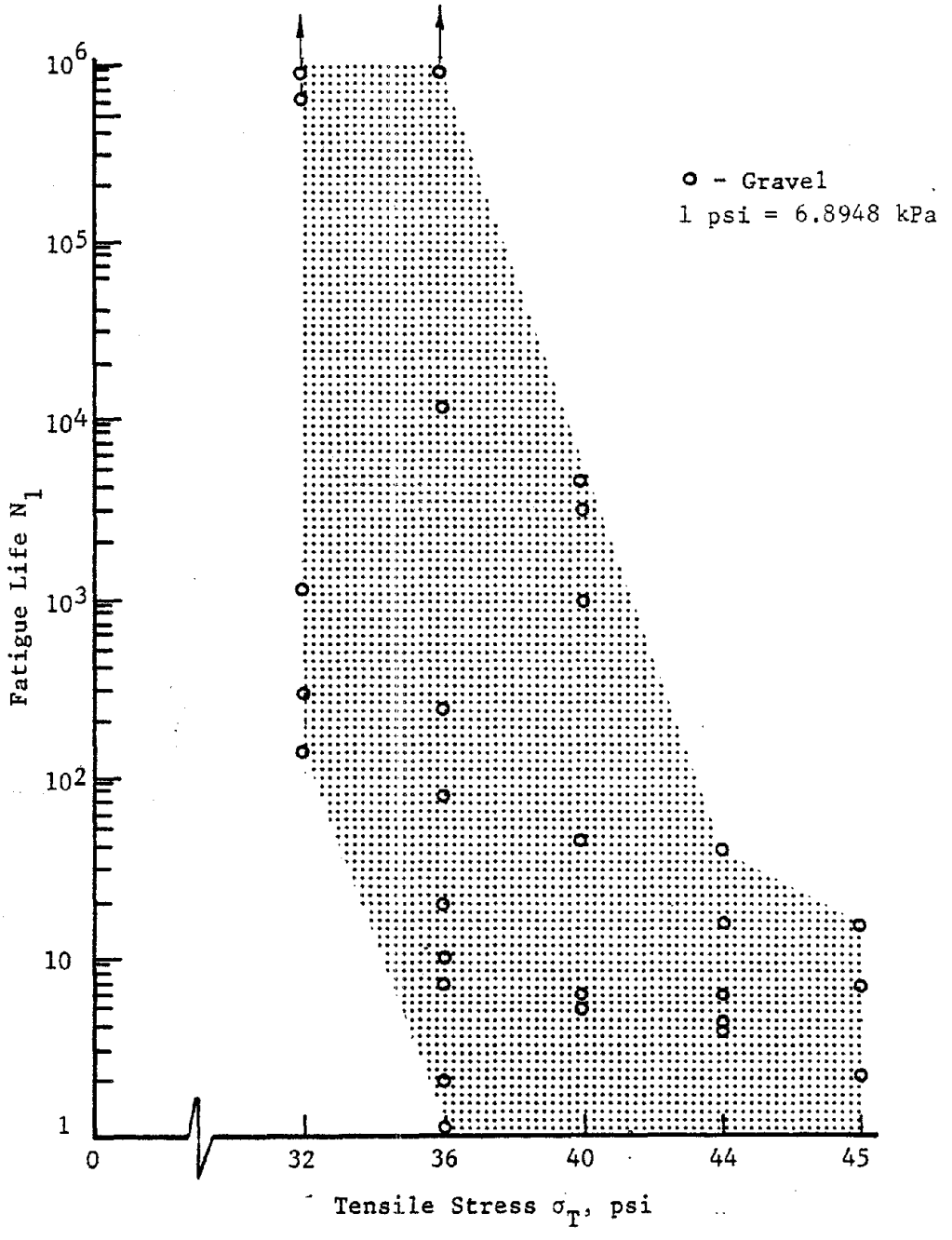

Figure 3.4. Relationships between logarithm of fatigue life and tensile stress for lime-treated materials (Ref. 36).

TABLE 3.1. MATERIAL PROPERTIES AFFECTING DISTRESSES IN PREMIUM COMPOSITE PAVEMENTS (REF. 4)

MATERIAL PROPERTY	COMPOSITE PAVEMENT DISTRESSES							
	CRACKING			RUTTING	COMPACTION-SWELLING**	RAVELING	STRIPPING	REDUCED SKID RESISTANCE
	FATIGUE	LOW TEMPERATURE	REFLECTION *					
Mixture Stiffness ***	a,b,c, d,e,f, g	a,b,d	a,b,c, d,e,f, g	a,b,c, d,e,f, g			a, c	
Fatigue Constants	a, b, c d, e		a					
Tensile Strength		a,b,d	a,b,d			a	a, c	
Shrinkage Characteristics		b,d	b,d					
Coeff. of Thermal Expansion		a,b,d	a,b,d					
Aggregate Characteristics			a				a, c	a
Compaction - Volume Change Characteristics					f, g			
Permanent Deformation Characteristics ***				a,c,f, g				
Frost Susceptibility					g			
Bond (Adhesion)						a	a, c	a

*Reflection cracking can be caused by any cracking of the Portland Cement Concrete layer. See the PCC pavement distress summary (Table 1-4) for a break-down of the various distress types.

**Compaction-Swelling refers to any large, relative vertical differential displacements.

***Includes stability, creep compliance, and elastic properties (E, v).

****LEGEND

a - asphalt concrete surface
b - portland cement concrete
c - asphalt-treated material
d - cement-treated material
e - lime-treated material
f - untreated granular material
g - subgrade soil

TABLE 3.2. MATERIAL PROPERTIES AFFECTING DISTRESSES IN PREMIUM FLEXIBLE PAVEMENTS (REF 4)

Material Property	FLEXIBLE PAVEMENT DISTRESS						
	Fatigue Cracking	Low-Temp. Cracking	Rutting	Compaction-Swelling*	Raveling	Stripping	Reduced Skid Resistance
Stiffness**	abcdef***	ab	abcdef			ab	
Coeff of Thermal Expansion		abd					
Tensile Strength		abd			a	ab	
Permanent Deformation Characteristics			abef				
Aggregate Characteristics						ab	a
Compaction-Volume Change Characteristics				ef			
Frost Susceptibility				ef			
Bond (Adhesion)					a	ab	a
Fatigue Constants	abcd						

*Large, Relative Vertical Differential Displacements

**Includes stability, creep compliance, and elastic properties (E, ν)

*** LEGEND

a - asphalt concrete surface
b - asphalt-treated material
c - lime-treated material
d - cement treated material
e - untreated granular material
f - subgrade soil

present the material properties, and development of each distress type for flexible and composite pavements, thus providing valuable insight for evaluating the significance of different materials for zero-maintenance. In order to determine preliminary requirements, a brief description of each material with respect to stress/strain behavior was completed. Then candidate cross-sections were selected which have high potentials to satisfy the zero-maintenance criteria.

IMPROVED SUBGRADE

As stated in the section on distress, soils which are highly susceptible to volume and strength changes (Figure 3.5, Ref. 45[1]) can cause severe roughness and accelerate the deterioration (fatigue cracking and rutting) of the structure when combined with repeated traffic loads. Generally, the resilient modulus of some soils are highly dependent on moisture and applied deviator stress, as illustrated in Figure 3.6. (Ref. 46[2]). Therefore, in some cases, the subgrade could be treated

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[6]Finn, F.N., "Factors Involved in the Design of Asphaltic Pavement Surfaces", National Cooperative Highway Research Program Report 39, Highway Research Board, 1967.

[7]Lottman, R.P., "Predicting Moisture-Induced Damage to Asphaltic Concrete", NCHRP Report No. 192, National Cooperative Highway Research Program, Transportation Research Board, 1978.

[8]Schmidt, R.J. and P.E. Graf, "The Effect of Water on the Resilient Modulus of Asphalt-Treated Mixes", Proceedings, Association of Asphalt Paving Technologists, Volume 41, 1972.

[9]Hadley, W.O., W.r. Hudson and T.W. Kennedy, "An Evaluation of Factors Affecting the Tensile Properties of Asphalt-Treated Materials", Center for Highway Research Report 98-2, The University of Texas at Austin, 1969.

[10]Treybig, H.J., B.F. McCullough, P. Smith and H. Von Quintus, "Overlay Design and Reflection Cracking Analysis for Rigid Pavements, Vol. 2, Design Procedures", Report No. FHWA-RD-77-67, Federal Highway Administration, August 1977.

[1]"Report on Committee on the Load-Carrying Capacity of Roads as Affected by Frost Action", HRB Bulletin 96, Highway Research Board, 1955.

[2]Treybig, H.J., et al., "Data Collection and Analysis Runway 4R-22L, O'Hare International Airport", Contract Report S-76-11, U.S. Army Waterways Experiment Station, Soils and Pavement Laboratory, September 1976.

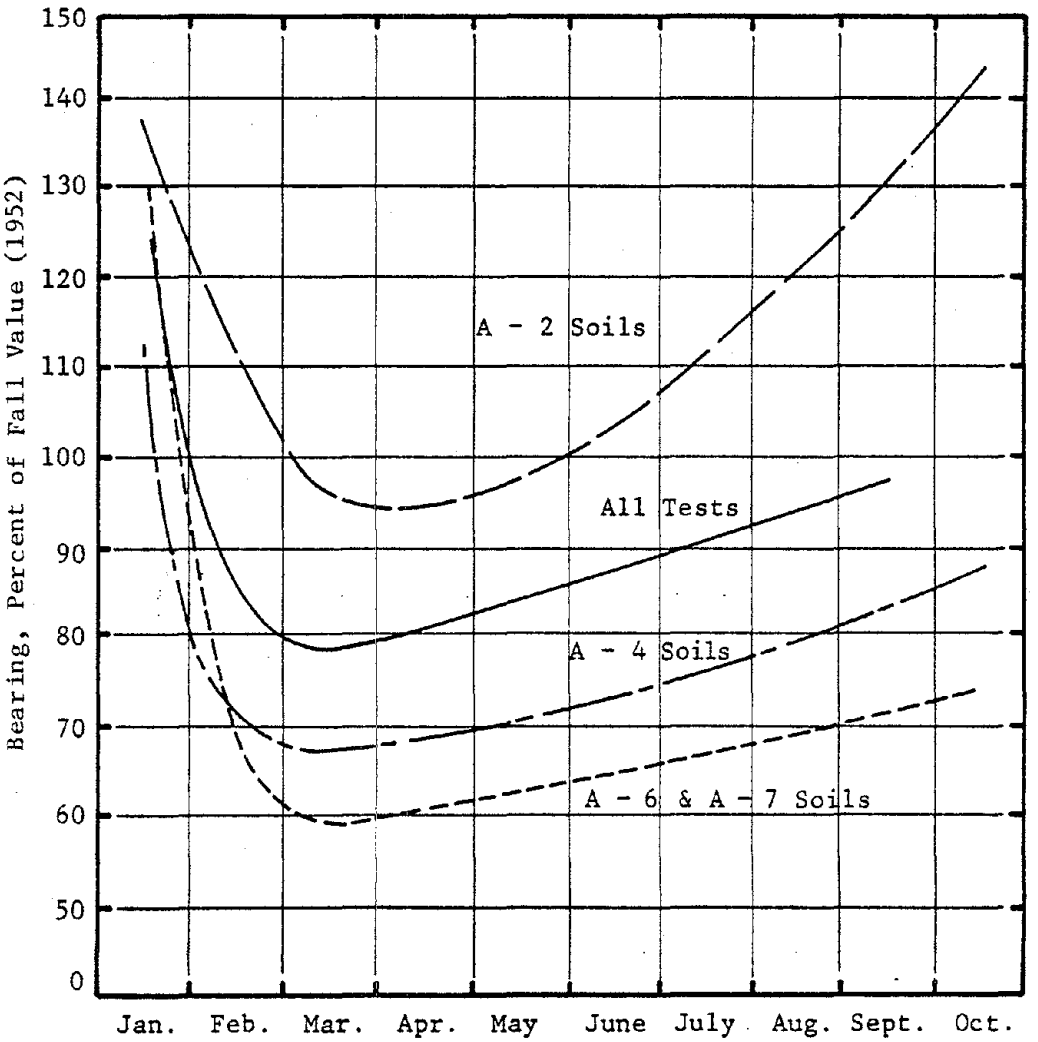

Figure 3.5. Seasonal changes in bearing capacity (Ref. 45).

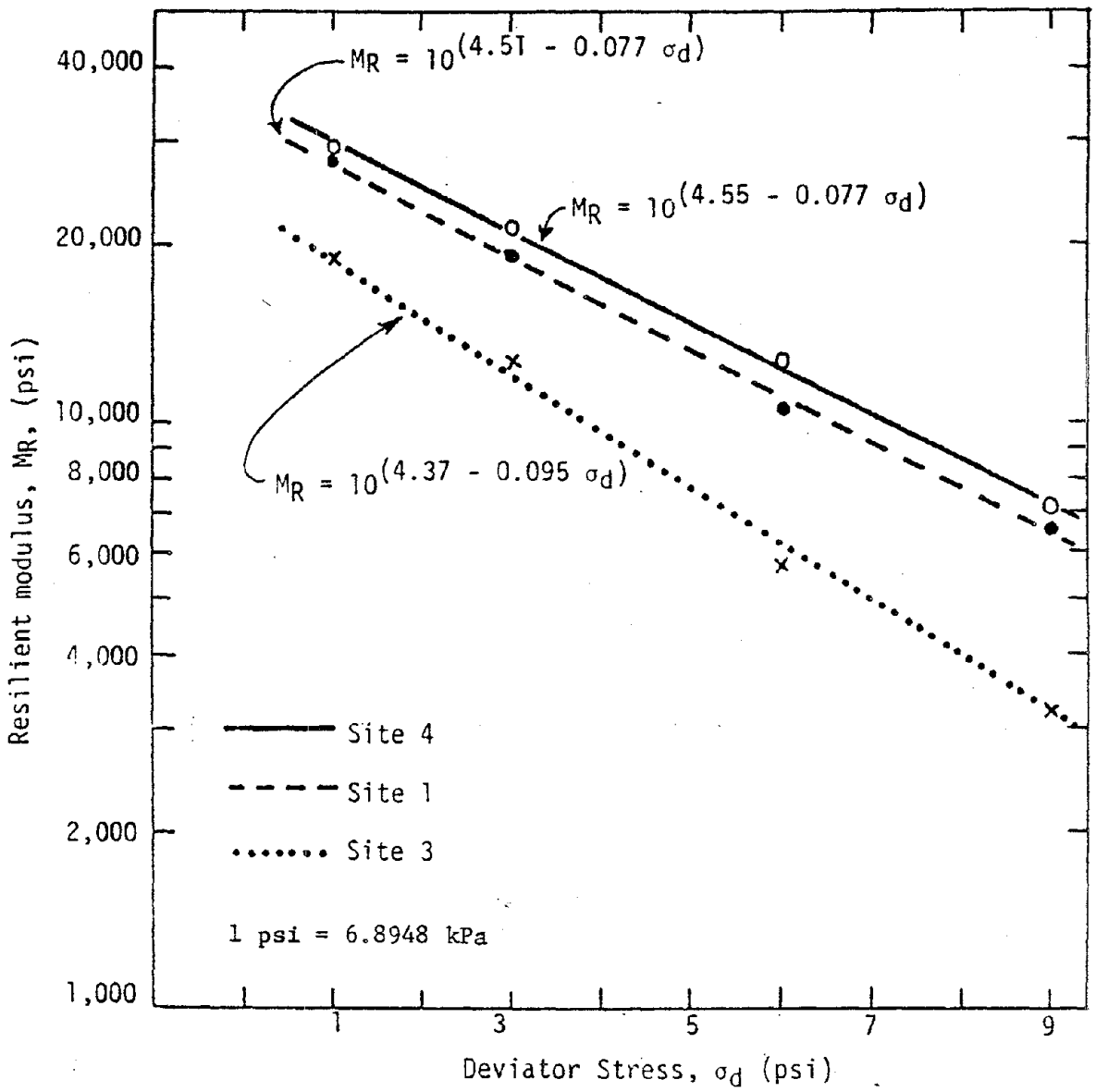

Figure 3.6. Laboratory test results of the subgrade material at a depth of 3.5 - 6 feet below the surface (Ref. 46). (Confining pressure = 4 psi).

with various methods to improve the drainage, strength and stiffness characteristics (Table 3.3). Example, to obtain a good construction platform in wet weather regions for various clay or fine-grained materials, lime stabilization could be used. For expansive frost-susceptible or compressible soils either densification, chemical stabilization, or subgrade encapsulation can be used to reduce the potential for swelling (Figure 3.7, Ref. 47[1]). Subgrade encapsulation in which an impervious material is used to protect a material from moisture changes will be discussed in a later section. Densification increases the stiffness of a material and normally does not significantly change the material characteristics (Ref. 48[2]). Lime, cement or asphalt stabilization have been normal means for controlling the swelling of soils and improving the strength characteristics of unsuitable materials (Ref. 49[3]). It is important to note that by changing or improving a material for a particular reason (to reduce the volume change potential of a highly expansive soil) could produce a material that is susceptible to other distress manifestations (frost susceptible materials) or behaves differently under load, moisture, or temperature.

Lime Stabilization

Lime or pozzolonic treatment of soils improves strength characteristics as illustrated in Figure 3.8 (Ref. 50[4]). Only fine-grained soils can be effectively stabilized with lime. Lime has been found most effective with clay soils containing montmorillonite, illite, and kaolinite. In treating certain soils with lime, some materials are produced which are subject to fatigue cracking, as illustrated from laboratory flexural fatigue test results shown in Figure 3.9 (Ref. 51[5]). The results were obtained by applying repeated flexural stresses ranging

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[1]Lambe, T.W., "Modification of Frost-Heaving of Soils with Additives", HRB Bulletin No. 135, Highway Research Board, 1956.

[2]Wahls, H.E., C.P. Fisher and L.J. Langfelder, "The Compaction of Soil and Rock Materials for Highway Purposes", Report No. FHWA-RD-73-8, Federal Highway Administration, August 1966.

[3]McKeen, R.G. and J.P. Nielsen, "Characterization of Expansive Soils for Airport Pavement Design", Interim Report No. FAA-RD-78-59, Federal Aviation Administration, August 1978.

[4]Neubaier. C.H., Jr. and M.R. Thompson, "Stability Properties of Uncured Lime-Treated Fine Grained Soils", Highway Research Record No. 381, Highway Research Board, 1972.

[5]Swanson, T.E. and M.R. Thompson, "Flexural Fatigue Strength of Lime-Soil Mixtures", Highway Research Record No. 198, Highway Research Board, 1967.

TABLE 3.3. SUMMARY OF SOIL CHARACTERISTICS AS A PAVEMENT MATERIAL (REF. 31)

Major Divisions		Name	Subgrade Strength When Not Subject to Frost Action	Potential Frost Action	Compressibility and Expansion	Drainage Characteristics
Gravel and Gravelly Soils	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	None to very slight	Almost None	Excellent
	GP	Poorly graded gravels or gravel-sand mixtures little or no fines	Good to excellent	None to very slight	Almost None	Excellent
	GM *d	Silty gravels, gravel-sand silt mixtures	Good to excellent	Slight to medium	Very slight	Fair to poor
	GM u		Good	Slight to medium	Slight	Poor to practically impervious
	GC	Clayey gravels, gravel-sand-clay mixture	Good	Slight to medium	Slight	Poor to practically impervious
Sand and Sandy Soils	SW	Well-graded sands or gravelly sands, little or no fines	Good	None to very slight	Almost None	Excellent
	SP	Poorly graded sands or gravelly sands, little or no fines	Fair to good	None to very slight	Almost None	Excellent
	SM *d	Silty sands, sand-silt mixtures	Fair to good	Slight to high	Very Slight	Fair to poor
	SM u		Fair	Slight to high	Slight to medium	Poor to practically impervious
	SC	Clayey sands, sand-clay mixtures	Poor to fair	Slight to high	Slight to medium	Poor to practically impervious

TABLE 3.3. SUMMARY OF SOIL CHARACTERISTICS AS A PAVEMENT MATERIAL (REF. 31) (CONTINUATION)

Major Divisions		Name	Subgrade Strength When Not Subject to Frost Action	Potential Frost Action	Compressibility and Expansion	Drainage Characteristics
Silts and Clays LL is less Than 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sand or clayey silts with slight plasticity	Poor to fair	Medium to very high	Slight to medium	Fair to poor
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays silty clays, lean clays	Poor to fair	Medium to high	Slight to medium	Practically impervious
	OL	Organic silts and organic silt-clays or low plasticity	Poor	Medium to high	Medium to high	Poor
Silts and Clays LL is greater Than 50	MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts	Poor	Medium to very high	High	Fair to poor
	CH	Inorganic clays of high plasticity, fat clays	Poor to fair	Medium to very high	High	Practically impervious
	OH	Organic clays of medium to high plasticity, organic silts	Poor to very poor	Medium	High	Practically impervious
Highly Organic Soils	Pt	Peal and other highly organic soils	Not suitable	Slight	Very high	Fair to poor

* d is used when the liquid limit is 25 or less and the plasticity index is 5 or less.
u is used in all other cases.

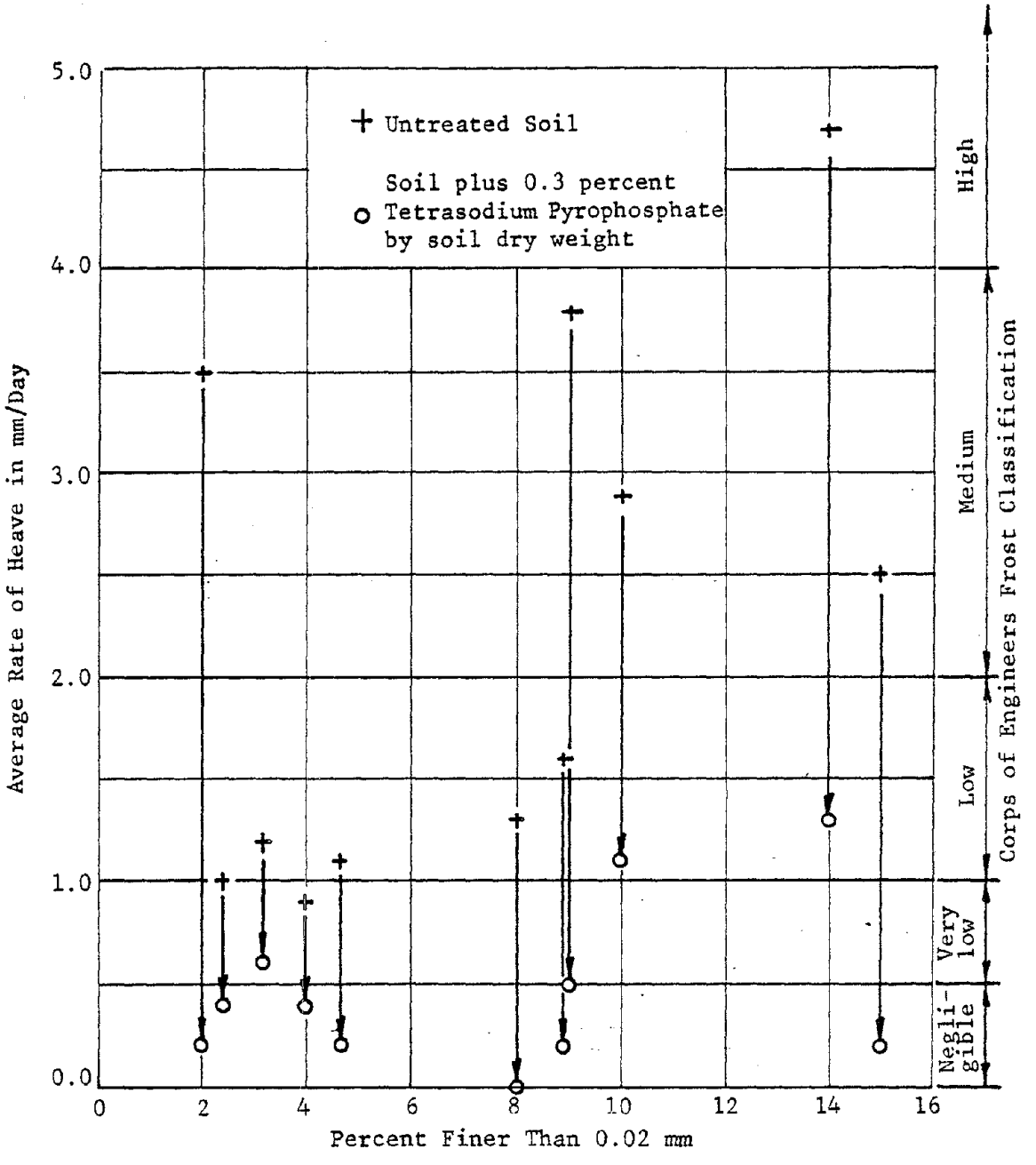

Figure 3.7. Illustration of the effect of treating a soil ("dirty" gravel) to reduce the frost heave under a pavement structure (Ref. 47).

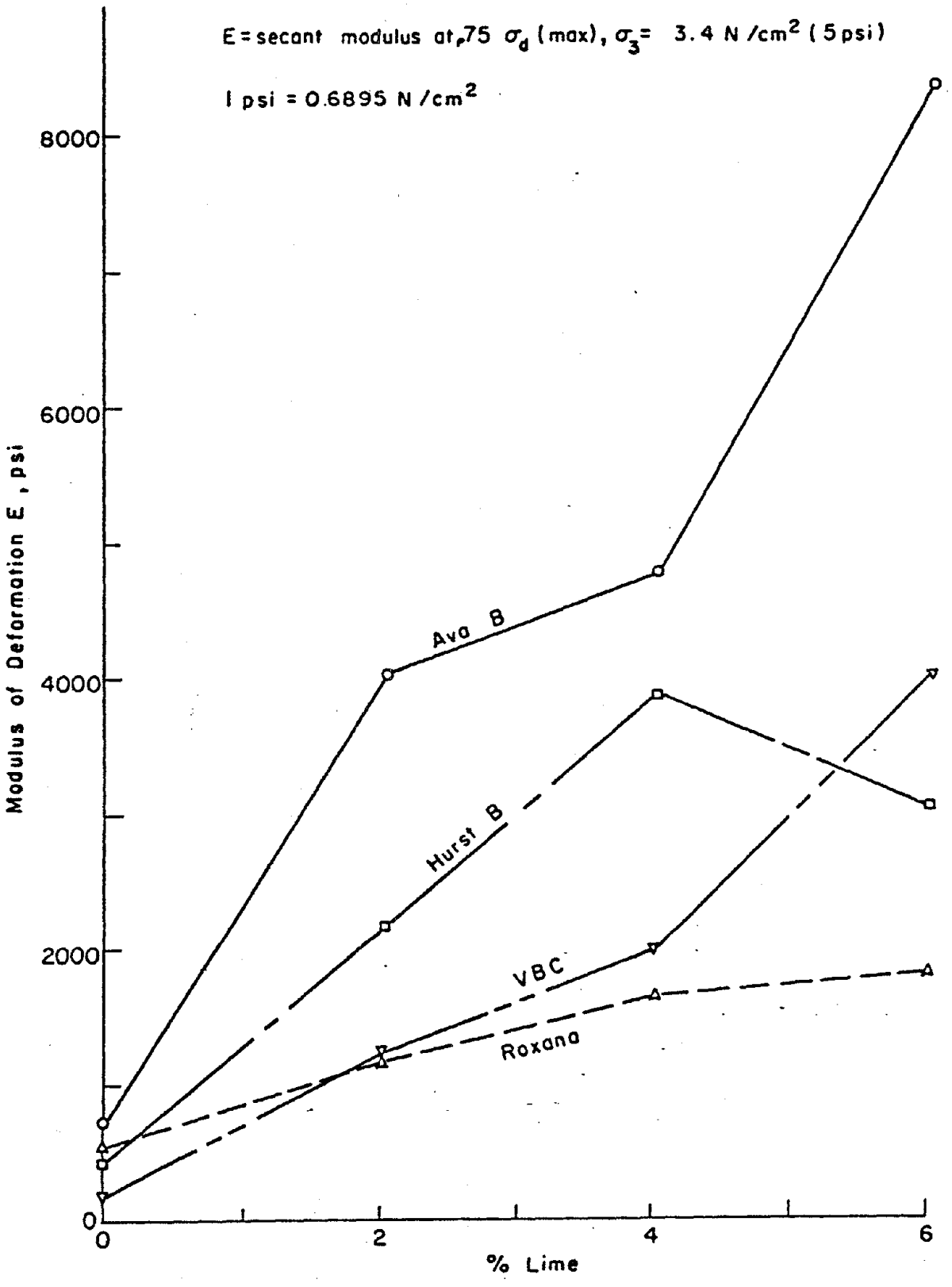

Figure 3.8. Immediate effects of lime treatment on modulus of deformation (Ref. 50).

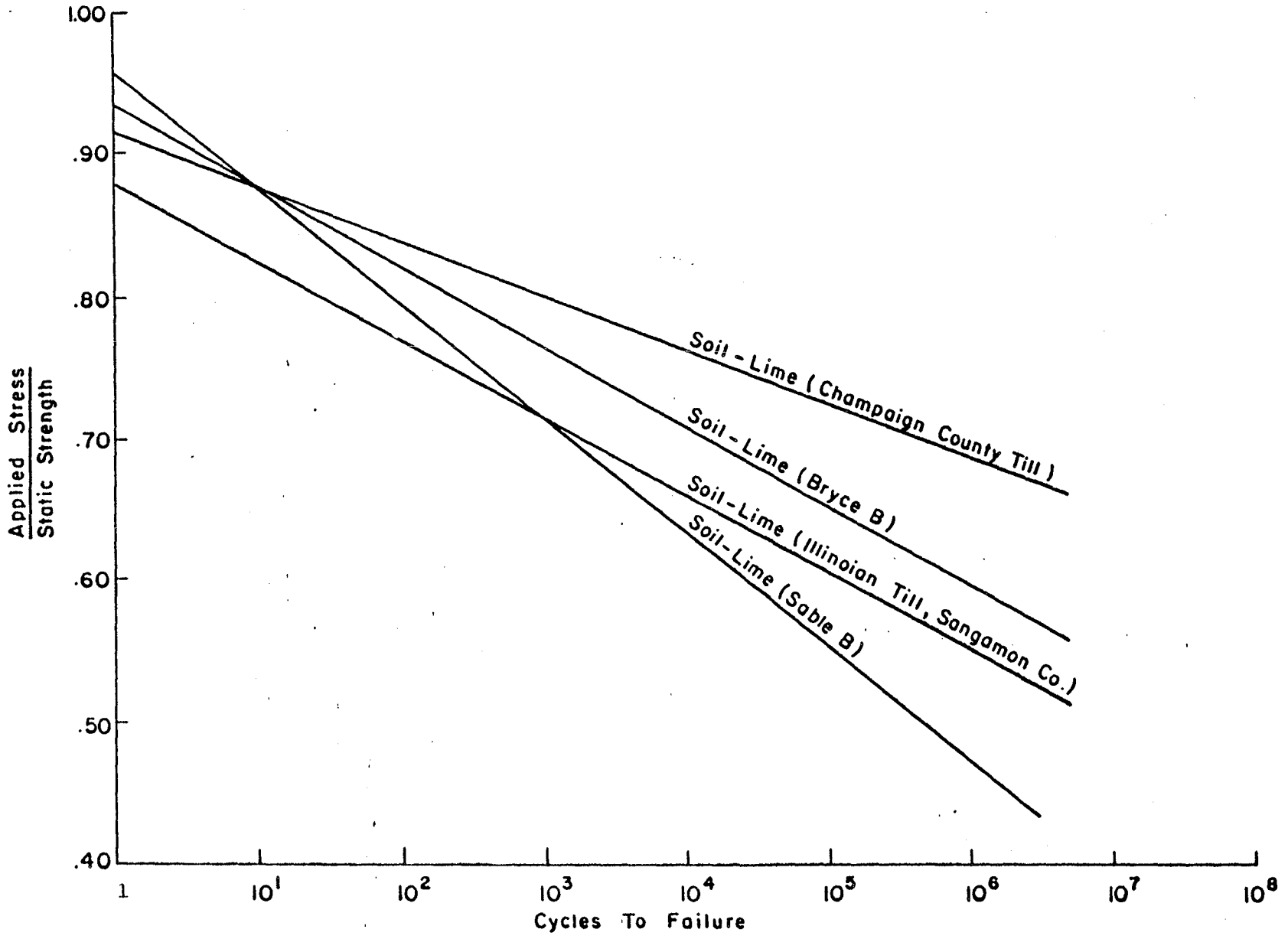

Figure 3.9. Flexural fatigue response curves (Ref. 51).

from 40 to 95% of the ultimate strength at approximately 12 cps to the specimen's third points using an equipment set-up described by Swanson and Thompson (Ref. 51). Some of the recorded experiences where performance of lime-treated, frost susceptible soils have been satisfactory were under freezing conditions. However, tests at the Colds Regions Research Engineering Laboratory (CRREL) have shown the lime treatment of clay soils can convert material that shows negligible to moderate frost heave into one that is highly susceptible to frost heave, acquiring characteristics more typically associated with silts. It has been suggested that this adverse effect has been caused by an insufficient curing period. Hence, it is emphasized that adequate curing is of critical importance. When lime-treated soils are properly treated, it has been concluded that the major durability consideration for lime-soil mixtures is resistance to cyclic freezing and thawing. Lime-flyash stabilization is applicable to a broader range of soil types because its cementing action is less dependent on fines contained within the soil. However, long-term durability studies of actual pavements of lime-flyash stabilization are limited.

Cement Stabilization

Portland cement is widely used for stabilizing medium-textured, sandy and granular materials to improve the engineering properties of strength and stiffness. Increasing the cement content increases the quality of the mixture. At low cement contents, the product is generally termed cement modified soil which is a soil with improved properties such as plasticity, expansive characteristics, and frost susceptibility. At higher cement contents, the end product is soil cement defined by the Portland Cement Association as a hardened, soft cement, structural material. Figure 3.10 illustrates laboratory test results from cement-treated soils in relation to fatigue cracking or reduction in stiffness with number of traffic applications as reported in the Shell Design Manual (Ref. 52[1]).

Asphalt Stabilization

Bituminous stabilized materials are generally used for base and subbase construction. Use of bitumen as a stabilizing agent produced different effects depending on the soil and may be divided into three principle groups: 1) sand-bitumen, which produces strength in cohesionless soils, such as clean sands, or acts as a binder or cementing agent, 2) soil-bitumen which stabilizes the moisture content of cohesive fine-grained soils, and 3) sand-gravel bitumen, which provides cohesive strength and waterproofs pit-run gravelly soils with inherent frictional strength. The durability of bitumen stabilized mixtures generally can be assessed by measurement of their water absorption characteristics.

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[1]"Shell Pavement Design Manual - Asphalt Pavements and Overlays for Road Traffic", prepared by Shell International Petroleum Company Limited, London, 1978.

Cemented Base	Mix Stiffness
Modulus, N/m^2 10^9 Thickness, mm 150	___ S1-50 ___S1-100
Subgrade modulus, N/m^2 2.5×10^7	w-MAAT, °C variable

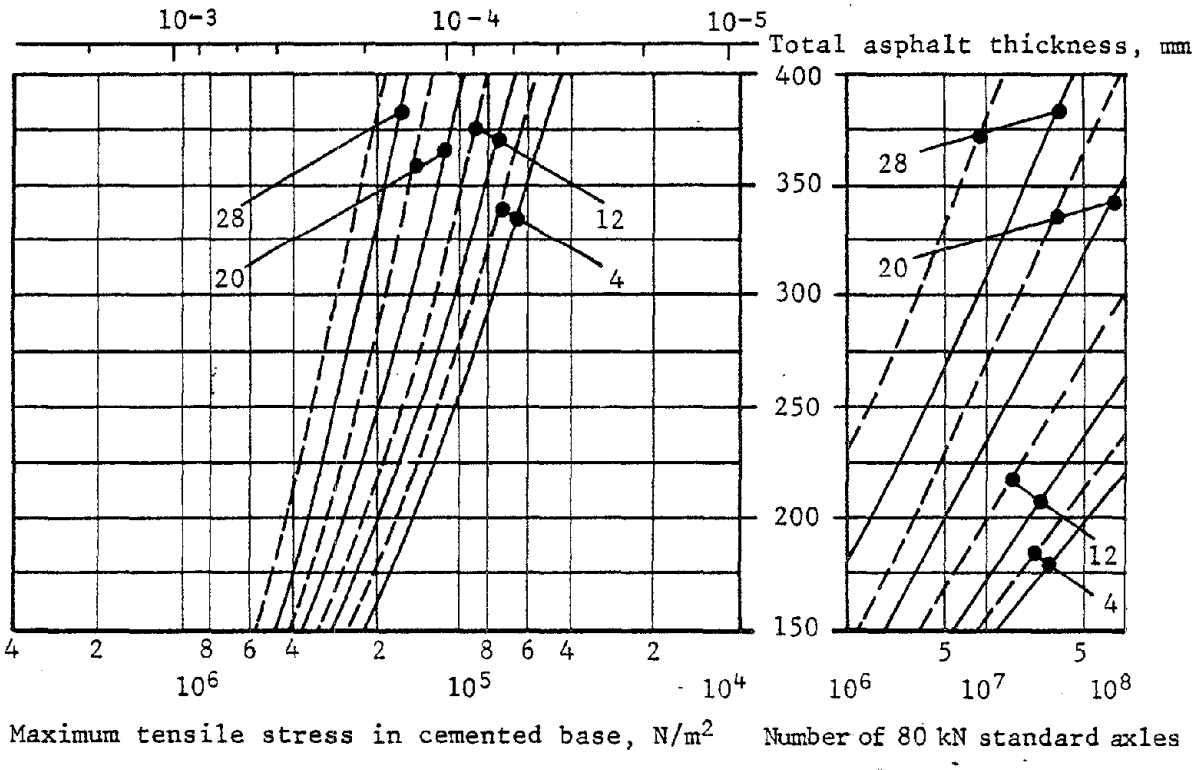

Figure 3.10 Design nomograph showing a fatigue relationship for a cement treated material (Ref. 52).

GRANULAR OR CRUSHED STONE AGGREGATE BASE

Bases are used in flexible or composite pavements for many reasons, some of which include: 1) to prevent pumping, 2) to control volume change in expansive clay subgrades, 3) to reduce frost heave, 4) to minimize loss of subgrade support during spring thaw, 5) to increase the modulus of subgrade reaction, 6) to provide a more stable working platform for construction equipment, 7) to replace soft and highly compressible subgrade soils, and 8) to provide drainage. Prevention of pumping through the use of a suitable base course is especially critical in pavements whose subgrades will experience freezing and thawing, which in most instances causes a sharp increase in moisture content during the spring. These detrimental heave and thaw weakening can be controlled and largely prevented by the use of thick granular bases.

Unbound base materials can also be used to reduce the amount of surface material required; although, a substantial thickness of base material would be required to significantly reduce the surface thickness as explained and illustrated in Chapter 6.

Another function of an unbound base course material is to minimize the formation of ice lenses under the pavement and subsequent heaving of the riding surface, as illustrated in Figure 3.11 (Ref. 53[1]) by restricting frost penetration. In the past, crushed stone base materials were believed to be resistant to the actions of frost and freeze/thaw and were expected to retain strength or load carrying ability when the frost in the base melted. However, based on the accumulation of evidence from past reports this may not be true; in fact, crushed stone bases can be very much susceptible to frost action. A recent Highway Research Board (HRB) Record No. 215 contains two such research reports documenting such evidence.

Some of the essential properties of base and subbase courses are strength, resistance to deformation, and consistency of volume. Deacon (Ref. 54[2]) showed that a key strength parameter for unbound granular bases is shear strength, whereas for asphalt treated bases, tensile and fatigue strength are the critical parameters. For cement-treated bases shear strength, flexural strength, and fatigue strength are the most important strength parameters. The resilient modulus of an unbound base course is a useful characterization property of the resistance to deformation and dependence on the confining pressure and moisture content

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[1]Penner, E., "Insulated Road Study", TRR No. 612, Transportation Research Board, 1976.

[2]Deacon, J.A., "Materials Characterization Experimental Behavior", HRB Special Report No. 126, Highway Research Board, 1971.

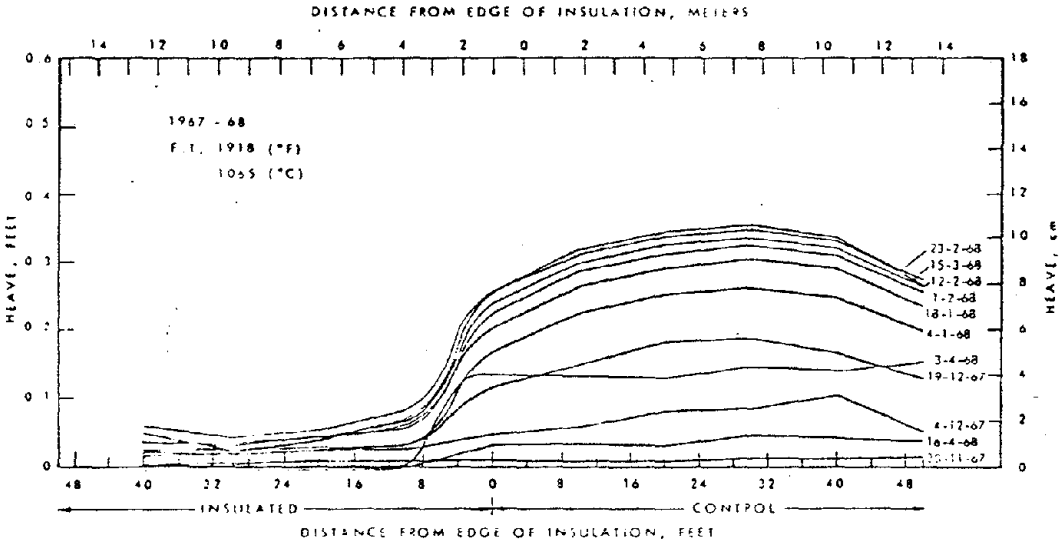

Figure 3.11. Example showing the potential of an insulation material in reducing frost heave in a frost-susceptible soil (Ref. 53).

as shown in Figure 3.12 (Ref. 55[1]). Degrees of saturation above 75 to 80% pore pressure buildup in unbound granular bases under repeated loads have been shown by various investigators (Ref. 56[2], 57[3], 58[4]) to severely reduce the resilient modulus. Therefore, in characterizing an unbound granular base the effect of a high degree of saturation must be accounted for. Also, the modulus of the underlying material must be accounted for since compaction and stiffness can only be achieved depending on the underlying materials stiffness as illustrated in Figure 3.13 (Ref. 59[5]).

ASPHALT CONCRETE AND ASPHALT TREATED BASES

Asphalt concrete is defined as a high quality mixture of asphalt cement and well graded, high quality aggregate, thoroughly compacted into a uniform mass. Asphalt concrete mixes must be designed to perform as a surface course and as a base course. As a base course, asphalt concrete must provide adequate load distribution properties; as a surface course, it must resist the polishing effects of heavy traffic. The material must also resist the adverse effects of aging and environment in some predictible fashion.

There is a limited amount of information relative to field performance to suggest that asphalt concrete mixes can perform over periods of

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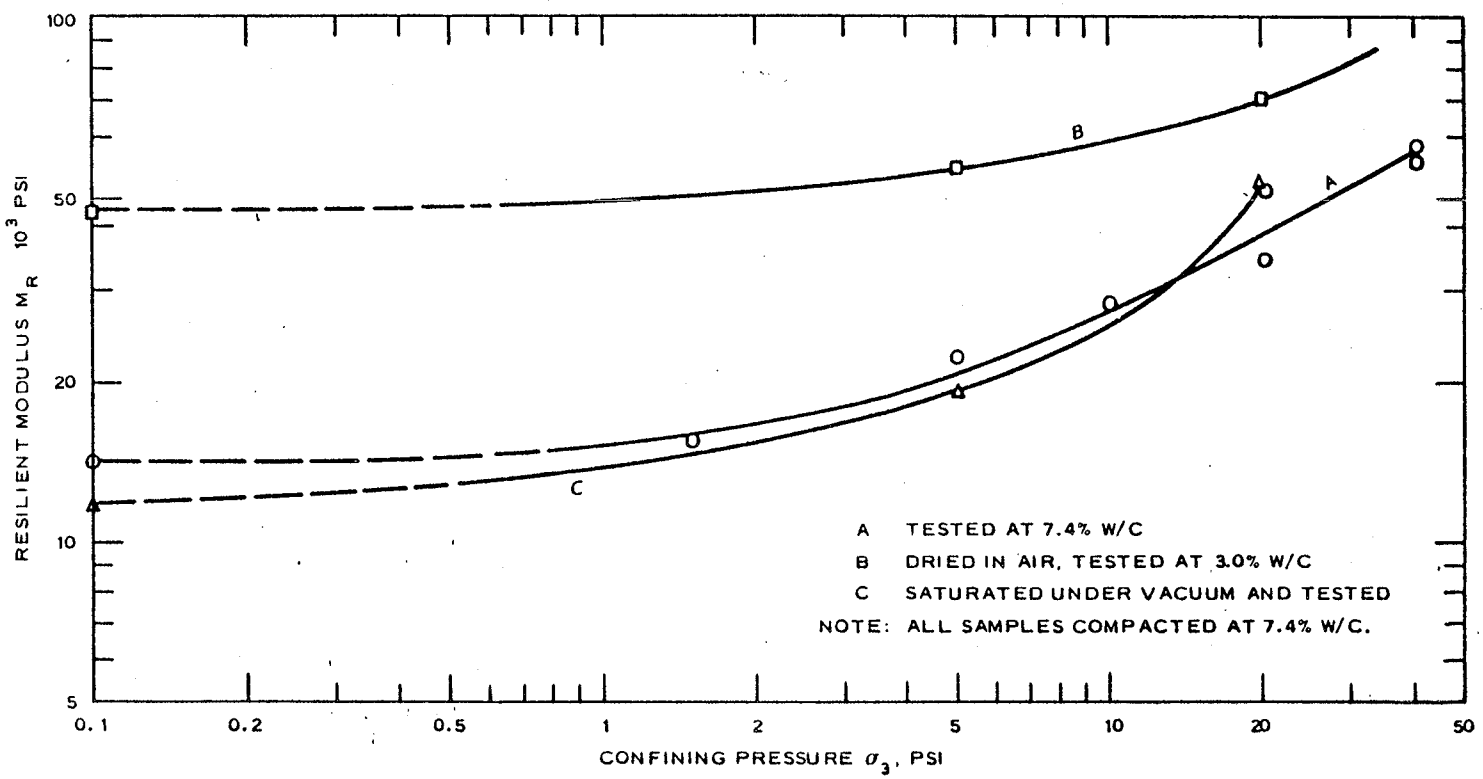
[1]Green, J.L., "Literature Review - Elastic Constants for Airport Pavement Materials", Report No. FAA-RD-76-138, prepared for U.S. DOT Federal Aviation Administration by the U.S. Army Engineer Waterways Experiment Station, March 1978.

[2]Haynes, J.H. and E.J. Yoder, "Effects of Repeated Loading on Gravel and Crushed Stone Base Course Materials Used in the AASHO Road Test", HRB Record No. 39, Highway Research Board, 1963.

[3]Kallas, B.F. and J.C. Riley, "Mechanical Properties of Asphalt Paving Materials", Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, University of Michigan, 1967.

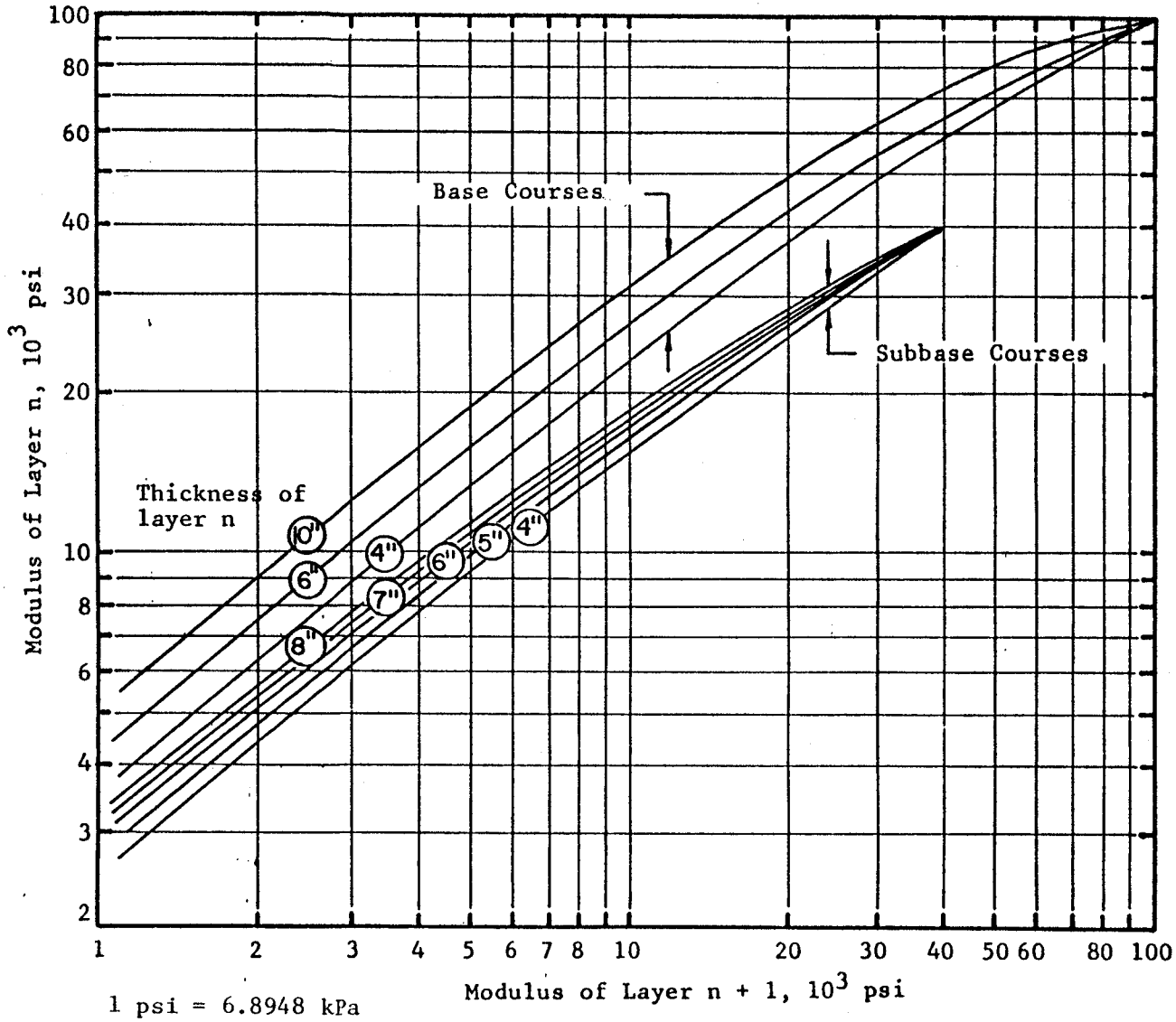
[4]Seed, H.B., F.G. Mitry, C.L. Monismith and C.K. Chan, "Prediction of Flexible Pavement Deflections from Laboratory Repeated-Load Tests", NCHRP Report 35, National Cooperative Highway Research Program, Highway Research Board, 1967.

[5]Barker, W.R. and W.N. Brabston, "Development of a Structural Design Procedure for Flexible Airport Pavements", FAA Report No. FAA-RD-74-199, U.S. Army Engineer Waterways Experiment Station, Federal Aviation Administration, September 1975.



1 psi = 6.8948 kPa

Figure 3.12. Resilient modulus versus confining pressure relationship for untreated base material (after Cook and Krukar[11]) (Ref. 55).



1 psi = 6.8948 kPa

Figure 3.13. Relationships between modulus of layer n and modulus of layer n + 1 for various thicknesses of layer n (Ref. 59).

20 to 25 years without some type of rehabilitation. Some researchers, such as Hoff, et al., (Ref. 7), considered asphalt concrete not to be a zero-maintenance candidate as experienced from its continuous maintenance. This, in part, results from the aging effects of the asphalt as illustrated in Figure 3.14 (Ref. 60[1]). However, experienced engineers in a number of states, as shown by Smith, et al., (Ref. 6), do concur that some percentage (approximately 10 percent) of flexible pavements do perform satisfactorily for 20 years or more.

Modifications to the basic asphalt aggregate combinations have been summarized in the literature. As much as possible, these modifications have been evaluated for potential use in premium pavement design. The most recent modification has been the various sulfur-asphalt combinations. These materials may eventually prove to be potential candidates as initially shown in the laboratory by Kennedy, et al., (Ref. 10); although Lytton (Ref. 61[2]) found the fatigue resistance of sulfur modified asphalt to be lower than conventional asphalt concrete. Therefore, since long-term experience in different environments is not now available, recommendations pertinent to this material have been deferred pending the availability of additional information.

Lime-modified asphalt is a material which has zero-maintenance potential as shown by Lottman's (Ref. 41) and Schmidt's (Ref. 42) laboratory studies. Figures 3.15 through 3.17 demonstrate that moisture has a very damaging effect to asphalt concrete, but when lime is added the effect of moisture is greatly reduced as observed in Figure 3.18-3.20. Therefore lime-modified asphalts were considered in wet regions for zero-maintenance potential.

PORTLAND CEMENT CONCRETE AND CEMENT TREATED BASE

This material can be used as a rigid base for composite pavements; either a continuously reinforced concrete pavement, a plain jointed concrete pavement, or a cement-aggregate mixture. For composite pavements the tensile stress at the bottom of the rigid base layer will be the response determinant. The portland cement concrete or cement aggregate mixture must be designed to provide adequate load distribution properties and to resist any adverse effects of the environment. Adequate

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[1]Roberts, J.M. and W.H. Gotolksi, "Paving Asphalt Properties and Pavement Durability", TRB Record No. 544, Transportation Research Board, National Research Council, 1975.

[2]Lytton, R.L., D. Saylak and D.E. Pickett, "Prediction of Sulphur-Asphalt Pavement Performance with VESYS IIM", Proceedings, Fourth International Conference on the Structural Design of Asphalt Pavements, Vol. I, August 1977.

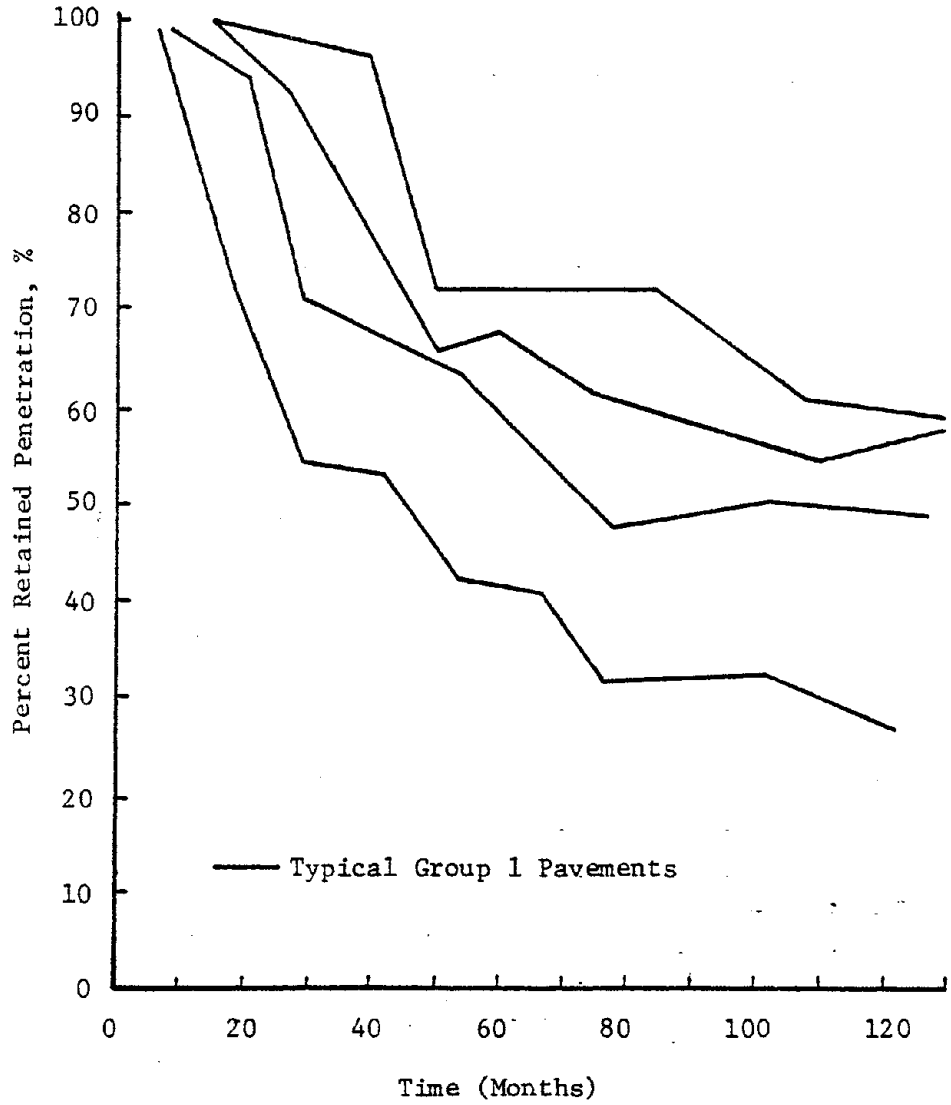


Figure 3.14 Percent of retained penetration versus time for Group 1 pavements (Ref. 60).

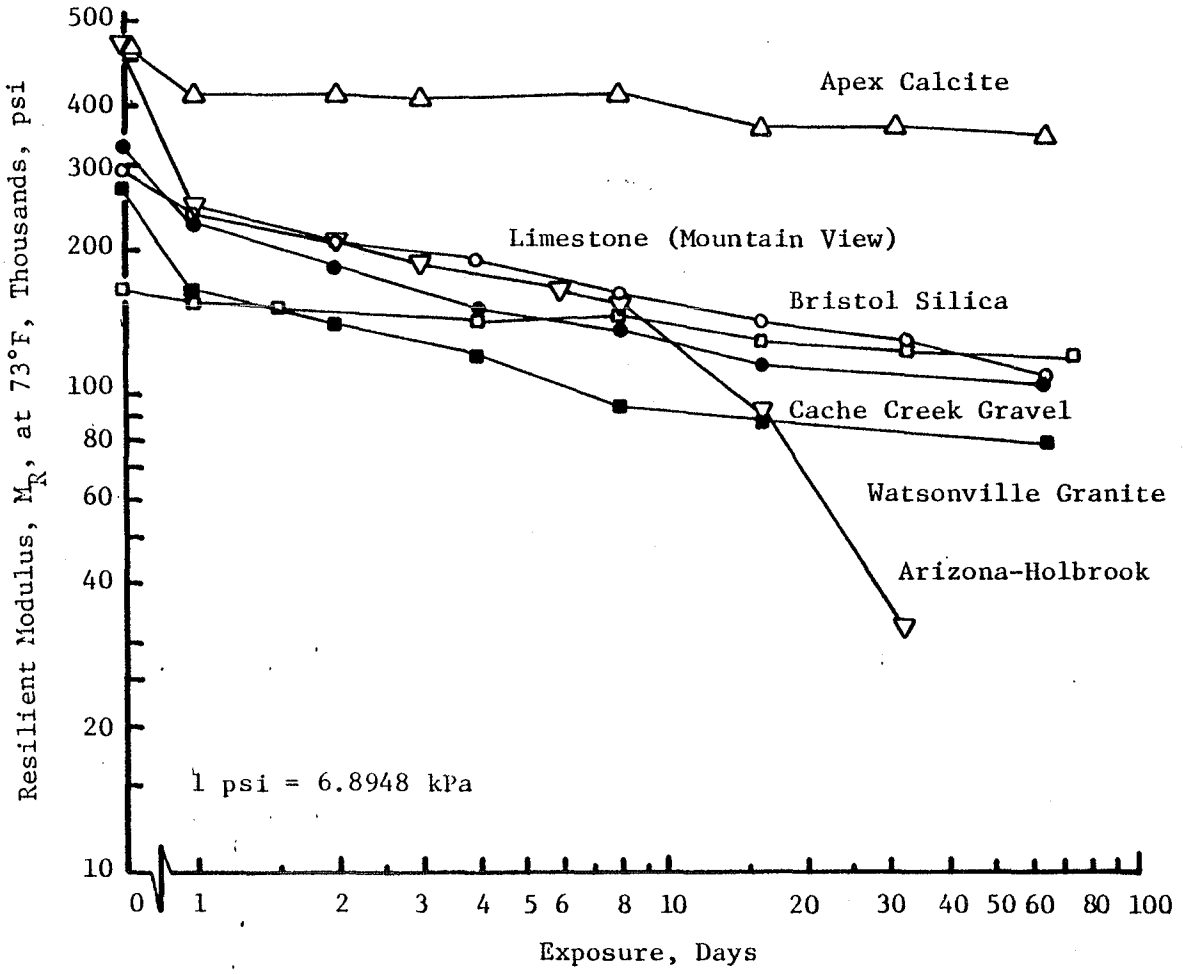
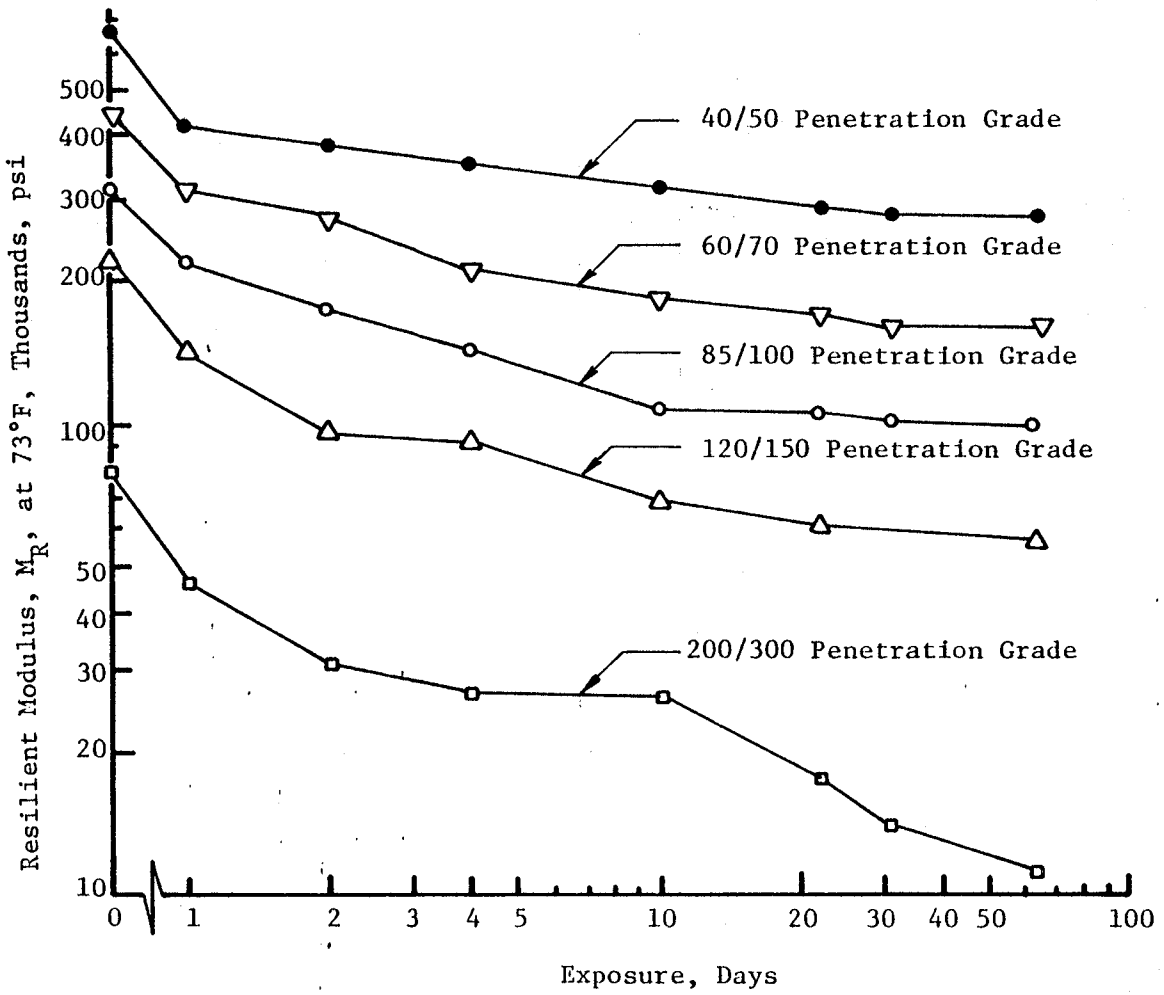


Figure 3.15. Effect of moisture on the resilient modulus of asphalt concrete for different types of aggregate (Ref. 42).



1 psi = 6.8948 kPa

Figure 3.16. Effect of moisture on the resilient modulus of asphalt concrete for different grades of asphalt (Ref. 42).

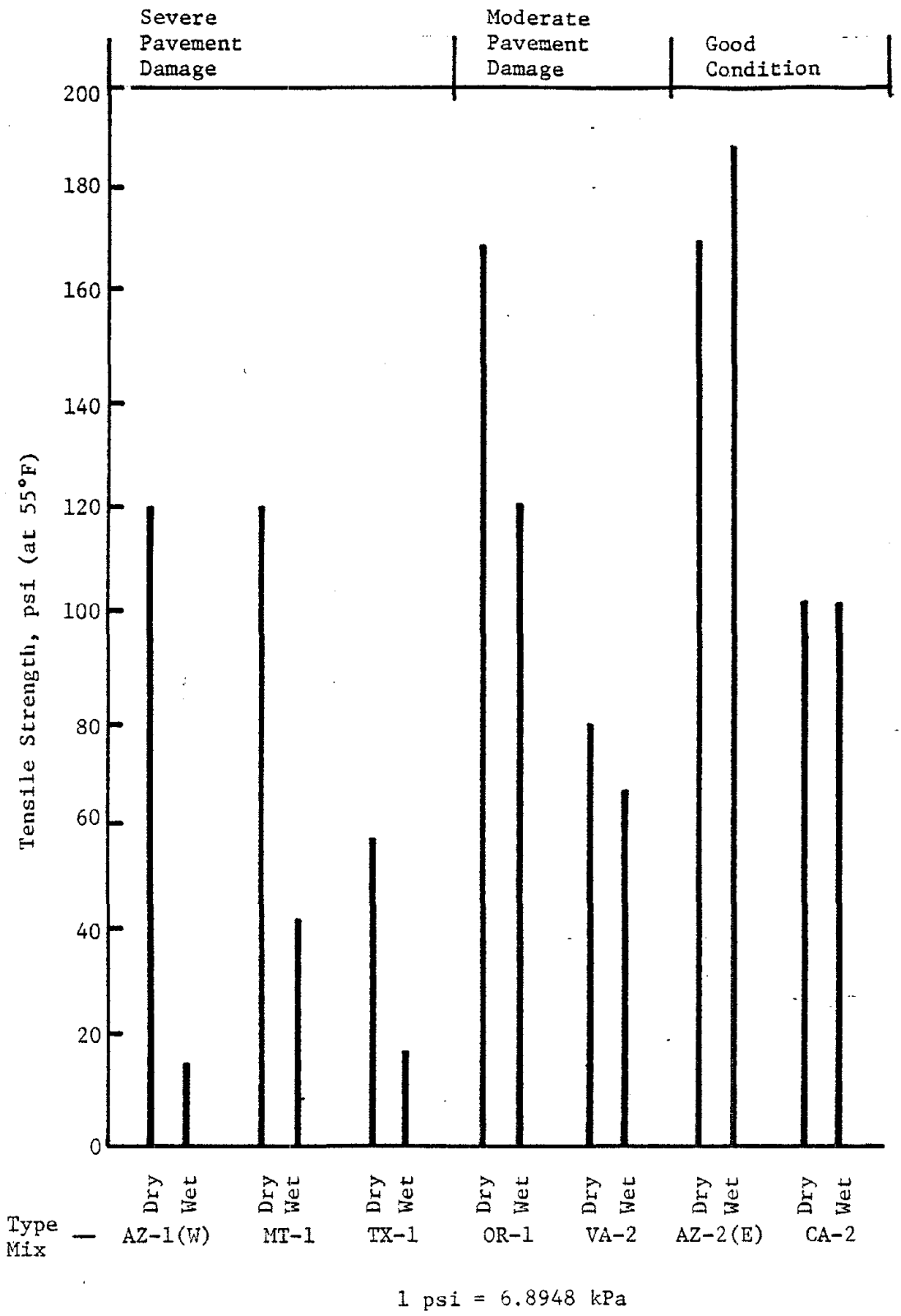
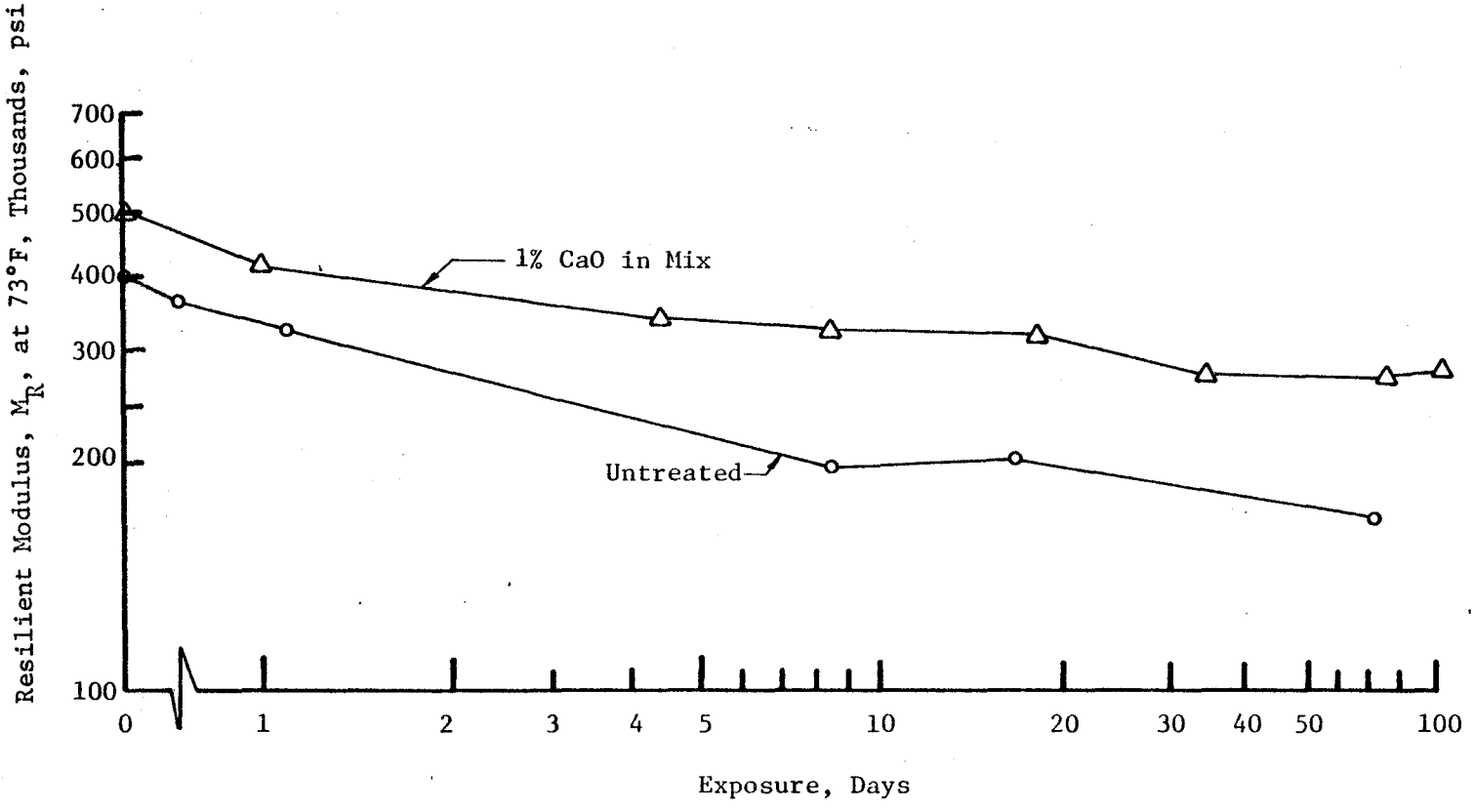
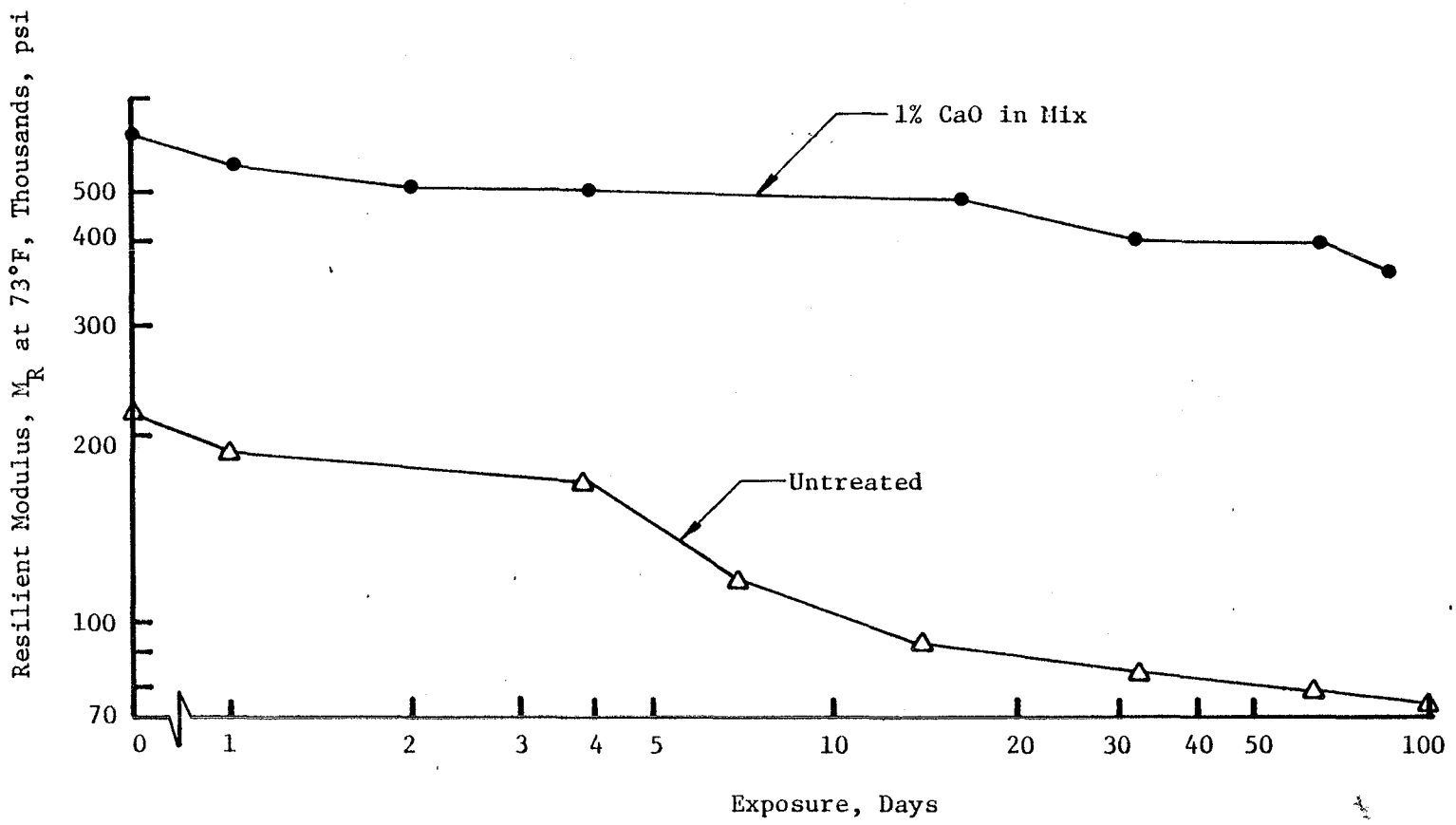


Figure 3.17. Effect of moisture exposure on the tensile strength of field cores (Ref. 41).



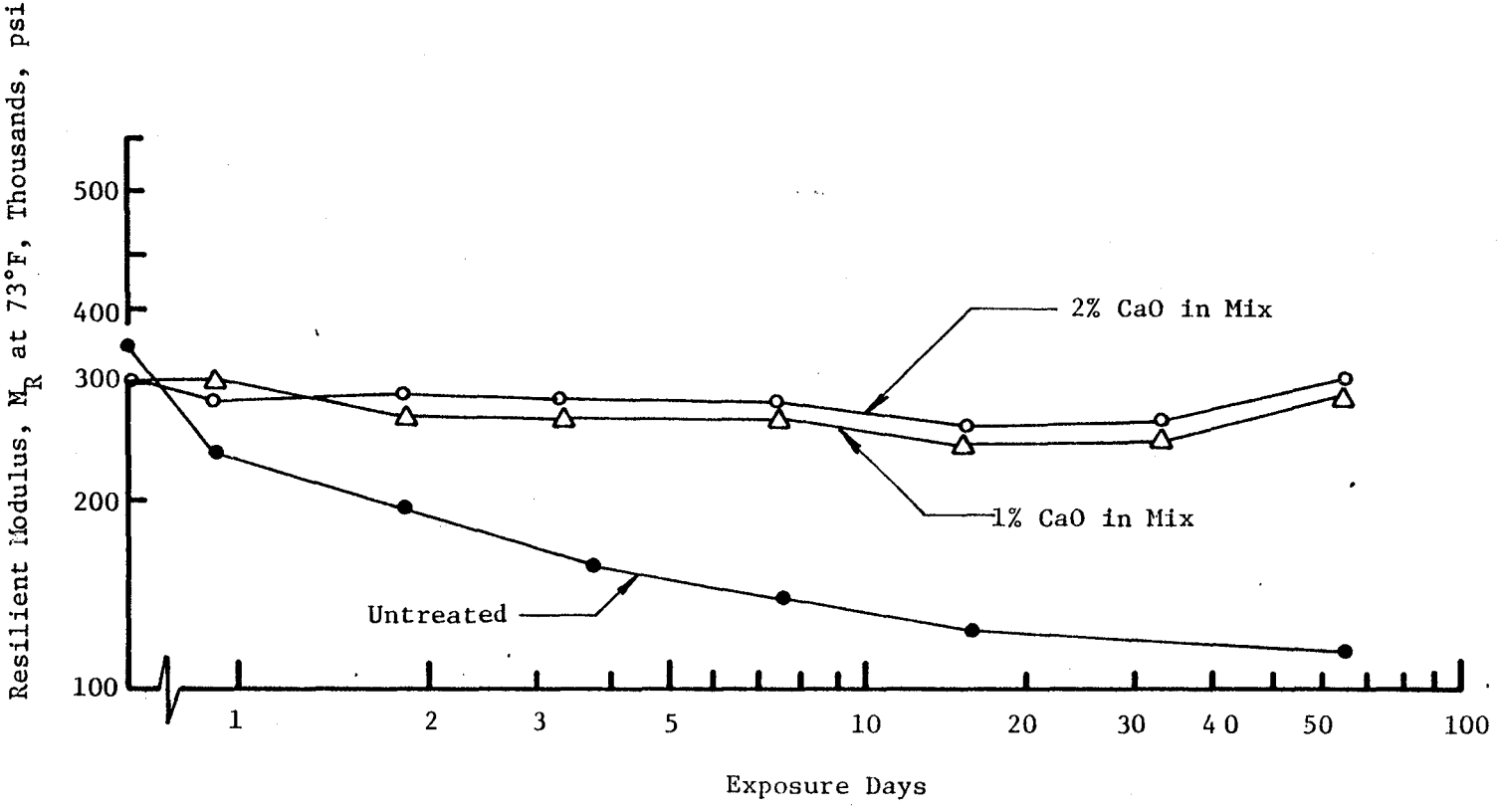
1 psi = 6.8948 kPa

Figure 3.18. Effect of moisture exposure on the resilient modulus of limestone mixes with and without lime (Ref. 42).



1 psi = 6.8948 kPa

Figure 3.19. Effect of moisture exposure in the M_R of watsonville granite mixes with and without lime (Ref. 42).



1 psi = 6.8948 kPa

Figure 3.20. Effect of moisture exposure on the M_R of Cache Creek gravel mixes with and without lime (Ref. 42).

information is available that indicates pavement layers constructed of these materials can meet the premium pavement requirements. Figure 3.21 illustrates the relationship between modulus and flexural strength for various concretes. A review of this figure indicates that a low-modulus, high strength concrete would be advantageous for zero-maintenance pavements. For most cases in this report, it is assumed that low modulus concrete has a modulus of 2 X 10^6 psi (13.78 X 10^6 kPa). However, this value can change depending on the type of aggregate and mix design requirements.

Therefore, both elastic layer theory and discrete element analysis were used to determine the effect of a variation in modulus or the minimum strength required for a different modulus (Figure 3.21). For a concrete pavement structure varying in thickness from 8 to 14 inches (20 to 36 cm) with a 4 inch (10 cm) asphalt treated drainage layer over subgrades with moduli varying from 5,000 to 20,000 psi (34,500 to 137,800 kPa), both elastic layer theory and discrete element analysis were used to compute the tensile stresses associated with varying concrete moduli. These stresses were then used to compute the 18 kip (80 kN) equivalent axle loads for each condition for concrete flexural strengths of 550, 500 and 450 psi (3790, 3450, and 3100 kPa) @ 28 days. The modulus was then varied and stresses recomputed for each flexural strength. Using the same 18 kip (80 kN) equivalent axle loads computed above, the minimum allowable concrete flexural strength was computed which produced equivalent performance in relation to fatigue cracking. These values were then plotted for the different types of concrete in relation to strength as shown in Figure 3.21. As a result, all strength values above the specific relationships of modulus versus strength will give equal or better performance in relation to fatigue cracking.

INTERLAYERS

Stress absorbing membrane interlayer (SAMI) or strain relieving interlayer and cushion courses, such as a crack relief mix or a coarse graded stone, are used to reduce the increase in stress or strain of the asphalt concrete layer from horizontal or vertical movements at underlying joints or cracks of the base material which result in reflection cracking. Basically, the SAMI layers and cushion courses have been proposed as a method to minimize the occurrence of reflection cracking in composite pavements (Figure 3.22) (Ref. 62[1]). Performance of pavement structures containing the SAMI layer are presented in References

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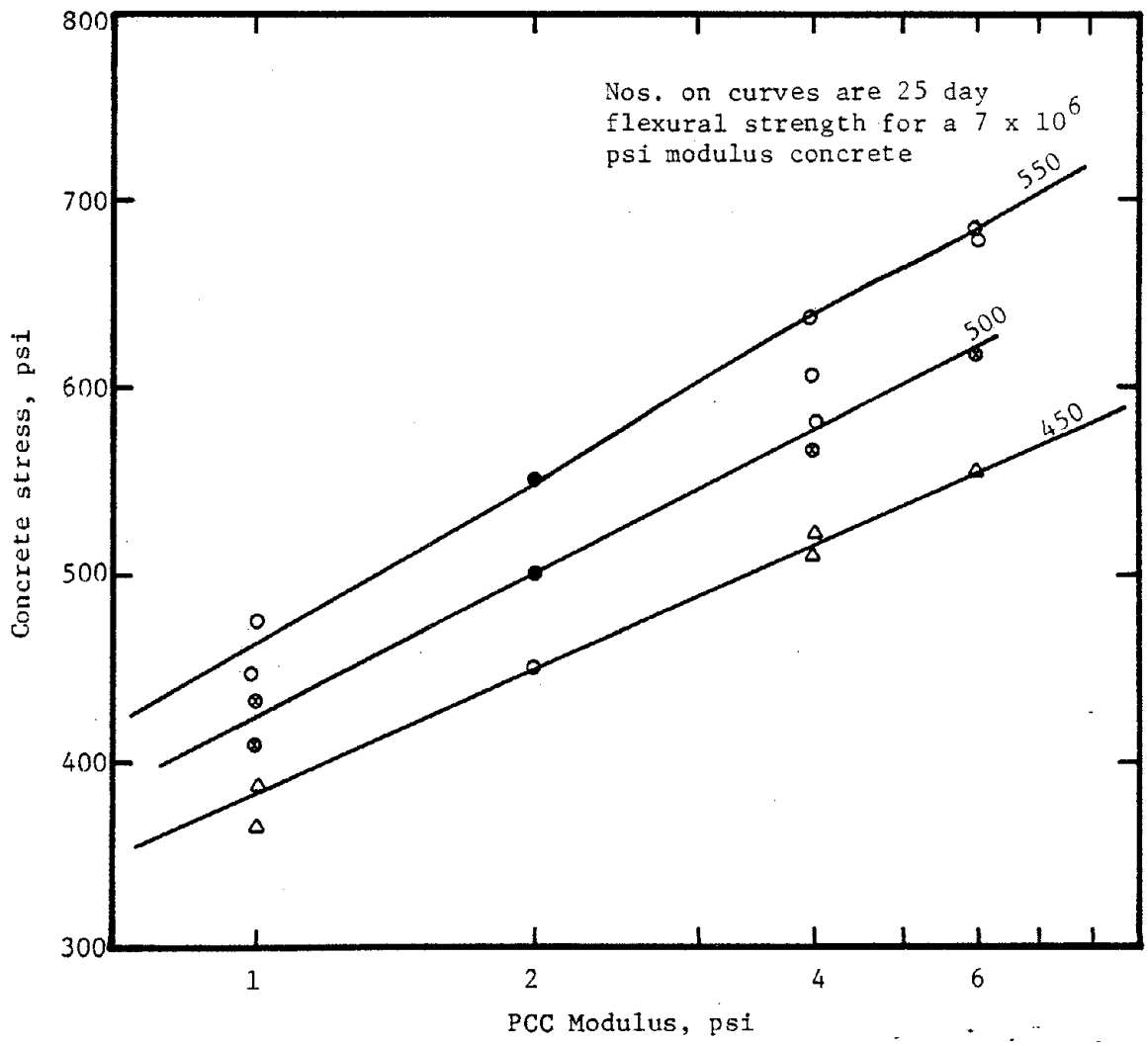
[1]Copple, F. and L.T. Oehler, "Michigan Investigation of Soil Aggregate Cushions and Reinforced Asphaltic Concrete for Preventing or Reducing Reflection Cracking of Resurfaced Pavements", HRR No.239, Highway Research Board, 1968.



1 psi = 6.8948 kPa

Figure 3.21 Illustration showing the theoretical relationship between modulus and stress in concrete to ensure that equal performance occurs as measured by fatigue cracking.

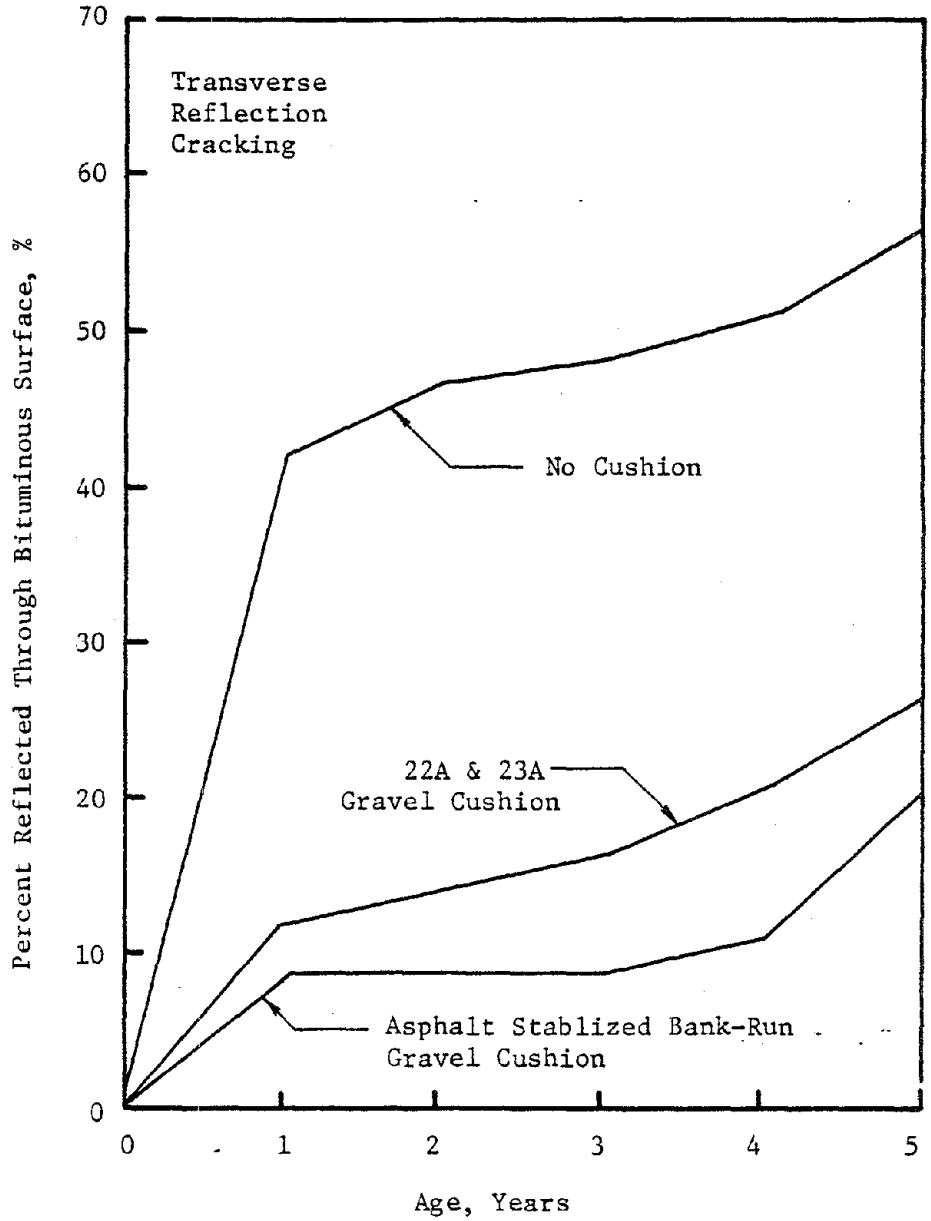


Figure 3.22. Transverse and longitudinal reflection cracking as influenced by an aggregate cushion course (Ref. 62).

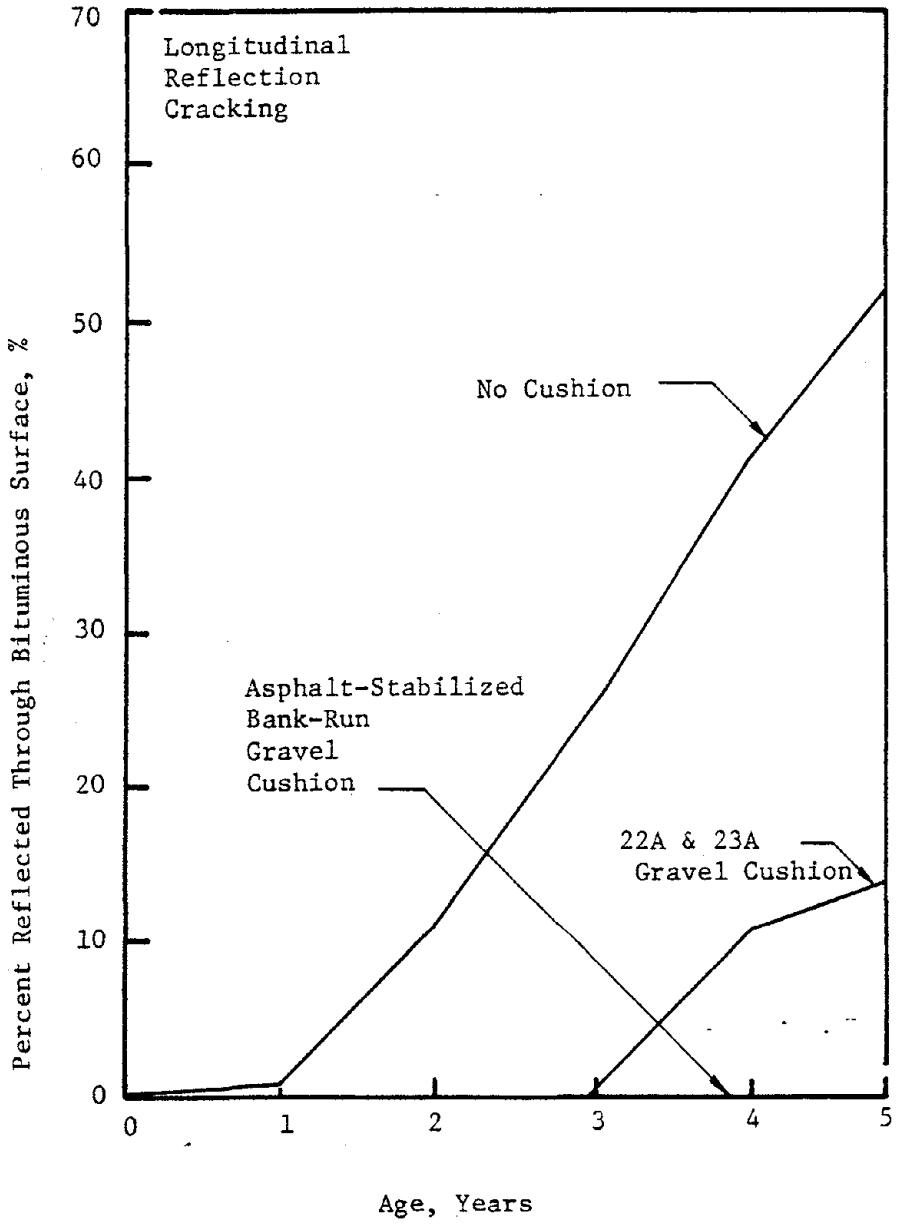


Figure 3.22. Transverse and longitudinal reflection cracking as influenced by an aggregate cushion course (Ref. 62). (Continued)

11, 12, and 63[1]. The effect of the cushion courses on the pavement structure is presented in References 64[2] and 62 as graphically illustrated in Figure 3.22. Construction specifications for the crack relief mix are given in Reference 64.

DRAINAGE LAYERS

Pavements are not usually designed for the rapid elimination of water. As water enters the structure, increased pore pressures can result, causing erosion and deterioration of the pavement structural layers. This deterioration can be represented by the effect moisture has on the resilient modulus of unbound materials. An example is presented in Figure 3.23 (Ref. 65[3]) which illustrates that an increase in moisture content can significantly decrease the resilient modulus resulting in increased fatigue cracking, rutting, and faulting. In order for pavements to meet the zero-maintenance criteria, either materials which are insensitive to moisture must be used, or all moisture migrations must be eliminated, or the infiltration of water must be quickly removed from the pavement structure. Since the first two are economic and construction impossibilities, the third method has been considered for premium pavements. Two drainage systems were considered for zero-maintenance pavements. The first is the use of a drainage system consisting of perforated pipes for the purpose of lowering a high water table. Underdrains or interceptor drains are the most widely employed technique for this objective. The second system consists of a drainage layer to rapidly remove surface moisture infiltration from the pavement structure (Figure 3.24) (Ref. 66[4]). References 67[5], 68[6], 69[7], and 70[8]

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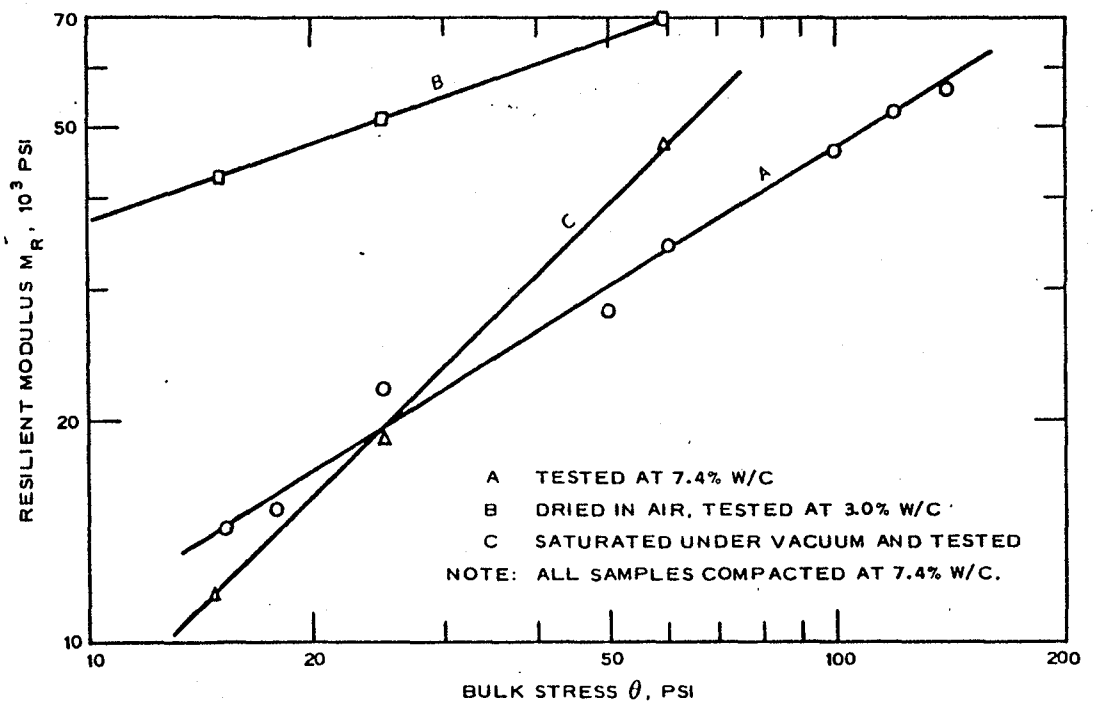
[1]Galloway, B.M., "Use of Rubber Aggregate in a Stress Relieving Interlayer for Arresting Reflection Cracking of Asphalt Concrete Pavements", Proceedings, International Symposium on Use of Rubber in Asphalt Pavements, 1971.

[2]"Preventing Reflection Cracking with an Asphalt Crack-Relief Layer", Construction Leaflet No. 16, The Asphalt Institute, December 1976.

[3]Cook, J.C. and M. Krukar, "Evaluation and Analysis of Results from Experimental Rings No. 1-4", Volume 5, Research Project Y-993 Supplement, Washington State University, Pullman, Washington, July 1971.

[4]Cedergren, H.R., "Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections", Report No. FHWA-RD-72-30, joint venture by Cedergren/KOA, June 1972.

[5]"Drainage of Asphalt Pavement Structures", MS-15, The Asphalt Institute, May 1966.



1 psi = 6.8948 kPa

Figure 3.23. Resilient modulus versus bulk stress relationship for untreated base material (after Cook and Krukarr) (Ref. 65).

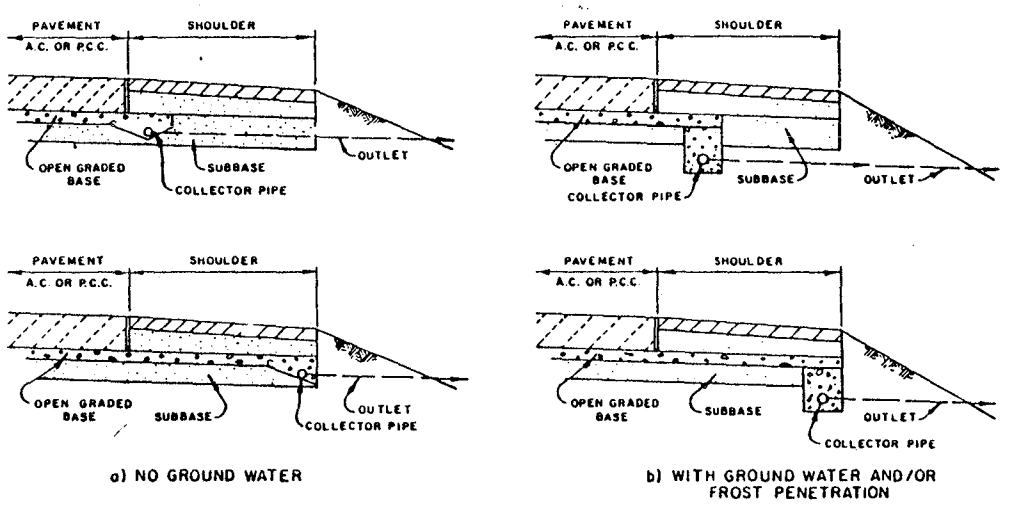


Figure 3.24. Typical cross sections of subdrainage systems (Ref. 66).

adequately explain the construction and purpose of each drainage system.

Effective drainage of unbound granular bases restricts transient pore pressure buildup under rapidly applied wheel loads and improves pavement performance under each distress mode - fatigue cracking, permanent deformation, and frost heave. Generally, substantial economies in the pavement layer structure may be possible if pore pressures in unbound granular base or subbase materials directly beneath the asphalt bound layers may be precluded or relieved. The drainage layer placed below the base in areas with high water tables should provide effective drainage when the base and subgrade are unfrozen but will not drain the base during midwinter, partial thaws, or during the early part of the spring thaw. When the upper part of the base thaws, drainage to the underlying layer would be blocked by the still frozen condition at the lower level. Ring in Reference 71[1] recognized this problem and advocated a drainage layer placed high within the drainage system. Others have also concluded that the open graded, drainage layer will have to be placed above the graded aggregate base where it will thaw and become functional as a drain before the base itself thaws and requires pore pressure relief. The essential function of the drainage layer in this case will be pore pressure relief; the work described in Reference 58 with drained triaxial tests on saturated gravel showed that the presence of water filling voids of the gravel specimen may not detrimentally affect the resilient modulus as long as pore pressure buildup is prevented. Cedergren has mentioned this concept of placement of the drainage layer directly below the dense graded asphalt surface course and has also advocated an open graded drainage layer of nearly uniform grain sizes, bound with 2 percent asphalt if necessary for stability (Ref. 68).

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[6]Cedergren, H.R., J.A. Arman and K.H. O'Brien, "Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections", Report No. FHWA-RD-73-14, Federal Highway Administration, February 1973.

[7]"Implementation Package for a Drainage Blanket in Highway Pavement Systems", Federal Highway Administration, May 1972.

[8]Cedergren, H.R., Drainage of Highway and Airfield Pavements, John Wiley and Sons, 1974.

[1]Ring, G.W., "Seasonal Strength of Pavements", Proceedings, Oslo Symposium on Frost Action in Roads, Vol. III, OECD, 1973.

It has been reported in References 32 and 72[1] that the spring reduction in subgrade reaction in plate bearing tests can be very large, indicating a reduction in the fatigue life. For rigid pavements, increase in moisture near joints or the pavement edge can create substantial void areas due to pumping thus increasing stress levels. For this condition adequate drainage is more essential in areas of seasonal frost than elsewhere because water is responsible for the majority of damaging effects at low temperatures.

SPECIAL ENVIRONMENTAL MATERIALS

Expansive and frost susceptible soils will be identified in accordance with References 18, 30, 49, and 73[2]. For frost susceptible soils, two techniques should be used depending on the frost penetration into the pavement structure. The first is encapsulating the material in a membrane to maintain the same moisture content during construction. This technique can be used in areas where the frost penetration is generally less than three feet. Figure 3.25 (Ref. 74[3]) illustrates the effect of encapsulation on deflection during different portions of the freeze/thaw cycle. The in-service and laboratory performance and construction of encapsulated materials are presented in References 75[4], 76[5], and 77[6].

In areas where the frost penetration is much greater than three feet, an insulating material may be warranted to protect the materials

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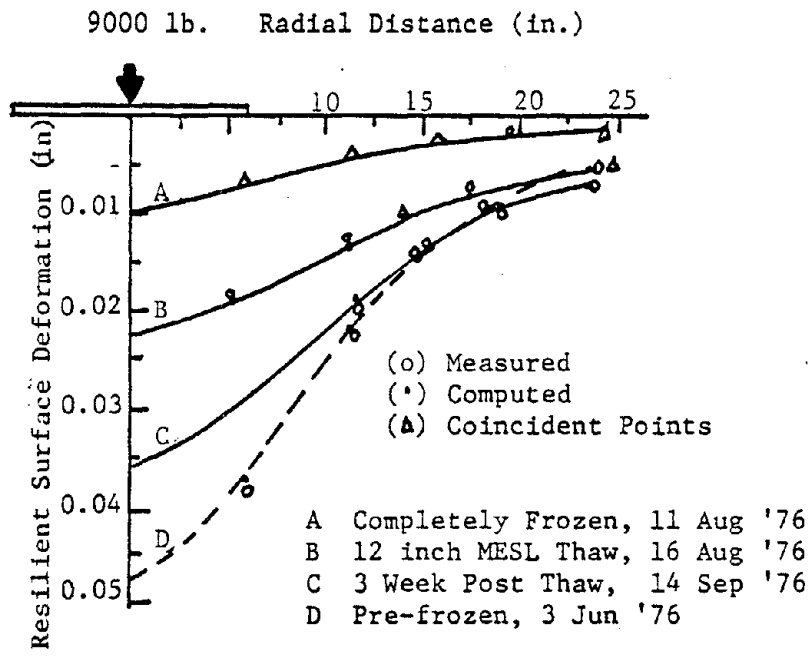
[1]Linell, K.A. and J.F. Haley, "Investigation of the Effect of Frost Action on Pavement Supporting Capacity", HRB Special Report 2, Highway Research Board, 1952.

[2]Snethen, D.R., L.D. Johnson and D.M. Patrick, "An Evaluation of Expedient Methodology for Identification of Potentially Expansive Soils", Report No. FHWA-RD-77-94, Federal Highway Administration, June 1977.

[3]Smith, N., R.A. Eaton and J. Stubstab, "Repetitive Loading Tests on Membrane-Enveloped Road Sections During Freeze-Thaw Cycles", CRREL Report 78-12, Cold Regions Research and Engineering Laboratory, May 1978.

[4]Sayward, J.M., "Evaluation of MESL Membrane-Puncture, Stiffness, Temperature, Solvents", CRREL Report 76-22, Cold Regions Research and Engineering Laboratory, June 1976.

[5]Smith, N. and D.A. Pazsint, "Field Test of a MESL (MembraneEnveloped) Soil Layer) Road Section in Central Alaska", CRREL Technical Report 260, Cold Regions Research and Engineering Laboratory, July 1975.



1 lb = 0.454 kg 1 in = 25.4 mm

Figure 3.25. Typical measured and computed resilient surface deformations of silt MESL (Ref. 74).

from differential frost heave. An example of the insulation's performance in reducing freezing temperatures and resulting frost heave is given by Penner (Ref. 53), and is illustrated in Figure 3.11.

Subgrade temperatures of insulated areas are usually warmer in the winter than in an uninsulated section. The temperature above the insulation is usually somewhat cooler due to the reduction of heat loss from below. The number of freeze/thaw cycles which the pavement experiences can be influenced by insulation also. Some of the shallowest insulated sections appear to experience about 80 percent of the freeze/thaw cycles which occur in uninsulated sections. One disadvantage of insulation is that freezing or icing of the pavement surface may develop more frequently and at a faster rate than without insulation, thereby increasing the safety hazards in terms of skid resistance. Also, from various deflection studies, deflection values were greater in the insulated section than in the uninsulated section at first. But with time, the deflections on the insulated section decreased below deflections on the uninsulated sections (Ref. 78[1]).

OTHER MATERIALS

Other materials were considered for use in the candidate zero-maintenance pavement cross-sections, but were not selected because of one or more of the following reasons: 1) inexperience in material performance, 2) expense in placing the materials, 3) insufficient material data, or 4) insufficient performance or test data in relation to distress type. Some of these materials include:

1. An open graded friction course was initially considered as a candidate material in terms of safety, because of the reduced "splash" and "hydroplaning" effects. This material was not used in the final cross-sections because of insufficient performance in relation to traffic and environmental loads.

2. Although gussasphalt has given relatively good performance in Europe, it has not been used to a large extent in the U.S. because of the cost to place this material. Therefore, performance and material information is very scarce, and hence gussasphalt was not selected in any of the candidate cross-sections.

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[6]Webster, S.L., "Implementation Package 74-2, Users' Manual for Membrane Encapsulated Pavement Sections (MEPS)", Federal Highway Administration, June 1974.

[1]Dunphy, W.J., Jr., "Experimental Insulation of a Subgrade in Hampden, Maine", Proceedings, Symposium on Frost Action on Road, Organization for Economics Co-operation and Development, 1974.

3. Carbon black asphalts were not used in any zero-maintenance cross-section because of insufficient performance and laboratory data.

4. Reinforced asphalt concrete was not selected because of the corrosiveness of steel and insufficient data concerning fabric reinforcement.

5. Bond breakers such as stone dust, sand, etc., were not used because of poor past performance. It has been observed that braking and acceleration forces can cause shoving in the asphalt concrete above the bond breaker in composite pavements. Also from past performance, these type of bond breakers do not insure that reflection cracking will not occur.

6. Fabrics, such as Petromat, Mirafi 140, Heavy Duty Bituthene, etc., were not used because performance results have either been inadequate or inconclusive as to their effects.

7. Jointed reinforced concrete pavements were not included as a candidate material because of past performance in relation to plain jointed and continuously reinforced concrete pavements.

8. Fiber reinforced concrete pavements were not selected because of costs and past performance in relation to plain jointed and continuously reinforced concrete pavements.

CANDIDATE CROSS-SECTIONS

A series of informal interviews were held with various state personnel and consultants concerning their views and candidate cross-sections for zero-maintenance composite and flexible pavement designs. Some personnel felt that a flexible or composite (flexible surface) pavement would not last the entire 20-year period without maintenance. Two important aspects included in many of the cross-sections were some kind of drainage system in the pavement structures and the inclusion of a crack relief layer for the composite pavements. Ideas from the 15 personal interviews were incorporated into the candidate cross-sections given below.

CANDIDATE FLEXIBLE CROSS-SECTIONS

Flexible cross-section A (Figure 3.26) was selected as a candidate cross-section because it uses a thick asphalt concrete as the primary load carrying layer, plus a surface layer of dense asphalt concrete which meets the polishing requirements of heavy traffic. A drainage layer is also included to remove water quickly from the pavement structure, which may infiltrate from the surface. The environmental layer is included because of economic reasons; i.e., it may be more economical to use insulation or to protect the subgrade from frost penetration. Cross-section B (Figure 3.26) was selected for the same reasons as

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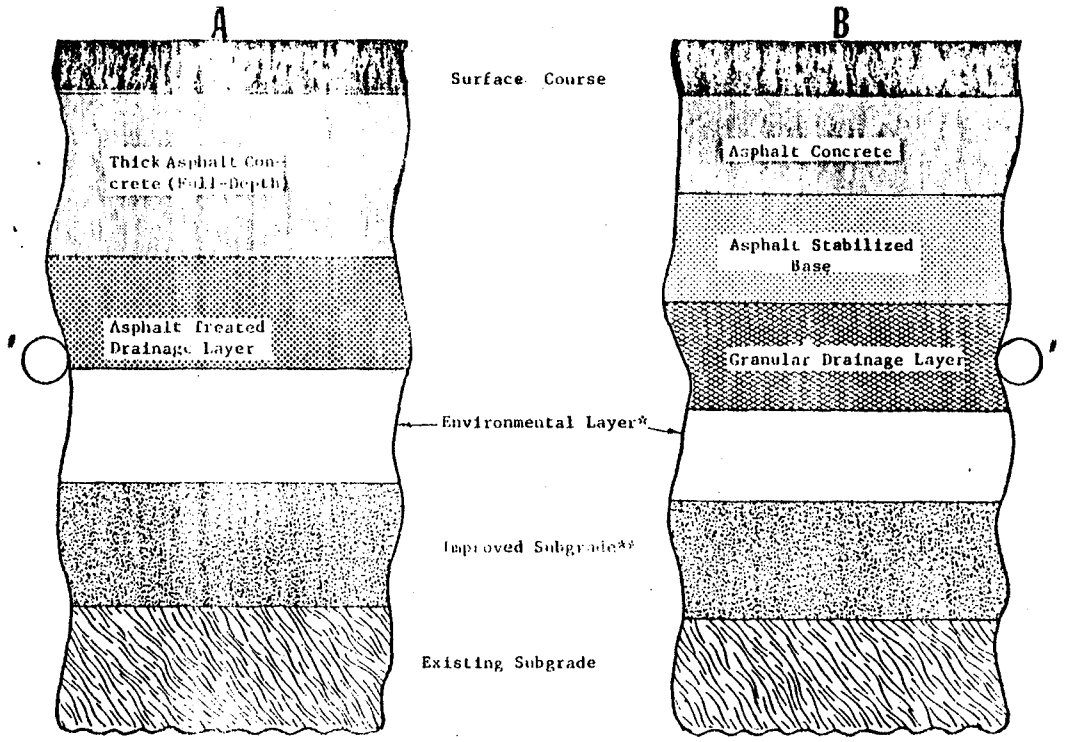
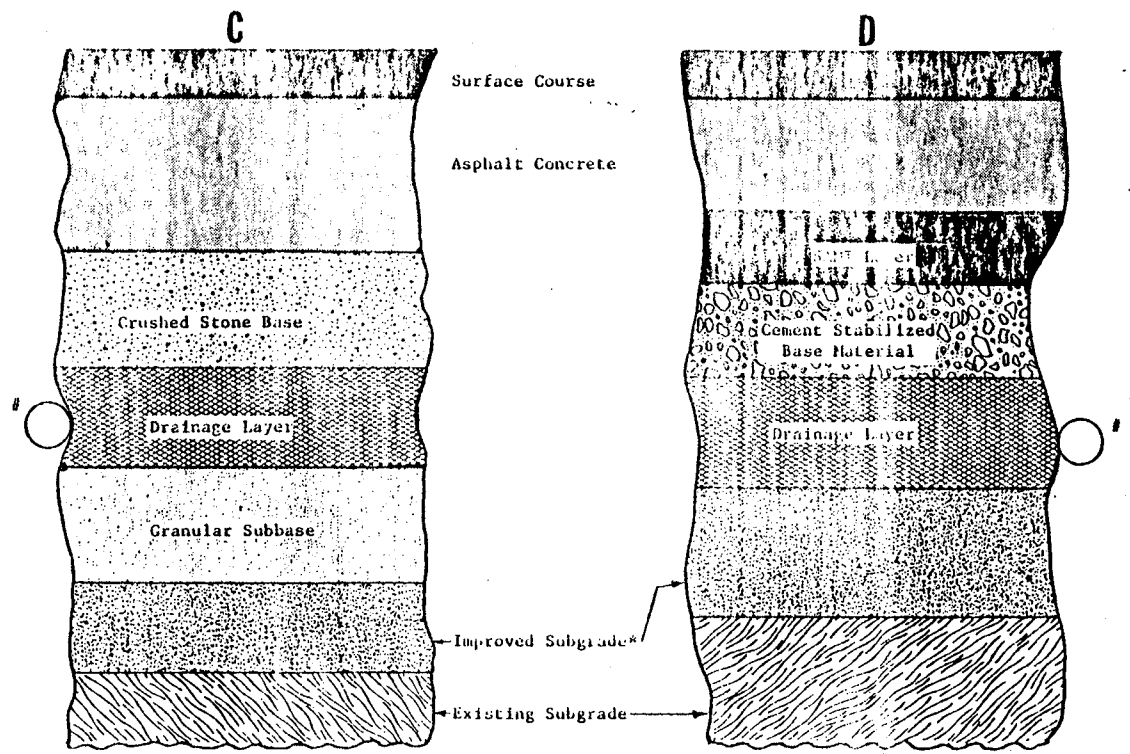


- Perforated Pipe Drainage System to be included
* - Environmental Layer Includes - 1) Insulation, 2) Encapsulated Material (MESL) or 3) Impervious Membrane
** - Subgrade Treatments include - 4) Stabilization, or 5) Compaction Requirements

Figure 3.26. Selected candidate cross-section and material for zero-maintenance flexible pavements.



- Perforated Pipe Drainage System to be Included
* - Environmental Layer Includes - 1) Insulation, 2) Encapsulated Material (MLSL) or 3) Impervious Membrane
** - Subgrade Treatments Include - 4) Stabilization, or 5) Compaction Requirements

Figure 3.26. Selected candidate cross-section and material for zero-maintenance flexible pavements (Continued).

cross-section A, except that an asphalt stabilized base and an asphalt concrete were used as the load carrying layers. The reason for using the asphalt stabilized base material is economics, in that the asphalt treated base is required based on a deflection and subgrade vertical strain criteria. Cross-section C was selected because of the load response behavior of a crushed stone. Normally the resilient modulus will increase as the bulk stress increases for constant moisture contents, thus providing a stronger or stiffer supporting layer at higher stress levels for the asphalt concrete if pumping or erosion do not occur. Cross-section C was also selected for conditions of shallow frost penetration, because it should be more economical to use a layer of a non-frost susceptible granular material than insulation or an encapsulated material to protect a frost susceptible subgrade. Cross-section D was selected because of the support provided by a cement stabilized base material, but is restricted to areas of stable soils and areas of relatively small temperature differentials due to the low-temperature cracking potential of the base material.

Full depth asphalt pavements as defined by the Asphalt Institute are pavements in which asphalt mixtures are employed for all courses above the subgrade or improved subgrade (Ref. 79[1]). By definition full depth asphaltic pavements contain no layers of pervious granular materials which in the past were widely held to be essential, especially in frost areas for drainage of the subgrade and base course. Also full depth pavements, especially those built in dense graded asphalt concrete represent another new concept for design of flexible pavements, particularly for those who have held that the need for strength, stiffness or stability in a particular layer of the pavement structure decreases as the depth of the layer below the surface increases. These two departures from traditional views of flexible pavement structures have left full depth asphalt pavements for frost areas, still a controversial concept.

COMPOSITE CROSS-SECTIONS

Cross-sections E and F (Figure 3.27) were selected as candidate composite cross-sections for basically the same reasons as cross-section A, except that the concrete is the primary load carrying layer. Both a continuously reinforced concrete (CRC) and plain dowelled jointed concrete pavement were selected. The plain jointed concrete pavement was selected because of Darter's findings (Ref. 1), whereas the CRC pavement was selected because of the following reasons. The performance of CRC pavement has been excellent in some cases, but in others spalling around cracks has required considerable maintenance. Therefore, by adding an asphalt concrete layer initially after construction of the CRC pavement,

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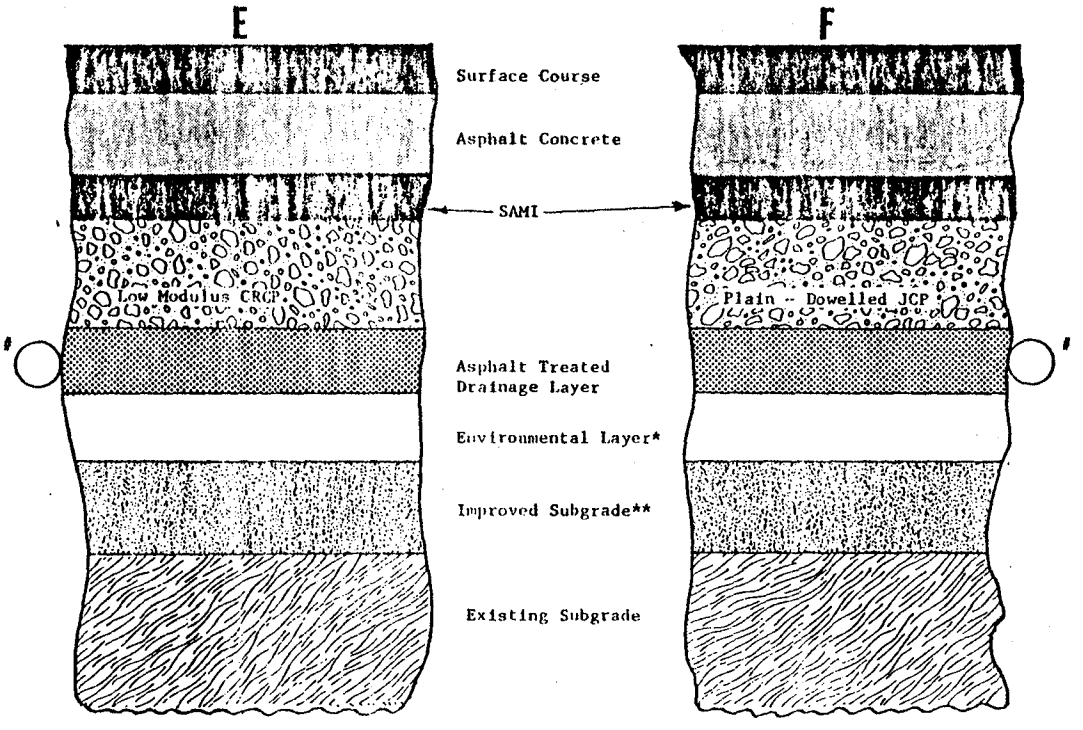
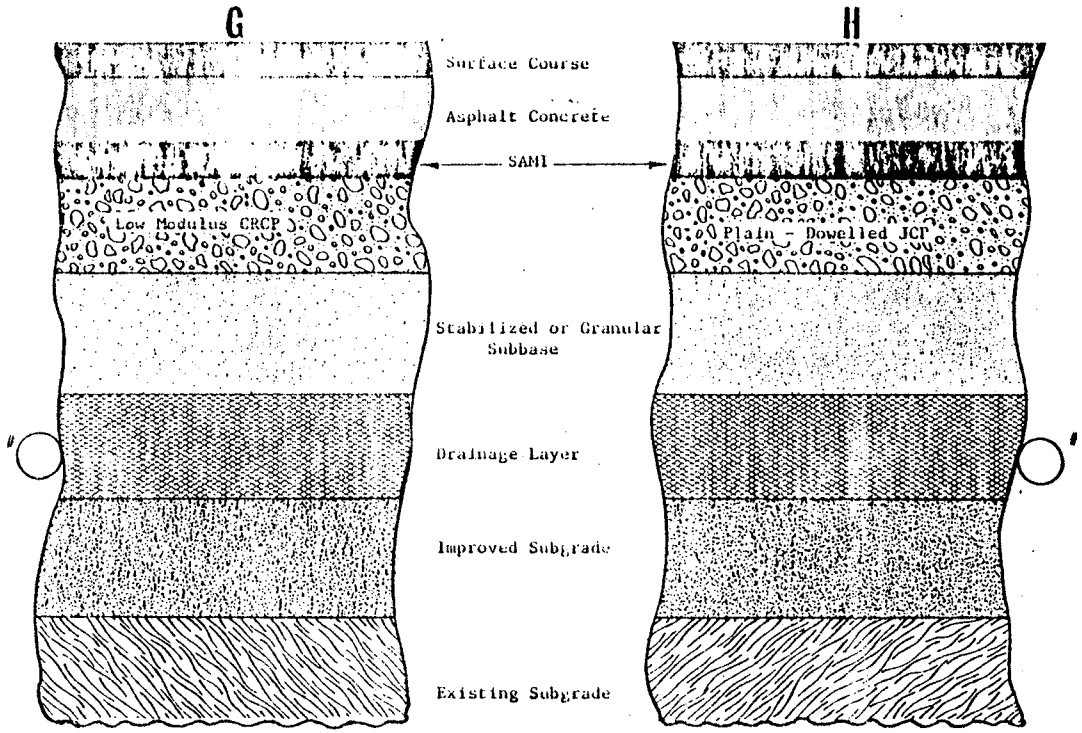
[1]"Thickness Design, Full Depth Asphalt Pavement Structures for Highways and Streets", MS-1, The Asphalt Institute, 1970.



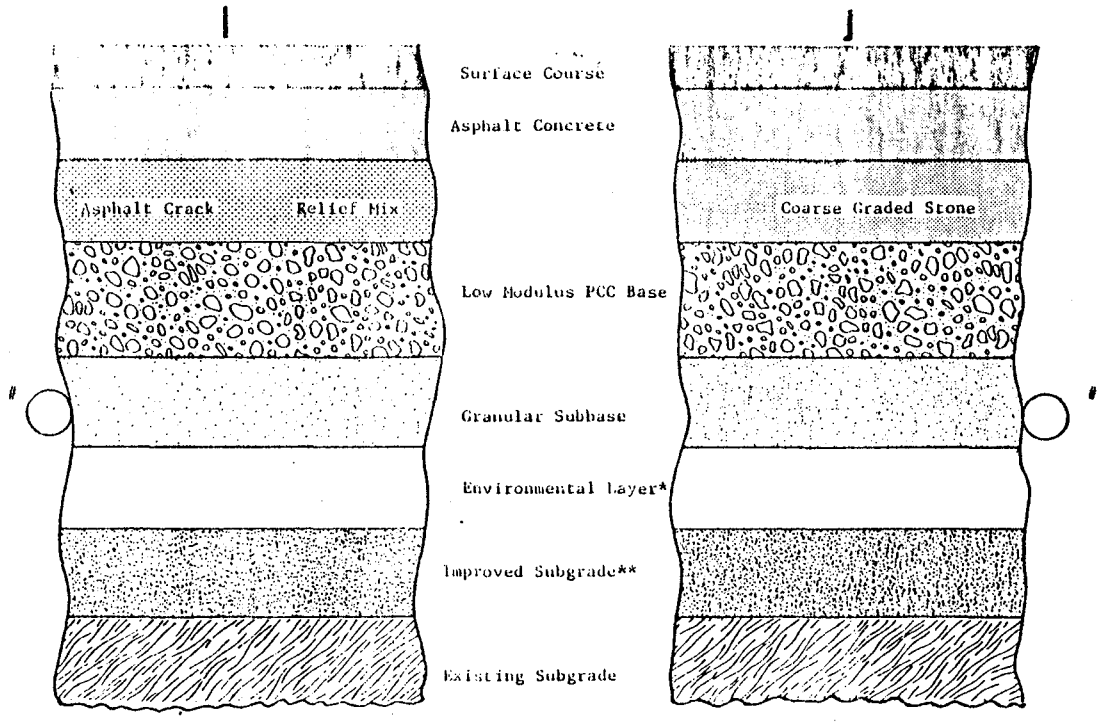
- Perforated Pipe Drainage System to be used
* - Environmental Layer Includes - 1) Insulation, 2) Encapsulated Material, or 3) Impervious Membrane
** - Subgrade Treatments Include - 4) Stabilization or 5) Compaction Requirements

Figure 3.27. Selected candidate cross-section and material for zero-maintenance composite pavements.



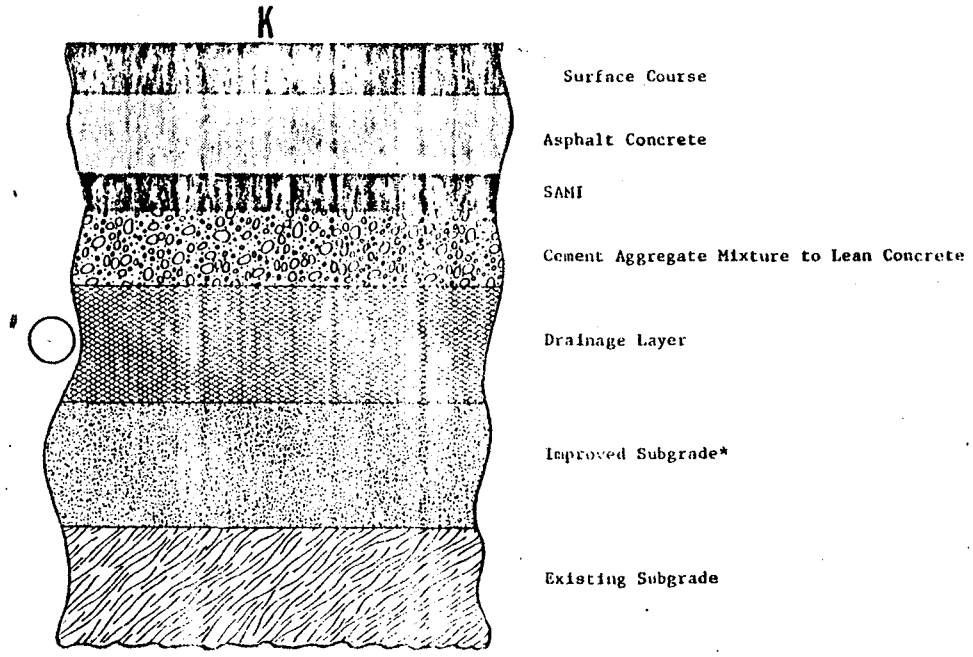
- Perforated Pipe Drainage System to be used
* - Environmental Layer Includes - 1) Insulation, 2) Encapsulated Material (MESL) or 3) Impervious Membrane
** - Subgrade Treatments Include - 4) Stabilization, or 5) Compaction Requirements

Figure 3.27. Selected candidate cross-sections and materials for zero-maintenance composite pavements (continued).



- Perforated Pipe Drainage System to be used
* - Environmental Layer Includes - 1) Insulation, 2) Encapsulated Material (MESL) or 3) Impervious Membrane
** - Subgrade Treatments Include - 4) Stabilization, or 5) Compaction Requirements

Figure 3.27. Selected candidate cross-sections and materials for zero-maintenance composite pavements (continued).



- Perforated Pipe Drainage System to be used
* - Environmental Layer Includes - 1) Insulation, 2) Encapsulated Material (MESL) or 3) Impervious Membrane
** - Subgrade Treatments Include - 4) Stabilization, or 5) Compaction Requirements

Figure 3.27. Selected candidate cross-sections and materials for zero-maintenance composite pavements (continued).

the potential for spalling should be reduced since the asphalt layer will distribute the load across the cracks. Also, the effects of temperature gradients which cause curling of the concrete slab should be greatly minimized because of the asphalt concrete surface. This should decrease the occurrence of edge distress and result in lower stresses at the bottom of the concrete slab. Hence, both were considered potential candidate cross-sections. A SAMI layer was also included in both cross-sections to reduce the potential for reflection cracking at joints or cracks. Several other cross-sections were considered for economic reasons. A low modulus portland cement concrete base that should provide adequate load carrying capacity and the cushion course (asphalt crack relief mix or coarse graded stone) that should prevent any differential vertical deflections from creating reflection cracking. Results of all these studies are given in Volume II of this report (Ref. 5).

CHAPTER 4. DESIGN PROCEDURES AND DISTRESS MODELS

With the more prominent distresses established and the candidate materials and pavement cross-sections identified, existing design procedures were reviewed in relation to the zero-maintenance criteria. Most pavement design equations in use in North American express required thickness and component layers of the pavement structure in relation to a certain level of performance. The equation parameters usually include traffic loads (magnitude and number), properties of construction materials, subgrade support values, and environmental coefficients, for example, the AASHTO Interim Design Guide (Ref. 106[1]). In as much as temperature and frost strongly affect all of these parameters except traffic load: assessments of temperature and frost defects must be essential parts in development of a suitable design for pavements. Performance requirements are necessary to control within tolerable limits the rates of pavement deterioration under all the applicable modes of distress.

DESIGN PROCEDURES

Various design procedures exist to determine the structural requirements of the pavement cross-section. Many of these are in use throughout the United States and have given satisfactory results for limited traffic levels and time periods. Some of these design procedures are presented in Table 4.1 (Ref. 124[2]) in relation to the design criteria. Most of these procedures have been studied rather thoroughly in past reports, hence, they were not dwelled upon in this project. As can be seen in Table 4.1 none of the design procedures plan for all the distress manifestations required for premium pavements. Some are even location or material dependent, in that they do not lend themselves for easy extrapolation to other areas and materials or combination of materials. Therefore, these procedures were eliminated as a basis for the zero-maintenance design procedure.

ANALYTICAL MODELS

There are many available models for predicting the behavior of a pavement structure. Many models were reviewed and studied concurrently with the identification of the major distress manifestations and related material properties and factors. Table 4.2 lists the available models

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[1] AASHTO Interim Guide for Design of Pavement Structure 1972, American Association of State Highway and Transportation Officials, Washington, D.C., 1974.

[2] "A Guide to the Structural Design of Flexible and Rigid Pavements in Canada", Roads and Transportation Association of Canada, 1971.

TABLE 4.1. DESIGN PROCEDURES

DESIGN BASED ON	AASHTO	CORPS OF ENGINEERS	NAVY	ASPHALT INSTITUTE	ACI	PCA	SHELL	POD	CANADA ONTARIO (Ref. 124)
Fatigue		X		X		X	X	X	
Rutting				X			X	X	
Low-Temperature Cracking							X		
Thermal Fatigue									
Reflection Cracking									
Deflection									X
Subgrade Vertical Compressive Strain		X							
Fracture Mechanics									
Elastic Layer Theory or Elastic Equations		X		X		X	X	X	
Discrete Element Analysis									
Finite Element Analysis									
Shrinkage Cracking									
Ravelling									
Empirical	X								X
Roughness	X			X					
Frost Penetration									

TABLE 4.2. ANALYTICAL MODELS AND PROCEDURES CONSIDERED FOR DESIGNING ZERO-MAINTENANCE FLEXIBLE AND COMPOSITE PAVEMENTS

Distress Type	Model	
	Flexible	Composite
Fatigue	VESYS IIIA (Ref. 80) Shell Method (Ref. 81) PDMAP (Ref. 82) OPAC (Ref. 83) and WATMODE (Ref. 84) ARE (Ref. 85)	VESYS IIIA SLAB 49 (Ref. 86) Shell Method Finite-Element Analysis (Ref. 87) RPOD (Refs. 44,27)
Rutting	VESYS IIIA Shell Method PDMAP Rutting Subsystem by Monismith (Ref. 88) DEVPAV (Ref. 89) Huschek Method (Ref. 90) Corps of Engineers (Ref. 59) OPAC	VESYS IIIA Shell Method
Reflection Cracking	--	RFLCR1 (Ref. 27) Majidzadeh (Ref. 91)
Low-Temperature Cracking	VESYS IIIA COLD (Ref. 92) Shahin-McCullough (Refs. 93, 38) OPAC and WATMODE	CRCP-2 (Refs. 94,95) JRCP-2 (Ref. 96) Shahin-McCullough (Ref. 38) VESYS IIIA Haas-Hajek (Ref. 97)

that were considered for this project (Refs. 27, 38, 44, 59, 80[1], 81[2], 82[3], 83[4], 84[5], 85[6], 86[7], 87[8], 88[9], 89[10], 90[11], 91[12], 92[13], 93[14], 94[15], 95[16], and 96[17]). The following items were considered in selecting the primary models for each distress predictor: output, simulation of actual conditions, input simplicity and requirements, computational efficiency and reliability or accuracy. The basic approach used was to review each model's capabilities, limitations, and boundary conditions. Also, when available from previous studies, sensitivity analyses were reviewed for use in determining the effects of material properties on the structural behavior of the pavement. The selection of these models was also based on a detailed review of each model completed by Rauhut, et al., in Reference 4. Using this approach specific models were initially selected for each distress and pavement type to simulate the pavement and estimate the maintenance-free life for different environmental conditions. The models reviewed are summarized in Table 4.3.

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[1]Kenis, W.J., "Predicted Design Procedures - A Design Method for Flexible Pavements Using the VESYS Structural Subsystem", Proceedings, Fourth International Conference on Stuctural Design of Asphalt Pavements, Vol. I, August 1977.

[2]Claessen, A.I.M., J.M. Edwards, P. Sommer, and P. Uge, "Asphalt Pavement Design - The Shell Method", Proceedings, Fourth International Conference on Structural Design of Asphalt Pavement, Vol. I, August 1977.

[3]Finn, F.N., C. Saraf, R. Kulkarni, K. Nair, W. Smith and A. Abdullah, "Development of Pavement Structural Subsystems", Final Report, NCHRP Project 1-10B, National Cooperative Highway Research Program, February 1977.

[4]Meyer, F.R.P., and R.C.G. Haas, "A Working Design Subsystem for Pavement Deformation in Asphalt Pavements", Proceedings, Fourth International Conference on Structural Design of Asphalt Pavements, Volume I, August 1977.

[5]Meyer, F.R.P., A. Cheetham and R.C.G. Haas, "A Coordinated Method for Structural Distress Predictions in Asphalt Pavements", a paper presented at the Meeting of the Associates of Asphalt Paving Technologists, Lake Buena Vista, Florida, February 1978.

[6]Austin Research Engineers Inc, "Asphalt Concrete Overlays of Flexible Pavements - Volume I, Development of New Design Criteria", Report No. FHWA-RD-75-75, Federal Highway Administration, June 1975.

[7]Treybig, H.J., W.R. Hudson, and A. Abou-Ayyash, "Applications of Slab Analysis Methods to Rigid Pavement Problems", Research Report 56-26, Center for Highway Research, The University of Texas at Austin, May 1972.

TABLE 4.3. DISTRESS MODELS REVIEWED

DISTRESS	VESYS	PDMAP	POD	TC-1	HAAS-HAJEK	RFLCR-1	CRCP-1	JRCP-1	COLD	OPAC	WATMODE	MAJIDZADEH	DEVPAV	HUSCHEK
Fatigue Cracking	X	X	X											
Rutting	X	X	X											
Low-Temperature Cracking	X	X		X	X		X	X	X					
Reflection Cracking						X						X		
Shrinkage Cracking							X	X						
Ravelling														
Frost Heave														
Thermal Fatigue				X										
Steel Rupture							X	X						
Empirical					X									
Swelling Soils														
Fracture Mechanics														
Elastic Layer Theory or Elastic Equation	X	X	X	X		X	X	X	X					
Discrete Element Analysis														
Finite Element Analysis												X		

The models selected are given in Table 4.4. These models were selected to be compatible with other FHWA projects and design work to minimize the variation in inputs and reported results.

VESYS III (Ref. 80)

A careful evaluation indicated that none of the models listed above for flexible and composite pavements have better capabilities for predicting rutting and fatigue cracking distress than VESYS A. As a result, the VESYS III computer program was selected as the primary structural model, because it has the capabilities of predicting fatigue cracking, rutting, low temperature cracking, and roughness. It also has the capabilities of an incremental breakdown of the axle load distribution by tire radius and corresponding time pressure, seasonal modification of material properties and variations in material properties in both strength and stiffness. The primary difference, between the VESYS formulations and others in common use, is that the VESYS equations have been expanded using probability theory so that the variability of pavements and environmental parameters can be considered in the predictions.

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[8]Huang, Y.H. and S.T. Wang, "Finite-Element Analysis of Concrete Slabs and Its Implications for Rigid Pavement Design", Highway Research Record 466, Highway Research Board, 1973.

[9]Monismith, C.L., K. Inkabi, C.R. Freena and D.E. McLean, "A Subsystem to Predict Rutting in Asphalt Concrete Pavement Structures", Proceedings, Fourth International Conference on Structural Design of Asphalt Pavements, Volume I, August 1977.

[10]Kirwan, R.W., M.N. Snaith and T.E. Glynn, "A Computer-Based Subsystem for the Prediction of Pavement Deformation", Proceedings, Fourth International Conference on Structural Design of Asphalt Pavements, Volume I, August 1977.

[11]Huschek, S., "Evaluation of Rutting Due to Viscous Flow in Asphalt Pavements", Proceedings, Fourth International Conference on Structural Design of Asphalt Pavements, Volume I, August 1977.

[12]Majidzadeh, K. and G.C. Suckarieh, "Analytical Modeling and Field Verification of Thermal Stresses in Overlays", TRR No. 632, Transportation Research Board, 1977.

[13]Christison, J.T., "Response of Asphalt Pavements to Low Temperatures", Ph.D. Dissertation, University of Alberta, 1972.

TABLE 4.4. MODELS SELECTED FOR DETERMINING THE MAINTENANCE-FREE LIFE FOR COMPOSITE AND FLEXIBLE PAVEMENTS AND EACH DISTRESS TYPE

Design Requirement	Pavement Type	
	Flexible	Composite
Structural Model	Elastic Layer Theory	Elastic Layer Theory plus Discrete Element Analysis
Fatigue	VESYS III-A	VESYS III-A
Rutting	VESYS III-A	VESYS III-A
Reflection Cracking		RFLCR1
Low-Temperature Cracking	Shahin-McCullough	Shahin-McCullough (Asphalt) CRCP-2 or JRCP-2 (Concrete)
Roughness PSI	VESYS III-A	VESYS III-A

In VESYS III, the fatigue cracking distress is predicted using the classical fatigue equation and linear summation of cycle ratios (Minor's Hypothesis) to predict damage at any point in time due to an established axle load distribution and traffic rate. Rutting is calculated as the difference in predicted total and elastic displacements at the surface. This is accomplished by separate sets of solutions from the elastic layer structural model, one with the normal elastic moduli and the other with the moduli modified by permanent deformation characterizations to delete the permanent strains. The permanent strains in each layer are accumulated through separate solutions with the layer stiffnesses modified. VESYS III also has the capability to predict with time the decrease in pavement serviceability as a function of applied loads and material variability. A detailed description of the VESYS III program is presented in References 37 and 80.

PROGRAM TC-1 (Ref. 38)

The Shahin-McCullough model (Program TC-1) was selected for prediction of low temperature of thermal cracking (described in detail in References 38 and 93) because it has the capability to estimate low-temperature cracking as a function of environmental and material properties for asphalt concrete surfaces.

The steps for predicting low temperature cracking in asphalt concrete pavements are, first the pavement temperature model, which is an improved version of the model developed by Barber (Ref. 98[1]), is used to

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[14]Shahin, M.Y., "Design System for Minimizing Asphalt Concrete Thermal Cracking", Proceedings, Fourth International Conference on Structural Design of Asphalt Pavements, August 1977.

[15]McCullough, B.F., A. Abou-Ayyash, W.R. Hudson and J.P. Randall, "Design of Continuously Reinforced Concrete Pavements for Highways", NCHRP 1-15, National Cooperative Highway Research Program, 1975.

[16]Ma, J.C.M., "CRCP-2, An Improved Computer Program for the Analysis of Continuously Reinforced Concrete Pavement", University of Texas at Austin, Master Thesis, August 1977.

[17]Rivero-Vallejo and B.F. McCullough, "Drying Shrinkage and Temperature Drop Stresses in Jointed Reinforced Concrete Pavement", Report 177-1, Center for Highway Research, The University of Texas at Austin, August 1975.

[1]Barber, E.S., "Calculations of Maximum Pavement Temperatures from Weather Reports", Bulletin 168, Highway Research Board, 1957.

predict an hourly pavement temperature as a function of air temperature, wind velocity, solar radiation, asphalt mixture thermal properties, and depth from the pavement surface. Second, using this temperature information the model calculates the thermal stresses and strains in the asphalt mixture as a function of its stiffness and the temperature changes. This model in turn consists of four interactive submodels for determining asphalt stiffness, asphalt aging, asphalt mixture stiffness, and thermal stresses and strains. Low-temperature cracking is predicted as the percent of surface area cracked using probabilistic methods to predict whether the thermal stresses will exceed the mixture strength. Both the thermal stress and the thermal strength are assumed to vary and these variabilities are defined in terms of their standard deviations. The thermal stress is a function of the asphalt mixture stiffness, which varies with temperature and loading time. The asphalt mixture strength is also a function of the same variables.

The thermal fatigue cracking model adds thermal fatigue cracking caused by the daily temperature cycling to the low temperature cracking. This thermal-fatigue cracking model grew out of the realization that thermal cracking of asphalt concrete pavements occurs in the temperate prone zones of the United States as well as the northern zones having lower temperatures. Study has attributed this occurrence of thermal cracking at relatively low levels of strain to the fatigue effects of daily temperature cycling.

The Shahin-McCullough model is more thorough than other low temperature cracking models. It not only provides predictions of pavement temperatures, but also provides predictions of asphalt cement stiffness at these temperatures, the changes in stiffness due to asphalt aging in time, and the asphalt mixture stiffness. Program COLD was not used for this study since it predicts only stresses in a pavement at a point and when a crack will occur. It does not include any predictions of crack spacing or area cracked, both of which may be obtained from the low temperature cracking model selected. The Hajek-Haas model for low-temperature cracking was also not selected since it has Canadian data as a basis. In general, the Shahin-McCullough model has a more thorough theoretical base and offers more generality for the studies required for this project.

SLAB49

One of the most important factors influencing the occurrence of fatigue cracking in composite or rigid pavements is the critical loading condition, regarding the boundary conditions. For concrete pavements three types of loadings can occur; interior, edge or corner conditions. In using VESYS III to predict the fatigue cracking associated with composite pavements, an interior loading condition is assumed since elastic layer theory is used. But, in actuality a more critical loading

or higher stress exists in the pavement due to edge effects or cracks and joints. Therefore, the discrete element analysis program (SLAB49) was selected to insure that the pavement would perform as designed by VESYS III. For a complete description of the SLAB49 program the reader is referred to Reference 80. In using VESYS III a fatigue curve representative of an interior condition must be used, and in using SLAB49 a fatigue curve representative of an edge or corner loading condition must be used. Both programs were selected for use in designing premium composite pavements.

Program RFLCR1 (Ref. 27)

The reflection cracking model RFLCR1 includes analysis of two types of distress mechanisms. One is a form of reflection cracking in the overlay due to horizontal movements of the rigid slab caused by temperature and moisture changes. The second is shear cracking due to differential vertical movements at joint or cracks in the underlying rigid pavement.

The reflection cracking analysis consists of evaluating overlay thickness using the following concept:

$$C_R = f\ (E_o,\ E'_o,\ D,\ \Delta T,\ \alpha,\ F_i,\ w_d,\ X_{BB})$$

where:

C_R = reflection cracking

E'_o = dynamic modulus of asphalt or concrete

E_o = creep modulus of asphalt or concrete

D = thickness of existing pavement or overlay

ΔT = temperature change of pavement materials

α = coefficient of volume change for pavement materials

F_i = force-movement relationship between pavement layers resulting from friction, adhesion, bearing, etc.

w_d = differential deflection at crack or joint

X_{BB} = width of bond breaker

The derivations for the reflection cracking prediction equations are quite complex and the reader is referred to Reference 58.

In studying the effects of material types and properties on reflective cracking, most of the above properties were reviewed in relation to minimizing transverse cracking. The most important properties were the

stiffness modulus and creep compliance for the flexible overlay, the thermal coefficient of the stiffer underlying layer, and the bond between the flexible and rigid layer.

Program CRCP-2

The dimensional changes in a continuously reinforced concrete pavement, caused by drying shrinkage of the concrete and temperature variation after curing, were investigated by McCullough et al., and the design method utilizing Program CRCP-1 was developed in the study described in Reference 94. This computer program was subsequently improved by Ma (Ref. 95). The method also considers stresses imposed by wheel loads. Due to the accumulated friction and the terminal treatments used in the construction, the slab model assumes an anchorage at each end so that the pavement within the anchorages will maintain a fixed length.

The difference in the thermal coefficients of the steel and the concrete together with the drying shrinkage of the concrete enable determination of the internal stress in the reinforced slab. Using the friction-movement characteristic of the slab, and the soil determined by controlled experiments, the degree of restraint due to the soil frictional resistance can be estimated. By assuming equilibrium in the system, the stress of one material can be correlated to the stress of the adjacent material. Finally, an incremental approach was adopted to predict the formation of the transverse cracks as a function of time by comparing the concrete stress with the strength of the concrete.

In the development of the model, the following assumptions were made which were reviewed by Rauhut, et al., in Reference 4:

1. A crack occurs when the concrete stress exceeds the concrete strength, and after cracking, the concrete stress at the location of the crack is zero.

2. The concrete and steel properties are linearly elastic.

3. In the fully bonded sections of the concrete slab, there is no relative movement between the steel and the concrete.

4. The force-displacement curve which characterizes the frictional resistance between the concrete slab and the underlying base is elastic.

5. Temperature variations and shrinkage due to drying are uniformly distributed throughout the slab, and hence, a one-dimensional axial structural model is adopted for the analysis of the problem.

6. Material properties are independent of space.

7. The effects of concrete creep and slab warping are neglected.

The spacing of transverse cracks that occur naturally in CRCP is perhaps the most important variable affecting the behavior of the pavement. Relatively large distances between cracks result in a higher accumulation of drag forces from the subgrade due to frictional resistance, thus producing high steel stress at the crack and wide crack widths. Closer crack spacing reduces the frictional restraint, thus decreasing the steel stress and the crack width. It is clear that the transverse cracks in CRCP are due to the thermal contraction and shrinkage of the concrete slab. Assumption 5 above is certainly questionable as there would be some temperature gradient in the slab and shrinkage is known to start at the surface and progress downward over a period of days or months. However, the one-dimensional axial model used in this method is the only rational model available which considers the internal forces caused by the difference in thermal coefficient between the concrete and the steel material and therefore is a valuable tool for the analysis of CRC pavements. For the comparative purposes of this project, the error from assumption 5 should not be important, since for composite pavements, an asphalt concrete layer exists at the surface.

To identify the requirements of zero-maintenance pavements, the CRCP-2 computer model is believed to be the best simulative pavement structural model available for continuously reinforced concrete pavement. This model deals directly with the external (wheel load) and internal (environmental) forces that cause the formation of the transverse cracks and influence the width of the crack. The predictions of crack width and the crack spacing using this model will allow designing for the effects of various material types and environments. By considering limiting criteria for crack width and the crack spacing based on field observations of distress and analysis, percent steels can be determined. Although distresses such as spalling, longitudinal cracking, etc., are not included in this model, they are closely correlated to crack width and crack spacing and may be optimized on that basis. The following equations were developed from the program to simplify its use in design:

$$\overline{X} = \frac{1.322\ (1 + \frac{f_t}{1000})^{6.695} \times (1.\frac{\alpha s}{2\alpha c})^{1.153} \times (1 + \emptyset)^{2.187}}{(1 + \frac{\sigma w}{1000})^{5.2007} \times (1 + p)^{4.6003} \times (1 + 1000z)^{1.7923}}$$

$$\Delta X = \frac{0.00932\ (1 + \frac{f_t}{1000})^{6.530} \times (1 + \emptyset)^{2.201}}{(1 + \frac{\sigma w}{1000})^{4.914} \times (1 + p)^{4.550}}$$

$$\sigma_s = \frac{47{,}280\ (1 + \frac{\Delta T_F}{100})^{0.4254} \times (1 + \frac{f_t}{1000})^{4.094}}{(1 + \frac{\sigma w}{1000})^{3.144} \times (1 + 1000z)^{0.494} \times (1 + p)^{2.742}}$$

Where: $\overline{X}$ = Crack Spacing, ft (0.3048 m/pt)
ΔX = Crack Width, in (25.4 mm/in)
σ_s = Steel Stress, psi (6.89 kPa/psi)

Program JRCP-2

Program JRCP-2 was developed by Rivero-Vallejo and McCullough (Ref. 96) and uses many of the concepts developed for CRCP-2. However, the geometry and boundary conditions for this model are considerably different from CRCP-2. JRCP-2 also considers the stresses in the concrete and reinforcing steel with time and location. These stresses are affected by the frictional resistance of the subbase; the stiffness, tensile strength, and the shrinkage coefficient of the concrete; the temperature drops anticipated in time; the slab length; the percent reinforcement; the bar diameter; the yield stress of the steel; the elastic modulus of the steel; the unit weight of the concrete; and the ages at which cracking is to be considered. Given this information, JRCP-2 theoretically will proceed with analysis until the first crack forms and then will restructure the problem for subsequent consideration for the formation of a second crack between the joint and the first crack. The output includes the time when the first crack is formed, concrete stress, steel stress, joint width, and crack width as a function of time, and the same data for second cracks if they are formed.

It is believed that Program JRCP-2 is the only thorough model available for the study of the effects of material properties on drying shrinkage and thermal cracking in JRCP. Unfortunately, our use of the model indicates that the computer program is not completely "debugged" in that it does not predict cracking in all cases where cracking has been observed. Since JRCP-2 appears to be theoretically correct and is the only suitable model for JRCP it has been selected for this study pending correction of the deficiencies.

CHAPTER 5. ENVIRONMENTAL FACTORS AND MODELS

As stated previously, the environmental factors of frost penetration, temperature and moisture are extremely important in relation to long-term performance of both the entire pavement structure and individual components. In order to establish structural thicknesses and select material property requirements for zero-maintenance pavements, these factors must be reliably predicted.

FROST PENETRATION

Frost action is a general term for the freezing and thawing of moisture in materials and the resultant effects on these materials and on structures of which they are a part or in contact. With protracted cold, freezing will extend into the subgrade and the pavement surface may rise; this is referred to as frost heave. Pavement damage associated with frost heave consists of bumps or waves in flexible pavements and cracks or faulted joints in rigid pavements. Frost action occurs when three conditions exist: (1) frost susceptible soils, (2) freezing temperatures, and (3) a supply of water. Therefore, to solve this problem one or more of these factors must be removed. The most common approach to minimizing the effect of frost action is to replace the frost susceptible soils with non-frost susceptible soils and provide adequate drainage. Other methods utilize a layer of high heat capacity (Ref. 31), a layer that cuts off moisture movements (Ref. 30), or chemicals that retard freezing of the soil water (Ref. 99[1]). In order to design against frost action, the depth of frost penetration must be known or predicted.

Although there are many important details to frost action, the most significant is temperature, since it can be directly related to the degree of frost penetration in a pavement structure. The severity of the frost damage is also a function of rate of temperature change, duration of freezing temperature, and temperature fluctuations (freeze-thaw cycles). In past studies intensity and duration of cold have been expressed in terms of "degree days" (Ref. 31). This value is obtained by subtracting average daily air temperatures from 0°C. Cumulative "degree days" is used to define "freezing index", which is a measure of the combined duration and magnitude of below-freezing temperatures occuring during any given freezing season (Ref. 30). This "freezing index" is used to predict the depth of frost penetration. The Corp of Engineers (Ref. 30) provides various design freezing index values depending on the probability of a particular value occurring in any one

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[1]Croney, D. and J.C. Jacobs, "The Frost Susceptibility of Soils and Road Materials", Road Research Laboratory Report LR 90, Crowthorne, England, 1967.

year of the design period. For the design of "zero-maintenance" pavements an average of the three coldest years in a 30 year period was selected, which is illustrated in Figure 5.1.

The following describes a procedure which is used by the Corp of Engineers for computing frost depth (Ref. 100[1]). Three terms or fundamental properties are used in the procedure: (1) Thermal Conductivity, (2) Volumetric Heat, and (3) Latent Heat.

Thermal Conductivity (K, Btu/(hr) (ft^2) (°F/ft)) expresses the rate of heat flow through a unit area under a unit thermal gradient. Tables 5.1-5.3 (Ref. 31, 101[2]) and Figures 5.2-5.3 illustrate typical values of K. In the procedure an average of the unfrozen and frozen values (Figure 5.2, Ref. 102[3]) is used, because at the commencement of freezing there is no frost penetration, and when maximum frost depth occurs all the material under analysis is frozen. Thus an average value is required in order to balance the two extreme conditions.

Volumetric Heat (C, Btu/($ft.^3$) (°F) is specific heat times mass, which expresses the change in thermal energy in a unit volume of soil per unit change in temperature. Typical values of C are given in Tables 5.2-5.4. Volumetric heat for soil and base-course materials can be calculated as follows:

$$C_o = \gamma_d \left(0.17 + \frac{W}{100}\right) \text{ Unfrozen Soil} \quad \ldots\ldots 5.1$$

$$C_o = \gamma_d \left(0.17 + 0.5\,\frac{W}{100}\right) \text{ Frozen Soil} \quad \ldots\ldots 5.2$$

where:

γ_d = Dry Density, pcf

W = Moisture Content, %

and 0.17 is the specific heat of dry soil material.

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[1]Aldrich, C.P., Jr., "Frost Penetration Below Highway and Airfield Pavements", HRB Bulletin No. 135, Highway Research Board, 1956.

[2]Berg, R.L. and R.W. McGaw, "Improved Drainage and Frost Action Criteria for New Jersey Pavement Design - Phase 2: Frost Action", Special Report 78-9, Federal Highway Administration, May 1978.

[3]Kersten, M.S., "Final Report, Laboratory Research for the Determination of the Thermal Properties of Soils", Engineering Experiment Station, University of Minnesota, 1949.

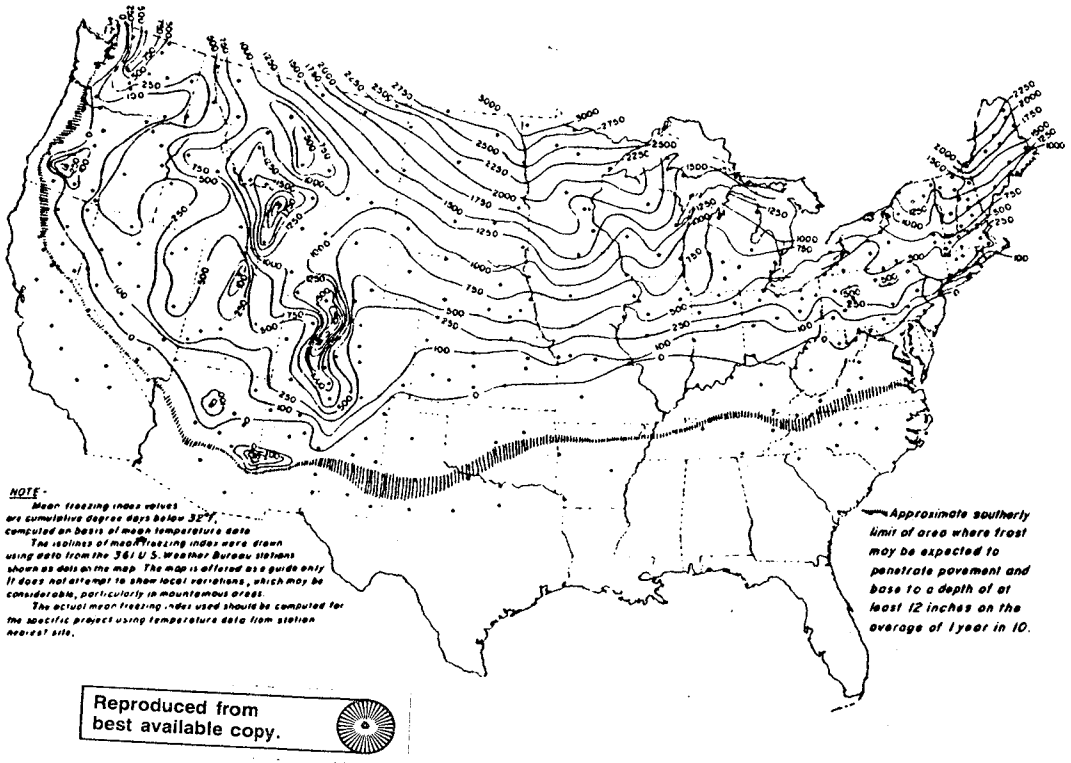

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Figure 5.1 Distribution of design air freezing index values in the Continental United States based on an average of the three coldest years in a 30 year period (Ref. 30).

TABLE 5.1. TYPICAL VALUES OF THERMAL CONDUCTIVITY FOR DIFFERENT MATERIALS (REF. 31).

Bituminous concrete	0.84
Air	0.014
Shale	0.90
Granite	1.60
Snow-	
Loose	0.06
Compact	0.20
Portland cement concrete	0.54
Water	0.35
Ice	1.30
Copper	225.00

Units: Btu/(hr.)(ft.^{2})(^{o}F)(ft.)

1 ft = 0.3048 m
1°F = 1.8°C -32

TABLE 5.2. THERMAL PROPERTIES FOR PAVEMENT SECTIONS WITHOUT A DRAINAGE LAYER (REF.101).

Material Property	γ_d	w	d	k_{dry}	k_{wet}	C_{dry}	C_{wet}	L_{dry}	L_{wet}
	lb/ft^3	%	ft	$\frac{Btu}{fthr^oF}$	$\frac{Btu}{fthr^oF}$	$\frac{Btu}{ft^3{}^oF}$	$\frac{Btu}{ft^3{}^oF}$	$\frac{Btu}{ft^3}$	$\frac{Btu}{ft^3}$
SECTION 1									
PCC	140	0	0.75	0.57	-	30.0	-	0	-
Base (1B)	129	6.7(9.7)	1.00	1.15	1.45	28.41	31.31	1245	1802
Subgrade (1C)	118	14.4	5.00	-	1.08	-	32.80	-	2447
SECTION 3									
PCC	140	0	0.75	0.57	-	30.0	-	-	-
Base (1A)	129	6.7(9.7)	0.50	1.15	1.45	28.41	31.31	1245	1082
Subbase (1C)	117	6.3(12.0)	0.50	1.17	1.81	25.42	30.42	1061	2022
Subgrade (1C)	118	14.4	10.00	-	1.08	-	32.80	-	2447
SECTION 5									
FABC	138	0	0.25	0.54	-	28.0	-	0	-
BSBC	138	0	0.50	0.53	-	28.0	-	0	-
Base (2B)	129	6.7(9.7)	0.42	1.15	1.45	28.41	31.31	1245	1802
Subbase (1B)	129	6.7(9.7)	0.42	1.15	1.45	28.41	30.42	1245	1802
Subgrade (1C)	118	14.4	5.00	-	1.08	-	32.80	-	2447
SECTION 6									
FABC	138	0	0.25	0.54	-	28.0	-	0	-
BSBC	138	0	0.50	0.53	-	28.0	-	0	-
Base (5A)	128	8.1(11.7)	0.50	0.99	1.05	29.54	37.54	1493	2157
Subbase (1B)	129	6.7(9.7)	0.92	1.15	1.45	28.41	31.31	1245	1802
Subgrade (1C)	118	14.4	15.00	-	1.08	-	32.80	-	2447
SECTION 7									
FABC	138	0	0.25	0.54	-	28.0	-	0	-
BSBC	138	0	0.50	0.53	-	28.0	-	0	-
Base (5A)	128	8.1(11.7)	0.50	0.99	1.05	29.54	32.99	1493	2157
Subbase (1C)	117	6.3(12.0)	1.00	1.17	1.81	25.42	30.42	1061	2022

Note: Moisture contents in parentheses are the "wet" values.

1 lb = 0.454 1°F = 1.8°C -32 1 ft = 0.348 m

TABLE 5.3. THERMAL PROPERTIES FOR PAVEMENT SECTIONS WITH A DRAINAGE LAYER (REF. 101).

Material	γ_d	w	d	k	C	L
	lb/ft^3	%	ft	$\frac{Btu}{fthr^oF}$	$\frac{Btu}{ft^3{}^oF}$	$\frac{Btu}{ft^3}$
SECTION 2A						
PCC	140	0	0.75	0.57	30.0	0
OGL (stabilized)	120	7.0	0.5	0.51	26.6	1204
Subbase (1B)	129	6.7	0.5	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.0	1.08	32.8	2447
SECTION 2B						
PCC	140	0	0.75	0.57	30.0	0
OGL (unstabilized)	121	4.5	0.50	0.40	25.9	784
Subbase (1B)	129	6.7	0.50	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.00	1.08	32.8	2447
SECTION 4A						
PCC	140	0	0.75	0.57	30.0	0
OGL (stabilized)	120	7.0	0.5	0.51	26.6	1204
Subbase (1A)	129	6.7	0.5	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.0	1.08	32.8	2447
SECTION (4B)						
PCC	140	0	0.75	0.57	30.0	0
OGL (unstabilized)	121	4.5	0.50	0.40	25.9	784
Subbase (1A)	129	6.7	0.50	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.00	1.08	32.8	2447
SECTION 8						
FABC	138	0	0.25	0.54	28.0	0
BSBC	138	0	0.50	0.54	28.0	0
OGL (stabilized)	120	7.0	0.42	0.51	26.6	1204
Subbase (1B)	129	6.7	0.42	1.15	28.4	1245
Subgrade (1C)	118	14.4	5.0	1.08	32.8	2447
SECTION 9						
FABC	138	0	0.25	0.54	28.0	0
BSBC	138	0	0.50	0.54	28.0	0
OGL (stabilized)	120	7.0	0.50	0.51	26.6	1204
Subgrade (1C)	118	14.4	5.0	1.08	32.8	2447

1 lb = 0.454 kg

1 ft = 0.3048 m

1 °F = 1.8°C -32

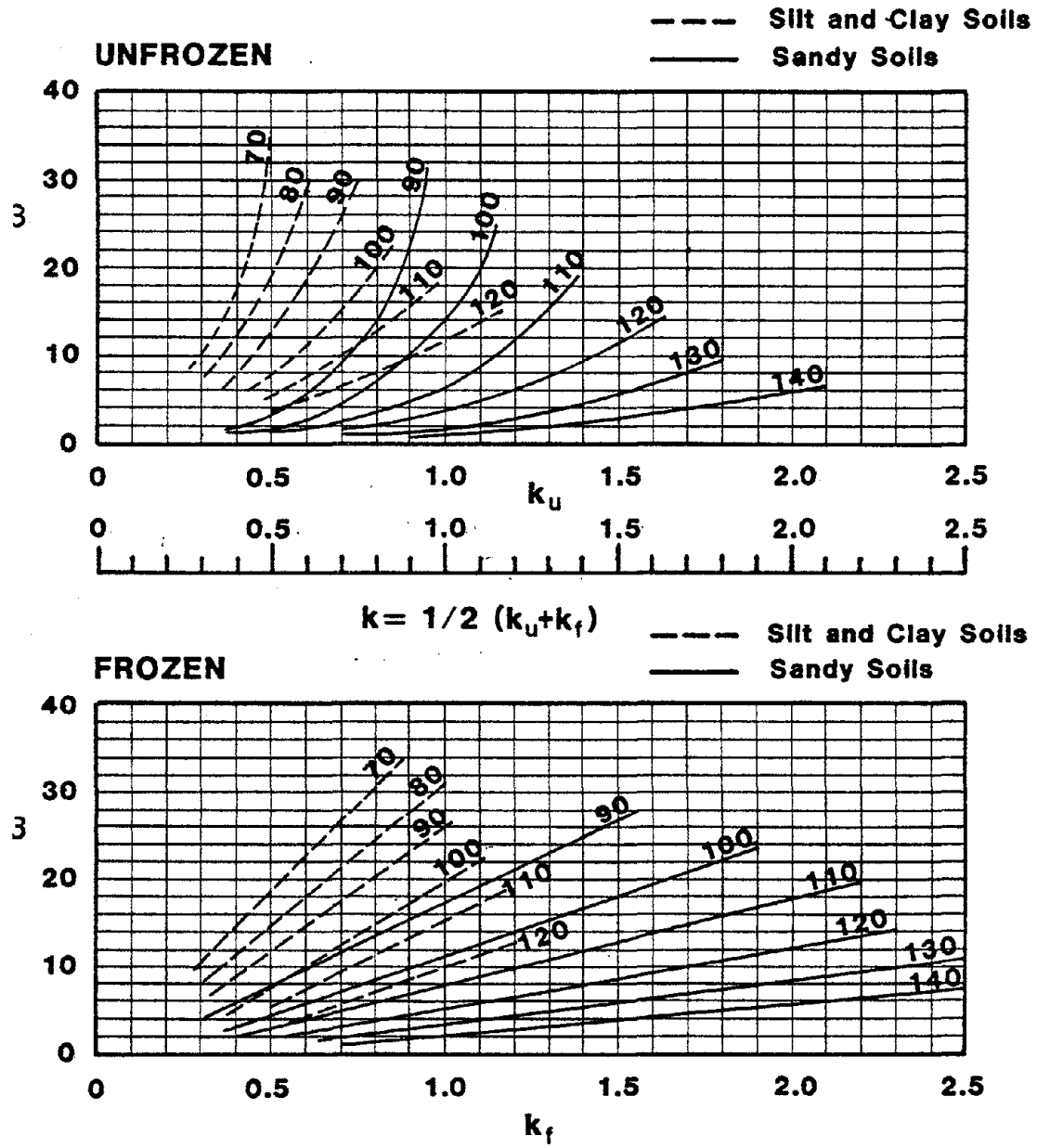

Figure 5.2. Thermal conductivity values for soils that are compacted at different moisture contents and densities (Ref. 102).

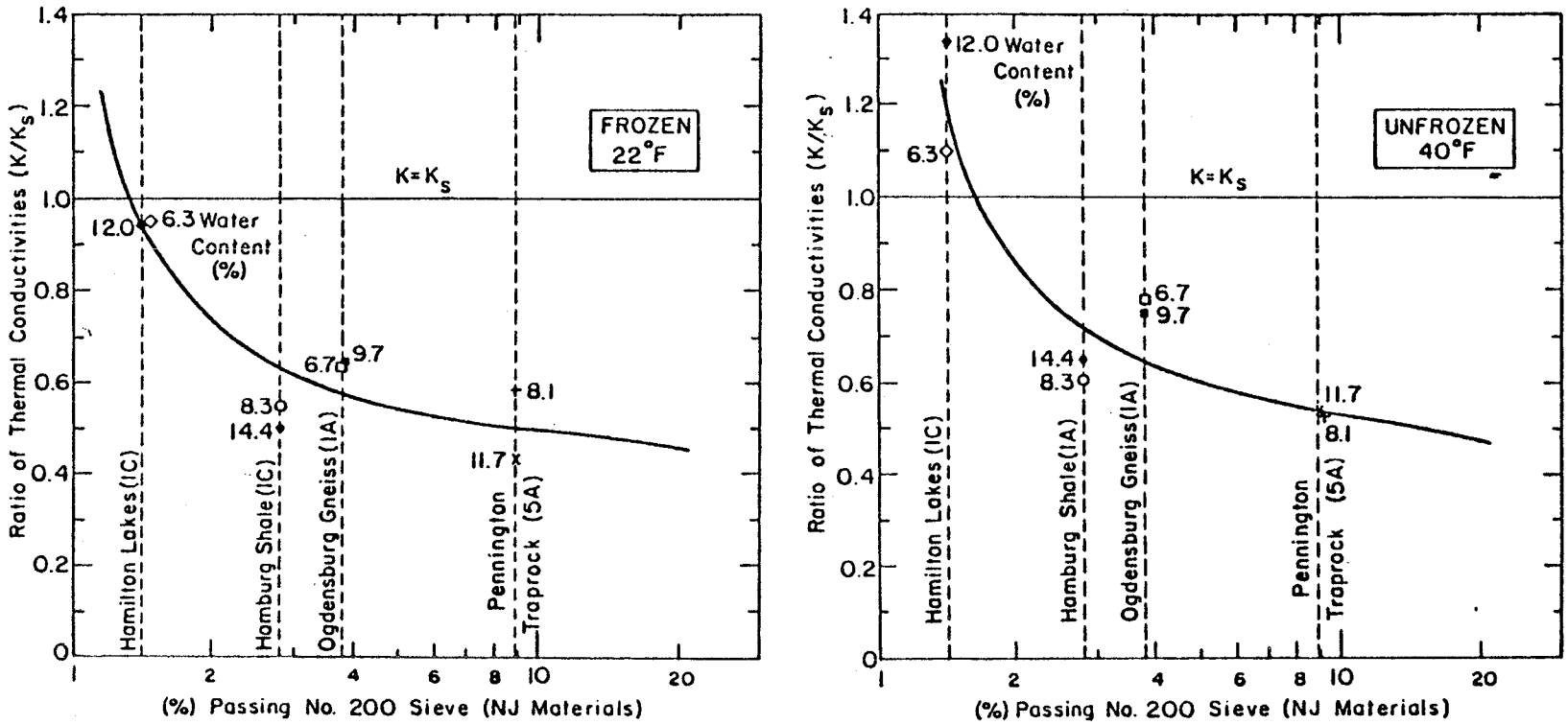

Figure 5.3. Comparison of thermal conductivity (K) of New Jersey soils with Kersten's values (K_s) for sandy soils, frozen and unfrozen (Ref. 101).

TABLE 5.4. TYPICAL VALUES OF VOLUMETRIC HEAT FOR DIFFERENT MATERIALS (REF. 31).

Water	62.4
Ice	28.0
Bituminous concrete	28.0
Portland Cement Concrete	30.0
Air	0.019

Units: BTU/(yd^3) (°F)

1 yd = 0.9144 m^3

1 °F = 1.8°C -32

Volumetric Heat has also been expressed as a function of specific heat, as stated by Eaton (Ref. 103[1]); the average of frozen and unfrozen volumetric heat can be calculated by equation 5.3:

$$C = \gamma_d \left(c + 0.75 \frac{W}{100}\right) \quad \text{.....5.3}$$

where:

c = specific heat

Latent heat (L, Btu/ft^3) expresses the change in thermal energy in a unit volume of soil when the soil moisture freezes or thaws at a constant temperature. Typical values of latent heat are given in Tables 5.2-5.3. Since latent heat depends only on the amount of water in a unit volume of soil, it can also be expressed as:

$$L = 1.434 \frac{W}{100} \gamma_d \quad \text{.....5.4}$$

In order to calculate frost penetration beneath a pavement, the Berggren formula as modified by Aldrich (Ref. 100) is used:

$$d = \gamma \left(\frac{48\eta FK}{L}\right)^{1/2} \quad \text{.....5.5}$$

where

d = Depth of frost penetration below the pavement surface, ft.
γ = A correction coefficient
F = Air Freezing Index, degree days
K = Thermal Conductivity, Btu/hr.ft^2 (°F/ft.)
η = the ratio of surface freezing index values to air freezing index values.

The correction coefficient, γ, is a function of two variables, the fusion parameter, μ, and thermal ratio, α, as shown in Figure 5.4 and defined below.

$$\mu = \frac{\eta FC}{tL} \quad \text{.....5.6}$$

$$\alpha = \frac{(MAAT-32)t}{\eta F} \quad \text{.....5.7}$$

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[1]Eaton, R.A. and R.H. Joubert, "Full-Depth Pavement Considerations in Seasonal Frost Areas", Reprint, Paper prepared for presentation at the Association of Asphalt Paving Technologists, February 1979.

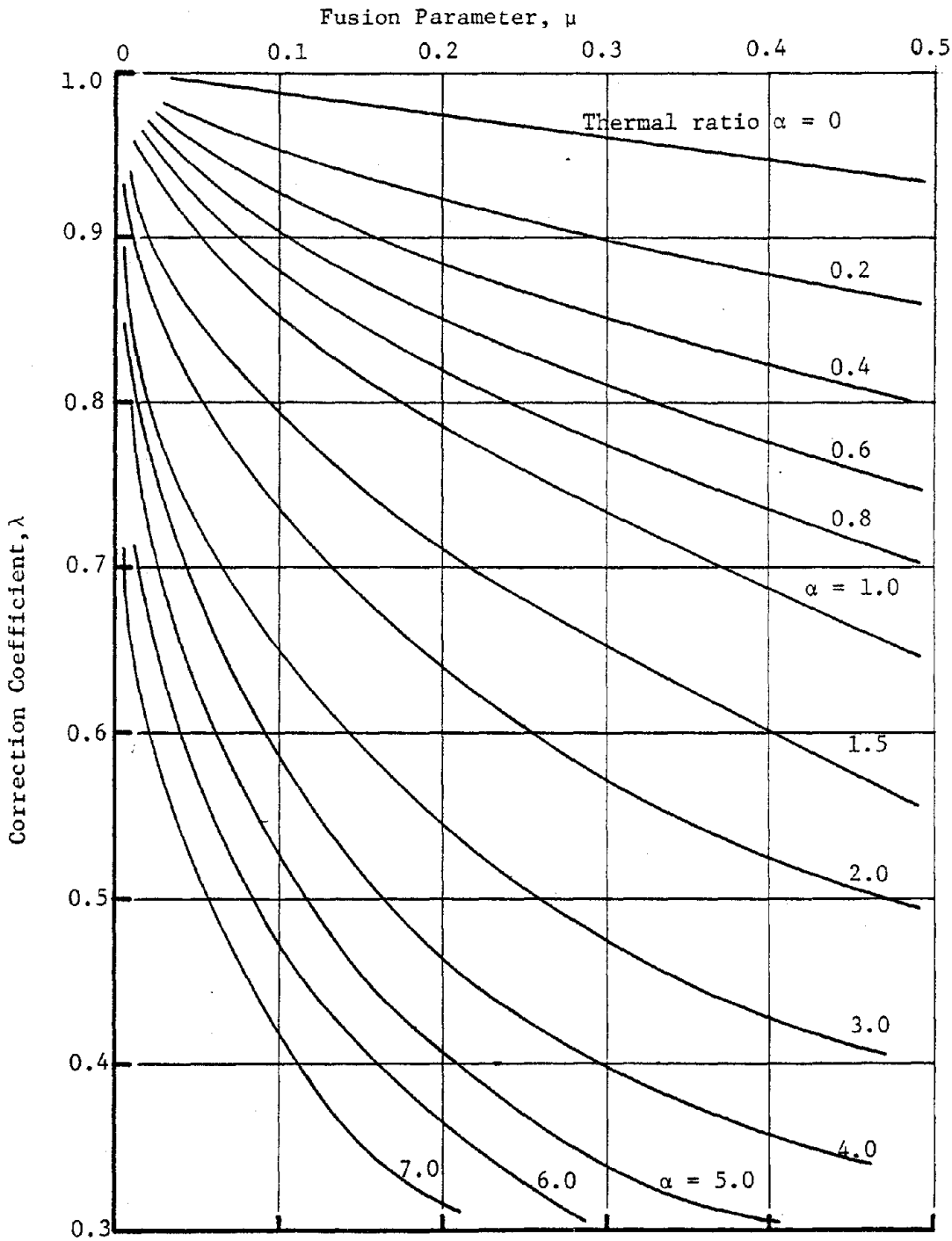

Figure 5.4. Correction coefficient in the modified Berggren formula (Ref. 100).

Where:

t = Average freezing duration, days

MAAT = Mean Annual Air Temperature, °F.

For natural soils, λ has been related to moisture and mean air freezing index values as shown in Figure 5.5. Environmental data accumulated by the Corps of Engineers concerning the average freezing duration (t) as related to the Mean Annual Air Temperature have been summarized and are shown in Figure 5.6 (Ref. 104[1]).

Using equation 5.5 for computing frost penetration results in an iterative procedure. First an initial assumption of frost depth (X) has to be made. For purposes of calculation, it is convenient to calculate an effective L/K which has been shown by Aldrich and Paynter (Ref. 105[2]) to be:

$$L/K = \frac{2}{X^2} \quad \frac{d_1}{K^1} \left(\frac{L_1 d_1}{2} + L_2 d_2 + \ldots + L_n d_n\right)$$

$$+ \frac{d_2}{K_2} \left(\frac{L_2 d_2}{2} + L_2 d_2 + \ldots + L_n d_n\right) + \ldots$$

$$\frac{d_n}{K_n} \quad \frac{L_n d_n}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.8$$

(where n is the number of the layer (counting down) in which the assumed depth X falls).

$$d_n = X - \sum_{i=1}^{n-1} d_i \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.9$$

The terms making up α and μ have been previously described except that it should be noted that values for volumetric heat and latent heat should be weighted as follows:

$$C_{wt} = \frac{C_1 d_1 + C_2 d_2 + \ldots + C_n d_n}{X} \quad \ldots\ldots\ldots\ldots\ldots\ldots 5.10$$

---

[1]Linell, K.A., "Frost Design Criteria for Pavements", HRB Bulletin 71, Highway Research Board, 1953.

[2]Aldrick, H.P. and H.M. Paynter, "Frost Interim Report, Analytical Studies of Freezing and Thawing in Soils", Arctic Construction and Frost Effects Laboratory, New England Division Corps of Engineers, Boston, June 1953.

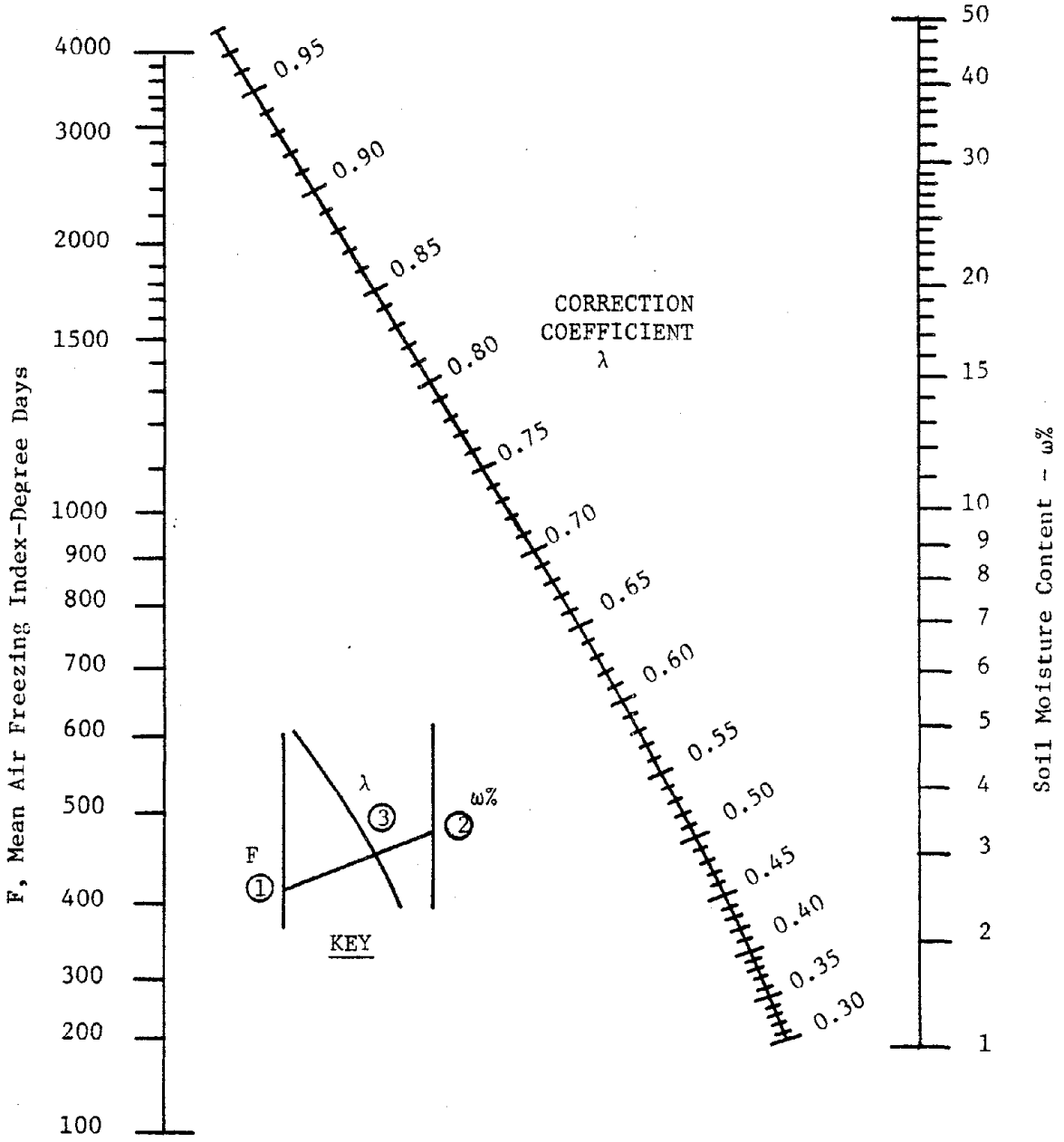

Figure 5.5. Graphical illustration of the modified Berggren Equation to determine λ (Ref. 105).

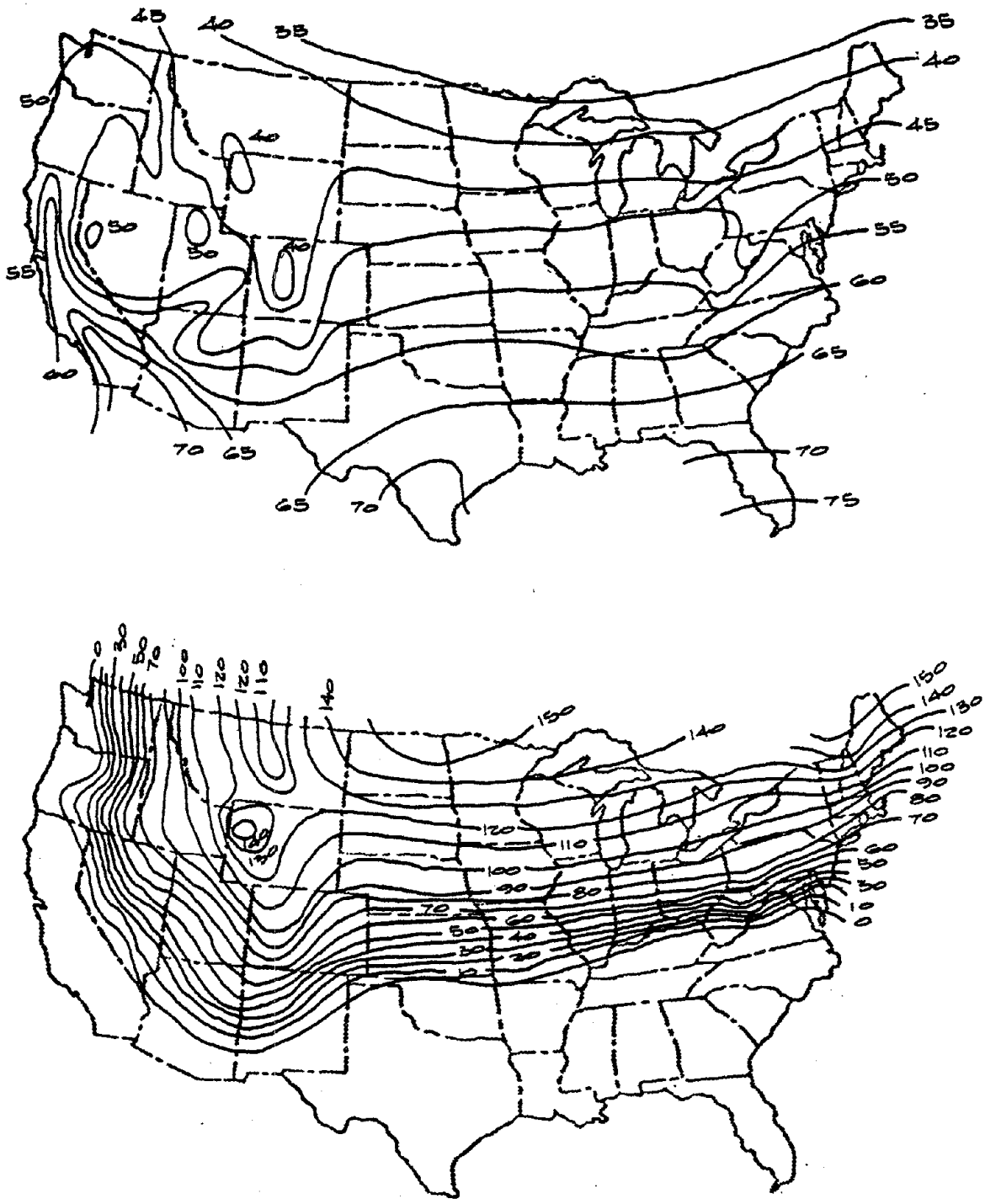

Figure 5.6. Mean annual air temperature, in degress F (upper); and duration of normal freezing index; in days (lower) (Ref. 104).

and

$$L_{wt} = \frac{L_1 d_1 + L_2 d_2 + \ldots L_n d_n}{X} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.11$$

Once d has been determined, it is compared to the initial estimated frost depth X, if they are unequal another frost depth is assumed and d is recalculated. This procedure is continued until both values become equal.

This procedure can be used to determine the depth at which the frost susceptible material must be removed; at what depth a moisture barrier must be placed; or evaluate the effect of an insulating layer, as shown in Figure 5.7 (Ref. 106[1]). Tables 5.5 and 5.6 summarize typical values reported in the literature used in equation 5.12, for various pavement materials and different environmental regions across the U.S.

PAVEMENT TEMPERATURE

As stated previously, temperature is considered for two reasons: 1) the effect temperature has on the asphalt concrete modulus and 2) the effect on low-temperature cracking. In order to predict the modulus of the asphalt concrete and low-temperature cracking of the asphalt concrete surface layer, the pavement temperature must be known throughout the pavement's thickness. There are existing procedures which can be used to predict the temperature at various points in the pavement structure. Some of these include the ones developed by Kentucky and the Asphalt Institute. Although each procedure has advantages, the one developed by Barber (Ref. 98) is the most general and allows temperature computation not only as a function of pavement depth and material type but also as a function of time and environment. Hence the procedure developed by Barber and as revised by Shahin (Ref. 107[2]) was used to compute pavement temperatures.

The other important factor considered in relation to zero-maintenance pavements is the effect curling stresses have on the performance of the PCC base layer. In order to restrict a limited temperature gradient in the PCC layer the temperature distribution in the asphalt concrete layer

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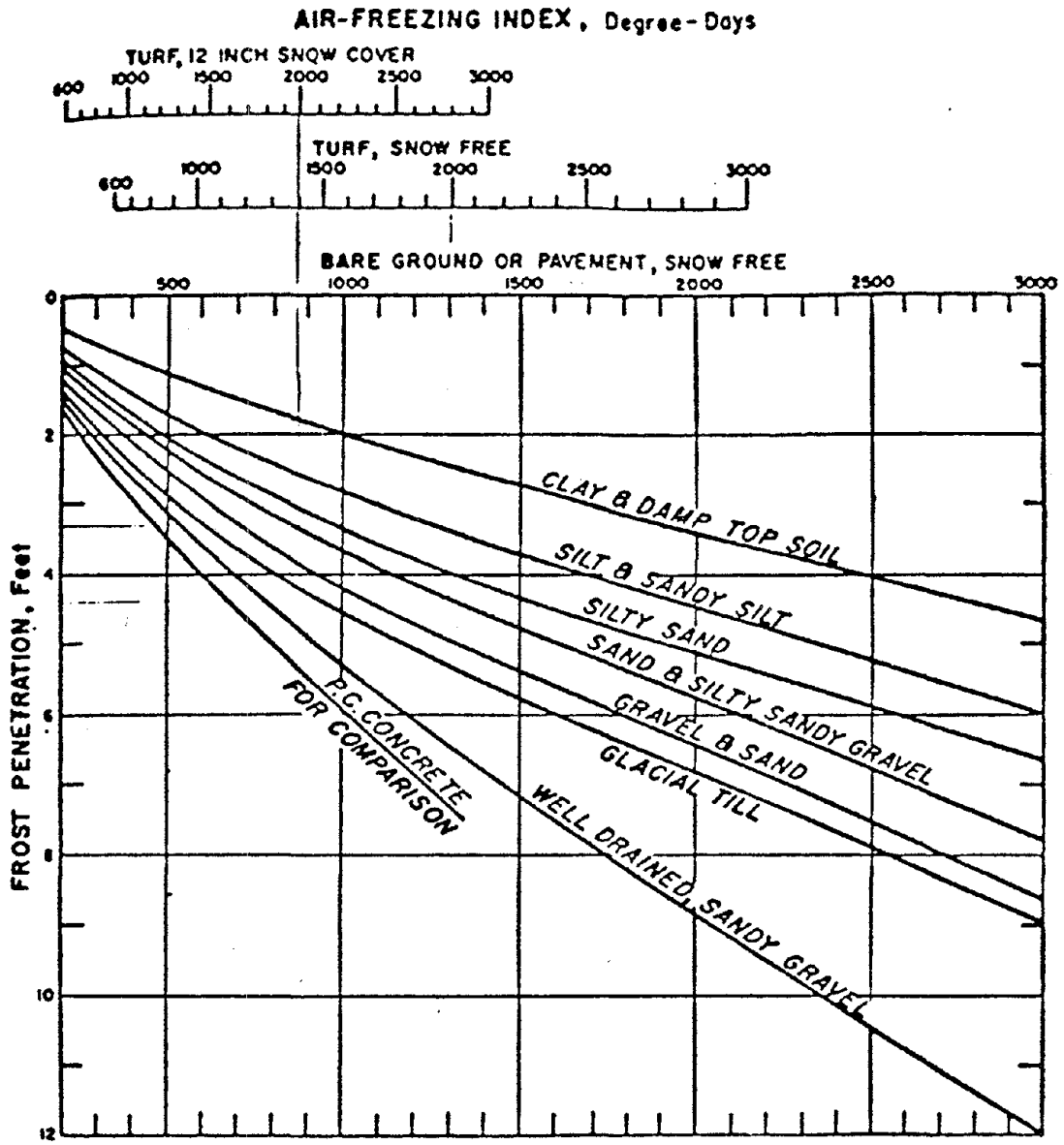
[1] AASHTO Interim Guide for Design of Pavement Structure 1972, American Association of State Highway and Transportation Officials, Washington, D.C., 1974.

[2] Shahin, M.Y. and B.F. McCullough, "Prediction of Low-Temperature and Thermal-Fatigue Cracking in Flexible Pavements", Report No. 123-14, Center for Highway Research, The University of Texas at Austin, August 1972.



1 in = 2.54 cm 1 ft = 0.348 m

Figure 5.7. Relationship between air freezing index, surface cover, and frost penetration into homogeneous soils (Ref. 106).

TABLE 5.5 SUMMARY OF TYPICAL VALUES FOUND IN THE LITERATURE FOR MOST MATERIALS IN A PAVEMENT STRUCTURE

Material		∂_d Density pcf	Moisture Content, %	Volumetric Latent Heat Btu/cu.ft.	Thermal Conductivity Btu/cu.ft.	Aug. Volumetric Heat Capacity	Specific Heat
AC	W	145	-	--	.80	30.4	.22
	D		-	--	.60	28.0	.22
Asphalt Drainage Layer	W	120	10%	1730	.50	29.4	.17*
	D*	120	5%	865	.40	24.9	
PCC	W	150	-	--			.20
	D		-	--	.54	30.0	
Stabilized Material	W	140	10	2010	1.5	31.5	.15*
	D		5	1005	1.2	26.3	
Crushed Stone or Granular Base	W	130	10%	1870	1.5	31.5	
	D		5%	935	1.2	26.3	
MEPS	Clay	120	10%	1728	.95	30.6	.18
	Silt/ Sand	125	10%	1800	1.85	28.1	.15
Insulation	W	10	4%		.20		
	D						
Clay & Silt Subgrade	W	110	20%	3168	1.15	36.3	.18
	D		10%	1584	.70	28.1	
Sandy Soils	W	110	20%	3168	1.85	33.0	.15
	D		10%	1584	1.2	24.8	

*Estimated or calculated from other data of similar conditions

1 cu ft = 0.0283 cu m 1 lb = 0.454 kg

TABLE 5.6 SUMMARY OF DESIGN AIR FREEZING INDEX VALVES AND FREEZING DEVIATION FOR EACH ENVIRONMENTAL REGION NUMBER

Environmental Region No.	MAAT °F	Design Freezing Index Air (3 Coldest In 30 Years)	Design Air Freezing Duration (Days)
1	50	750	125
	55	500	80
	60	250	40
	65	20	5
	70	0	-
2	35	3500	170
	40	2500	150
	45	2000	130
	50	1250	100
	55	500	60
	60	100	20
	65	50	10
3 (Wet)	40	2000	150
	45	1500	130
	50	1000	110
	55	500	90
3 (Dry)	40	2000	200
	45	1500	150
	50	1000	100
	55	500	80

1°F = 1.8°C -32

must be predicted. Using the equation developed by Barber, the required thickness of asphalt concrete can be determined to limit the amount of curling stresses or temperature differential in the PCC layer.

The following equations provide a procedure by which the pavement temperature at arbitrary depth and time can be estimated for various conditions, using available weather records. The temperature in a semi-infinite mass in contact with air may be expressed by:

$$T = T_M + T_V \left(\frac{He^{-xC}}{((H+C)^2 + C^2)^{0.5}} \right) \sin(S) \quad \text{.....5.12}$$

where

$$T_V = 0.5T_R + 3R \quad \text{.....5.13}$$

$$T_M = T_A + R \quad \text{.....5.14}$$

if $\sin(S) \geq 0$, and

$$T_V = 0.5T_R \quad \text{.....5.15}$$

$$T_M = T_A + 0.5R \quad \text{.....5.16}$$

if $\sin(S) < 0$. Other variables in the equation are:

$$H = h/k \quad \text{.....5.17}$$

h = surface coefficient, BTU per square foot per hour

k = thermal conductivity, BTU per square foot per hour per (degree F per foot)

$$C = (.131/c)^{0.5} \quad \text{.....5.18}$$

c = diffusivity, square feet per hour = k/sw 5.19

s = specific heat, BTU per pound per °F

w = density, pounds per cubic foot

x = depth at which the temperature T is desired, feet

T_M = mean effective air temperature, °F

T_V = maximum variation from the effective mean (the half-amplitude of the effective air temperature), °F

T = desired temperature, °F

$$S = .262t - xC - \arctan\left(\frac{C}{H+C}\right) \quad \text{.....5.20}$$

t = time after beginning of cycle (24 hour cycles), hours

T_A = mean daily air temperature, °F

T_R = daily air temperature range, °F

R = average contribution of solar radiation to the effective air temperature; calculated from

$$R = \frac{2}{3} \left(\frac{3.69L}{24}\right) \frac{b}{h} \quad \text{.....5.21}$$

where

b = surface absorptivity to solar radiation
L = solar radiation in Langleys per day

As used in program TC, the above equations are modified, in that the relation for S is replaced by several for different periods of the day, in which the constants have been adjusted to provide both continuity and agreement with observations.

Equation 5.12 produces pavement temperatures which are dependent not only on the material thermal properties, but also on the environmental conditions in which the pavement exists. In reviewing the climatological data (Ref. 108[1]) it was determined that particular environmental zones exist across the U.S. which exhibit similar relationships between mean annual air temperature and average yearly temperature change. These are presented in Figure 5.8. Therefore, environmental regions were characterized using the two temperatures as shown in Figure 5.8. The limits or areas which represent each environmental region are illustrated in Figure 5.9.

In order to predict the amount of low-temperature cracking, the average minimum pavement temperature must be known. This temperature can be estimated using equation 5.12 or the following simplified procedure based on Figure 5.8:

1) The final part of equation 5.12 is material dependent and is independent of changes in the environment. Therefore, set a constant for the specific pavement surface material:

$$M = \frac{He^{-xC}}{((H+C)^2+C^2)^{0.5}} \sin(S) \quad \text{.....5.22}$$

where again

$$S = .262t - xC - \arctan\left(\frac{C}{C+H}\right) \quad \text{.....5.20}$$

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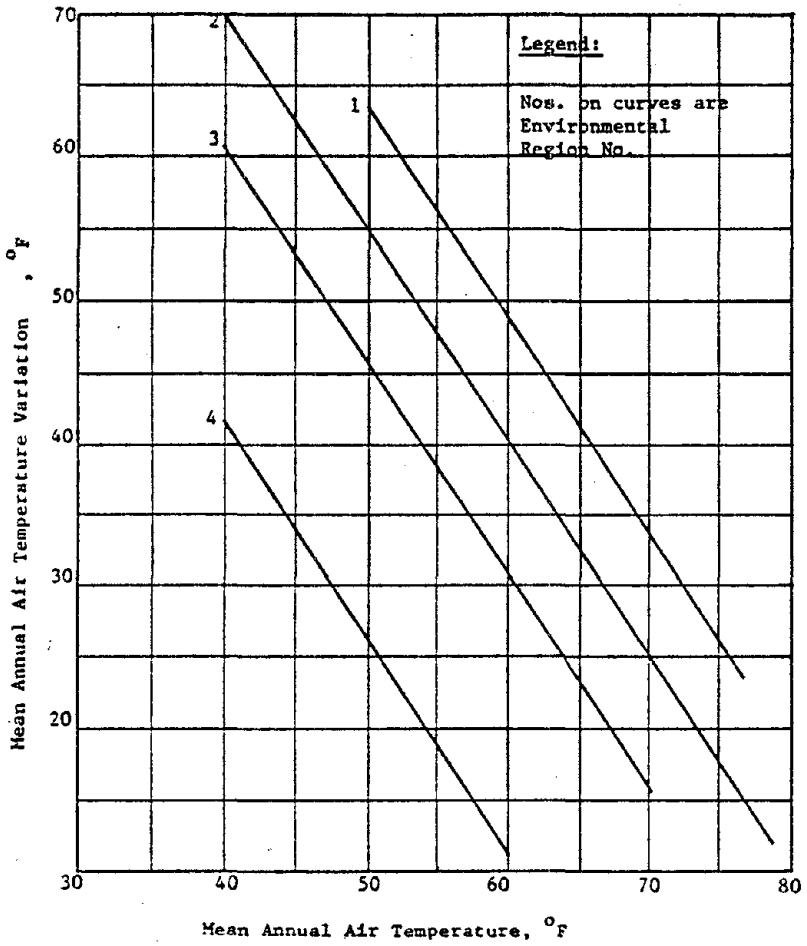
[1]Local Climatological Data - Annual Summaries for 1977, Part I and II, National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, North Carolina, 1977.



°C = (°F - 32)/1.8

Figure 5.8. Determination of the environmental region number.

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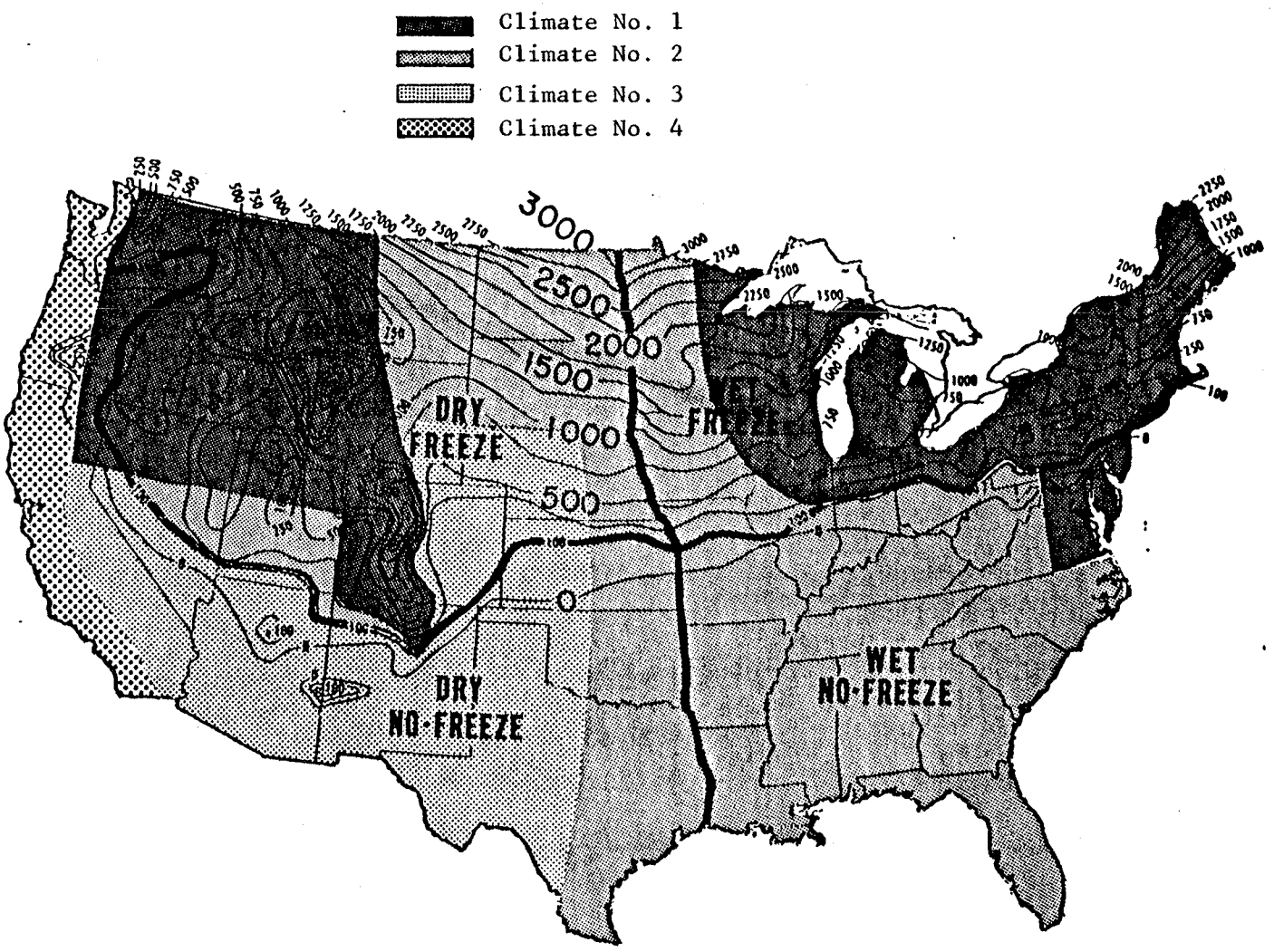


Figure 5.9. Creation of the environmental regions.

and S is evaluated at that time t for which sin(S) is minimum. The entire expression is calculated for depth x equal to zero, that is, at the surface of the pavement, where low temperature cracking begins.

2) Compute the mean winter temperature using equation 5.23 and Figure 5.8 or determine the environmental region number from Figure 5.8 using:

$$T_W = MAAT - 1/2\Delta MAAT \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 5.23$$

where:

MAAT = mean annual air temperature, °F

ΔMAAT = average yearly temperature change (°F), which is the high monthly average temperature minus the low monthly average temperature

T_W = mean winter temperature, °F

3) Using the above equations and substituting in equation 5.12 results in an estimate for the average minimum pavement temperature (S < 0):

$$T_{Min.} = (T_W + 0.5R) + 0.5\Delta T_W \cdot M \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 5.24$$

where:

ΔT_W = the winter average daily temperature change, °F

From a review of the same climatological data as mentioned above, Tables 5.7-5.10 summarize the average daily temperature changes with season and environmental region numbers and Table 5.11 summarizes the solar radiation values.

Similarly, the average pavement temperature can be obtained for each month or season of the year using equation 5.12, Figure 5.8 and Tables 5.7-5.11.

MOISTURE

At the present there is no mechanistic model which can accurately predict the moisture migrations below the pavement surface. Although, moisture can have a very detrimental effect on pavement performance, a model which can predict this damage was not used. Figure 5.10 (Ref. 48) illustrates the general increase in moisture content of the existing subgrade with time. This increase in moisture is a result of many

TABLE 5.7. SUMMARY OF THE AVERAGE DAILY TEMPERATURE CHANGE FOR ENVIRONMENTAL REGION NO. 1 (°F)

Climate	Season	Mean Annual Air Temperature, °F				
		40	50	60	70	75
Wet*	Winter	21	21	21	21	21
	Spring	23	23	23	23	23
	Summer	22	22	22	22	22
	Fall	25	25	25	20	18
Dry**	Winter	32	32	28	22	22
	Spring	30	30	30	30	30
	Summer	30	30	30	30	30
	Fall	35	35	30	25	25

*Wet - Average yearly rainfall > 12 inches
**Dry - Average yearly rainfall < 12 inches

1°F = 1.8°C -32

TABLE 5.8. SUMMARY OF THE AVERAGE DAILY TEMPERATURE CHANGE FOR ENVIRONMENTAL REGION NO. 2 (°F)

Climate	Season	Mean Annual Air Temperature, °F				
		40	50	60	70	75
Wet*	Winter	21	21	21	21	21
	Spring	23	23	23	23	23
	Summer	25	23	21	19	18
	Fall	30	27	25	20	15
Dry**	Winter	32	27	20		
	Spring	30	30	30	30	
	Summer	37	35	33	30	
	Fall	40	35	30	25	

*Wet - Average yearly rainfall > 12 inches
**Dry - Average yearly rainfall < 12 inches

1°F = 1.8°C -32

TABLE 5.9. SUMMARY OF THE AVERAGE DAILY TEMPERATURE CHANGE FOR ENVIRONMENTAL REGION NO. 3 (^{o}F)

Climate	Season	Mean Annual Air Temperature, ^{o}F				
		40	50	60	70	75
Wet*	Winter	20	20	20	15	10
	Spring	20	20	20	15	10
	Summer	24	24	20	15	10
	Fall	23	23	20	15	10
Dry**	Winter	20	20	15		
	Spring	25	25	25		
	Summer	30	30	30		
	Fall	29	29	29		

*Wet - Average yearly rainfall > 12 inches
**Dry - Average yearly rainfall < 12 inches

1°F = 1.8°C -32

TABLE 5.10. SUMMARY OF THE AVERAGE DAILY TEMPERATURE CHANGE FOR ENVIRONMENTAL REGION NO. 4 (°F)

Climate	Season	Mean Annual Air Temperature, °F				
		40	50	60	70	75
Wet*	Winter	15	15	12		
	Spring	15	15	15		
	Summer	20	20	20		
	Fall	15	15	15		
Dry**	Winter	15	15	12		
	Spring	20	20	20		
	Summer	20	20	20		
	Fall	22	22	22		

*Wet - Average yearly rainfall > 12 inches
**Dry - Average yearly rainfall < 12 inches

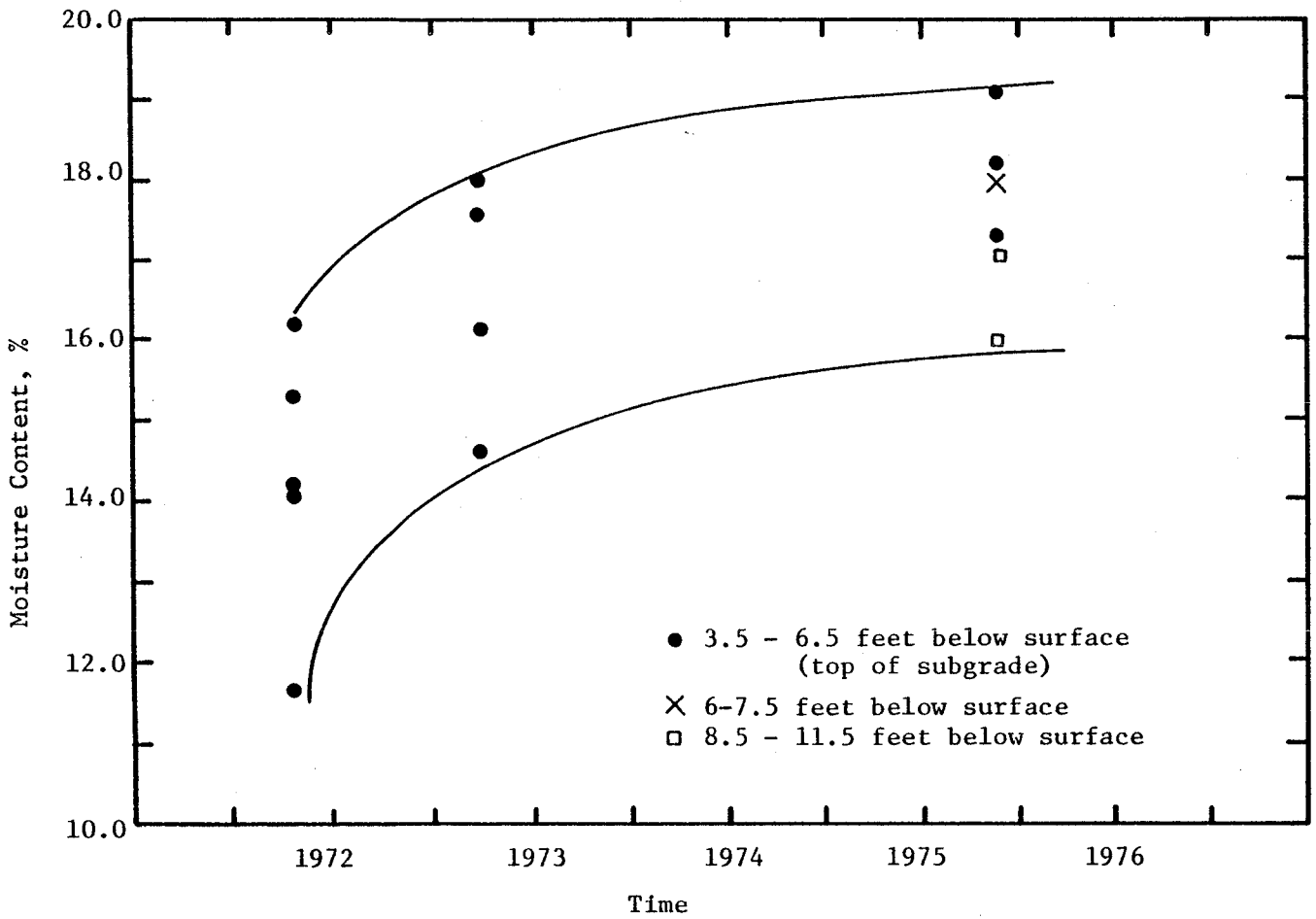
1°F = 1.8°C -32

TABLE 5.11. SUMMARY OF SOLAR RADIATION VALUES FOR EACH ENVIRONMENTAL REGION

Environmental Region No.	MAAT* (°F)	Winter	Summer	Spring	Fall
2	40	160	500	400	200
	50	180	500	430	260
	60	220	500	460	320
	70	280	500	500	380
	75	330	500	500	420
1	50	280	650	550	350
	60	290	650	600	400
	70	300	650	650	430
	75	305	650	650	450
3 (Dry)**	40	160	600	420	250
	50	180	600	480	350
	60	220	600	540	400
	70	280	600	600	430
3 (Wet)***	40	140	500	340	200
	50	160	500	380	230
	50	180	500	420	260
4	40	200	600	400	230
	50	250	600	480	310
	60	300	600	600	400

*MAAT = Mean Annual Air Temperature
**Dry = Average Yearly Rainfall < 12 inches
***Wet = Average Yearly Rainfall > 12 inches

1°F = 1.8°C -32



1 ft = 0.3048 m

Figure 5.10. General increase in moisture content with time on Runway 4R-22L at O'Hare International Airport (Ref. 48).

variables including the effect of hydrogenesis, which is not considered directly in this report. Therefore, in order to include the effect of increased moisture under a pavement structure, the characterization and resilient modulus test procedures are offered as an alternative. This allows the increase in moisture content to be considered during initial design in the form of a reduced modulus of the existing natural soil or fill material. The TRRL uses soil suction to predict equilibrium moisture under pavements; however, these procedures have not been confirmed in the U.S.

CHAPTER 6. LIMITING DESIGN CRITERIA

The serviceability of a pavement refers to the level of service it provides to road users at any particular time. Deterioration caused by traffic and the environment reduces the serviceability of the pavement continuously, but not necessarily at a constant rate. As the serviceability reaches a lower limiting level the pavement is often improved or reconstructed. Performance is measured in part as the rate of such deterioration and the designer seeks to optimize the performance within some restraints of adopted strategy. A measure of pavement serviceability is the serviceability index established by the AASHO Road Tests (Ref. 109[1]). The principal parameter in the PSI is longitudinal slope variance in the two wheel paths; minor terms are measures of cracking, patching, and either rutting for flexible pavements or spalling for rigid pavements. The PSI is accordingly an expression of the degree to which the surface is smooth, intact, and undeformed. Each distress manifestation will be selected at some limiting level to ensure that structural maintenance is not required either directly or indirectly from wheel loads or some environmental factors.

FATIGUE CRACKING

Asphalt Concrete

There have been numerous and extensive laboratory and field fatigue studies on asphalt concrete mixtures throughout the world, which have been adequately summarized and presented in References 40 and 110[2]. In the laboratory, fatigue failure can be a defined point related to the ability of the material to perform as a load carrying entity under repetitive loading. Fatigue failure in the field is not as well defined because it is associated with a certain level or amount of cracking, which is dependent on the pavement's function. The various variables affecting the fatigue life can be classified into three general categories:

1) Load
2) Environment
3) Mixture

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[1]The AASHO Road Test, Report 5 - Pavement Research", HRB Special Report 61E, Highway Research Board, 1962.

[2]"Fatigue of Compacted Bituminous Aggregate Mixtures", ASTM Special Publication 50-8, American Society for Testing and Materials, 1971.

Therefore, to develop a fatigue cracking failure criteria for zero-maintenance pavements, the level of cracking and variables of load, environment and mixture must be defined for asphalt concrete.

Although laboratory tests provide valuable information on the relative performance of different materials, the relation to performance in practice under triaxial loadings conditions is doubtful, since crack initiation does not mean functional failure of the pavement or that maintenance is required. It has generally been accepted that fatigue in pavements is less severe than in laboratory conditions due to strain relaxation or healing between loads; although, Monismith et al., (Ref. 111[1]) and Bergan (Ref. 112[2]) have shown, through a limited number of cases, that crack initiation observed in the field correlates with laboratory test results. Therefore, crack initiation, termed "Fracture Life", can be based on laboratory fatigue results, but performance or functional failure, termed "Service Life", should be based on in-field performance data.

In determining the fatigue failure characteristics and criteria of an asphalt concrete mixture, the type of loading (controlled stress or controlled strain mode) which the asphalt pavement experiences must be known. It has been shown in Reference 111 and in others that based on the controlled stress mode of loading, for a given stress level the fatigue resistance of an asphalt mixture increases with an increase in asphalt modulus; whereas, for the controlled strain mode of loading, for a given strain value the fatigue resistance decreases with an increase in modulus. A recent analysis completed by Monismith and Deacon (Ref. 113[3]), indicates that the controlled stress mode is never experienced by the pavement, but is approached in thick (greater than 6 inches (15 cm) as stated in Ref. 114[4]), stiff asphalt concrete layers. Therefore it is recommended that although the controlled stress condition is not fully experienced by the asphalt concrete layer, this mode of loading should be used in determining the service life for zero-maintenance pavements, since it should provide a conservative estimate of fatigue life.

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[1]Monismith, C.L., J.A. Epps, D.A. Kasianchuk and D.B. McLean, "Asphalt Mixture Behavior in Repeated Flexure", Institute of Transportation and Traffic Engineering, Report No. TE 70-5, University of California, Berkeley, 1972.

[2]Bergan, A.T., "Some Considerations in the Design of Asphalt Concrete Pavements for Cold Regions", Ph.D. Dissertation, University of California, Berkeley, 1972.

[3]Monismith, C.L. and J.A. Deacon in Proceedings of ASCE, Transportation Engineering Journal of ASCE, TPEJA, Vol. 95, No. TE2, May 1969.

[4]Mills, R.E. and R.F. Dawson, "Fatigue of Concrete", Proceedings, Seventh Annual Meeting, Highway Research Board, Washington, D.C., 1928.

Figure 6.1 (Ref. 117[1]) illustrates various fatigue curves that have been developed for asphalt concrete pavements. Numerous others exist, but are of the same general form as illustrated in Figure 6.1.

In order to illustrate the "Service Life" or fatigue criteria to be used for zero-maintenance pavements, elastic layer theory (ELSYM5) was used along with fatigue curves developed for different levels of performance (cracking) for in-service pavements (Refs. 11, 82, 115[2]). Figures 6.2-6.5 represent the required asphalt concrete (AC) surface thicknesses over a range of base thicknesses for a standard granular base flexible pavement cross-section, with various base moduli.

All fatigue equations or results were developed based on the AASHO Road Test data except Shell's (Ref. 81). Figure 6.6 combines information from Figures 6.2-6.5 for a specific base condition to compare the relative differences between different levels of cracking. Figure 6.7 illustrates the general decrease in required AC thickness by allowing a greater percentage of surface fatigue cracking. This demonstrates that a small increase in AC thickness will significantly decrease the amount of cracking, 50% to approximately 10%, whereas a large increase in thickness is required to reduce the amount of cracking from 10% to crack initiation.

Based on the results obtained by Darter (Ref. 1) a cracking value of 50% would definitely require an overlay or major maintenance, whereas no maintenance would be anticipated for crack initiation. Therefore, based on past performance information, the 10% amount of cracking was selected as the failure criteria at 20 years (no maintenance). Beyond 20 years, routine maintenance would be anticipated because of the increase in cracking with time, as theoretically shown in Figure 6.8.

The 10% cracking criteria applies to the wheel path area and can be considered equivalent to 5% of the total area. Based on pavement condition survey reports developed by a number of state agencies this amount of fatigue cracking would not be sufficient to require maintenance or rehabilitation. Also, data from the AASHO Road Test indicates that the functional performance (PSI) of a pavement would not be significantly reduced by this amount of cracking.

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[1]Rauhut, J.B., J.C. O'Quin, and W.R. Hudson, "Sensitivity Analysis of FHWA Structural Model VESYS II", Vol. I, Preparatory and Related Studies, Report No. FHWA-RD-76-24, Federal Highway Administration, March 1976.

[2]"Asphalt Concrete Overlays of Flexible Pavements, Vol. I - Development of New Design Criteria", Report No. FHWA-RD-75-75, Federal Highway Administration, June 1975.

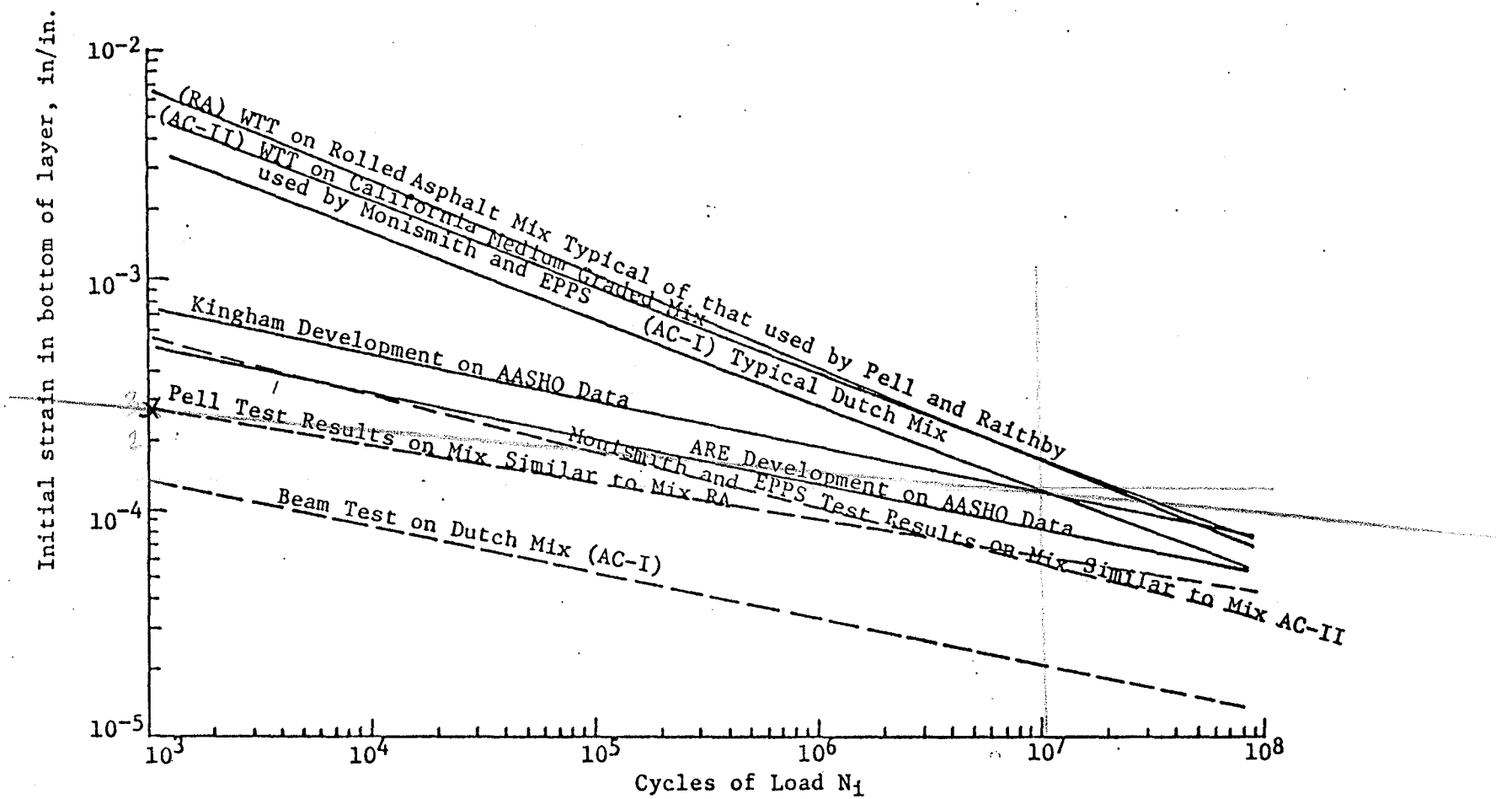
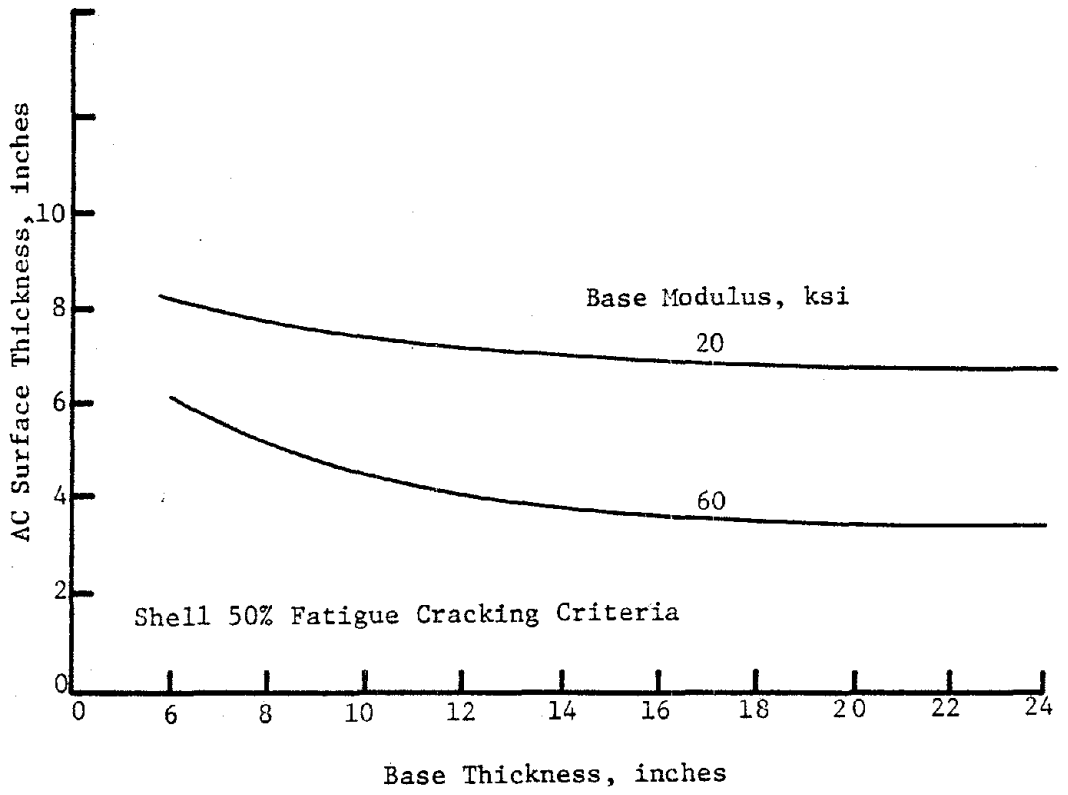
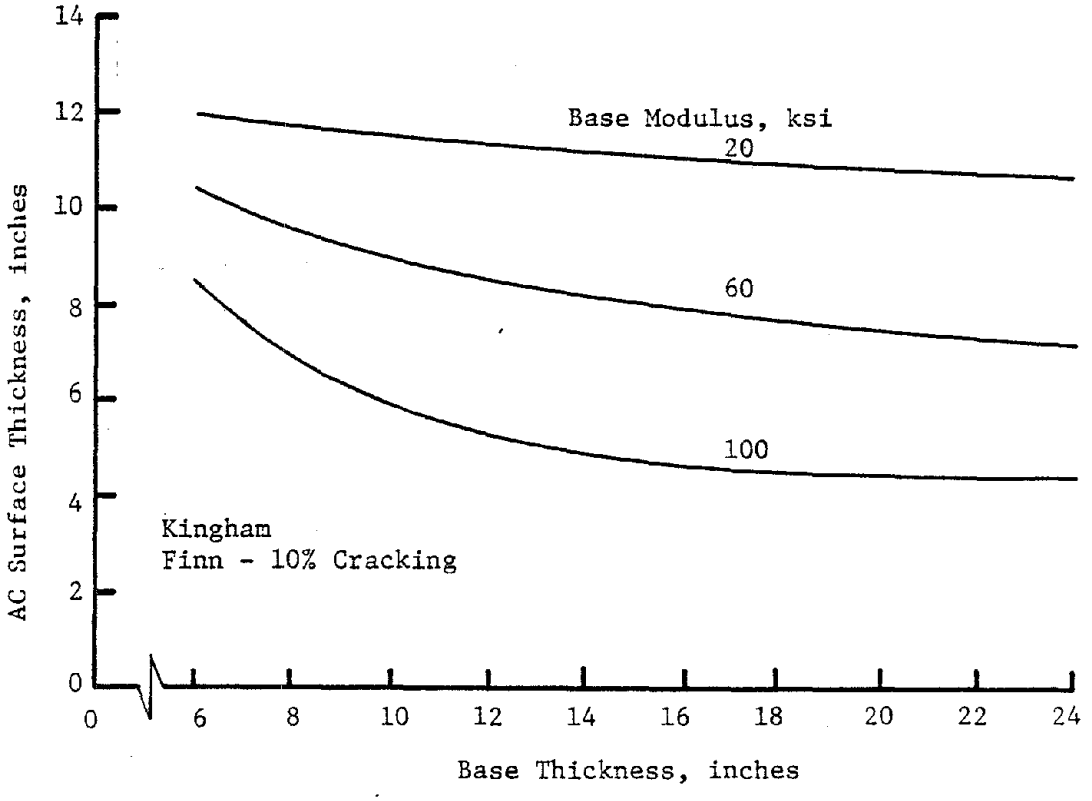


Figure 6.1. Plots for fatigue relationships from wheel tracking tests, analytical developments based on AASHO Test data and beam tests (Ref. 117).



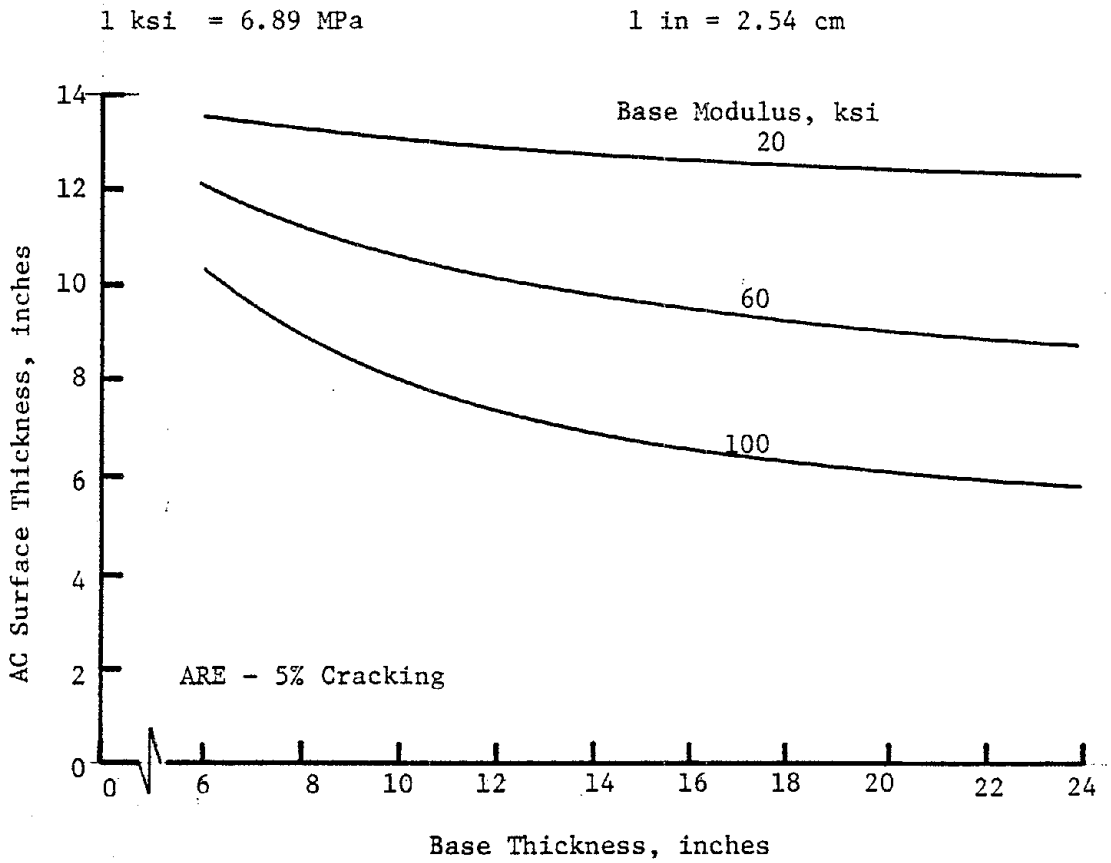
1 ksi = 6.89 MPa 1 in = 2.54 cm

Figure 6.2. Illustration of the required AC thickness for different base conditions based on the fatigue curve for 50% cracking as developed by Shell.



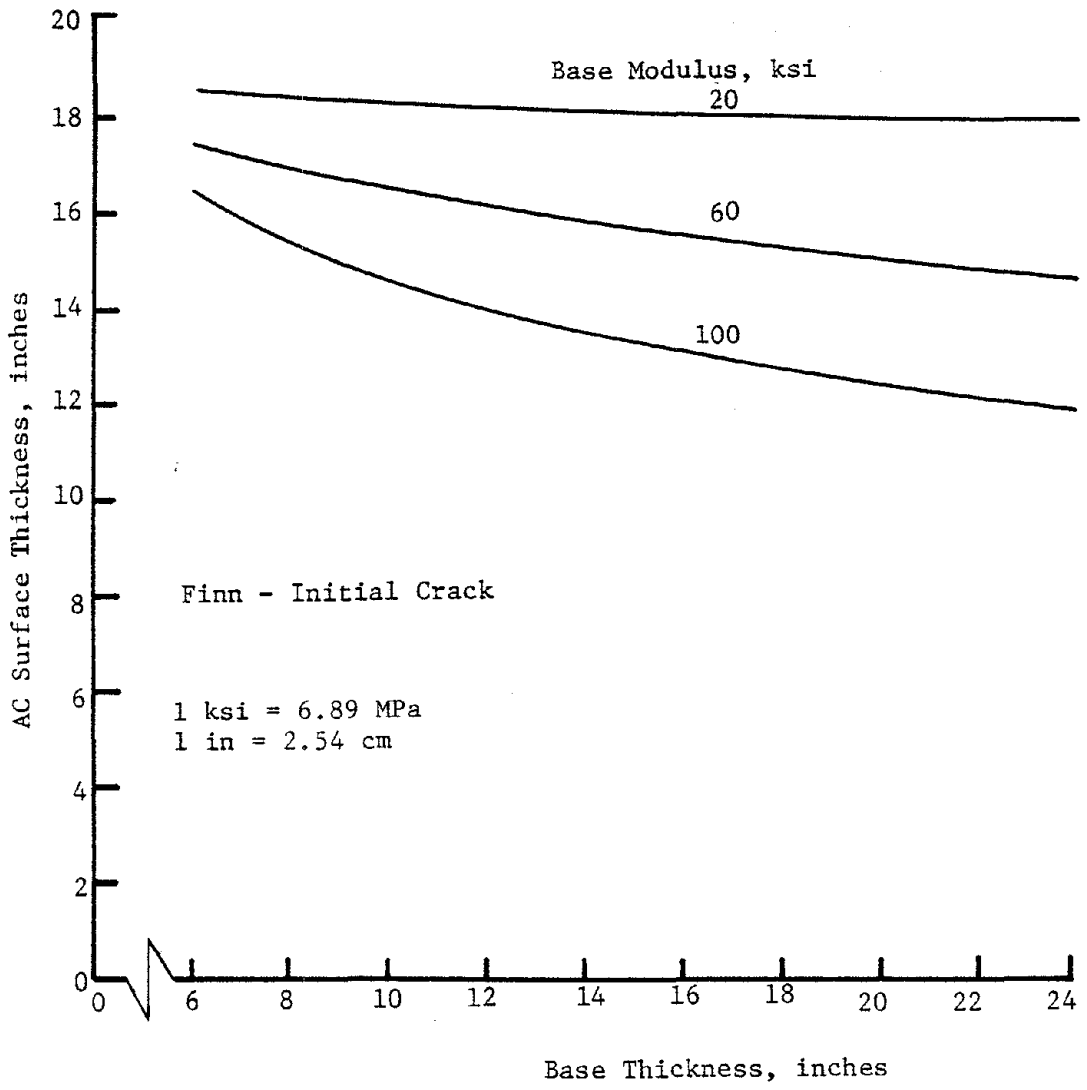
1 ksi = 6.89 MPa 1 in = 2.54 cm

Figure 6.3. Illustration of the required AC thickness for different base conditions based on the fatigue curve for 10% cracking.



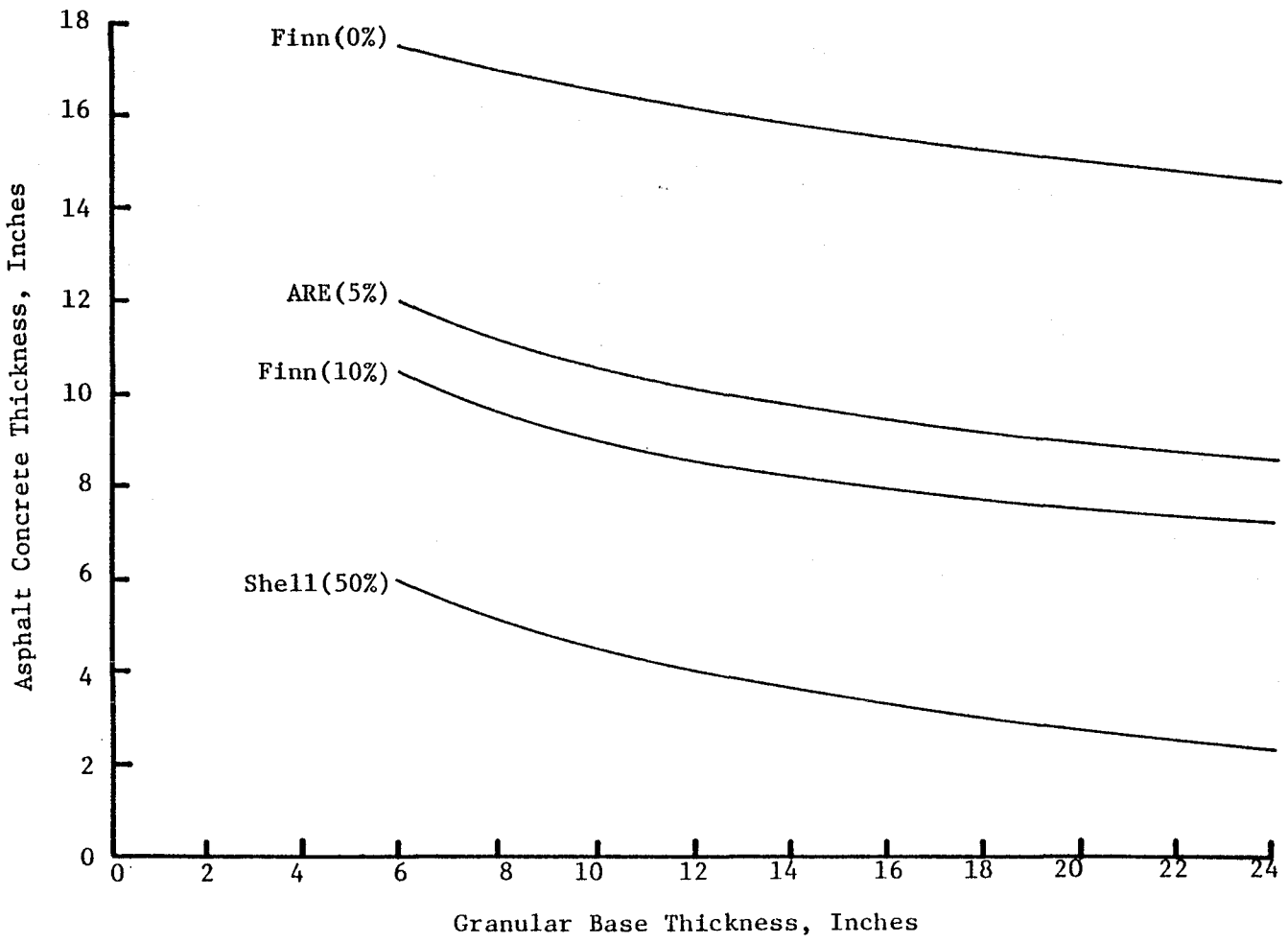
Required Surface and Base Thickness to Meet Fatigue Criteria

Figure 6.4. Illustration of the required AC thickness for different base conditions based on the fatigue curve for 5% cracking.



Required Surface and Base Thickness to Meet Fatigue Criteria

Figure 6.5 Illustration of the required AC thickness for different base conditions based on the fatigue curve for crack initiation.



1 in = 2.54 cm

Figure 6.6. Comparison of the different failure levels for different granular base thicknesses.

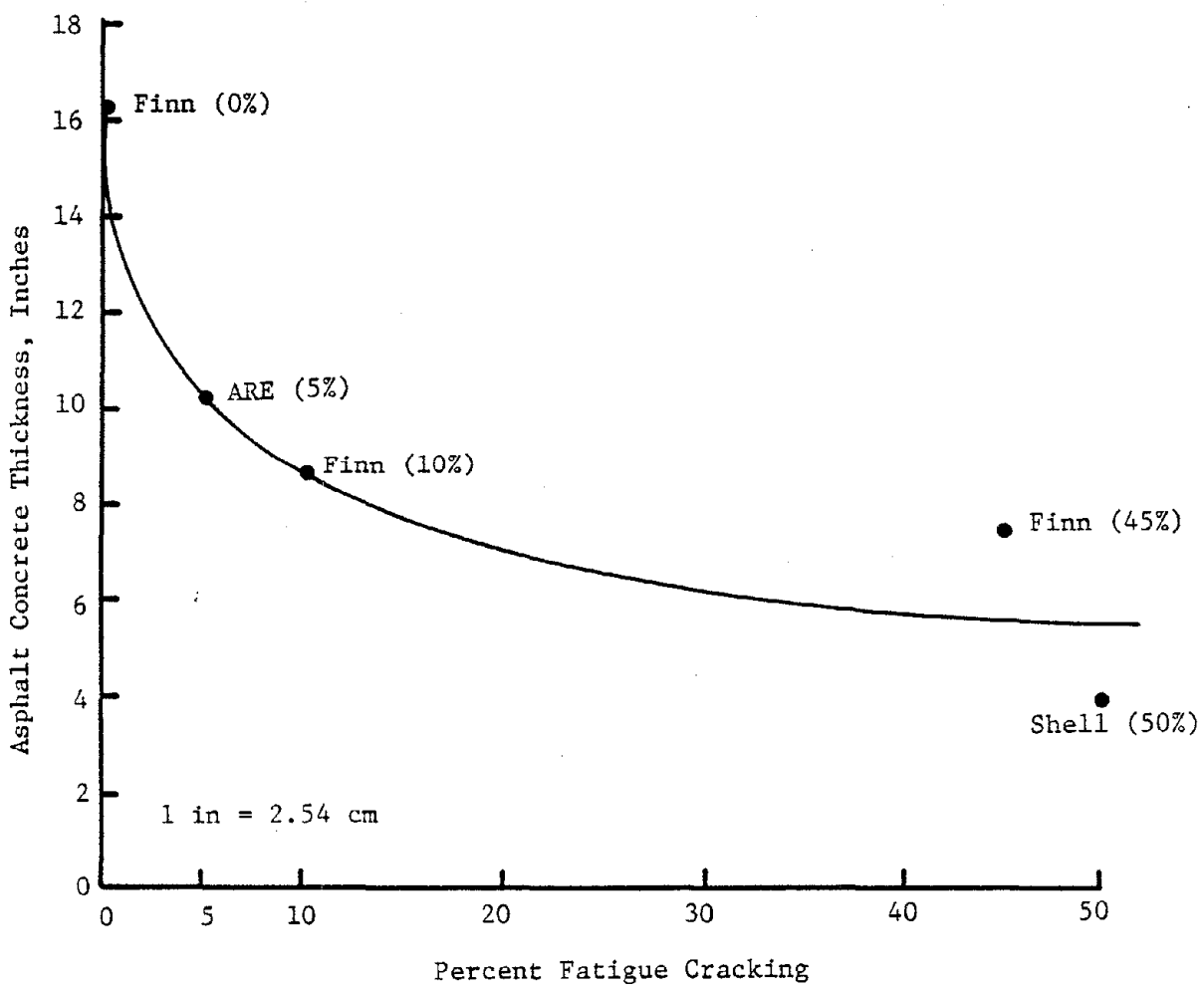
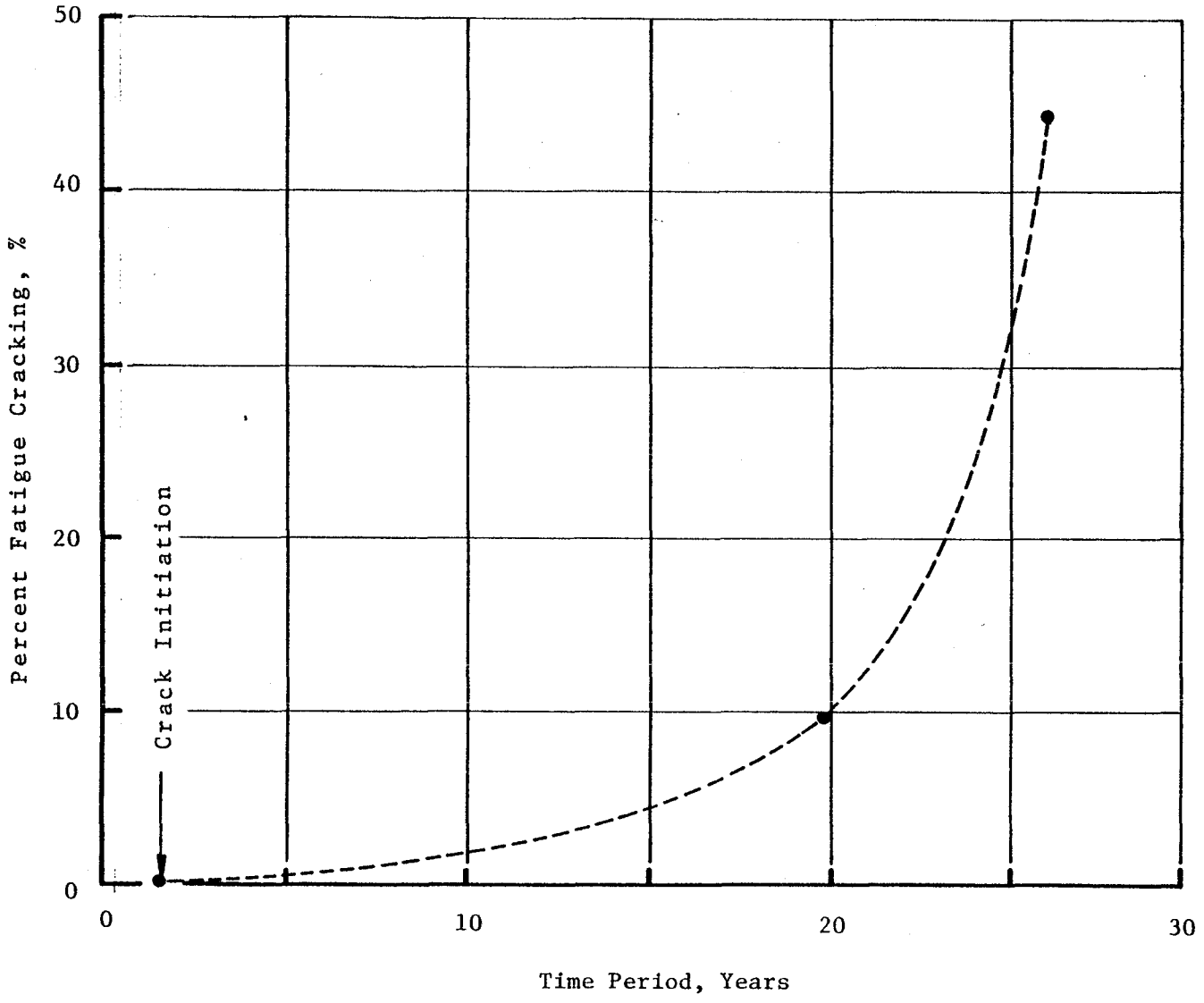


Figure 6.7. Illustration showing the effect of asphalt concrete thickness on the percent of fatigue cracking.



Assumption: Linear Increase in Traffic with Time 14 x 10^6 applications at 20 years

Figure 6.8. Illustration showing the increase in cracking with time (traffic), assuming no maintenance, using the fatigue criteria developed by Finn.

Based on these findings it is recommended that the equation developed by Finn be used to estimate the fatigue cracking failure of the asphalt concrete layer as follows (Ref. 82):

$$\text{Log } N_f = 15.947 - 3.291 \log (\varepsilon 10^{-6}) - 0.854 \text{ Log } (E^*/10^3) \quad \ldots\ldots \quad 6.1$$

Where:

N_f = Allowable number of 18-Kip (80 Kn) single axle loads for a 10% fatigue cracking criteria.

E^* = Dynamic modulus of the asphalt concrete, psi

ε = Horizontal tensile strain at the bottom of the asphalt concrete layer, in/in.

After this calculated number of traffic applications (N_f), routine maintenance would be required in order for the pavement to maintain an acceptable level of serviceability (less than 45% cracking) for an additional 10 to 20 years as illustrated in Figure 6.8. Equation 6.1 is illustrated in Figure 6.9.

Using equation 6.1 and the VESYS III computer program to evaluate flexible pavements, the fatigue damage of the asphalt concrete layer can be evaluated throughout the year by using fatigue damage factors. These damage factors can be developed by simply taking the damage throughout the year or as a function of the asphalt concrete modulus and equating to the damage at a constant or effective asphalt concrete modulus. In relating temperature, material properties, and loading rate, the regression equation developed by Witczak (Ref. 116[1]) was applied to simplify the number of inputs and interrelationships between variables. This was completed for the fatigue equation and assuming that equal levels of traffic would occur in four equal seasons during the year, projected results are shown in Figure 6.10. Hence, Figure 6.10 allows a simplified design criteria for fatigue cracking. It must be understood that if the traffic changes with season or if other fatigue curves are used, Figure 6.10 is not applicable.

Portland Cement Concrete

One of the major distress mechanisms associated with rigid pavements is fatigue cracking, defined as "the process of progressive localized permanent structural change occurring in the material subjected to conditions which produce fluctuating stresses and strains at some point or points and which may culminate in cracks or complete fracture after

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[1]Witczak, M.W., "Development of Regression Model for Asphalt Concrete Modulus for Use in MS-1 Study", January 1978.

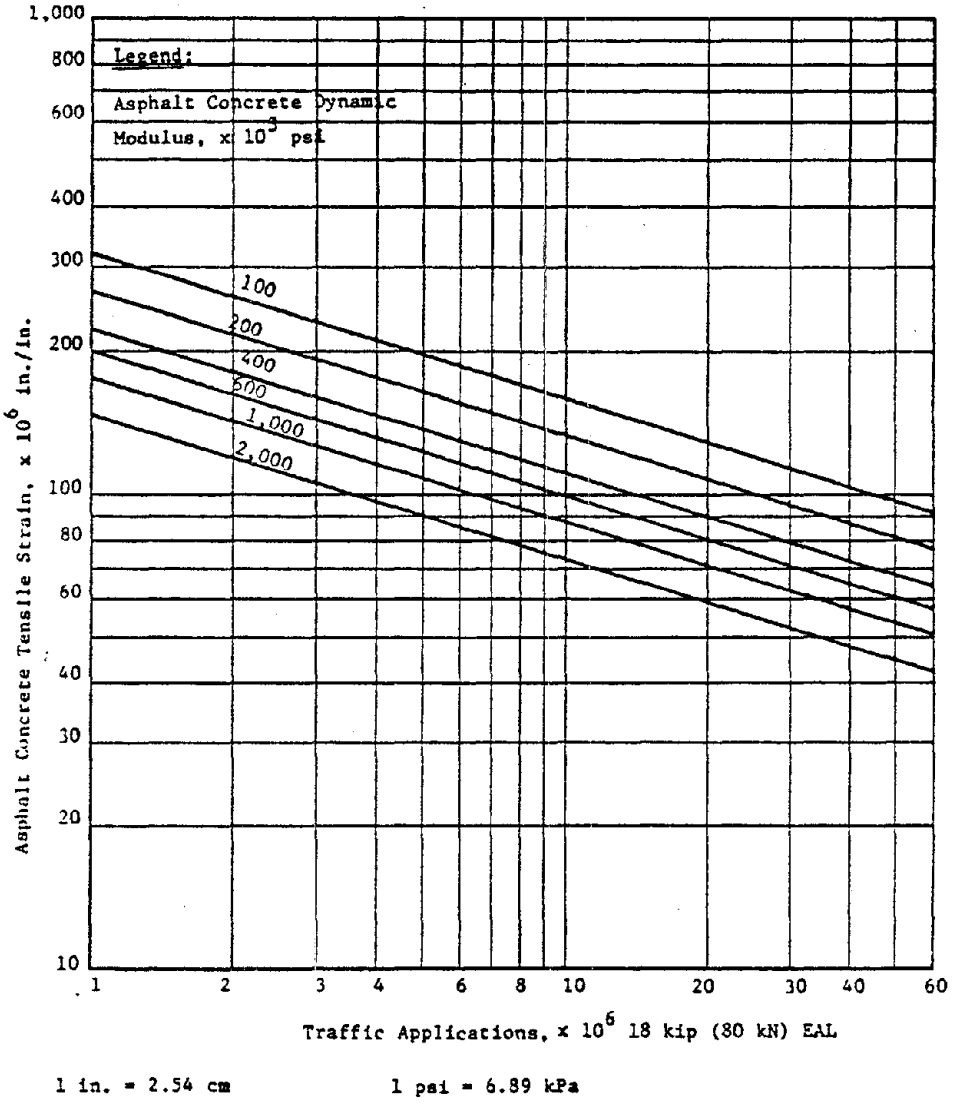

Figure 6.9. Asphalt concrete fatigue curves used to establish the fatigue cracking criteria (Ref. 5).

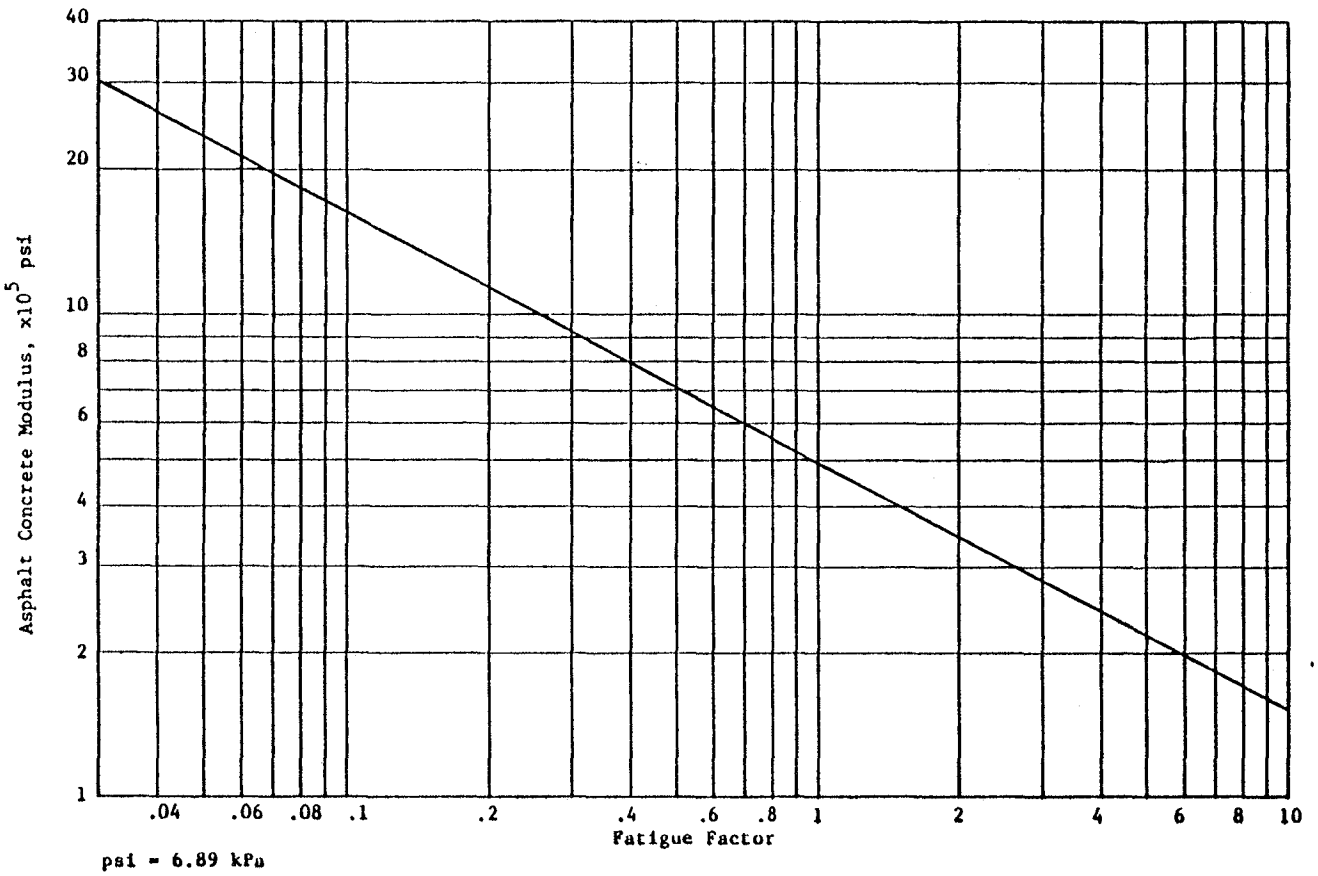

Figure 6.10. Estimation of the fatigue factor.

sufficient number of fluctuations" (Ref. 114). The basis for using cracking alone as a design criteria is inadequate since the formation of a crack does not necessarily imply a functional failure as illustrated in Figure 6.11 (Refs. 125[1], 126[2], 127[3]). These fatigue relationships were determined from data collected on reinforced and non-reinforced jointed concrete pavements, to different levels of service (Class 3 and 4 cracking, and a Present Serviceability Index of 2.5). In order to develop a fatigue failure criteria for zero-maintenance pavements, specific items must be investigated as to their effect on fatigue cracking of concrete layers. These are given below:

1. Failure Level
2. Pavement Type
3. Concrete Type
4. Laboratory Data

Failure Level

In establishing a failure criteria, it becomes questionable whether the same fatigue relationships, based on performance data obtained from concrete surface layers (Figure 6.11), are valid for composite pavements. For composite pavements the concrete layer is no longer the riding surface, hence fatigue characteristics, may be different for a failure criteria based on a PSI of composite pavements. Although the fatigue characteristics for a failure criteria of cracking should also be different for composite pavements, the difference should be much smaller, since cracking is primarily related to concrete stress and strength magnitudes, whereas PSI is related to roughness, cracking, and patching as determined from the AASHO Road Test. Also, since performance of a composite pavement is directly related to the cracking of the rigid base layer, cracking was selected as the failure criteria for the concrete layer.

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[1]Hudson, W.R. and F.H. Scrivner, "AASHO Road Test Principal Relationships - Performance with Stress in Rigid Pavements", Special Report 73, Highway Research Board, Washington, D.C., 1962.

[2]Vesic, A.S. and S.K. Saxena, "Analysis of Structural Behavior of Road Test Rigid Pavements", NCHRP Project 1-41(1), National Cooperative Highway Research Program, 1968.

[3]"The AASHO Road Test, Proceedings of a Conference held May 16-18, 1962, St. Louis, MO", Special Report 73, Highway Research Board, Washington, D.C., 1962.

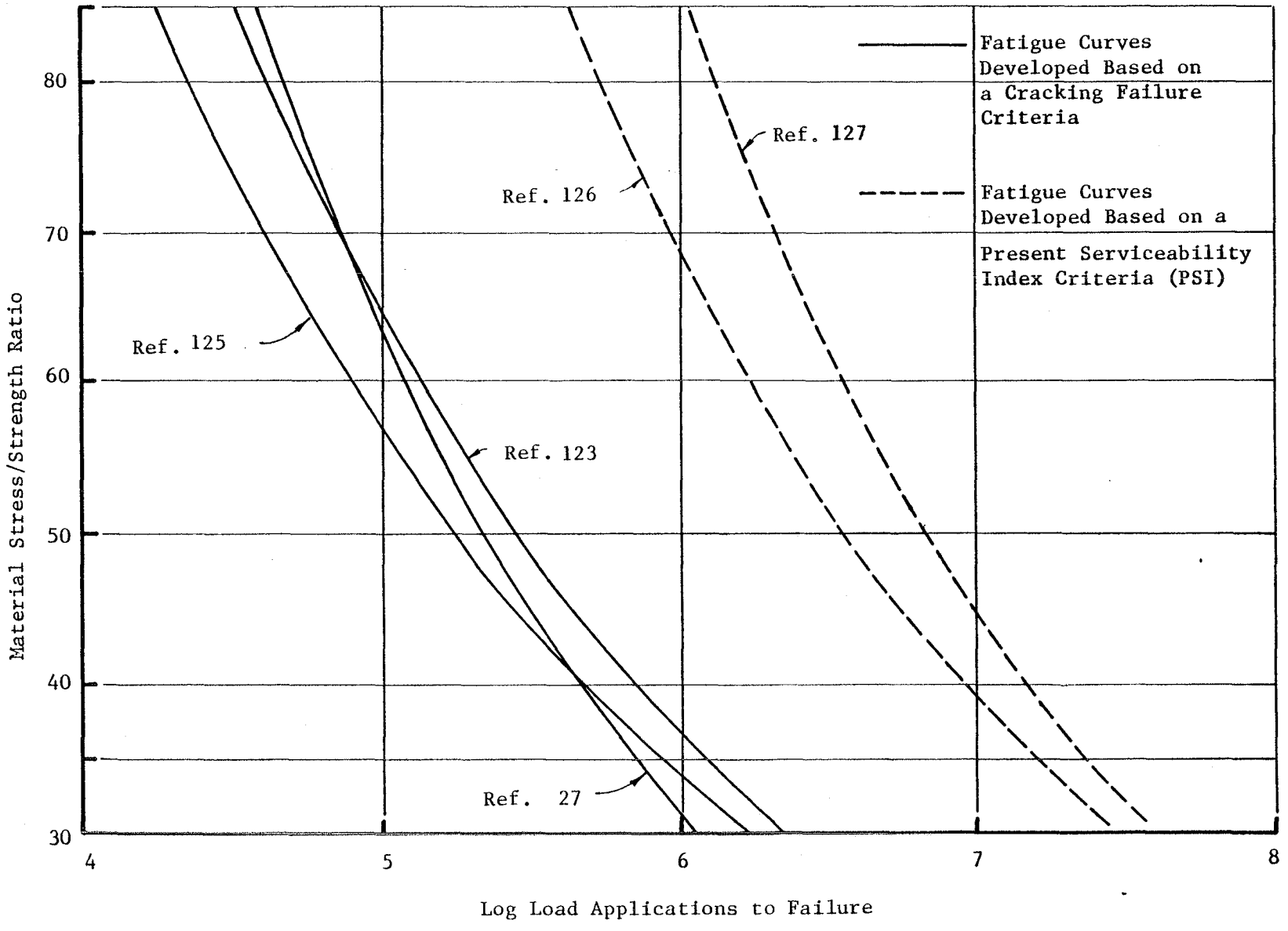

Figure 6.11. Fatigue curves for jointed concrete pavements to different failure definitions based on field performance data.

Pavement Type

The fatigue relationships presented in Figure 6.11 were based on the performance of reinforced and non-reinforced jointed concrete pavements. In studies completed by Treybig, et al., (Ref. 27) and Yimprasert, et al., (Ref. 122[1]), of the AASHO Road Test, it was observed that essentially no difference existed in the sections which failed; although, the non-reinforced sections had a greater probability of survival and shorter time span between the different levels of cracking. Therefore, based on these analyses and since no other fatigue cracking studies exist comparing reinforced to non-reinforced pavements, the same fatigue cracking criteria will be applied to all concrete pavement types. Only a difference of loading conditions (edge, corner or interior) will be used to predict the applied flexural stress in the concrete layer.

Concrete Type

The fatigue relationships presented in Figure 6.11 represent the same concrete pavement but according to different failure definitions, except for the curve developed in Reference 123[2] which is based on airfield failures of a jointed concrete pavement. Therefore, it becomes questionable whether these same fatigue relationships can be applied to other concrete mixtures; such as lean concrete and econocrete mixtures, given in Table 6.1 (Ref. 120[3]), etc. Figure 6.12 presents laboratory fatigue results illustrating the effect of curing time and mixture properties on the fatigue relationship. As shown, all mixtures exhibit approximately the same relationship between stress/strength ratio and cycles to failure. Figure 6.13 (Refs. 128[4], 129[5], 130[6], 131[7]) presents laboratory fatigue results for specimens ranging from portland cement concrete to lean concrete. As shown, there is no distinct relationship for each type of concrete; i.e., lean concrete relationship falls between relationships determined for PCC. Figure 6.14 demonstrates that the loading frequency of concrete beams does not have a large effect on the fatigue life of concrete. Therefore, based on the above information

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[1]Yimprasert, P. and B.F. McCullough, "Fatigue and Stress Analysis Concepts for Modifying the Rigid Pavement Design System", Research Report No. 123-16, Center for Highway Research, The University of Texas at Austin, January 1973.

[2]Treybig, H.J., B.F. McCullough and W.R. Hudson, "Tests of Existing Pavements and Synthesis of Design Methods", Vol. I, CRC Airfield Pavements, Air Force Weapons Laboratory, December 1973.

[3]Raithby, K.D. and J.W. Galloway, "Effects of Moisture Condition, Age, and Rate of Loading on Fatigue of Plain Concrete", Abeles Symposium - Fatigue of Concrete, ACI Publication SP-41, American Concrete Institute.

TABLE 6.1. DETAILS OF CONCRETE MIXTURES PRESENTED IN FIGURE 6.12 (REF. 120).

Mix			PQ1		PQ2		LC1	
Aggregate	Type		Uncrushed Flint		Crushed Limestone		Uncrushed Flint	
	Particle Shape		Irregular		Angular		Irregular	
	Surface Texture		Smooth		Fairly Rough		Smooth	
	Water Absorption % and mix proportions %	19 - 9.5 mm	1.9	41	0.8	17	1.9	35
		9.5 - 4.8 mm	3.3	20	0.7	39	3.3	25
		Passing 4.8 mm	1.1	39	0.5	44	1.1	40
	BS 882 Grading of Fines		ZONE 2		ZONE 1		ZONE 2	
Mix Proportions	Aggregate/cement ratio		6.7		5.2		18.0	
	Free water/cement ratio		0.5		0.62		6½% of total oven dry weight of aggregate and cement	
Trial Mix Details	Workability		Compacting Factor 0.84		'V-B' Test 7 secs. (equivalent to C.F. of 0.84)		Compacting Factor 0.79	
	28 day indirect tensile strength (152 mm x 102 mm diameter cyclinders)		3.50 MN/m^2		3.48 MN/m^2		2.00 MN/m^2	
	28 day compressive strength (102 mm cubes)		44.80 MN/m^2		33.30 MN/m^2		21.00 MN/m^2	

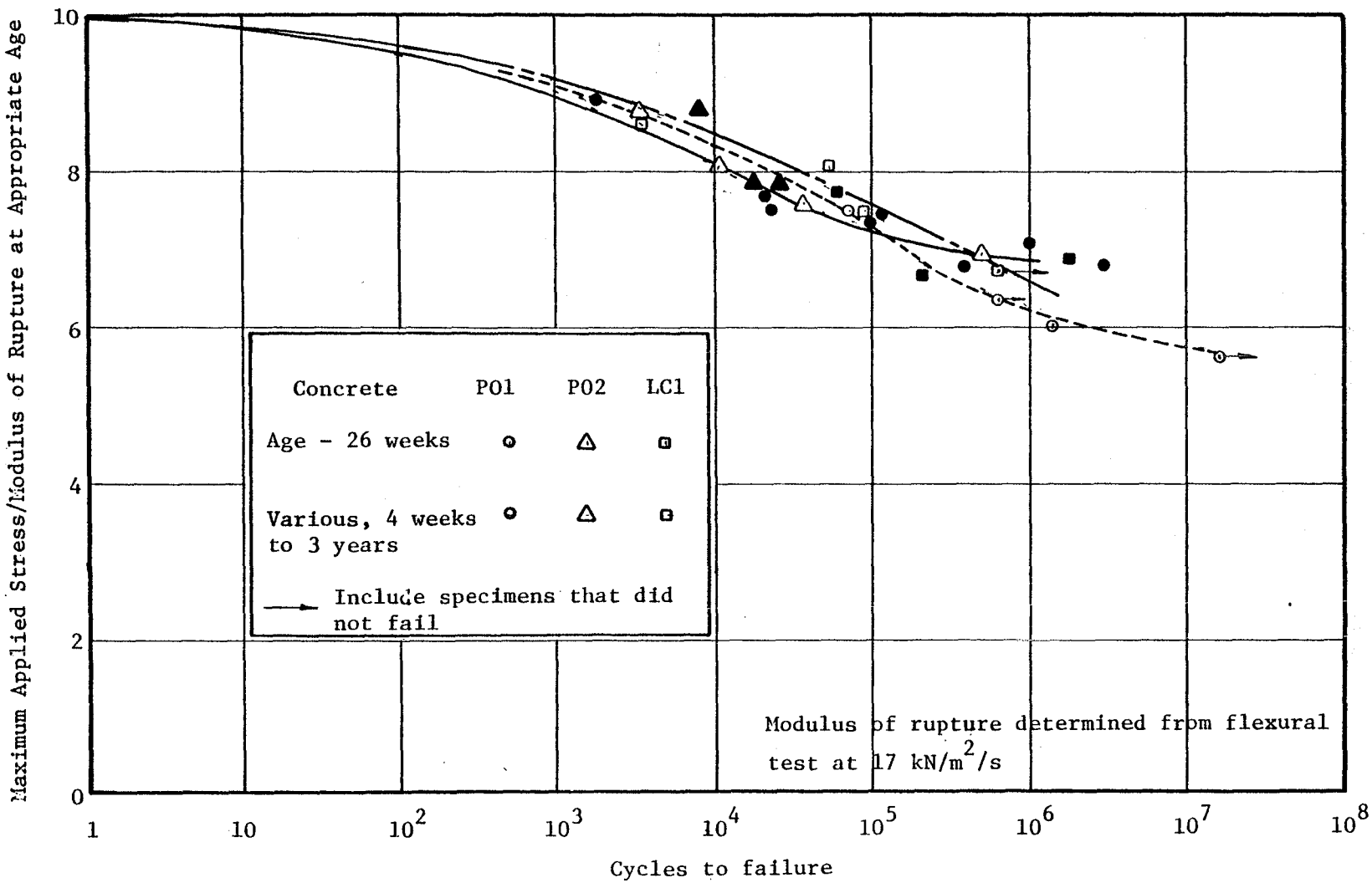


Figure 6.12. Laboratory fatigue results of different concrete mixtures given in Table 1.1 (Ref. 120).

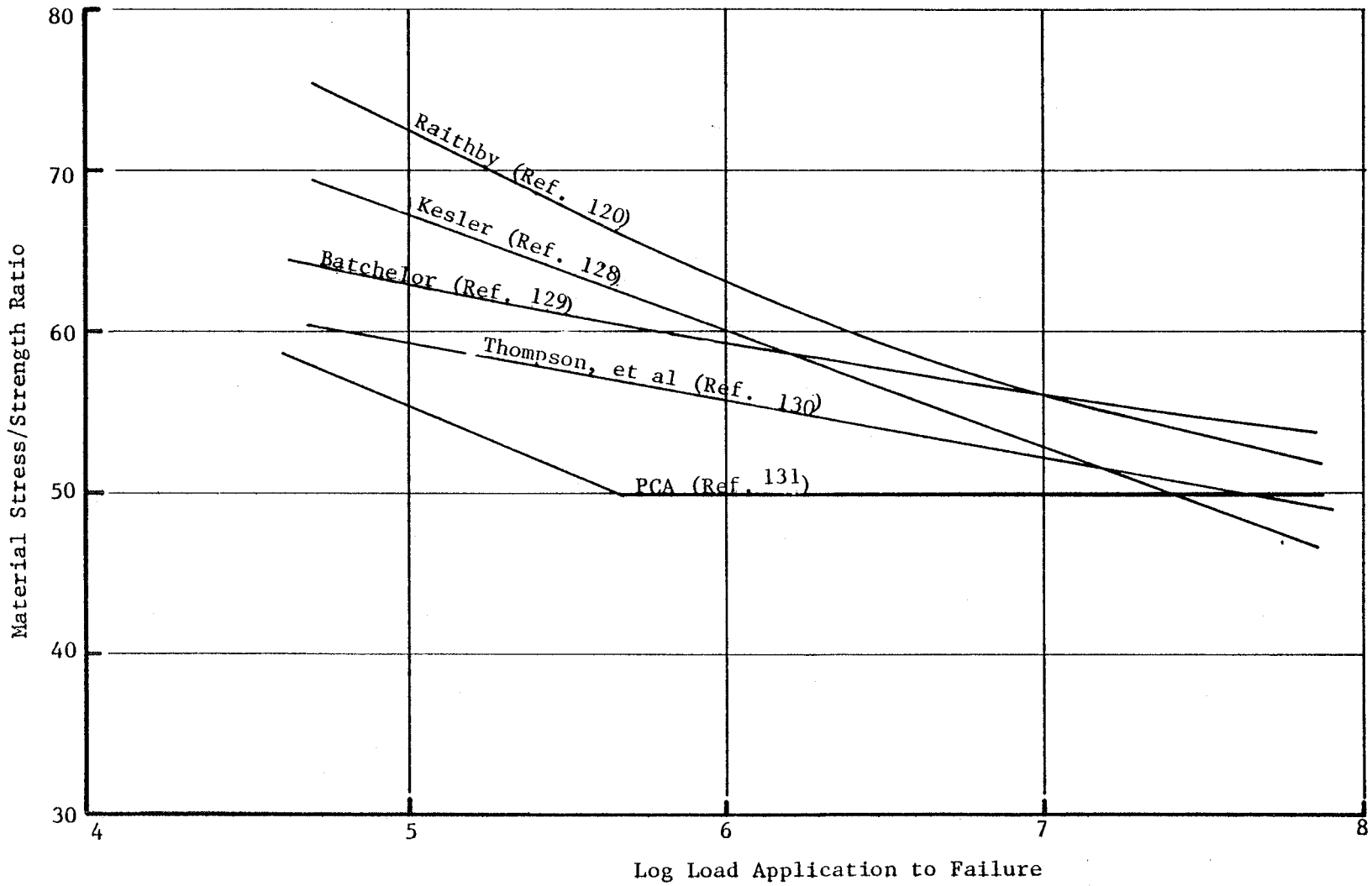
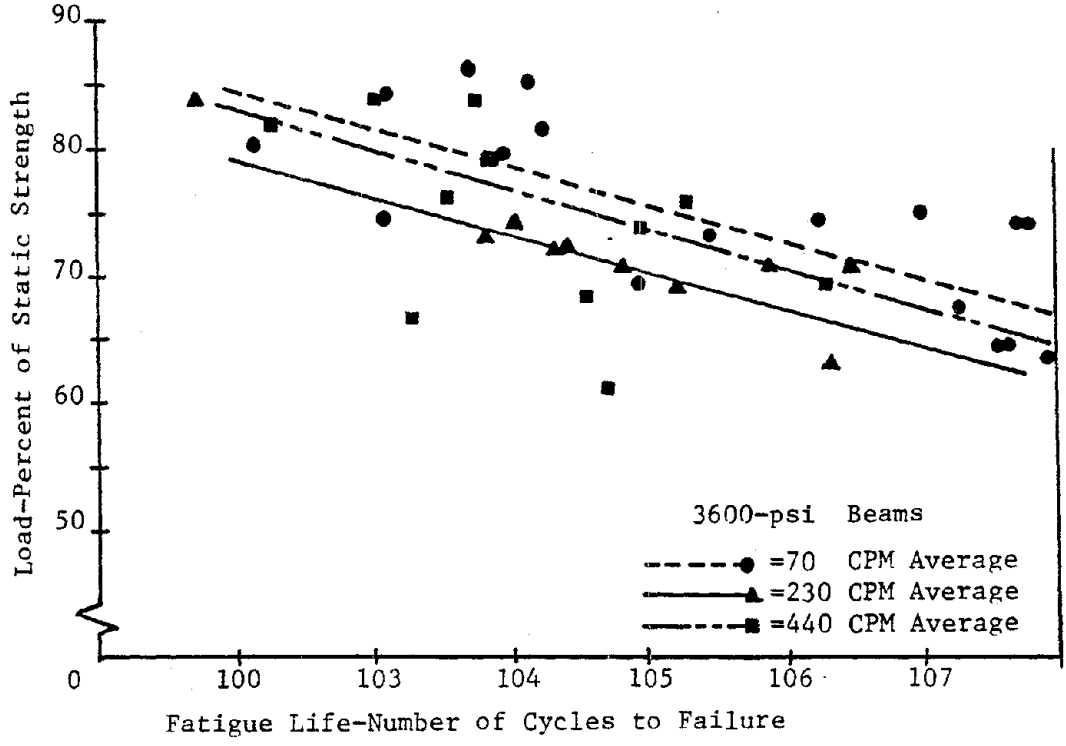
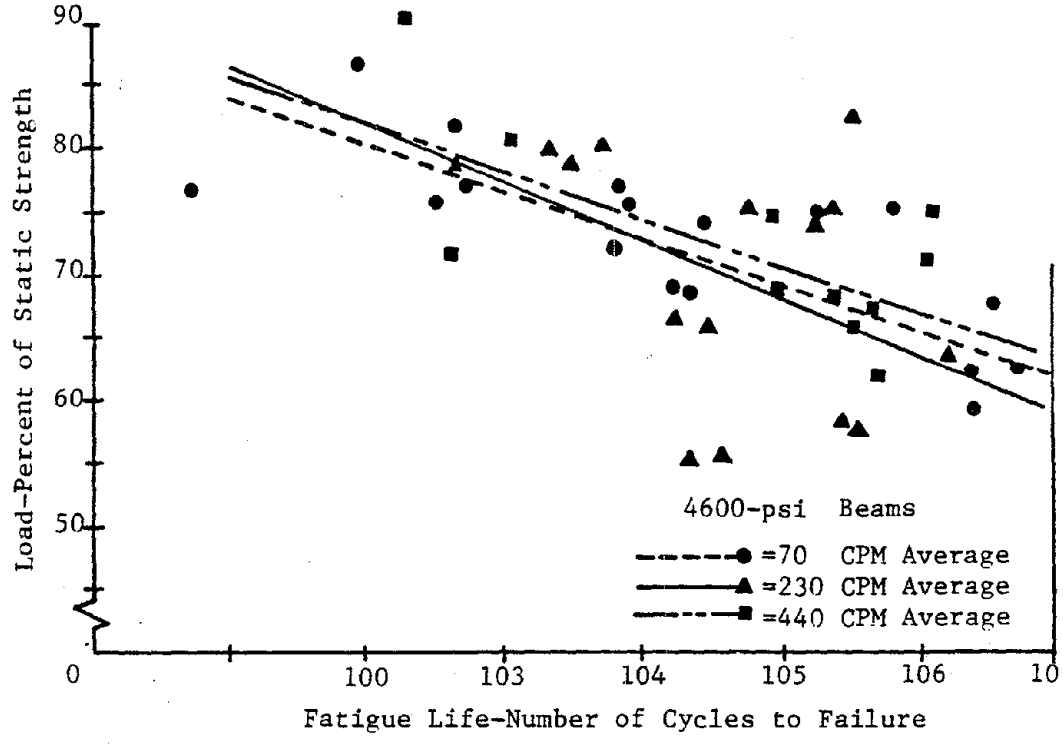


Figure 6.13. Laboratory fatigue curves for different concrete mixtures.



(a)



(b)

1 psi = 6.89 kPa

Figure 6.14. Effect of loading frequency on the fatigue of two concrete mixtures (Ref. 120).

it will be assumed that one fatigue curve can be used to describe the cracking failure criteria for all types of concrete mixtures. Although, this might not be true if the concrete were the surface layer and failure was based on the present serviceability index.

Laboratory Data

As shown by Figures 6.13 and 6.14 there have been numerous laboratory studies concerning the fatigue resistance of concrete. By comparing these laboratory results to the fatigue characteristics determined from pavement performance (Figure 6.11) it is observed that a difference exists. For large stress/strength ratios, a larger number of load applications would be expected based on field results. This is probably due to the interval between initial concrete cracking and some severity level of cracking, indicating pavement failure. For small stress/strength ratios, a smaller number of load applications would be expected, based on field results as compared to laboratory results. This may be a result of environmental factors, dynamic effect of loads, and other damage factors which may have been underestimated in developing the field fatigue curves (Figure 6.11). Therefore, based on the above information and until more reliable fatigue information can be obtained, the fatigue curves describing infield performance will be used to determine fatigue failures for low modulus concrete.

In summary, it is recommended, based on the information reviewed, that failure of the concrete layer for composite pavements be based on cracking severity determined from pavement performance data. Using this failure criteria zero-maintenance should not be required for a 20-year traffic period, when cracking will approach a condition where routine maintenance will be required to provide an additional 10 to 20 years of service.

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[4]Kesler, C.E., "Fatigue and Fracture of Concrete", Stanton Walker Lecture Series on the Material Sciences, University of Maryland, College Park, MD.

[5]Batchelor, B. and B.E. Hewitt, "Are Composite Bridge Slabs Too Conservatively Designed? - Fatigue Studies", Abeles Symposium - Fatigue of Concrete, ACI Publication SP-41, American Concrete Institute, 1974.

[6]Thompson, P.D., D. Croney, and E.W.H. Currer, "The Alconbury Hill Experiment and Its Relation to Flexible Pavement Design", Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, London, England, September 1972.

[7]"Fatigue - Coverage Concepts Applied to Concrete Airport Pavements Designs", Portland Cement Association, Technical Paper, Skokie, Ill., 1973.

Of equal importance to fatigue resistance are the boundary conditions associated with the applied wheel loads. In using VESYS III, a fatigue curve developed from elastic layer theory must be used, because of the interior loading or boundary conditions associated with the program. But in using SLAB49, a different fatigue curve must be used, since edge or corner loading conditions can be simulated of actual boundary conditions. Therefore, since the data used to develop the fatigue curves in Figure 6.11 are representative of an edge loading condition, but were developed with elastic layer theory, they are applicable to VESYS III, as long as an edge loading condition exists on the actual pavement. In using SLAB49 the present fatigue curves are not applicable, and hence must be modified. The same procedure as used by Rauhut et al., in Reference 4 was used to adjust the elastic theory based fatigue curve to account for the edge loading condition and curling stresses based on previous SLAB49 solutions. This adjustment resulted in two fatigue curves developed from the same data but with different theoretical boundary conditions. These fatigue curves are presented in Figure 6.15.

RUTTING

The state of the art in the analysis of permanent deformation under repeated loads is less advanced than the existing fatigue formulations. For flexible pavements two criteria were selected which must be satisfied, permanent deformation of the surface layer and subgrade vertical compressive strain. Based on performance studies by Darter (Ref. 1) and Shell (Ref. 81) of in-service pavements, a value of 0.40 inches (1.0 cm) rut depth was found not to require any maintenance. Therefore, a rut depth of 0.4 inches (1.0 cm) was selected as the failure criteria for zero-maintenance flexible and composite pavements. The additional failure criteria of subgrade vertical strain is only applicable to flexible pavements as shown in Figure 6.16. Based on the review of previous research studies presented in Figure 6.16, numerous failure curves have been developed for various conditions. The failure curve developed by the Corps of Engineers takes into account variations in subgrade modulus, unlike the other curves shown in Figure 6.16. Although this curve was developed based on information gathered from airfield pavements, it was selected as a failure criteria for premium pavements. By using this curve subgrade modulus, traffic applications and load magnitudes will be considered. Figure 6.17 illustrates this relationship as developed by the Corps of Engineers. Some extrapolation of subgrade modulus curves was necessary.

For unbound base and subbase courses, Reference 28 suggests as design criteria that the horizontal tensile stress be limited to 0.5 times the vertical stress plus the horizontal overburden pressure. Hence, the stress strain condition in each layer must be analyzed to ensure that the contribution of each layer to surface rutting will be minimized.

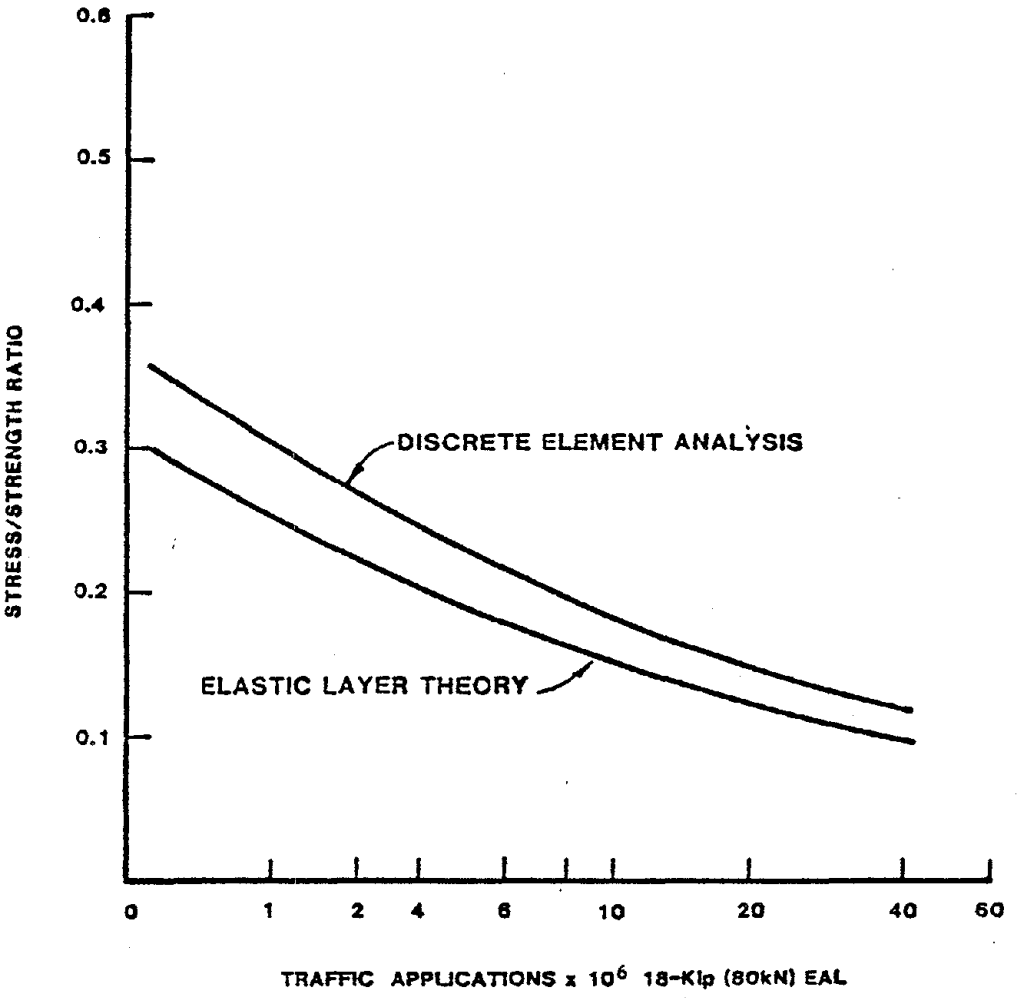


Figure 6.15. Portland cement concrete fatigue curves used to establish the fatigue cracking criteria.

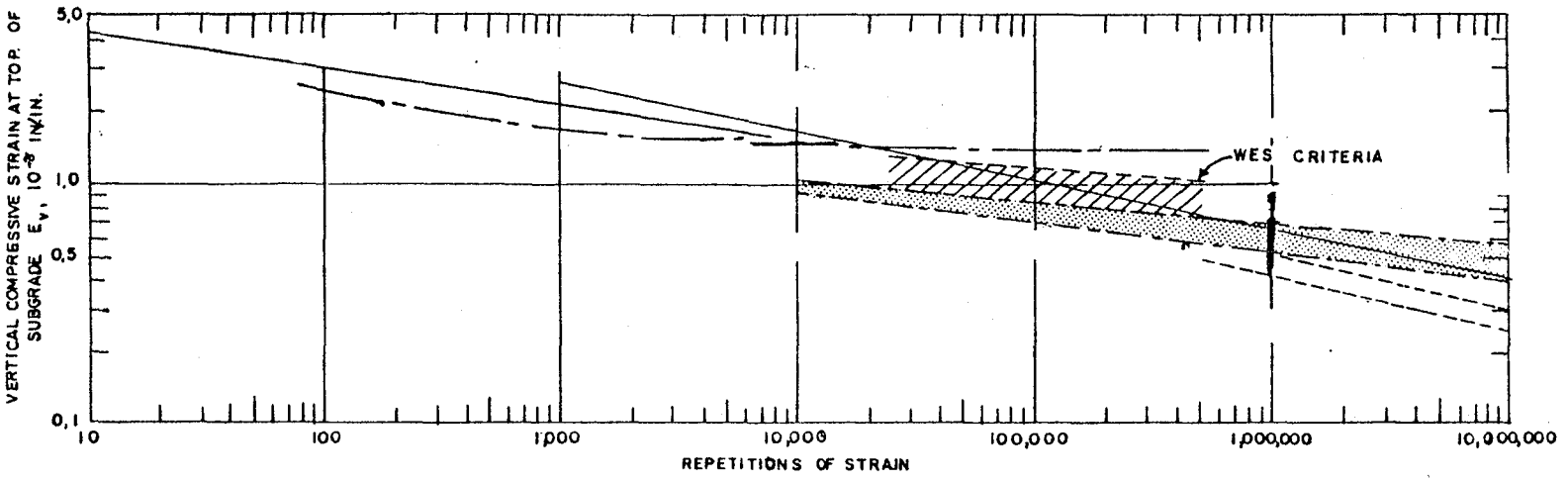
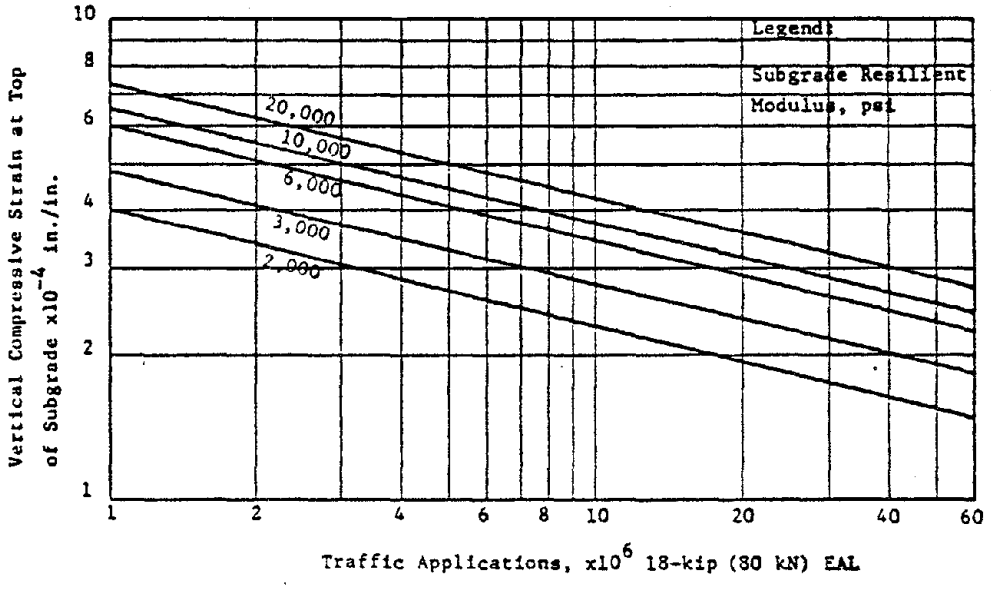


Figure 6.16. Comparison of subgrade strain criteria (Ref. 59).



1 in. = 2.54 cm
1 psi = 6.89 kPa

Figure 6.17. Allowable subgrade vertical strain used to develop the structural cross-section for the zero-maintenance pavement (Ref. 59).

TRANSVERSE CRACKING

There is almost no data available to establish a failure criteria for transverse cracking (including low-temperature and reflection cracking). Based on Darter's observations of in-service pavements, he established a 10 to 30 foot (3.0 to 9.0 m) crack spacing as a limiting value. Therefore, based on this information and the project staff's experience, an average crack spacing of 30 feet (90 m) was selected as the failure criteria for transverse cracking in the surface layer.

For composite pavements, the continuously reinforced concrete layer's performance has also been observed to be related to crack spacing. Based on previous observations and performance studies a crack spacing greater than 4 feet (1.2 m) but less than 8 feet (1.4 m) has been observed to provide adequate serviceability.

DEFLECTION

Surface deflection has also been related to field performance, as shown in Figures 6.18 and 6.19 (Ref. 118[1]). In fact, a large number of design and evaluation procedures are based on surface deflection. Therefore, surface deflection was selected as an additional limiting design criteria for zero-maintenance pavements. Emphasizing surface deflection allowed the use or consideration of a large number of studies and observations of past pavement performance (Figures 6.18 and 6.19). Therefore, based on a review of existing data, the relationships in Figure 6.20 (Ref. 119[2]) were selected to develop the pavement structural cross-section to limit surface deflections.

ROUGHNESS

The other important performance parameter used in establishing limiting criteria for zero-maintenance pavements was roughness or PSI (present serviceability index). Based on previous studies conducted on flexible and composite pavements, a PSI lower than 3.0 will require maintenance. Hence, a limiting value of PSI of 3.0 was selected in establishing the pavement cross-section to meet the zero-maintenance criteria.

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[1] Roads and Transportation Association of Canada, Pavement Management Guide, published by RTAC, Ottawa, 1977.

[2] Lister, N.W. and C.K. Kennedy, "A System for the Prediction of Pavement Life and Design of Pavement Strengthening", Volume I, Fourth International Conference Structural Design and Asphalt Pavements, The University of Michigan, August 1979.

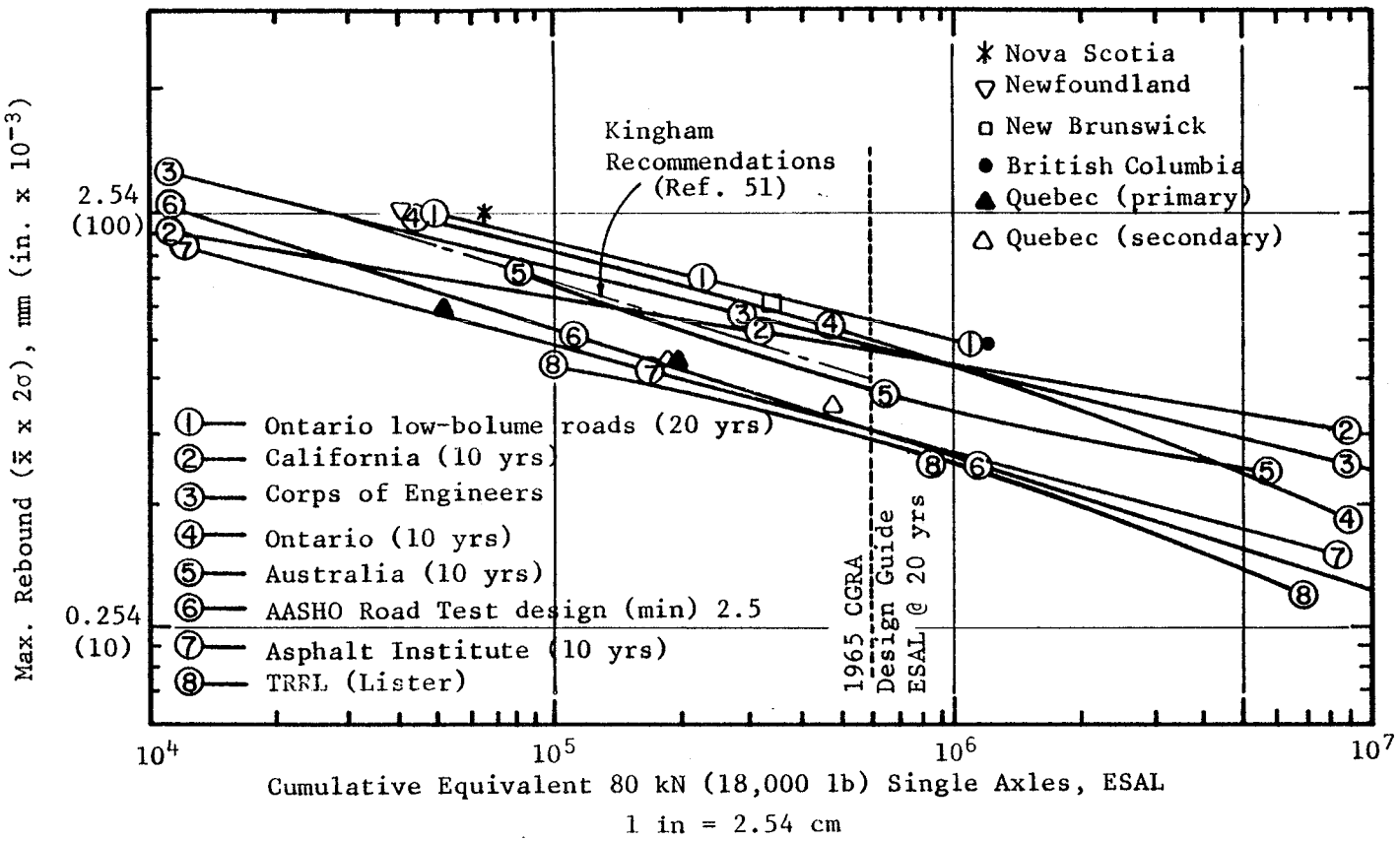
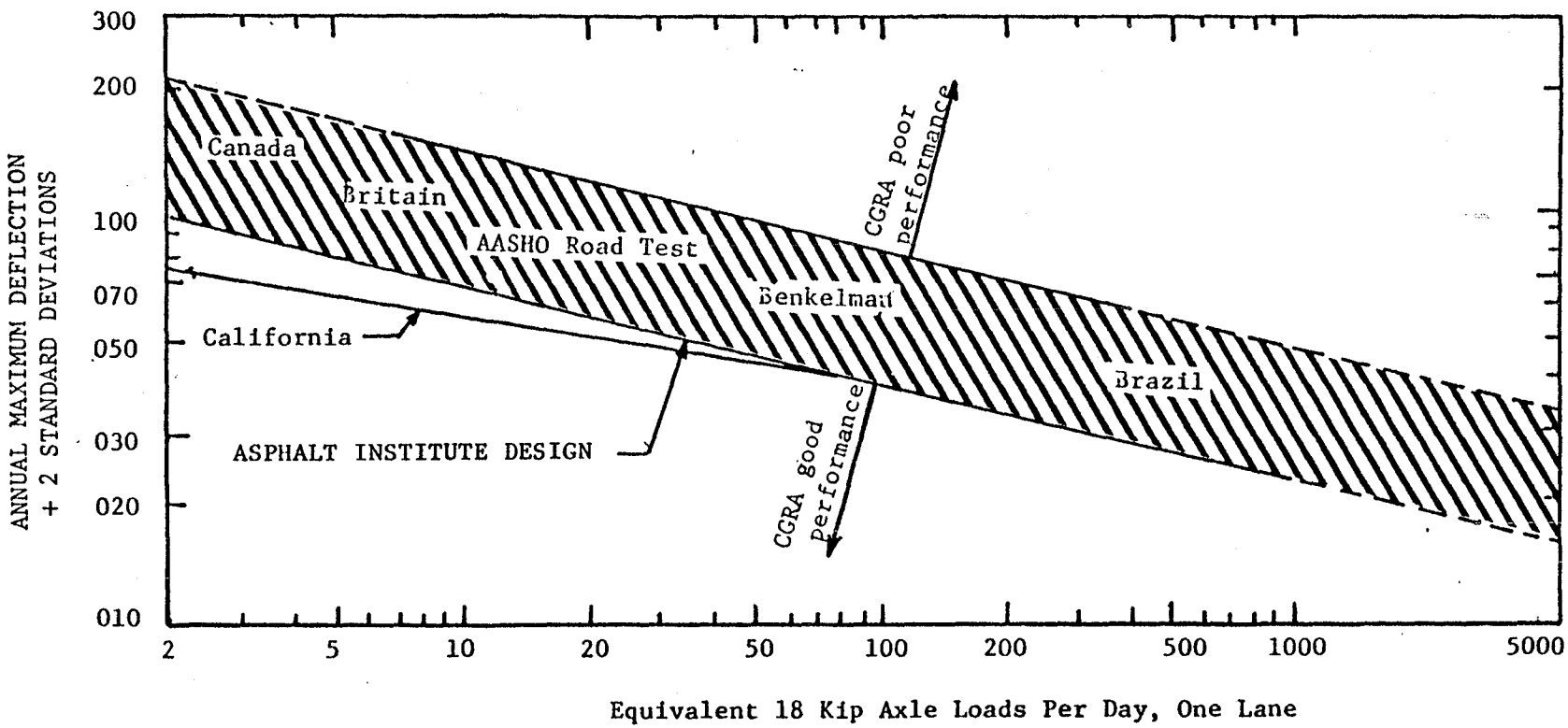


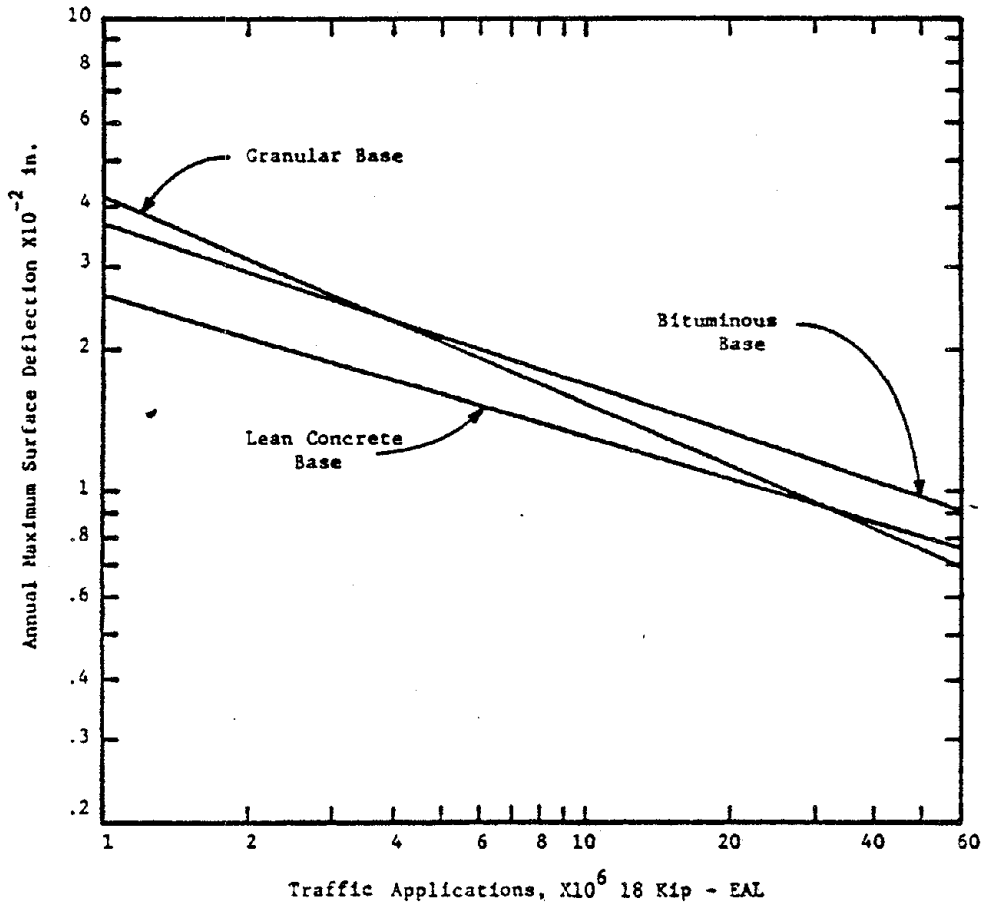
Figure 6.18. Maximum Benkelman Beam rebound ($\bar{x}$ + 2σ) versus cumulative equivalent 80 kN (Ref. 118).



1 in = 2.54 cm

18 kips = 80 kN

Figure 6.19. Deflection criteria check based on past performance (Ref. 87).



1 in = 2.54 cm
18 kips = 80 kN

Figure 6.20. Allowable maximum surface deflections used to develop the structural cross-section for zero-maintenance pavements (Ref. 120).

CHAPTER 7. PREMIUM PAVEMENT DESIGN PROCEDURE

The basic steps of the design procedure for premium pavements developed are shown in Figure 7.1 in flow chart form. Fatigue, low-temperature and reflection cracking, rutting, and roughness are the significant distress factor which were considered in developing a premium pavement structure. The models discussed in Chapter 4 were used to develop the thickness and specification requirements for each of the materials recommended in Chapter 3.

The procedure consists of a series of steps using results from the various models as well as interactions with the engineer. The final products are thicknesses and materials combinations that will minimize the occurrence of each of the distress types listed. The following sections describe the high level of inputs required for premium pavements. Using this general procedure, each step has been summarized into a complete graphical solution technique which is presented in Appendix E.

BASIC INPUTS

Inputs describing the environment and soil are used to select materials and the number of layers that are required for a basic or "skeleton" pavement cross-section. Figure 7.2 contains a flow chart showing the use of environmental and soil inputs to estimate the need for drainage and frost protection. Figure 7.2 illustrates logic patterns that can be used to determine if the soil conditions require that treatments be used to alter some of the detrimental effects of these soil conditions so that zero-maintenance performance can be provided.

To ensure premium performance, the frost susceptibility and volume change potential of the existing soil must be carefully considered. The Corps of Engineers test procedure can be used to determine if a soil is frost susceptible and to estimate the heave and strength loss during freeze and spring thaw, respectively. Classification and swelling potential tests can be used to determine if a soil is expansive and if it should be treated to reduce the volume change potential. Pressure-swell tests or other available means should be used for recommendations on the type or types of stabilizer which can be used to reduce the expansive nature of the soil. Simple techniques used by the Corps of Engineers and Federal Aviation Administration can be used to determine if the soil is expansive and/or frost susceptible. These are presented in Appendix E.

Once the existing soil has been evaluated and an appropriate treatment selected, the effect of changes in moisture on the stiffness of the treated and untreated subgrade soils should be estimated. A procedure similar that is used by the Corps of Engineers has been adopted and is presented in Volume II. This procedure can be used to estimate the stiffness of most unbound materials, including granular base materials

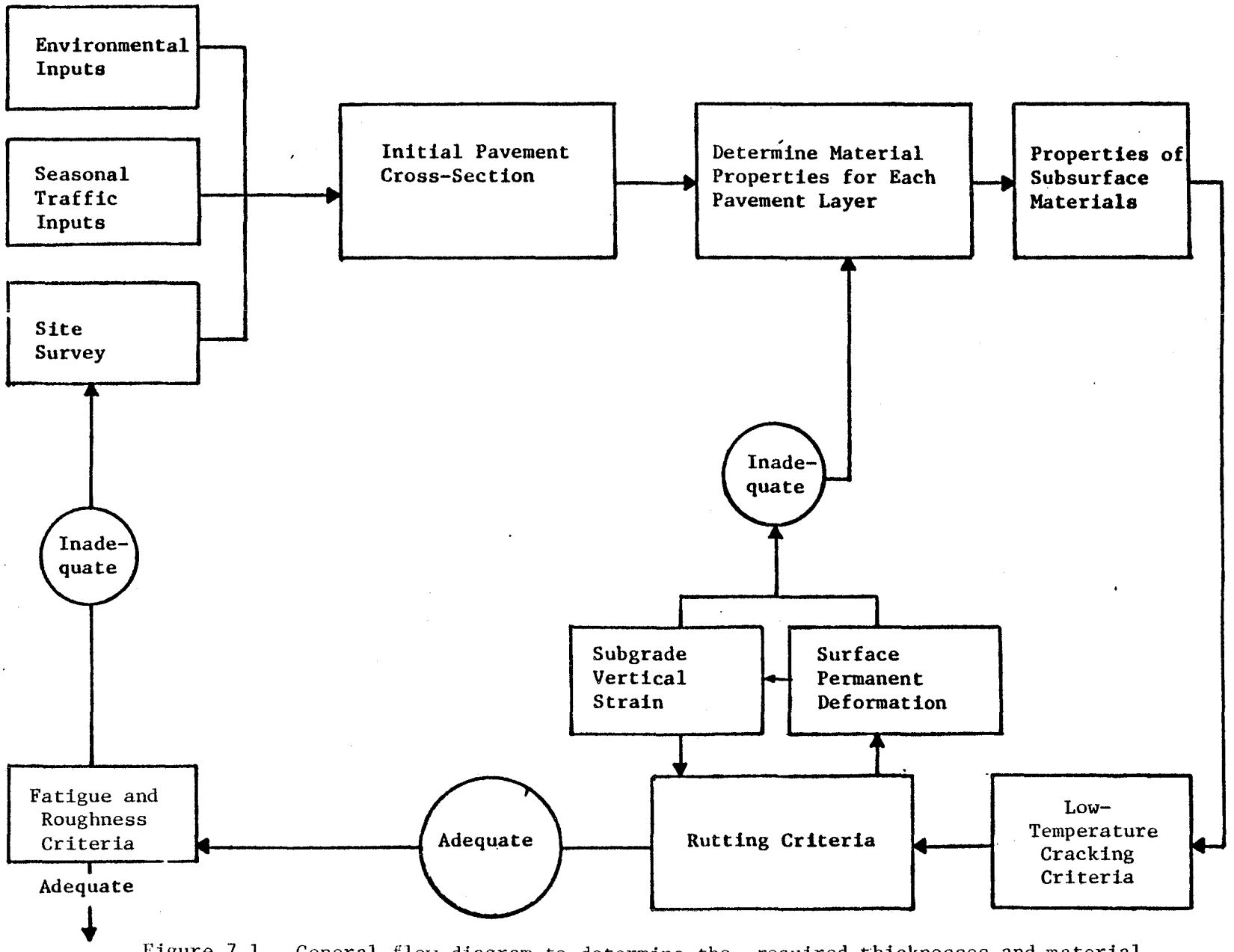


Figure 7.1. General flow diagram to determine the required thicknesses and material properties to meet the premium pavement requirements.

Figure 7.1 (continued)

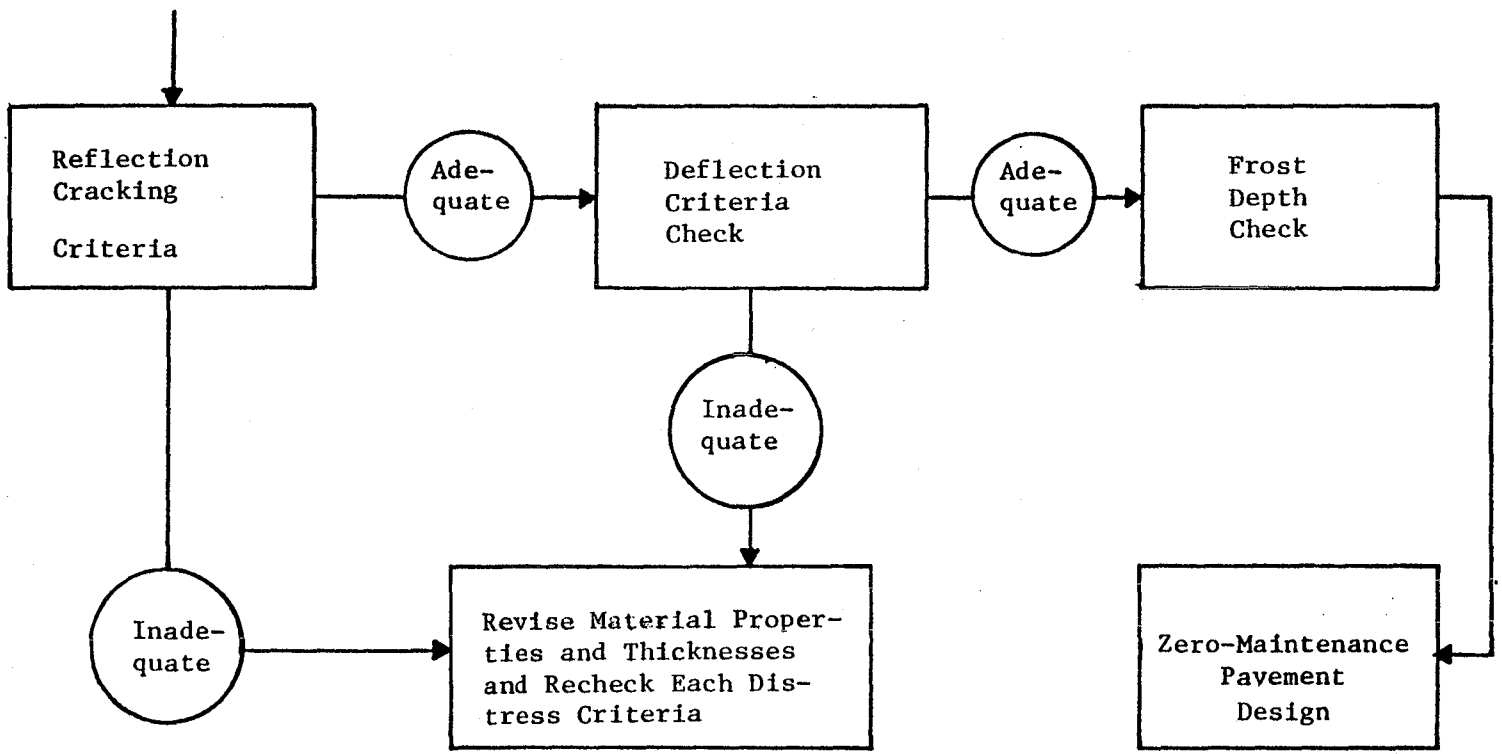


Figure 7.1. General flow diagram to determine the required thicknesses and material properties to meet the premium pavement requirements (continued).

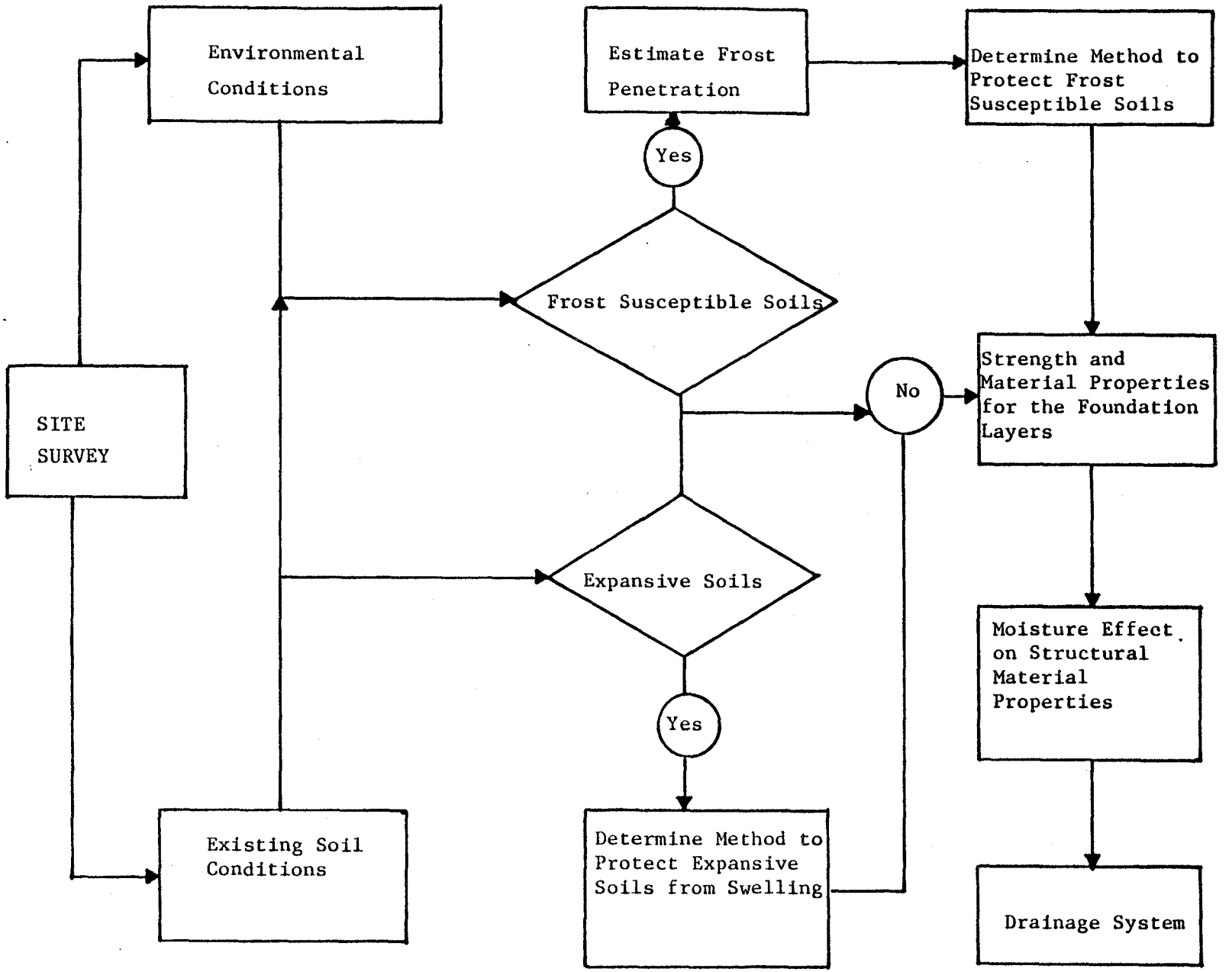


Figure 7.2. Flow diagram to determine the drainage and environmental requirements (stabilization, insulation, encapsulated material, etc.) for the existing subgrade material.

at various levels of applied stress. Tests should be performed on both the treated and untreated soils to adequately define the material characteristics, including stress-sensitivity, and to evaluate the effects of the treatment on moisture and frost susceptibility. In addition to soil information, required environmental inputs include temperature, rainfall, freezing indexes and other factors discussed in Chapters 4 and 5.

Traffic should be projected over a 20 year period to determine the number of 18-kip (80 kn) single axle loads expected during the design period. Generally, the equivalence factors derived from the AASHO Road Test can be used along with the actual traffic applications for each truck classification estimated for the design period.

MATERIAL CHARACTERIZATION

Based on the initial subgrade characterizations, an initial cross-section is selected to be either a composite or flexible pavement. For each material being considered for use in the pavement cross-section, the stiffness, temperature and moisture effects, strength, and stress-sensitivity must be available for the analyses.

SPECIAL DESIGN CONSIDERATIONS

Stringent criteria must be met for a pavement structure to serve as a premium pavement for zero-maintenance. Not only must the strength and thicknesses of the structural layer be adequate, but all components must be carefully designed in order to satisfy the criteria. Special considerations should also be directed toward the adequacy of: 1) shoulders, 2) reinforcement in composite pavements, 3) joints in composite pavements, 4) sub-surface and surface drainage, and 5) anchorage of concrete layers. Of particular importance to the thickness design is drainage and reinforcement. Reinforcement will be considered in a later section.

Some experience indicates that inadequate drainage is probably responsible for as much pavement distress as inadequate structural or material design. Consequently, effective drainage is essential to good premium pavement performance, and is a requirement of this design procedure. For drainage consideration, the discharge of surface runoff and the control of sub-surface water must both be considered. Surface runoff is readily controlled by the pavements crown or slope. However, a relative permeable base often overlies a relatively impermeable sub-base. Detailed discussions on the design and construction of drainage layers is contained in References 68-70. For this design procedure, a drainage blanket (Ref. 70) has been recommended for use. This drainage system consists of a layer extending full width of the entire pavement cross-section.

Other items which should be considered in relation to the performance of composite pavements is the thickness of asphalt concrete to

significantly reduce the curling stress in the concrete slab. Basically, the procedure outlined in Chapter 5 is used to compute the temperature differential of the concrete slab as a function of asphalt concrete thickness, as illustrated in Figure 7.3. For the majority of the conditions investigated, 3 inches (7.6 cm) of asphalt concrete significantly reduced the temperature differential of the concrete slab and the resulting curling stresses.

LOW-TEMPERATURE CRACKING

The low-temperature cracking criteria is initially used to define the material properties for the asphalt concrete or continuously reinforced concrete. Both programs TC-1 and CRCP-2 are used to define the materials necessary to achieve proper performance. Regression equations 6.6 can be used to design for the required percent steel for the CRC rigid base layers instead of the CRCP-2 program. To determine the allowable asphalt cement grade, information from Chapters 4-6 have been used to develop a graphical solution which is presented in Appendix C. This includes Figures E.3, E.4, and E.10. Once these material properties have been defined, the thicknesses to resist fatigue cracking and rutting can be determined. Programs TC-1 and CRCP-2 have been briefly discussed in Chapters 4 and 5 and thoroughly reviewed in Reference 4.

FATIGUE CRACKING

This step determines the thickness of the surface layers required to limit the amount of fatigue cracking. Using the fatigue cracking criteria presented in Chapter 6, the VESYS III computer program can be used to determine the thickness of asphalt concrete or low-modulus layers. Using the material properties and simplifying assumptions presented in Chapters 3 and 4, and the criteria in Chapter 6, Figures E.19-E.22, E.27-E.28, and E.32 presented in Appendix E were developed to graphically determine the required asphalt concrete thickness for flexible pavements and low-modulus concrete thickness for composite pavements. In determining the asphalt concrete thickness for granular base cross-sections an additional step is applied. Since most unbound granular materials are stress dependent, and since the stiffness of the material depends on the stiffness of the material of the layer below it, the Corps of Engineers chart for estimating the modulus is recommended (Figure 3.13). Given these thicknesses and moduli values the VESYS program is used to predict the required asphalt concrete thickness. Once the thickness has been determined the stresses are analyzed in the granular base to determine if a correct modulus value was selected based on the laboratory test results as indicated in Chapter 3. Concurrently, rutting and roughness are also used to ensure that these criteria are satisfied.

RUTTING AND ROUGHNESS

Once the thicknesses have been determined to resist fatigue cracking, rutting and roughness as predicted by the VESYS III program, to

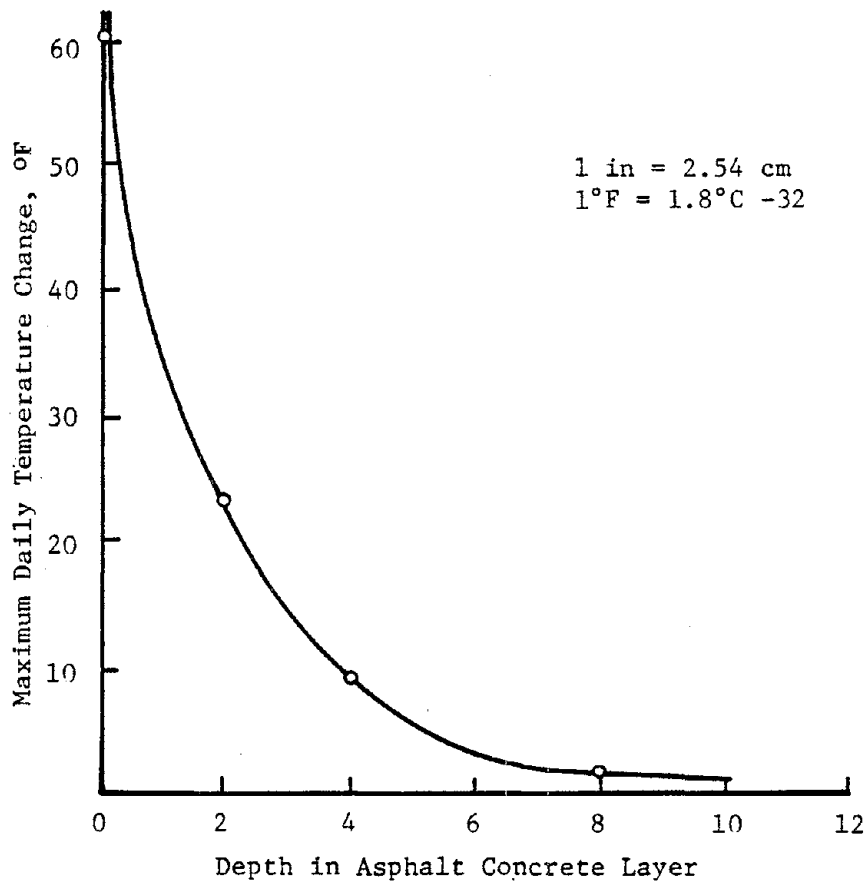


Figure 7.3. Maximum daily temperature change in asphalt concrete surface layer as a function of depth.

ensure that these criteria are met. If the rut depth or roughness exceed allowable values, then other thicknesses are selected and VESYS III is rerun until all three criteria are simultaneously satisfied.

REFLECTION CRACKING

The computer program "RFLCR-1" is used to determine if reflection cracking will occur in composite pavements. Based on numerous studies, it was determined that an asphalt crack relief layer would provide great potential for minimizing the distress manifestation of reflection cracking.

DEFLECTION AND SUBGRADE COMPRESSIVE STRAIN

Using the criteria presented in Chapter 6, VESYS III is used to compute surface deflection and subgrade compressive strains for the particular pavement cross-section that satisfies these criteria. If the pavement structure does not meet the criteria established, then a stabilized base material is used to reduce the two pavement response variables. This has been graphically summarized in Appendix E (Figures E.23-E.26, E.29-E.31, and E.38-E.39).

FROST PENETRATION

Once the pavement structural thicknesses have been determined, the environmental factor, frost action, must be considered. Using the procedure described in Chapter 5 the frost penetration can be predicted, if a frost susceptible material exists. If the frost penetrates a frost susceptible material then additional thicknesses or an insulating type of material must be used to prevent frost heave or spring-thaw strength reductions from occurring. Guidelines for determining the direction or treatment method to be used have been supplied in Appendix C. Also, based on the information supplied in Chapters 3 and 5, Figures E.40-E.42 were developed to simplify this step if the variables satisfy the users conditions.

SUMMARY

The procedure outlined in this Chapter is an interative one which has many user and model interactions and can be complicated. In order for the procedure to accurately define or design premium pavements, the user must have adequate information concerning the materials in his specific environment. Without this information, no model or procedure can be expected to reliably design for proper performance. It is also noted, that if the user selects to use Appendix E instead of each particular computer program, he must ensure that the simplifying assumptions, described in Chapter 6 and as discussed by Rauhut in Reference 4, are applicable to his particular conditions and materials.

CHAPTER 8. ECONOMIC ANALYSIS AND USER COSTS

BACKGROUND

This report is concerned with the development of materials, layer thicknesses or construction procedures which will produce premium pavements, i.e., pavements which require minimum routine maintenance, and no structural rehabilitation for an extended period of time. Such pavements will have higher initial costs than currently experienced under present design systems.

Assuming that premium pavements which require virtually no structural maintenance can be developed, the next question must inevitably be; what is the initial cost, and can we afford such costs? If the cost increase is only 10 percent more than conventional designs for the 25 year design period, justification could be based on savings in routine maintenance costs. However, if the cost is double that of conventional designs, reduced maintenance can probably not justify premium pavements, especially if the maintenance costs are discounted to present worth at an interest rate above inflation. If increased initial costs cannot be justified by reduced maintenance costs, they must be justified by benefits to the highway user.

Butler and Boos (Ref. 132[1]) presented four factors that produce economic impacts for the highway user when a road is closed for roadway maintenance:

Vehicle operating costs
Time cost to the user
Accident costs
Pollution

These investigators conducted a very comprehensive evaluation of each of the factors, developed the basic relationships and models and included them in a computer program that performs an Economic Analysis of Roadway Occupancy for Maintenance and Rehabilitation (EAROMAR). In this study Butler and Boos draw on a number of sources for basic data and in so far as that data is reliable the program seems to be very useful.

Highways in many of the urban and suburban area throughout the country are subjected to unusually heavy traffic volumes and traffic loadings. Often these pavements are distressed early in their life with the resulting deterioration which creates a need for maintenance. This maintenance early in the life of the pavement disrupts traffic both by creating congestion points on the highway and by exposing the motorist

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[1]Butler, B.C. and J.V. Boos, "Economic Analysis of Roadway Occupancy for Freeway Pavement Maintenance and Rehabilitation", Report No. FHWA-RD-76-13, 1976.

to the maintenance crews who occupy the roads to perform the maintenance. This maintenance is costly for a number of reasons.

To minimize the exposure of the maintenance crews to heavy traffic volumes, emergency type repairs are frequently done in a hasty manner with resulting poor quality work that frequently must be repatched. To occupy the road long enough to adequately perform the work, crews must either operate during the four or five hours between traffic peak volumes or schedule night-time operations. This gain in occupancy time, however, is offset by the increased danger to the maintenance personnel and the motoring public. Plus, an increased cost is associated with setting up to perform maintenance at night which requires extensive traffic control and lighting plus premium pay to the workers. Because of the unusually costly requirements for traffic control that must be borne anytime the road is occupied, maintenance work is frequently deferred. This deferred maintenance produces two problems. First, the deterioration process continues or may even accelerate with the result that a complete structural failure may occur with the need for future reconstruction. The second problem is produced by a lower level of service provided to the user with a corresponding increase in vehicle operating cost on the facility. These costs are made up of the running costs of the vehicle, accident costs associated with increased hazards, and the loss of time. All of the components are affected when maintenance crews occupy a roadway to perform structural maintenance.

When discussing justification of the additional cost for premium pavements, all who have input in such a decision must be considered. The decision must inevitably start at the highest administrative level of the Highway Department or DOT. However, it does not stop there. The legislature, the taxpayer, the industrial user, truckers, automobile associations will all likely respond to discussions of the need for and cost of premium pavements, especially if increases in gas tax, approving bond issues or transferring monies from the general fund are required.

The chief highway administrator could probably be convinced of the need for premium pavements if the initial increase in costs could be offset by reduced maintenance. If such is not the case, funds must be shifted from expenditures in less visable portions of the network or services must be reduced in some way. The justification for adjusting the budget will likely be found by offsetting costs for the user.

An alternative funding route lies with a direct approach to the legislature based on the potential cost savings to the highway user. Thus, the identification and quantification of user costs, as described by Butler and Boos, would play a major role in justifying premium pavements for heavily trafficked urban freeways.

User cost benefits are the opposite of excess user costs often included in various pavement management systems. These benefits are produced by reductions of costs associated with; (1) reduced traffic delays for routine or major maintenance, (2) speed slowdowns related to

pavement riding quality, and (3) accidents due to maintenance associated lane closures.

Until recently user costs have been considered in route selections for most major highways in accordance with criteria and information provided in the AASHO Red Book. However, there is no evidence that such criteria has ever been used for the selection of pavement types or designs. Intuitively some highway departments have shown a preference for portland cement concrete pavements for urban freeways based on the assumption that reduction in maintenance frequently results. However, so-called premium pavements will likely be considerably more expensive and require stronger economic justification.

REVIEW OF THE STATE OF THE ART

Winfrey, Hudson, et al (1970), Lytton and McFarland (1975), Smith (1974), Darter and Barenberg (1976) and many others have made significant contributions to the technology of economic evaluations considering user costs. One problem with application of the results of these studies arises because the investigator had to make a series of key assumptions. The layman is not particularly impressed by the assumption of a Poisson distribution for arriving traffic into a maintenance area, or the complicated calculations required to predict pavement performance. To substantiate these estimates, real examples with real data are required. The participation of the average driver, the truck operators (large and small), the insurance agencies and air pollution experts must be acquired. The problem must be reduced from an academic exercise to a realistic, straight forward documented example.

Smith (Ref. 133[1]) estimates that the average time value for truck is $9.30 per hour and for each person in a passenger car is $3.20 per person hour. By making adjustments for the number of cars (10 percent) and passengers (1.4 per vehicle) the time value for each car could be $4.50 per hour. Do these savings in time have an impact on the economy? Just how real are these user cost savings, decisions makers from Governor to taxpayer ask? An appropriate question for the user might be; "Would you pay a $9.30 toll to avoid a delay of 1 hour produced by a lane closure for major maintenance?" More realistically, "Would you be willing to increase the gasoline tax to avoid a 10 minute, 20 minute, 30 minute or 1 hour delay at 10 year intervals?" Toll roads survive because users pay a toll to save time and make driving experience more pleasant; but very few toll roads have been built since 1970.

ECONOMIC ANALYSIS FOR RIGID PAVEMENTS

In Reference 1 Darter does not attempt to economically justify zero-maintenance pavements, but rather uses excess user costs produced

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[1]Smith, W., "A Flexible Pavement Maintenance Management System", Ph.D. Dissertation, University of California, Berkeley, 1974.

by increasing roughness to identify a lower limit for terminal serviceability. The excess user costs are thus defined as the additional operating and time cost produced by traveling on a road with a serviceability index lower than the initial value. In calculating these excess user costs, Darter ignored speed change and excess accident costs due to increasing roughness but considered the time cost (varying with speed) and the vehicle operating costs (varying with speed and PSI). These operating costs include fuel, oil, tire wear, repair and maintenance, and depreciation. Consumption rates are based on work by Winfrey and Claffey; roughness correction factors are based on a report by Kher (Ref. 134[1]); and time cost are based on average wage rates.

In References 2 and 3 Darter concludes that zero-maintenance pavements must be justified on a life cycle economic analysis and uses EAROMAR to compute maintenance and rehabilitation costs for a conventional 20 year design. Two pavements are designed (conventional and zero-maintenance) with the same inputs (ie., traffic, environment, materials, etc.); initial construction costs estimated; and maintenance costs (including user costs) based on EAROMAR prediction. Result of the economic analysis are shown in Table 8.1.

It must be noted, however, that nowhere in the actual cost analysis is roughness considered, i.e., EAROMAR estimates only user costs resulting from roadway maintenance activities.

ROUGHNESS FACTORS IN USER COSTS

Smith (Ref. 133) considered three user cost components resulting from major maintenance, routine maintenance and pavement roughness. Predictions of user costs from major and routine maintenance were based on Reference 134 (with slight modifications) and predictions of user costs from roughness were based on a modification of McFarland's work (Ref. 135[2]). Smith assumes speed-serviceability relationship based on a combination of a small amount of field data and subjective estimates. Vehicle operating costs are based on Winfrey and Claffey data updated for inflation and roughness unit costs developed by McFarland. Delay costs are based on wage rates and the percentage of the vehicles being used for business purposes. Excess accident cost and discomfort costs are excluded, primarily as a result of the lack of the ability to accurately estimate them at the present time.

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[1]Kher, R.K. and W.A. Phang, "Road User Cost as a Function of Pavement Roughness", a paper offered at the TRB Annual Meeting, January 1976.

[2]McFarland, W.R., "Benefit Analysis for Pavement Design Systems", Research Report No. 123-13, Highway Design Division Research Section, Texas Highway Department, Texas Transportation Institute-Texas A&M University, Center for Highway Research-The University of Texas at Austin, April 1972.

TABLE 8.1 PRESENT WORTH COST IN DOLLARS PER MILE FOR TWO DESIGNS (REF. 2,3)

Design Type	Initial Construction	Maintenance and User	Total
Conventional Design	1,050,522	350,000	1,400,522
Zero-Maintenance Design	1,180,291	0	1,180,291
	Savings		220,231

Darter makes a comparison between zero-maintenance and conventional pavements, but fails to consider roughness as effecting the user costs. Without considering the excess user costs generated by roughness, zero-maintenance pavements will be difficult to justify. If roughness has no significant effect on excess user cost, alternatives to zero-maintenance pavement should be considered. Foster suggests the use of extra wide shoulders that might be more economical than an expensive premium pavement. During maintenance, these shoulders could be used as traffic lanes along with permanent crossovers so that traffic can be diverted and carried with very little delay while a section is being maintained.

USER COST PREDICTION MODELS AND WARRANTS FOR PREMIUM PAVEMENTS

Although initial and maintenance costs have typically been a significant consideration in the design and construction of pavements, the costs that these activities impose on the user has either been totally ignored or only partially accounted for. These costs are difficult to determine and some agencies are reluctant to consider them because of their magnitude.

Several models have been reviewed, evaluated, and are discussed briefly in the following sections. These models generally predict the cost to the user resulting from maintenance activities and reduced serviceability. These costs are calculated for each year of the design period, converted to either present worth or annual cost, and summed at the beginning of the design period.

Maintenance Models

The models available for calculating maintenance related user costs are EAROMAR (Ref. 132) and USERCST (Ref. 136[1]). EAROMAR was developed to predict user costs from traffic interruption associated with pavement maintenance activities including accidents resulting from speed changes. EAROMAR is currently being revised for the FHWA by Research Contract with MIT. The delay cost model, USERCST, has recently been updated for inflation and with little additional programming effort, can be made compatible with EAROMAR to account for the extra delay caused by the excess accidents.

Serviceability Model

The model presented in Reference 135 accounts for the delay, discomfort, accident and vehicle operating costs that are produced by a decrease in serviceability (PSI).

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[1]Daniel, J., "USERCST", Unpublished Report, Center for Highway Research at the University of Texas, April 1978.

McFarland combines data from two studies to develop a relationship between PSI and accident rates for rural areas of Texas. Review of the prediction from the model show that the excess accident costs are not large for 4 lane or more divided rural roads. A relationship for multi-lane urban roads has not yet been developed.

DISCUSSION

Darter and Barenberg (1976) postulated that premium pavements have a flatter serviceability curve than conventional pavements do and hence, the benefits of a smoother pavement for longer periods of time would accrue to users of premium pavements. These investigators conclude that "...unlike vehicle operating costs, travel time cost is not a function of roughness; i.e., at a uniform average speed, travel time costs will not change at varying levels of roughness". These investigators indicate that vehicle operating costs will be affected by roughness with the result that a rapid increase in user costs occurs as the serviceability level decreases. Thus, if the serviceability level is high by design, rather than by maintenance, substantial user cost benefits can occur. It might be argued that high levels of serviceability could also be obtained by frequent thin overlays which would require lower agency costs, but these frequent overlays would produce large user delay costs due to congestion during construction.

Smith (1974) concludes that for conventional pavements "...the level of maintenance in an optimum maintenance strategy for each pavement category increased as the value placed on excess user costs increased". This implies that if excess costs are sufficiently high, additional maintenance can be justified or conversely more could be spent initially, by constructing premium pavements, with an attendant decrease in excess user costs.

Finn, et. al., (Ref. 137[1]), in developing a pavement management system framework for the state of Washington, concluded that excess user costs associated with major maintenance would completely dominate the optimum maintenance strategy producing recommendations for high levels of maintenance at infrequent intervals. Because of its impact and the uncertainty of the basic speed profiles, excess user costs were deleted from the Washington system until more documentation on their real worth could be obtained.

The Butler and Boos report (Ref. 132) provides a basic program that can be used to evaluate the economic impact of roadway closures. However, much of the current input is obtained through programmed default

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[1]Finn, F.N., R. Kulkarni, and K. Nair, "Pavement Management System-Feasibility Study", prepared for Washington Highway Commission, Olympia, Washington, 1974.

assumptions. To be useful for design, these assumptions must be translated into reliable data.

Do user costs provide sufficient justification for premium pavements? The inference is that user costs can be used to identify those situations for which premium pavements should be considered; however, considerable effort must be expended to generate more complete field data. Engineers in the past have often justified their decisions using statements like, "...it has long been our experience", or "...in my opinion". Such decisions were, in all probability, pretty reliable; however, they are no longer acceptable when several agencies are competing for transportation funds. If user costs can indeed be used to justify premium pavements, we must prove it.

CHAPTER 9. SUMMARY AND CONCLUSION

The materials presented in this report identifies and examines the factors and requirements for establishing zero-maintenance pavements. This report examines the primary distress manifestations and corresponding distress mechanisms which govern the behavior of flexible and composite pavements. Materials and material properties were reviewed in relation to providing the maximum performance required by the premium pavement definition.

From this information and experience candidate premium pavement cross-sections were established which have strong potential to function for twenty or more years with minimum maintenance. Specific models were selected which, in the opinion of the research team, have the highest potential reliability to predict actual performance based on current pavement technology. Using these models and past performance information, design criteria for each distress manifestation were established for designing flexible and composite structures. Subsequently, a design procedure was developed and organized such that both environmental and traffic induced damage would be considered in design.

In accomplishing this, some extrapolation beyond the normal range of experience was required. Because of this extrapolation, there are certain aspects of the design procedure which are not as reliable as others. Other areas must remain under the direct influence of the engineer and no matter how well proven the concepts have been established, the engineer must provide the correct input and control, or the pavement's performance will be less than that desired. For example, quality control is probably the most important item of concern to the engineer in zero-maintenance pavements. Any pavement not properly constructed will fail to provide the high level of performance required for the minimum 20 year design. Proper inspection and control to correct material or construction deficiencies must be accomplished in the field.

Other areas factors not well established in terms of performance and of critical concern to the design engineer include asphalt concrete and low modulus portland cement concrete durability, environmental considerations and pavement subsurface drainage. In this study, every attempt has been made to investigate available performance data and to synthesize a reasonably and coherent design procedure, which considers the effects of changing environmental and material conditions. In conclusion, we recommend that the concepts and criteria presented herein and implemented in the designs presented in Volume II (Ref. 5) be reviewed and observed in future implementation studies by practical and experienced design engineers to increase the reliability of this procedure and to provide additional performance data for future revisions.

APPENDIX A. DEFINITION OF TERMS

The definitions of terms listed below are consistent with the definitions used by Rauhut, et. al., in Reference 4 to ensure compatibility and interpretation between projects.

1. A Pavement Structure is an organized combination of materials constructed in layers over a natural soil.

2. Material Properties are those definitive descriptive measures of the quality of the material (Ref. 138[1]).

3. Response is the reaction of a pavement structure to load and environment.

4. Primary Responses are those responses which, when carried past some limiting value, initiate distress.

5. Other Responses are those responses which do not contribute directly to distress.

6. Distress is a condition of a pavement structure which reduces serviceability or leads to a reduction of serviceability.

7. Distress Manifestations are the visible consequences of various distress mechanisms, which usually lead to a reduction of serviceability (Ref. 19).

8. A Mechanism is the physical or chemical process responsible for an action, reaction, or other natural phenomenon.

9. A Response Mechanism is the physical or chemical process responsible for the response of a pavement structure.

10. A Distress Mechanism is the physical or chemical process involved in or responsible for distress in pavements.

11. Serviceability is the ability of a specific section of pavement to serve traffic in its existing condition (Ref. 139[2]).

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[1]Murphy, G., Properties of Engineering Materials, International Textbook Company, 1957.

[2]Hudson, W.R. and F.N. Finn, "A General Framework for Pavement Rehabilitation", Report No. FHWA-RD-74-60, Federal Highway Administration, June 1974.

12. Riding Comfort Index is a numerical estimate of serviceability based on pavement roughness.

13. Serviceability History is the time-based representation of the serviceability of the pavement. Serviceability history is sometimes termed performance and is often specified with a performance index as suggested by Carey and Irick (Ref. 142[1]). As such, it is a direct function of the present serviceability history of the pavement (Ref. 19).

14. Performance is a measure of the accumulated service provided by a facility; i.e., the degree to which a pavement fulfills its purpose.

15. Functional Failure is a level of serviceability below which the service provided by the pavement is unsatisfactory to users.

16. Structural Failure is a fracture or distortion which may or may not cause an immediate reduction of serviceability, but which will lead to a future loss of serviceability.

17. Fracture is the state of a pavement material being broken.

18. Distortion is a permanent change in the shape of the pavement or pavement component.

19. Disintegration is the state of being decomposed or abraded into constitutive elements (Ref. 19).

20. Bleeding is the condition of free bitumen on the surface of the pavement due to excessive bitumen and/or insufficient void space.

21. Reflection Cracks are cracks occurring in the surface course of a pavement that coincide with and are caused by the relative movement of cracks or joints in underlying layers.

22. Low-Temperature Cracks are cracks (generally transverse) caused when tensile stresses induced by frictional resistance of the underlying layer to thermal contraction of the surface layer exceeds the tensile strength of the surface material.

23. Ravelling is the progressive disintegration of an asphalt concrete layer from the surface downward by the dislodgement of aggregate particles. This may be caused by insufficient amount of binder in the mix, hardening of the asphalt binder, wet or dirty aggregate or aggregate with smooth surface texture.

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[1]Winfrey, R., Economic Analysis of Highways, International Textbook Company, Scranton, Penn., 1969.

24. Ruts are longitudinal depressions that form in the wheel paths of flexible or composite pavements, resulting from compaction or lateral migration of one or more of the pavement layer materials under the action of traffic and environment.

25. Reduced Skid Resistance is the reduction of frictional resistance between tires and a pavement surface. This reduction is generally due to abrasive wear of aggregates by traffic.

26. Shrinkage Cracks are generally transverse cracks caused when tensile stresses induced by frictional resistance of the underlying layer to drying contraction of the surface layer exceeds the tensile strength of the surface material. These cracks generally occur in portland cement concrete and other cement treated materials.

27. Spalling is cracking, breaking, or chipping of a rigid pavement along joints, edges, or cracks in which small portions of the slab are dislodged.

28. Faulting is a difference in the elevation of two adjacent rigid slabs at the joint or crack interface due to consolidation or swelling or underlying material, inadequate load transfer, or pumping.

29. "D" Cracking is a series of fine, crescent-shaped hairline cracks in a rigid slab surface, usually paralleling a joint or major crack.

30. Steel Rupture is the occurrence of a tensile fracture failure in the reinforcing steel when excessive stress is transferred upon fracture of adjacent concrete.

31. Polished Aggregates are surface aggregate particles having smooth, rounded surfaces with fine microtexture, either as original condition or after abrasive wear by traffic.

32. Punchouts are blocks of rigid pavement that are cracked around their periphery and displaced downward relative to the rest of the slab. Punchouts usually occur between closely-spaced transverse cracks that are subsequently connected by longitudinal cracks.

33. Fatigue Cracks are cracks in a pavement layer caused by the combination of repetitive strains and apparent reduction of tensile strength due to fatiguing of the layer material. The repetitive strains causing fatigue are usually caused by passing wheel loads, but may include repetitive thermally-induced or other strains.

34. Pavement Structure Models are mathematical formulations that use material property and load inputs to predict pavement responses.

35. Distress Models are formulations that use the calculated responses from pavement structure models along with other parametric input to predict pavement distress.

36. Performance Models are mathematical formulations that use the calculated distresses to predict pavement performance in terms of serviceability with time and/or service life.

37. A Parameter is a physical property whose value affects the behavior of a pavement structure.

38. Model Inputs are values of parameters affecting calculated predictions of pavement responses, distress, or performance.

39. Model Outputs are calculated predictions of pavement response, distress, or performance.

40. Period of Weakening is an interval of the years which starts at the beginning of the frost melting period and ends when the subgrade strength has returned to normal summer values.

41. Frost-Melting is an interval of the year during which the ice and foundation materials is returning to a liquid state. It ends when all the ice in the ground has melted or when freezing is resumed. Although in the generalized case there is visualized only one frost melting period, beginning during the general rise of air temperatures in the spring, or more significant frost melting may occur during a winter season.

42. Frost Heave is the rising of a surface due to formation of ice in the underlying soil.

43. Ice Segregation is the growth of ice as distinct lenses, layers, veins, and masses in soils, commonly, but not always, oriented normal to the direction of heat loss.

44. Frost Action is a general term for freezing and thawing of moisture in materials and the resultant effects on these materials and on structures for which they are a part or with which they are in contact.

APPENDIX B. DISTRESS MANIFESTATIONS AND RELATED MATERIAL PROPERTIES FOR EACH PAVEMENT TYPE

APPENDIX B. MATERIAL PROPERTIES AND FACTORS (REF. 4)

Table B.1 Factors Affecting Material Properties That Affect Distress in Flexible Pavements

	Flexible Pavements	Fatigue Cracking Engineering Properties									
		Stiffness						Fatigue Constants			
	Fracture / Factors Affecting Properties	Asphalt Concrete Surface	Asphalt-Treated Material	Cement-Treated Material	Lime-Treated Material	Untreated Granular Material	Subgrade Soil	Asphalt Concrete Surface	Asphalt-Treated Material	Cement-Treated Material	Lime-Treated Material
Environ.	Temperature										
	Moisture Content	x	x	x	x	x	x	x	x	x	x
	Solar Radiation	x	x	x	x	x	x	x	x	x	x
	Time (Age)										
Material	Asphalt Type	x	x					x	x		
	% Asphalt*	x	x					x	x		
	Cement Type			x						x	
	% Cement*			x						x	
	Lime Type				x						x
	% Lime*				x						x
	Aggregate Type#	x	x	x		x		x	x	x	
	Aggregate Gradation#	x	x	x		x		x	x	x	
	Soil Type#			x	x		x			x	x
	Type of Clay Mineral				x						x
	% Clay or Pozzolanic				x						x
	Soil Gradation#			x	x		x			x	x
Construction	Mixing Temperature	x	x					x	x		
	Mixing, % Water			x	x					x	x
	Extent of Mixing			x	x					x	x
	Time of Mixing			x	x					x	x
	Compaction Type	x	x	x	x	x	x	x	x	x	x
	Degree of Compaction	x	x	x	x	x	x	x	x	x	x
	Compaction Temperature	x	x					x	x		
	Compaction, % Water			x	x	x	x			x	x
	Curing, % Water			x	x					x	x
	Curing Temperature			x	x					x	x
	Length of Curing			x	x					x	x
Traffic	Magnitude of Loads	x	x	x	x	x	x	—	—	—	—
	Number of Loads	x	x	x	x	x	x	—	—	—	—
		x									
		x	x		x		x				

Table B.1 Continued

Flexible Pavements		Low Temperature Cracking Engineering Properties									
		Coefficient of Thermal Expansion				Creep Stiffness		Tensile Strength			
Fracture	Factors Affecting Properties	Asphalt Concrete Surface	Asphalt-Treated Material	Cement-Treated Material		Asphalt Concrete Surface	Asphalt-Treated Material	Asphalt Concrete Surface	Asphalt-Treated Material		
Environ.	Temperature										
	Moisture Content					x	x	x	x		
	Solar Radiation	x	x	x		x	x	x	x		
	Time (Age)										
Material	Asphalt Type	x	x			x	x	x	x		
	% Asphalt*	x	x			x	x	x	x		
	Cement Type										
	% Cement*			x							
	Lime Type										
	% Lime*										
	Aggregate Type#	x	x	x		x	x	x	x		
	Aggregate Gradation#					x	x	x	x		
	Soil Type#			x							
	Type of Clay Mineral			x							
	% Clay			x							
	Soil Gradation#										
Construction	Mixing Temperature					x	x	x	x		
	Mixing, % Water										
	Extent of Mixing										
	Time of Mixing										
	Compaction Type					x	x	x	x		
	Degree of Compaction					x	x	x	x		
	Compaction Temperature					x	x	x	x		
	Compaction, % Water										
	Curing, % Water										
	Curing Temperature										
	Length of Curing										
Traffic	Magnitude of Loads					x	x	x	x		
	Number of Loads										
	Load Duration					x		x			
						x	x	x	x		
						x	x	x	x		

Table B.1 Continued

	Flexible Pavements	Slippage Cracking Engineering Properties									
		Tensile Strength		Ductility							
	Fracture	Asphalt Concrete Surface			Asphalt Concrete Surface						
	Factors Affecting Properties										
Environ.	Temperature	x			x						
	Moisture Content	x									
Material	Asphalt Type	x			x						
	% Asphalt*	x			x						
	Cement Type										
	% Cement*										
	Lime Type										
	% Lime*										
	Aggregate Type#	x			x						
	Aggregate Gradation#	x			x						
	Soil Type#										
	Type of Clay Mineral										
	% Clay or Pozzolanic										
	Soil Gradation#										
Construction	Mixing Temperature	x			x						
	Mixing, % Water										
	Extent of Mixing										
	Time of Mixing										
	Compaction Type	x									
	Degree of Compaction	x									
	Compaction Temperature	x									
	Compaction, % Water										
	Curing, % Water										
	Curing Temperature										
	Length of Curing										
Traffic	Magnitude of Loads	x									
	Number of Loads	x									
	Solar Radiation	x									
	Time (Age)	x									

Table B.1 Continued

Flexible Pavements		Permanent Deformation (rutting) Engineering Properties									
		Stiffness						Creep Comp.		Alpha & Gnu	
	Distortion / Factors Affecting Properties	Asphalt Concrete Surface	Asphalt-Treated Material	Cement-Treated Material	Lime-Treated Material	Untreated Granular Material	Subgrade Soil	Asphalt Concrete Surface	Asphalt-Treated Material	Asphalt Concrete Surface	Asphalt-Treated Materials
Environ.	Temperature	x	x	x	x	x		x	x	x	x
	Moisture Content	x	x	x	x	x		x	x		
Material	Asphalt Type	x	x					x	x	x	x
	% Asphalt*	x	x					x	x	x	x
	Cement Type			x							
	% Cement*			x							
	Lime Type				x						
	% Lime*				x						
	Aggregate Type#	x	x			x		x	x	x	x
	Aggregate Gradation#	x	x			x		x	x	x	x
	Soil Type#			x	x		x				
	Type of Clay Mineral				x						
	% Clay or Pozzolanic				x						
	Soil Gradation#			x	x		x				
Construction	Mixing Temperature	x	x					x	x	x	x
	Mixing, % Water			x	x						
	Extent of Mixing			x	x						
	Time of Mixing			x	x						
	Compaction Type	x	x	x	x	x	x	x	x	x	x
	Degree of Compaction	x	x	x	x	x	x	x	x	x	x
	Compaction Temperature	x	x					x	x	x	x
	Compaction, % Water			x	x	x	x				
	Curing, % Water			x	x						
	During Temperature			x	x						
	Length of Curing			x	x						
Traffic	Magnitude of Loads	x	x	x	x	x	x	x	x	x	x
	Number of Loads	x	x	x	x	x	x	x	x	x	x
	Solar Radiation	x						x			
	Time (Age)	x	x		x		x	x	x		
	Type of Stress	x	x							x	x

Table B.1 Continued

	Flexible Pavements	Shoving Engineering Properties									
		Permanent Deform. Characteristics				Stability		Modulus of Elasticity			
	Distortion / Factors Affecting Properties	Asphalt Concrete Surface	Asphalt-Treated Material	Untreated Granular Material	Subgrade Soil	Asphalt Concrete Surface		Asphalt Concrete Surface	Asphalt-Treated Material		
Environ.	Temperature	x	x			x		x	x		
	Moisture Content	x	x	x	x	x		x	x		
Material	Asphalt Type	x	x			x		x	x		
	% Asphalt*	x	x			x		x	x		
	Cement Type										
	% Cement*										
	Lime Type										
	% Lime*										
	Aggregate Type#	x	x	x		x		x	x		
	Aggregate Gradation#	x	x	x		x		x	x		
	Soil Type#				x						
	Type of Clay Mineral/Fines			x							
	% Clay			x							
	Soil Gradation#				x						
Construction	Mixing Temperature							x	x		
	Mixing, % Water										
	Extent of Mixing										
	Time of Mixing										
	Compaction Type	x	x	x	x	x		x	x		
	Degree of Compaction	x	x	x	x	x		x	x		
	Compaction Temperature							x	x		
	Compaction, % Water			x	x						
	Curing, % Water										
	Curing Temperature										
	Length of Curing										
Traffic	Magnitude of Loads							x	x		
	Number of Loads							x	x		
	Solar Radiation							x	x		
	Time (Age)							x	x		
	Type of Stress	x	x					x	x		

Table B.1 Continued

Flexible Pavements		Large, relative, vertical differential displacements Engineering Properties									
		Volume Change Cons. Dens.			Characteristics Swelling			Frost Susceptibility			
	Distortion / Factors Affecting Properties		Subgrade Soil			Subgrade Soil			Subgrade Soil		
Environ.	Temperature								x		
	Moisture Content		x			x			x		
Material	Asphalt Type										
	% Asphalt*										
	Cement Type										
	% Cement*										
	Lime Type										
	% Lime*										
	Aggregate Type#										
	Aggregate Gradation#										
	Soil Type#		x						x		
	Type of Clay Mineral					x					
	% Clay or Pozzolanic					x					
	Soil Gradation#		x								
Construction	Mixing Temperature										
	Mixing, % Water										
	Extent of Mixing										
	Time of Mixing										
	Compaction Type		x			x					
	Degree of Compaction		x			x					
	Compaction, Temperature		x			x					
	Compaction, % Water										
	Curing, % Water										
	Curing Temperature										
	Length of Curing										
Traffic	Magnitude of Loads										
	Number of Loads										
	Solar Radiation										
	Time (Age)										

Table B.1 Continued

	Flexible Pavements	Raveling Engineering Properties					Stripping Engineering Properties				
		Tensile Strength					Tensile Strength				
	Disintegration										
	Factors Affecting Properties			Asphalt Concrete Surface				Asphalt Concrete Surface	Asphalt-Treated Material		
Environ.	Temperature			x							
	Moisture Content							x	x		
Material	Asphalt Type			x				x	x		
	% Asphalt*			x				x	x		
	Cement Type										
	% Cement*										
	Lime Type										
	% Lime*										
	Aggregate Type#			x				x	x		
	Aggregate Gradation#										
	Aggregate Surface Coating			x							
	Type of Clay Mineral										
	% Clay or Pozzolanic										
	Soil Gradation#										
Construction	Mixing Temperature			x				x	x		
	Mixing, % Water							x	x		
	Extent of Mixing										
	Time of Mixing										
	Compaction Type										
	Degree of Compaction							x	x		
	Compaction Temperature			x				x	x		
	Compaction, % Water										
	Curing, % Water										
	Curing Temperature										
	Length of Curing										
Traffic	Magnitude of Loads (type of traffic)			x							
	Number of Loads (traffic vol)			x							
	Solar Radiation			x							
	Time (Age)			x							

Table B.2 Factors Affecting Material Properties That Affect Distress in Composite Pavements

	Composite Pavements	Fatigue Cracking													
		ENGINEERING PROPERTIES													
		Fatigue Constants					Stiffness Modulus								
	Fracture FACTORS AFFECTING PROPERTIES		Asphalt Concrete Surface	Portland Cement Concrete	Asphalt Treated Materials		Asphalt Concrete Surface	Portland Cement Concrete	Asphalt Treated Materials	Cement Treated Materials	Lime-Treated Materials	Untreated Granular Materials	Subgrade Soil		
Envir.	Temperature		x		x		x		x						
	Moisture		x	x	x		x	x	x	x	x	x	x		
	Freezing Temperature									x	x	x	x		
	Solar Radiation						x								
	Degree of Oxidation														
	Time-Temperature Relationship														
Material	Asphalt Type		x		x		x		x						
	% Asphalt		x		x		x		x						
	Cement Type			x				x		x					
	% Cement									x					
	Lime Type										x				
	% Lime										x				
	Aggregate Type		x	x	x		x	x	x						
	Aggregate Gradation		x	x	x		x	x	x						
	Aggregate Surface Coating														
	Aggregate Moisture Content														
	Soil Type									x	x	x	x		
	Soil Gradation									x	x	x	x		
	Type of Clay														
	% Clay														
	Volume of Aggregate or Paste			x				x							
	Water Cement Ratio			x				x							
Construction	Mixing Temperature		x		x		x		x						
	Mixing, % Water									x	x				
	Extent of Mixing									x	x				
	Time of Mixing									x	x				
	Compaction Type		x		x		x		x	x	x		x		
	Degree of Compaction		x		x		x		x	x	x	x	x		
	Compaction Temperature		x		x		x		x						
	Compaction, % Water									x	x	x	x		
	Curing, % Water			x				x		x	x				
	Curing Temperature			x				x		x	x				
	Length of Curing			x				x		x	x				
Traffic	Magnitude of Loads						x	x	x		x	x	x		
	Types of Stress														
	Stress-Strength Ratio			x											
	Number of Loads						x	x	x		x	x	x		
	Time (age)						x								
	Load Duration														

Table B. 2 Continued

	COMPOSITE PAVEMENTS	Shoving													
		ENGINEERING PROPERTIES													
		Permanent Deformation Characteristics						Stability			Stiffness Modulus				
	Distortion														
	FACTORS AFFECTING PROPERTIES			Asphalt Concrete Surface					Asphalt Concrete Surface			Asphalt Conc. & Asphalt Treated Materials			
Envir.	Temperature			x					x			x			
	Moisture			x					x			x			
	Freezing Temperature														
	Solar Radiation											x			
	Degree of Oxidation														
	Time-Temperature Relationship														
Material	Asphalt Type			x					x			x			
	% Asphalt			x					x			x			
	Cement Type														
	% Cement														
	Lime Type														
	% Lime														
	Aggregate Type			x					x			x			
	Aggregate Gradation			x					x			x			
	Aggregate Surface Coating														
	Aggregate Moisture Content														
	Soil Type														
	Soil Gradation														
	Type of Clay														
	% Clay														
	Volume of Aggregate or Paste														
	Water Cement Ratio														
Construction	Mixing Temperature											x			
	Mixing, % Water														
	Extent of Mixing														
	Time of Mixing														
	Compaction Type			x					x			x			
	Degree of Compaction			x					x			x			
	Compaction Temperature											x			
	Compaction, % Water														
	Curing, % Water														
	Curing Temperature														
	Length of Curing														
Traffic	Magnitude of Loads											x			
	Types of Stress			x											
	Stress-Strength Ratio														
	Number of Loads											x			
	Time (age)											x			
	Load Duration														

Table B.2 Continued

	COMPOSITE PAVEMENTS	Large, Relative Vertical Differential Displacement													
		ENGINEERING PROPERTIES													
		Volume Change Characteristics				Frost Susceptibility of the Subgrade									
	Distortion / FACTORS AFFECTING PROPERTIES		Subgrade Soil						Subgrade Soil						
Envir.	Temperature														
	Moisture		x						x						
	Freezing Temperature														
	Solar Radiation														
	Degree of Oxidation														
	Time-Temperature Relationship								x						
Material	Asphalt Type														
	% Asphalt														
	Cement Type														
	% Cement														
	Lime Type														
	% Lime														
	Aggregate Type														
	Aggregate Gradation														
	Aggregate Surface Coating														
	Aggregate Moisture Content														
	Soil Type		x						x						
	Soil Gradation		x												
	Type of Clay		x						x						
	% Clay		x						x						
	Volume of Aggregate or Paste														
	Water Cement Ratio														
Construction	Mixing Temperature														
	Mixing, % Water														
	Extent of Mixing														
	Time of Mixing														
	Compaction Type		x												
	Degree of Compaction		x												
	Compaction Temperature														
	Compaction, % Water		x												
	Curing, % Water														
	Curing Temperature														
	Length of Curing														
Traffic	Magnitude of Loads														
	Types of Stress														
	Stress-Strength Ratio														
	Number of Loads														
	Time (age)														
	Load Duration														

Table B.2 Continued

	COMPOSITE PAVEMENTS	Rutting												
		ENGINEERING PROPERTIES												
		Modulus			Creep Compliance			Permanent Deformation Characteristics-						
	Distortion / FACTORS AFFECTING PROPERTIES		Asphalt Concrete Surface				Asphalt Concrete Surface					Asphalt Concrete Surface			
Envir.	Temperature		x				x					x			
	Moisture		x				x								
	Freezing Temperature														
	Solar Radiation		x												
	Degree of Oxidation														
	Time-Temperature Relationship														
Material	Asphalt Type		x				x					x			
	% Asphalt		x				x					x			
	Cement Type														
	% Cement														
	Lime Type														
	% Lime														
	Aggregate Type		x				x					x			
	Aggregate Gradation		x				x					x			
	Aggregate Surface Coating														
	Aggregate Moisture Content														
	Soil Type														
	Soil Gradation														
	Type of Clay														
	% Clay														
	Volume of Aggregate or Paste														
	Water Cement Ratio														
Construction	Mixing Temperature		x				x					x			
	Mixing, % Water														
	Extent of Mixing														
	Time of Mixing														
	Compaction Type		x				x					x			
	Degree of Compaction		x				x					x			
	Compaction Temperature		x				x					x			
	Compaction, % Water														
	Curing, % Water														
	Curing Temperature														
	Length of Curing														
Traffic	Magnitude of Loads		x				x					x			
	Types of Stress											x			
	Stress-Strength Ratio														
	Number of Loads		x												
	Time (age)		x												
	Load Duration						x					x			

Table B.2 Continued

	COMPOSITE PAVEMENTS	Slippage Cracking						Reflection Cracking						
		ENGINEERING PROPERTIES						ENGINEERING PROPERTIES						
		Tensile Strength						Tensile Strength			Ductility (flexibility)			
	Fracture / FACTORS AFFECTING PROPERTIES		Asphalt Concrete Surface						Asphalt Concrete Surface			Asphalt Concrete Surface		
Envir.	Temperature		x						x			x		
	Moisture		x						x					
	Freezing Temperature													
	Solar Radiation		x						x					
	Degree of Oxidation													
	Time-Temperature Relationship													
Material	Asphalt Type		x						x			x		
	% Asphalt		x						x			x		
	Cement Type													
	% Cement													
	Lime Type													
	% Lime													
	Aggregate Type		x						x			x		
	Aggregate Gradation		x						x					
	Aggregate Surface Coating													
	Aggregate Moisture Content													
	Soil Type													
	Soil Gradation													
	Type of Clay													
	% Clay													
	Volume of Aggregate or Paste													
	Water Cement Ratio													
Construction	Mixing Temperature		x						x			x		
	Mixing, % Water													
	Extent of Mixing													
	Time of Mixing													
	Compaction Type		x						x					
	Degree of Compaction		x						x					
	Compaction Temperature		x						x					
	Compaction, % Water													
	Curing, % Water													
	Curing Temperature													
	Length of Curing													
Traffic	Magnitude of Loads		x						x					
	Types of Stress													
	Stress-Strength Ratio													
	Number of Loads		x						x					
	Time (age)		x						x					
	Load Duration													

Table B.2 Continued

COMPOSITE PAVEMENTS		Low Temperature Cracking												
		ENGINEERING PROPERTIES												
		Thermal Diffusivity				Coef. of Thermal Expansion				Thermal Conductivity				
	Fracture / FACTORS AFFECTING PROPERTIES		Asphalt Concrete Surface	Portland Cement Concrete			Asphalt Concrete Surface	Portland Cement Concrete				Asphalt Concrete Surface	Portland Cement Concrete	
Envir.	Temperature													
	Moisture			x			x	x					x	
	Freezing Temperature													
	Solar Radiation													
	Degree of Oxidation													
	Time-Temperature Relationship													
Material	Asphalt Type						x	x						
	% Asphalt						x							
	Cement Type			x									x	
	% Cement													
	Lime Type													
	% Lime													
	Aggregate Type			x			x	x					x	
	Aggregate Gradation													
	Aggregate Surface Coating													
	Aggregate Moisture Content													
	Soil Type													
	Soil Gradation													
	Type of Clay													
	% Clay													
	Volume of Aggregate or Paste			x				x					x	
	Water Cement Ratio			x				x					x	
Construction	Mixing Temperature													
	Mixing, % Water													
	Extent of Mixing													
	Time of Mixing													
	Compaction Type													
	Degree of Compaction													
	Compaction Temperature													
	Compaction, % Water													
	During, % Water													
	Curing Temperature													
	Length of Curing													
Traffic	Magnitude of Loads													
	Types of Stress													
	Stress-Strength Ratio													
	Number of Loads													
	Time (age)													
	Load Duration													

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Table B.2 Continued

	COMPOSITE PAVEMENTS	Low Temperature Cracking												
		ENGINEERING PROPERTIES												
		Tensile Strength			Temperature Susceptibility				Creep Stiffness		Stiffness Modulus			
	Fracture FACTORS AFFECTING PROPERTIES	Asphalt Concrete Surface	Asphalt-treated Materials			Asphalt Concrete Surface				Asphalt Concrete Surface		Asphalt Concrete Surface	Portland Cement Surface		
Envir.	Temperature	x	x							x		x			
	Moisture	x	x							x		x	x		
	Freezing Temperature														
	Solar Radiation	x	x							x		x			
	Degree of Oxidation														
	Time-Temperature Relationship														
Material	Asphalt Type	x	x			x				x		x			
	% Asphalt	x	x							x		x			
	Cement Type												x		
	% Cement														
	Lime Type														
	% Lime														
	Aggregate Type	x	x							x		x	x		
	Aggregate Gradation	x	x							x		x	x		
	Aggregate Surface Coating														
	Aggregate Moisture Content														
	Soil Type														
	Soil Gradation														
	Type of Clay														
	% Clay														
	Volume of Aggregate or Paste												x		
	Water Cement Ratio												x		
Construction	Mixing Temperature	x	x							x		x			
	Mixing, % Water														
	Extent of Mixing														
	Time of Mixing														
	Compaction Type	x	x							x		x			
	Degree of Compaction	x	x							x		x			
	Compaction Temperature	x	x							x		x			
	Compaction, % Water														
	Curing, % Water												x		
	Curing Temperature												x		
	Length of Curing												x		
Traffic	Magnitude of Loads	x	x							x		x	x		
	Types of Stress														
	Stress-Strength Ratio														
	Number of Loads	x	x									x	x		
	Time (age)	x	x							x		x			
	Load Duration									x					

Table B.2 Continued

	COMPOSITE PAVEMENTS	Raveling						Stripping							
		ENGINEERING PROPERTIES						ENGINEERING PROPERTIES							
		Tensile Strength Bonding of Asphalt						Tensile Strength Bond Of Asphalt Amount							
	Disintegration			Asphalt Concrete Surface						Asphalt Concrete Surface					
	FACTORS AFFECTING PROPERTIES														
Environ.	Temperature			x											
	Moisture									x					
	Freezing Temperature														
	Solar Radiation			x											
	Degree of Oxidation			x											
	Time-Temperature Relationship														
Material	Asphalt Type									x					
	% Asphalt			x						x					
	Cement Type														
	% Cement														
	Lime Type														
	% Lime														
	Aggregate Type			x						x					
	Aggregate Gradation														
	Aggregate Surface Coating			x											
	Aggregate Moisture Content									x					
	Soil Type														
	Soil Gradation														
	Type of Clay														
	% Clay														
	Volume of Aggregate or Paste														
	Water Cement Ratio														
Construction	Mixing Temperature														
	Mixing, % Water														
	Extent of Mixing														
	Time of Mixing														
	Compaction Type									x					
	Degree of Compaction									x					
	Compaction Temperature			x											
	Compaction, % Water														
	Curing, % Water														
	Curing Temperature														
	Length of Curing														
Traffic	Magnitude of Loads			x											
	Types of Stress														
	Stress-Strength Ratio														
	Number of Loads			x											
	Time (age)			x											
	Load Duration														

APPENDIX C

POSSIBLE MODELS TO PREDICT THE EFFECTS OF ROUGHNESS ON USER COSTS

In trying to choose a model, criteria must be developed in order to make a valid comparison. Figure C.1 shows the factors considered to be of importance in determining the excess user costs resulting from pavement roughness. The following user costs and variables should not vary with roughness:

1. Insurance
2. Licenses
3. Taxes
4. Altitude
5. Grade
6. Curvature
7. Atmospheric Conditions
8. Parking Fees and Tolls

Table C.1 (Refs. 141[1], 142[2], 143[3], 144[4], 145[5], 146[6], 147[7], 148[8], and 149[9]) provides brief summaries of each reference reviewed indicating the applicability of the reference to this study.

Review of Table C.1 indicates that the method proposed by McFarland (Ref. 135) appears to be the best choice for predicting the user costs resulting from pavement roughness. Although the Canadian model (Ref. 141) also predicts these costs, it was developed for 2-lane rural roads while McFarland's considers rural and urban, 2-lane, 4-lane unidivided, and 4 or more lane divided roads.

A discussion of each of the relationships required to estimate values of each component of Figure C.1 follows:

Measurement of Roughness

Current roughness is measured in terms of Present Serviceability Index (PSI). There are several models which can be used to predict the PSI of a pavement given the appropriate design variables. The standard model used is contained in the AASHTO Interim Guide, 1972 (Ref. 106) as developed from data collected from the AASHO Road Test.

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[1]Karan, M.A., R. Haas, and R. Kher, "The Effects of Pavement Roughness on Vehicle Speeds", A paper offered for presentation to the Transportation Research Board, Washington, D.C., January 1976.

[2]Winfrey, R., Economic Analysis of Highways, International Textbook Company, Scranton, Penn., 1969.

[3]Claffey, P.J. and P.J. Claffey and Associates, "Running Costs of Motor Vehicles as Affected by Road Design and Traffic", National Cooperative Highway Research Program Report No. 111, 1971.

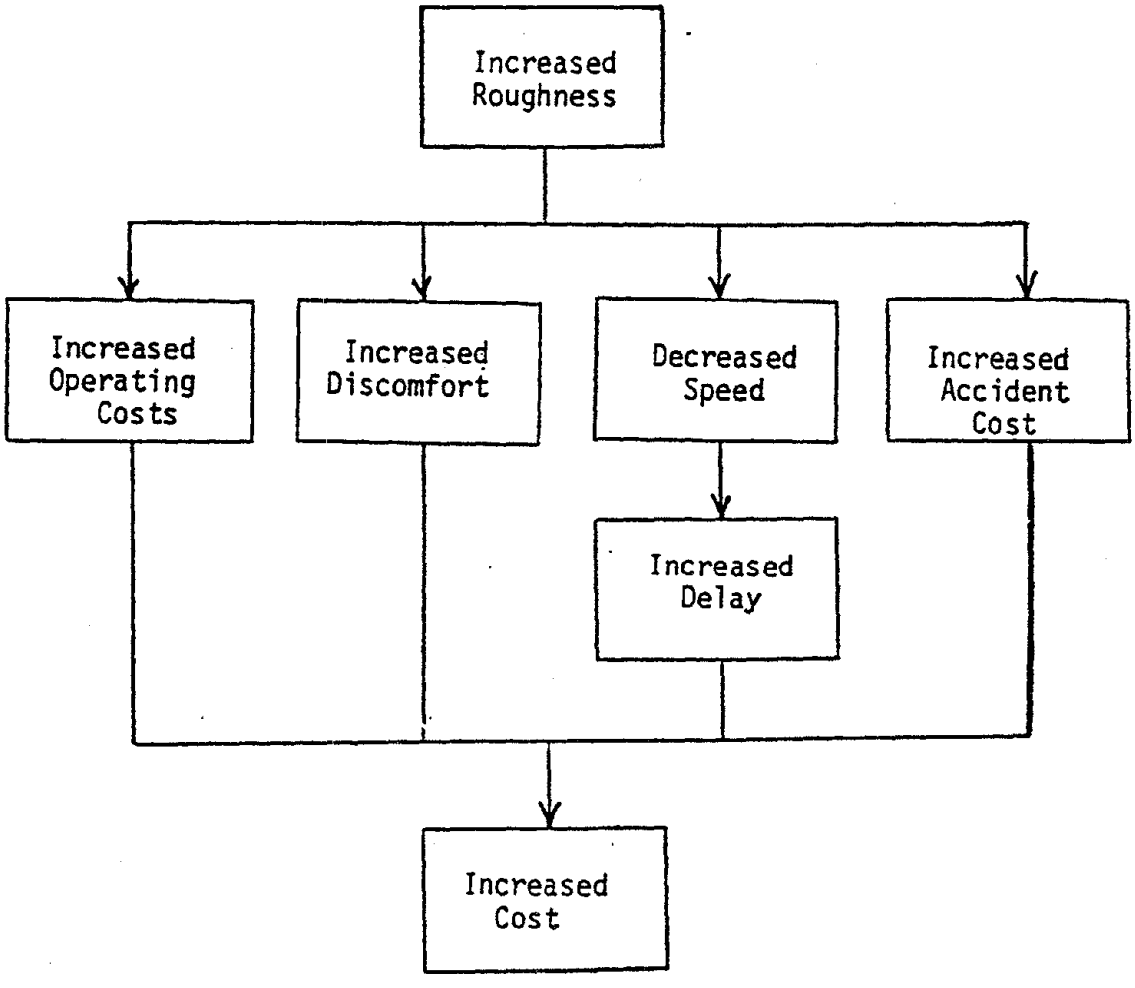

Figure C.1. Effects of roughness on user costs.

TABLE C.1. SUMMARY OF LITERATURE REVIEWED

Reference	Comments
"Benefit Analysis for Pavement Design Systems," William F. McFarland Reference 135	This report presents a complete model relating user costs to pavement roughness. However, most of the relationships are derived from literature and best guesses rather than actual data. The cost data must be updated for inflation. The model considers the effect of urban, rural, 2-lane, 4-lane undivided and 4-lane or more divided highway configurations.
"The Effects of Pavement Roughness on Vehicle Speeds," M.A. Karan, R.C.G. Haas, and Ramesh Kher Reference 141	This report describes a model that can be used to predict user costs as a function of roughness; It is based on field studies but only predicts for 2-lane rural highways; the majority of the effort was spent on development of PSI vs speed relationship.
"Economic Analysis for Highways," Robley Winfrey Reference 142	Winfrey has compiled data from Claffey, Sawhill and Firey, personal test results, and information obtained from correspondence with individuals in the automotive industry. From this data he has compiled consumption rate tables as a function of vehicle type, speed, horizontal curvature, vertical curvature, and speed changes. He includes depreciation, oil, gas, tires, and maintenance in his operating costs but does not consider roughness as a factor. These results are used by McFarland and Haas in the development of their models.
"Running Costs of Motor Vehicles as Affected by Road Design and Traffic," Claffey & Associates Reference 143	Claffey conducted various field tests to determine the effect of gravel roads and paved roads on consumption rates. These results are used by McFarland and Haas to relate roughness of paved roads to operating cost.
"The Kenya Road Transport Cost Study: Research on Road Deterioration," J.W. Hodges, J. Rolt, and T.E. Jones Reference 144	The roads surveyed in these studies are not comparable to roads in this country because of the nature of the subgrade and quality of the roads.

TABLE C.1. SUMMARY OF LITERATURE REVIEWED (Continued)

Reference	Comments
"User Delay Cost Model for Highway Rehabilitation," M.A. Karan and R.C.G. Haas Reference 145	This model was not designed to predict costs resulting from pavement roughness. Part of the output contains speed vs operating cost tables. That may be more accurate than using Winfrey and Claffey's data updated by inflation factors, but may not be justified because of extensive input data required to run the program.
"USERCST," Jim Daniels Reference 136	This program was pulled out of the RPS program and updated (by inflation factors), to reflect current costs but does not relate cost to serviceability. The costs computed are a result of the delay during maintenance activities and are based primarily on Winfrey and Claffey's data.
FPS, Reference 146 RPS, Reference 147 SAMP6, Reference 148 LVR, Reference 149	These models were designed for use as a pavement management system. They all include user cost subroutines but none consider the cost due to roughness. These costs, like Daniel's, are delay costs based on Winfrey and Claffey.

Speed vs PSI

A relationship between speed and PSI is presented in References 135 and 141, herein referred to as McFarland's and the Canadian study, respectively. As a part of the Canadian study the following formula was obtained from regression analysis on actual speed data from 2-lane rural highways:

$$S = 30.7368 + 1.0375\,(\overline{RCI}) - 11.2421(V/C) + .0062(\overline{SL})^2$$

where:

S = Average highway speed, mph

$\overline{RCI}$ = Riding Comfort Index = 2 (PSI)

$\overline{SL}$ = Speed limit, mph

V/C = Volume Capacity Ratio

McFarland's study is based on another field study (Ref. 150[1]) and can not be easily verified, but presents, in graphical form, the PSI vs Speed relationship for 2-lane, 4-lane divided, and 4 or more lane undivided roadways in both urban and rural conditions (See Figure C.2).

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[4]Hodges, J.W., J. Rolt, and T.E. Jones, "The Kenya Road Transport Cost Study: Research on Road Deterioration", Transport and Road Research Laboratory, Digest of Report, 1975.

[5]Karan, M.A. and R.C.G. Haas, "User Delay Cost Model for Highway Rehabilitation", University of Waterloo, Project W-30, Ontario Joint Transportation and Communications Research Program, April 1974.

[6]Hudson, W.R. and B.F. McCullough, "Flexible Pavement Design and Management Systems Formulation", NCHRP 139, Washington, D.C., 1973.

[7]Kher, R.K., W.R. Hudson, and B.F. McCullough, "A Systems Analysis of Rigid Pavement Design", Federal Highway Administration, Department of Transportation Report #123-5, November 1970.

[8]Lytton, R.L., W.F. McFarland, and D.L. Schafer, "Flexible Pavement Design and Management Systems Approach Implementation", NCHRP 160, Washington, D.C., 1975.

[9]Sullivan, E.C., "Vehicle Operating Cost Model - User's Guide 2nd Edition", U.S. Forest Service, University of California, Berkeley, June 1977.

[1]Jorgenson, R. and Associates and Estate Research Analysts, Inc., "Evaluation of Criteria for Safety on the Highways, 1966.

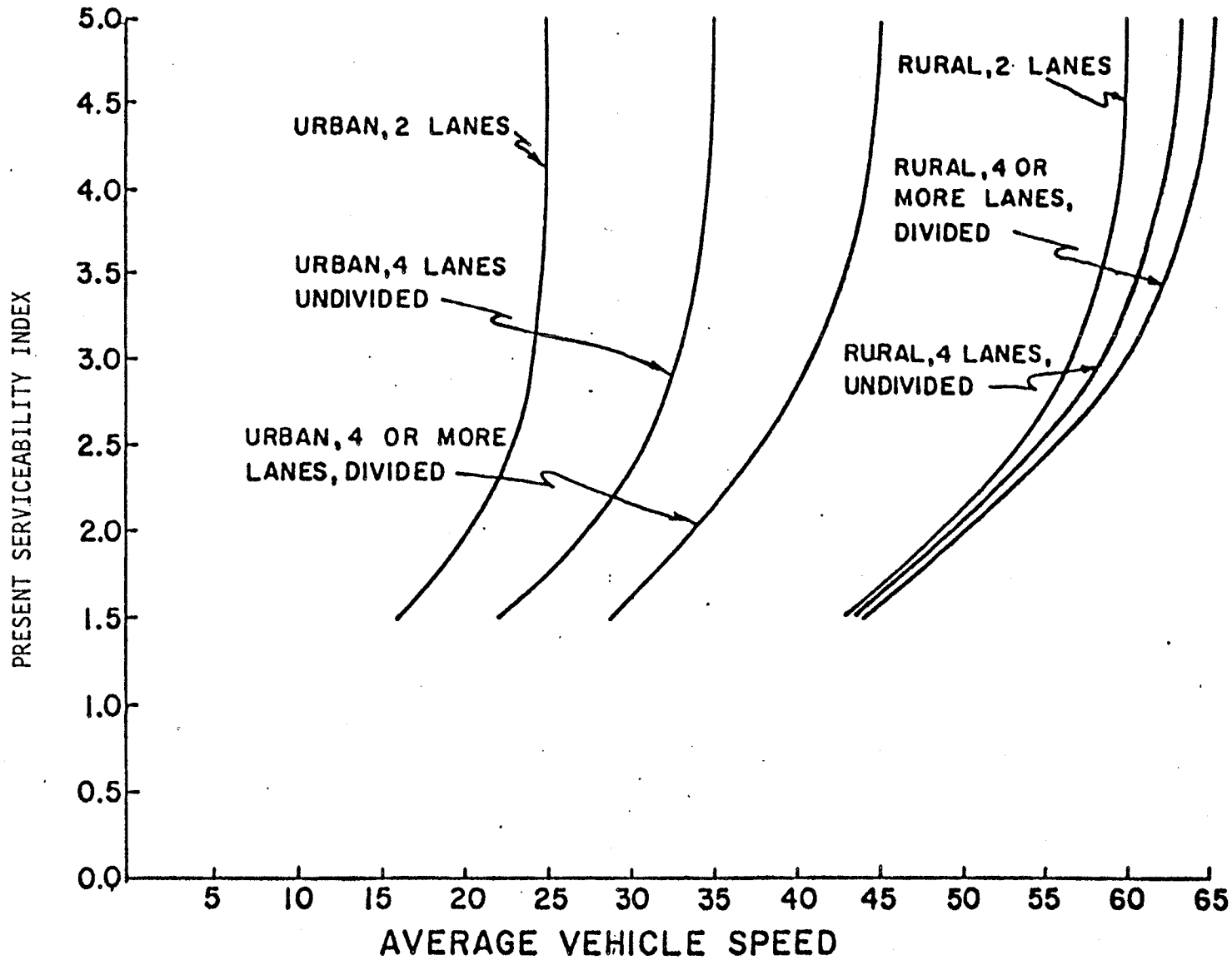

Figure C.2. Average vehicle speed related to pavement serviceability index by type of road (Ref. 135).

In an attempt to verify McFarland's study, typical expected values were assumed for variables in the Canadian formula and results were compared to McFarland's curves. The maximum variation in speed between the two techniques was 2% for PSI $\geq$ 2.0 (see appendix for calculations). Since the Texas relationship is the only published model which considers rural, urban, 2-lane, 4-lane divided and 4 or more lane undivided conditions, and it corresponds favorably with the Canadian speed model, it seems to be the reasonable choice for this study.

Roughness vs Accident Rate

McFarland combines two prior studies (Refs. 150 and 151[1]) to derive a relationship between PSI and accident rates in Texas. This relationship assumes a decreasing coefficient of friction as PSI decreases, which would not be true with some materials. This is, however, the only model available that does consider accident costs due to roughness. These costs are illustrated in Table C.2.

Roughness vs Operating Costs

Operating costs are used to include the costs of oil, gasoline, tires, maintenance, and depreciation. McFarland uses Claffey's findings (Ref. 143) along with The Red Book (Ref. 152[2]) and a study by Bonney and Stevens (Ref. 153[3]) to determine correction factors to modify vehicle operating costs reported by Winfrey. These costs are illustrated in Table C.3, where operating costs are for a composite vehicle.[4]

Roughness vs Delay Costs

Given the relationship between speed and roughness, delay costs can be calculated after determination of the value of time. Again, several references were reviewed by McFarland. McFarland chose to use the value used in the Texas Pavement Design System. This system considers different values for commercial and passenger vehicles, as well as, rural and urban. These costs are illustrated in Table C.4.

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[1]Burke, D., "Highway Accident Costs and Rates in Texas", Research Report 144-1, Texas Transportation Institute, 1970.

[2]American Association of State Highway Officials, "Road User Benefit Analysis for Highway Improvements", AASHO, Washington, D.C., 1960.

[3]Boney, R.S.P. and N.F. Stevens, "Vehicle Operating Costs on Bituminous, Gravel, and Earth Roads in East and Central Africa", Road Research Laboratory Report 76, 1967.

[4]$$\text{Composite Vehicle Cost} = \sum_{i=1}^{n} \begin{matrix}\text{Operating Cost} \\ \text{for Vehicle}_i\end{matrix} \times \begin{matrix}\text{Percentage} \\ \text{of Vehicle}_i\end{matrix}$$

TABLE C.2. RURAL ACCIDENT RATES, IN ACCIDENTS PER HUNDRED MILLION VEHICLE MILES, BY TYPE OF ROAD AND LEVEL OF SERVICEABILITY INDEX (REF. 135).

Present Serviceability Index	2-lane highway	4-lane undivided highway	4 or more lane divided highway
1.5	276	904	143
2.0	241	666	123
2.5	218	500	111
3.0	204	404	104
3.5	196	345	100
4.0	192	321	98
4.5, 5.0	190	311	97

TABLE C.3. VEHICLE OPERATING COSTS, IN CENTS PER VEHICLE MILE, BY TYPE OF AREA, TYPE OF ROAD, AND PRESENT SERVICEABILITY INDEX (REF. 135).

Pavement Serviceability Index	Type of Area and Type of Road					
	URBAN			RURAL		
	2-lane	4-lane undivided	4 or more lane divided	2-lane	4-lane undivided	4 or more lane divided
1.5	5.90	5.60	5.66	7.95	8.05	8.14
2.0	5.43	5.30	5.45	7.84	8.00	8.10
2.5	5.03	5.10	5.26	7.73	7.96	8.05
3.0	4.91	4.94	5.12	7.37	7.70	7.97
3.5	4.86	4.85	5.02	7.06	7.34	7.65
4.0	4.83	4.77	4.94	6.75	7.03	7.30
4.5	4.82	4.74	4.88	6.58	6.84	7.11
5.0	4.82	4.73	4.84	6.43	6.67	6.88

TABLE C.4. TRAVEL TIME COSTS, IN CENTS PER VEHICLE MILE, RELATED TO SERVICEABILITY INDEX, BY TYPE OF AREA AND TYPE OF ROAD (REF. 135).

Pavement Serviceability Index	Travel Time Costs, in Cents per Vehicle Mile, by Type of Area and Type of Road					
	Urban			Rural		
	2-lane	4-lane, undivided	4 or more lanes, divided	2-lane	4-lane, undivided	4 or more lanes, divided
1.5	24.74	18.00	13.66	9.86	9.75	9.64
2.0	19.80	14.67	11.65	8.74	8.57	8.48
2.5	17.22	12.77	10.42	7.93	7.78	7.65
3.0	16.50	12.00	9.66	7.50	7.25	7.07
3.5	15.84	11.65	9.21	7.25	7.01	6.78
4.0	15.84	11.48	9.00	7.13	6.84	6.63
4.5	15.84	11.31	8.80	7.07	6.73	6.52
5.0	15.84	11.31	8.80	7.07	6.73	6.52

Roughness vs Discomfort Costs

McFarland investigated several sources to determine the value of discomfort. The measurement of this cost was handled differently in each of the different sources. One study by Lisco (Ref. 154[1]) suggests that the discomfort costs can be measured by the cost incurred by driving one's own vehicle rather than using the public transit when available. The Red Book (Ref. 152) based its discomfort cost estimate on road surface quality. McFarland used the Red Book's estimate for discomfort costs on an unsurfaced road and a loose surfaced road as a starting point, updated those costs for inflation and assumed a decrease in cost with increasing PSI. These costs are illustrated in Figure C.3.

Roughness vs Total Cost

The total costs have been summarized by McFarland such that, given the PSI of the road, the total cost in cents per vehicle mile can be found (see Tables C.5, C.6, and C.7). The economical benefit of operating on better roads can then be evaluated by the difference in costs at the different PSI values. These costs must be updated for inflation before use in this project.

PURPOSE: To compare results from Canadian Model with those obtained in McFarland's model in order to verify McFarland's model.

The Canadian model is based on actual field studies while McFarland's represents an estimate from a compilation of many unrelated studies.

PROCEDURE: Assume typical values for variables required for Canadian model and compare the computed speeds with those corresponding to the same conditions in McFarland's model. Since the Canadian model predicts only 2-lane speeds, a 2-lane curve is the only check. If the models compare favorably, then McFarland's could be assumed to be accurate until future research refines the model.

COMPUTATIONS: The Canadian model uses the following formula to compute the speed as a function of roughness:

$$S = 30.7386 + 1.0375(\overline{RCI}) - 11.2421(V/C) + .0062(\overline{SL})^2$$

where:

S = Average highway speed
RCI = 2 PSI
V/C = .2 (Free-flow conditions)
$\overline{SL}$ = 55 mph (88 kph)

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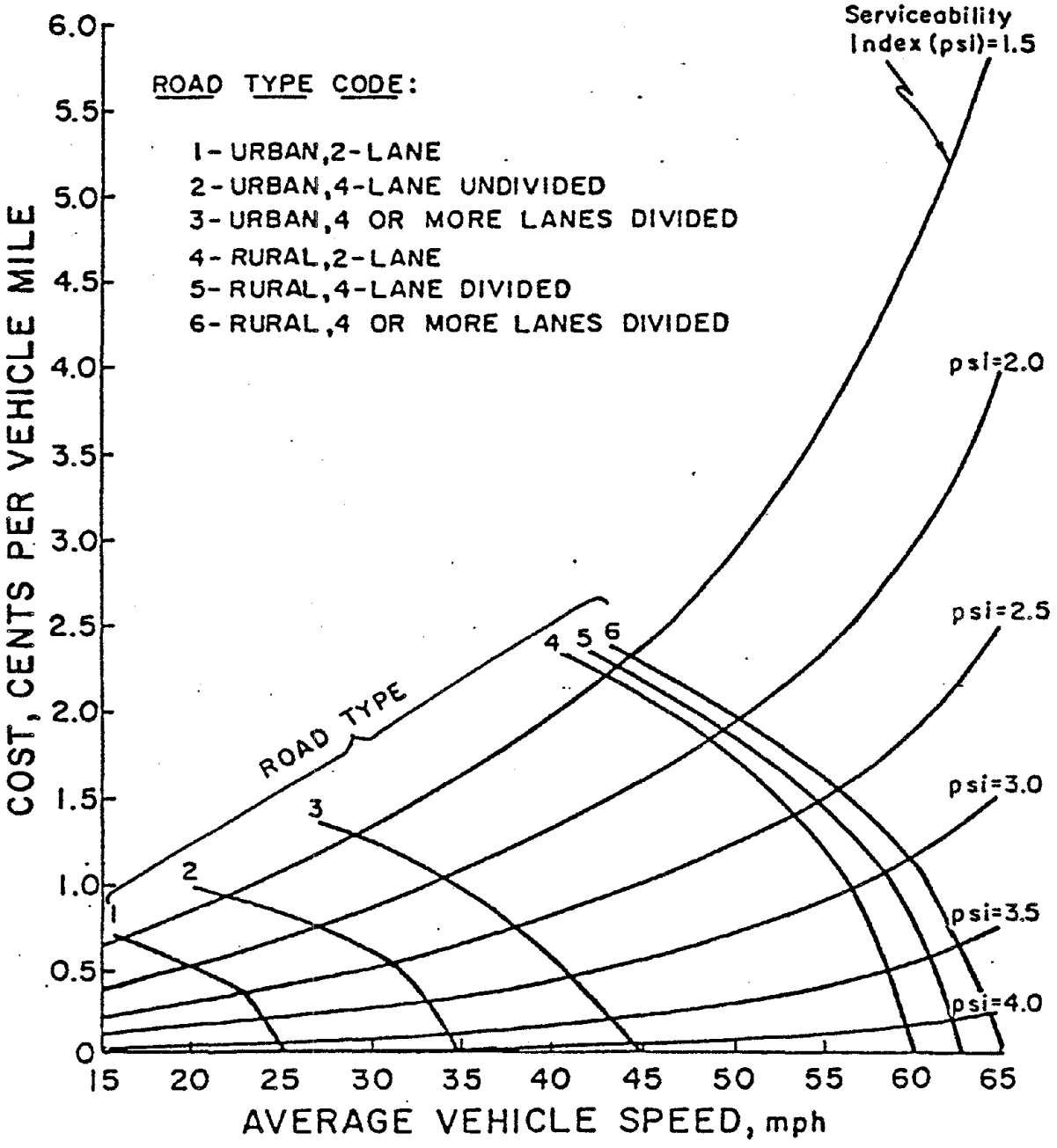
[1]Lisco, T.E., "The Value of Commuter's Travel Time: A Study in Urban Transportation", paper presented at the Forty-Eight Annual Meeting of the Highway Research Board, January 1969.



1 mph = 1.6 kph

Figure C.3. Discomfort costs as a function of present serviceability index, average vehicle speed, and road type (Ref. 135).

TABLE C.5. USER COSTS, IN CENTS PER VEHICLE MILE, RELATED TO THE PRESENT SERVICEABILITY INDEX, FOR RURUAL ROADS WITH 2 LANES (REF. 135).

Present Serviceability Index	Type of Cost				
	Time	Operating	Accident	Discomfort	Total
1.5	9.86	7.95	0.86	2.20	20.87
2.0	8.74	7.84	0.75	1.80	19.13
2.5	7.93	7.73	0.68	1.40	17.74
3.0	7.50	7.37	0.63	0.95	16.45
3.5	7.25	7.06	0.61	0.50	15.42
4.0	7.13	6.75	0.60	0.20	14.68
4.5	7.07	6.58	0.59	0.00	14.24
5.0	7.07	6.43	0.59	0.00	14.09

TABLE C.6. USER COSTS, IN CENTS PER VEHICLE MILE, RELATED TO THE PRESENT SERVICEABILITY INDEX, FOR UNDIVIDED RURAL ROADS WITH 4 LANES (REF. 135).

Present Serviceability Index	Type of Cost				
	Time	Operating	Accident	Discomfort	Total
1.5	9.75	8.05	2.80	2.25	22.85
2.0	8.57	8.00	2.06	1.90	20.53
2.5	7.71	7.96	1.55	1.50	18.72
3.0	7.25	7.70	1.25	1.05	17.25
3.5	7.01	7.34	1.07	0.55	15.97
4.0	6.84	7.03	1.00	0.20	15.07
4.5	6.73	6.84	0.96	0.00	14.53
5.0	6.73	6.67	0.96	0.00	14.36

TABLE C.7. USER COSTS, IN CENTS PER VEHICLE MILE, RELATED TO THE PRESENT SERVICEABILITY INDEX, FOR DIVIDED URBAN ROADS WITH 4 OR MORE LANES (REF. 135).

Present Serviceability Index	Type of Cost			
	Time	Operating	Discomfort	Total
1.5	13.66	5.66	1.25	20.57
2.0	11.65	5.45	1.05	18.15
2.5	10.42	5.26	0.75	16.43
3.0	9.66	5.12	0.45	15.23
3.5	9.21	5.02	0.20	14.43
4.0	9.00	4.94	0.05	13.99
4.5	8.80	4.88	0.00	13.68
5.0	8.80	4.84	0.00	13.64

PSI	S Calculated (mph) McFarland*	Canadian	Δ	%Δ
1.5	43.9	52.4	-9.4	-20%
2.0	48.5	53.4	-5.4	-12%
2.5	53.5	54.4	-0.9	- 2%
3.0	56.5	55.5	1.0	2%
3.5	58.5	56.5	2.0	3%
4.0	59.5	57.5	2.0	3%
4.5	60.0	58.6	1.4	2%
5.0	60.0	59.6	0.4	0%

CONCLUSIONS: It is reasonable to assume that premium pavement PSI will never drop below 2.0, therefore ±2% difference is an acceptable variation. McFarland's curves appear to be relatively accurate.

*From Table C.2, 2-lane Rural curve.

Summary

The cost model presented by McFarland in Benefit Analysis for Pavement Design Systems is the most comprehensive study to date. The model lacks the research necessary to verify the accuracy of the predictions, but is the only model that considers all the variables currently associated with costs attributable to roughness. At a later date, certain portions of this procedure may be replaced as verified models are made available.

APPENDIX D

EVALUATION OF MODELS FOR THE PREDICTION OF USER COSTS RESULTING FROM PAVEMENT MAINTENANCE ACTIVITIES

Introduction

Before discussion of any of the models available, the definition of user costs must be clarified. The user costs discussed in this memorandum are confined to those extra costs incurred by the vehicle operator resulting from the delay caused by the presence of maintenance activities on the highway. These costs are made up of the excess cost of operating the vehicle at a slower speed. The cost of decelerating, perhaps stopping, and then accelerating, and the travel time delay cost to the operator. It should be noted that none of the models consider the excess cost of operation over a rougher or lower quality surface.

The inputs, calculated values and type of cost data used are summarized in Table D.1. A more lengthy discussion of each model follows.

Canadian Model (Ref. 145)

Upon initial inspection, the Canadian Model developed by R.C.G. Haas and M.A. Karan appears to be the most comprehensive model. There is a trade off, however: a more accurate estimate at the expense of excessive amount of input. Further review of the program, after we obtain a copy, may reveal that much of the input can be incorporated as fixed constants that may be updated on request by us or some other mechanism. The final decision on whether or not this model will be used will be deferred to a later time when we have the program, and can determine whether the extensive input results in a significantly more accurate estimation of the excess user costs.

The major benefit in using this model is the capability of inputting the current costs of various products and services associated with vehicle operating costs. Choosing these costs may present a problem, but since we are dealing with excess user costs; national averages reported by consumer and business magazines should prove accurate enough. In addition to the actual user cost estimates, the program also prints out cost of operating tables for each vehicle type which may not be necessary for this project, but could be useful at some other point in time. This output could be deleted in the interests of keeping the program to a minimum length.

Another unique feature included in this program is the ability to account for seasonal cost differences and overtime, if required. This capability is not in any of the other models and could conceivably make a significant cost difference.

TABLE D.1 SUMMARY OF USER COST MODELS

Model	Values Calculated	Cost Data	Input Required
Canadian (Ref. 145	COST OF: Gas, Oil, Tire Wear, Maintenance Delay (seasonal) Variation and Overtime considered)	Input Unit Costs	Time required for maintenance operations Time (season, day, hour) Unit costs Traffic (Extensive Counting Sta. Data) Roadway Geometrics of Normal condition and maintenance condition Growth factors Interest Rate
Accident and Speed Studies (Ref. 155	Gas Consumption Delay Time	Input Unit Costs	Time required for maintenance operations Urban or rural Traffic (AADT only) Geometrics (length of influence zone)
EAROMAR (Ref. 156	COST OF: Gas, Oil, Tire Wear, Maintenance Delay	Input Unit Cost or default and update with inflation factor	These inputs are difficult to define since these user cost models are a part of other programs and many of the inputs are calculated in the main program.
LVR, FPS RPS, SAMP6 (Ref. 157	COST OF: Gas, Oil Tire Wear Maintenance, Delay	Needs to be updated with inflation factor	
USERCST (Ref. 136	COST OF: Gas, Oil, Tire Wear, Maintenance, Delay, Type of overlay	Reflects current prices as is.	Working hours Urban or Rural Traffic (ADT) Geometrics of normal and maintenance conditions (speed, lengths, etc.) Overlay size, type (flexible or rigid) and rate Growth Factor Interest rate

The geometric design of the roadway before and during construction is an input required of all the models. Traffic characteristics required are also similar, with the exception of this Canadian Model needing an extra 300 cards associated with permanent counting station data and Highway Capacity Manual volume/capacity charts. It is probable that the HCM charts could be incorporated into the program permanently, without having to input them each time.

Since calculation of the user costs will be only a small portion of the entire FH-6 program the extensive input required for the Canadian Model will have to be justified by either being able to shorten the input and/or a considerable increase in reliability and accuracy over the other models.

Accident and Speed Studies Model (Ref. 155[1])

Another study was published in June, 1977, by the Midwest Research Institute entitled Accident and Speed Studies in Construction Zones. The purpose of the study was not primarily to predict user costs but to improve the safety factor through construction zones. In performing the primary task, however, enough data was gathered to derive equations for delay time and excess gasoline consumption. Oil, tires and excess vehicle maintenance due to rehabilitation operations were neglected. The equations assume a 10% truck population and the required input is the average daily traffic, length of the influence zone, number of working hours per day, number of days of construction and method of traffic handling through construction zone. This model could be adapted for our use, but would require derivation of equations and some additional programming effort since this model is not computerized. The advantage of being able to use current time delay and gasoline costs along with simple input data would be very desirable, but the neglect of oil, tires, and maintenance would seem to outweigh those benefits.

EAROMAR (Ref. 156[2])

The initial drawback in the use of the EAROMAR model is that it does not stand alone. It is part of a larger program that is poorly documented and thus hard to determine the actual procedure. Use of the EAROMAR user cost model would require rather extensive programming effort to separate it from the rest of the model, determine all inputs and write the driver for running it separately. Previous reviews of

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[1]Graham, J.L., R.J. Paulsen, and J.C. Glennon, "Accident and Speed Studies in Construction Zones", Midwest Research Institute, FHWA, NTIS, Springfield, Va., June 1977.

[2]Butler, B.C., Jr., "Economic Analysis of Roadway Occupancy for Freeway Pavement Maintenance and Rehabilitation", Federal Highway Administration, Washington, D.C., October 1974.

EAROMAR have indicated that this programming effort would not be worthwhile, i.e., instead of performing the required maintenance, it would probably be more cost effective to either use another model or even develop a new one rather than trying to "decode" this one.

SAMP6, FPS, RPS, LVR (Ref. 157[1])

These models are all pavement design optimization programs which have user cost calculations incorporated into them. They basically use the same input and generate the same output. The differences in the programs are that SAMP6 and FPS are for flexible pavements, RPS is for rigid, and LVR is for paved or gravel roads. There is really no need to discuss these programs any further since the user cost model has been extracted, costs updated to 1978 prices, and a driver written by Jim Daniel in the program USERCST (Ref. 136).

USERCST (Ref. 136)

As previously stated, USERCST has been updated to reflect current prices and can stand alone. The input required (see Table D.1) will be available at the time the model is used, and therefore, will present no problem.

The printed output is expressed as user cost of the overlay in dollars per square yard of overlay with consideration of the type of overlay used (i.e., flexible or rigid). This form of output could be very useful because of its flexibility.

Summary and Conclusions

The Canadian model requires such extensive data that it would probably not be practical to use. The Accident and Speed Studies model neglects the excess cost incurred from maintenance, oil consumption, and tire wear. Therefore, it should also be ruled out. EAROMAR would require an excessive amount of modification and because of poor documentation, should not be used. The models used in SAMP6, LVR, FPS, and RPS are all basically the same and have already been extracted in the form of the USERCST program. Therefore, it is proposed to use the USERCST model since it reflects current prices and can stand alone without any further programming.

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[1]McCullough, B.F., Pavement Design Lecture, The University of Texas at Austin, C.E., Spring 1978.

APPENDIX E

THICKNESS DESIGN PROCEDURE

This appendix has been prepared to provide the engineer with a simplified graphical solution for determining thicknesses of zero-maintenance pavements using the computer programs and information discussed in Chapters 4-6 and the design procedure outlined in Chapter 7. In order to handle each situation a series of design charts have been included in this appendix that will enable an engineer to develop a cross-section design utilizing the following types of information:

1. Environmental factors (Temperature and Air Freezing Index)
2. Natural and Existing Soil Properties
3. Traffic in 18-kip (80 kN) equivalent axle loads
4. Subgrade Modulus
5. Material Type

The structural design charts included in this Appendix are presented in a form where a combination of traffic applications and asphalt concrete modulus or portland cement concrete flexural strength are used to determine layer thicknesses. The first set of charts are the principal thickness charts which include asphalt concrete thickness or portland cement concrete thicknesses as a function of traffic and subgrade modulus. These charts indicate the structural thickness required to satisfy the design criteria of fatigue cracking, rutting and roughness. The second set of charts are the stabilized base thickness requirements as a function of traffic and subgrade modulus. These charts indicate the additional thickness required to satisfy the deflection and subgrade vertical compressive strain criteria.

The following presents this series of charts and graphs for designing the thicknesses of a pavement cross-section capable of providing the high quality of serviceability required for premium pavements.

DESIGN INPUTS

In order to use this design manual, the designer must collect basic information on the items listed above. A worksheet for tabulating this information (Worksheet A) is given in Figure E.1.

Environmental Factors

In this procedure temperatures are used to modify asphalt concrete properties and determine depth of frost penetration. The temperature values required are: 1) mean annual air temperature (MAAT), 2) warmest mean monthly air temperature (WMMAT), and 3) the coldest mean monthly air temperature (CMMAT). These air temperatures are obtained from

"ZERO-MAINTENANCE" DESIGN

INPUT-WORKSHEET A

Project:_____ Date:_____

Location:_____

ENVIRONMENTAL DATA

(1) Mean Annual Air Temperature (MAAT), °F	
(2) Warmest Mean Monthly Air Temperature (WMMAT), °F . .	
(3) Coldest Mean Monthly Air Temperature (CMMAT), °F . .	
(4) Mean Monthly Air Temperature Change, (MMATC), °F (MMATC = WMMAT - CMMAT)	
(5) Design Air Freezing Index (Figure 2.2)	

NATURAL SOIL DATA

	Soil I	Soil II	Soil III
(6) Boring Number			
(7) Boring Location			
(8) Soil Type: Unified Soil Classification and P. I. ( ) . . .	( )	( )	( )
Percent finer than 0.02 mm . . .			
Frost Soil Classification (Table 2.2, Figure 2.3)			
(9) Subgrade Stiffness (Appendix A) .			
(10) Water Table Depth, ft.			

TRAFFIC DATA

(11) Design Applications 18-kip (80 kN) Equivalent Axle Loads (EAL) in Design Lane (Worksheet B, Figure 2.4)	

Figure E.1. Worksheet tabulating the input information

climatological records (Ref. 108) and then recorded on Worksheet A (Fig. E.1). These temperature values are used to reflect the effects of other environmental factors, such as solar radiation, daily temperature variations and minimum design temperatures for specific areas in the U.S. Figure E.2 is used to obtain this particular environmental region. Figures E.3 and E.4 are then used to predict the annual minimum pavement temperature for that particular region.

Air freezing index values are less readily available than the temperature information. Hence, Figure E.5 can be used to estimate air freezing index for various locations in the United States. This value is recorded on Worksheet A (Fig. E.1).

Natural Soil Properties

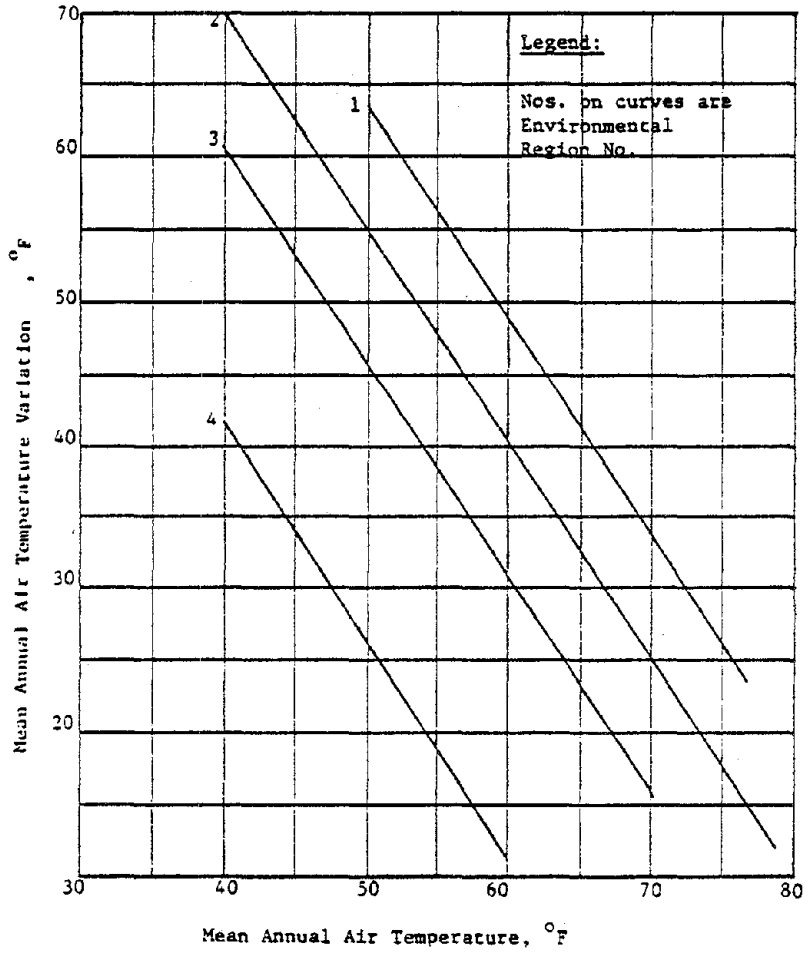
The subgrade modulus is used as one of the principal design parameters. Since the modulus of most subgrade soils is dependent on the state of stress, the subgrade modulus should be based on in-situ or expected field conditions, e.g., a moisture content and density which the soil is likely to reach under the pavement structure. This equilibrium moisture content is usually similar to that found at a depth of about 3 feet (1 m) in the natural soil. An appropriate method for determining the modulus of the subgrade is described in Volume II of this study (Ref. 5). The resilient modulus is determined a function of stress and confining pressure. The test should be performed on undisturbed specimens or on specimens which are recompacted to a moisture content and density which are similar to in-situ conditions under the actual pavement. Suggested steps which can be followed to obtain information on natural soil properties are presented below:

1. Make a complete and thorough investigation of the topographic and subgrade conditions.

2. Conduct exploratory borings at a spacing and depth prescribed by the engineer. The spacing and depth of these borings are dependent on the variability of the existing soil conditions, both vertically and horizontally. These borings should also be used to determine the water table depth. Take sufficient and appropriate auger, split tube, or undisturbed samples at all representative subsoil layers. Prepare boring logs and soil profiles.

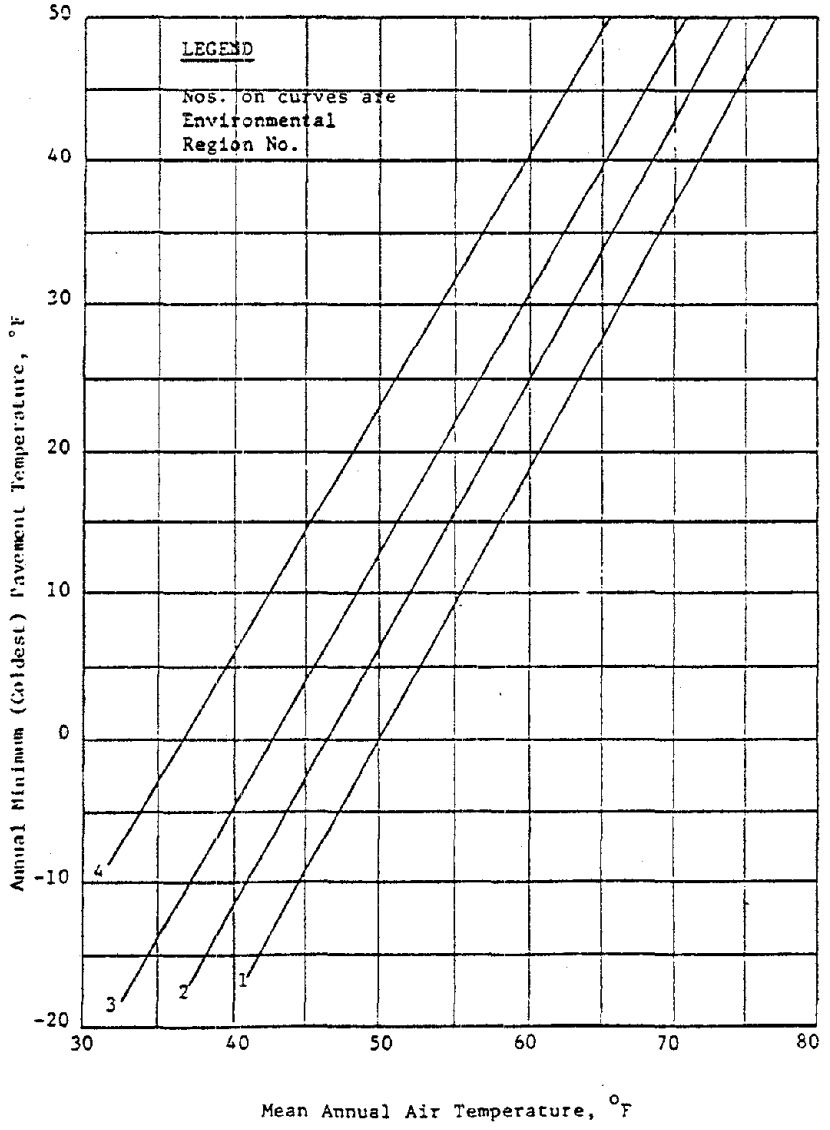
3. Classify all soils using the Unified soil classification system. The modified Proctor moisture-density test should be used to determine the compaction characteristics for soil and untreated pavement materials. The degree of compaction required or the in-place density should be expressed as a percentage of the maximum density from the modified Proctor test.

Example:
Mean monthly maximum temperature - 81°F
Mean monthly minimum temperature - 53°F
Mean monthly variation - 28°F
Mean annual temperature - 72°F
Environmental Region 1



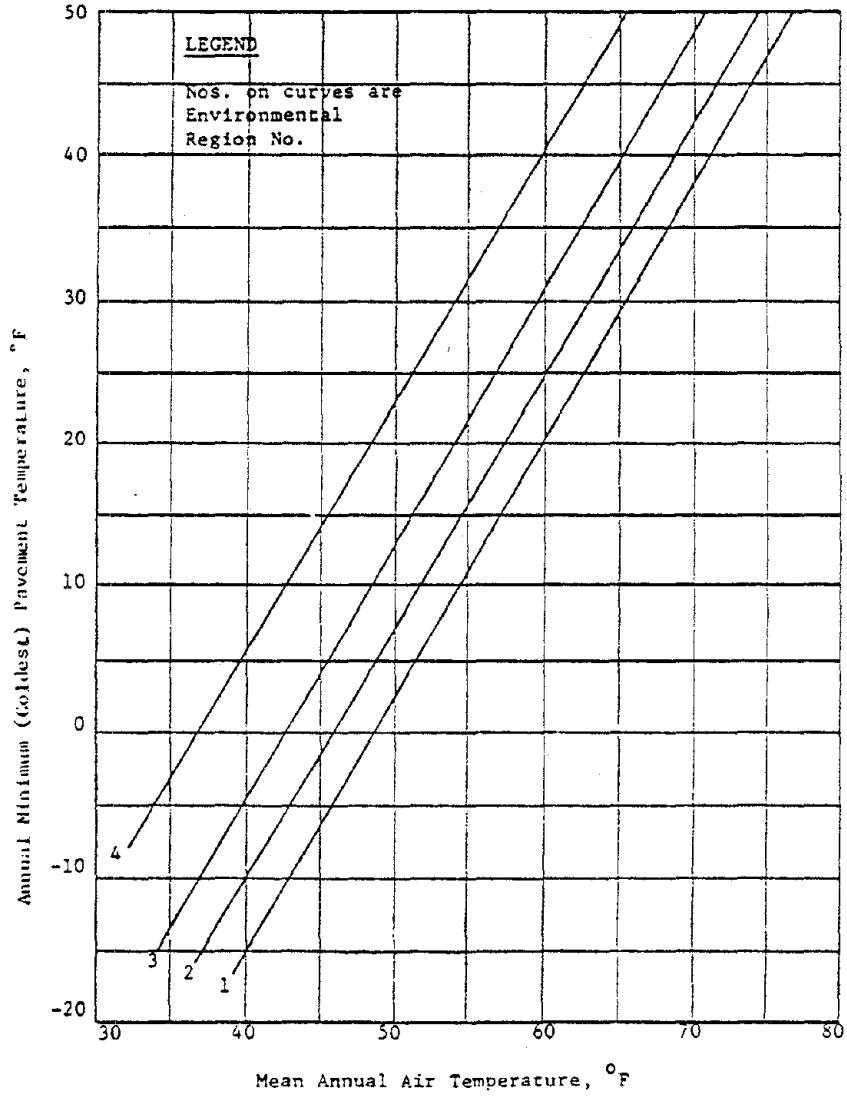
°C = (°F - 32)/1.8

Figure E.2. Determination of the Environmental Region Number



°C = (°F - 32)/1.8

Figure E.3. Estimation of the Annual Minimum (Coldest) Pavement Temperature for Dry Environments (Average Yearly Rainfall < 12 in.).



°C = (°F - 32)/1.8

Figure E.4. Estimation of the Annual Minimum (Coldest) Pavement Temperature for Wet Environments (Average Rainfall > 12 in.).

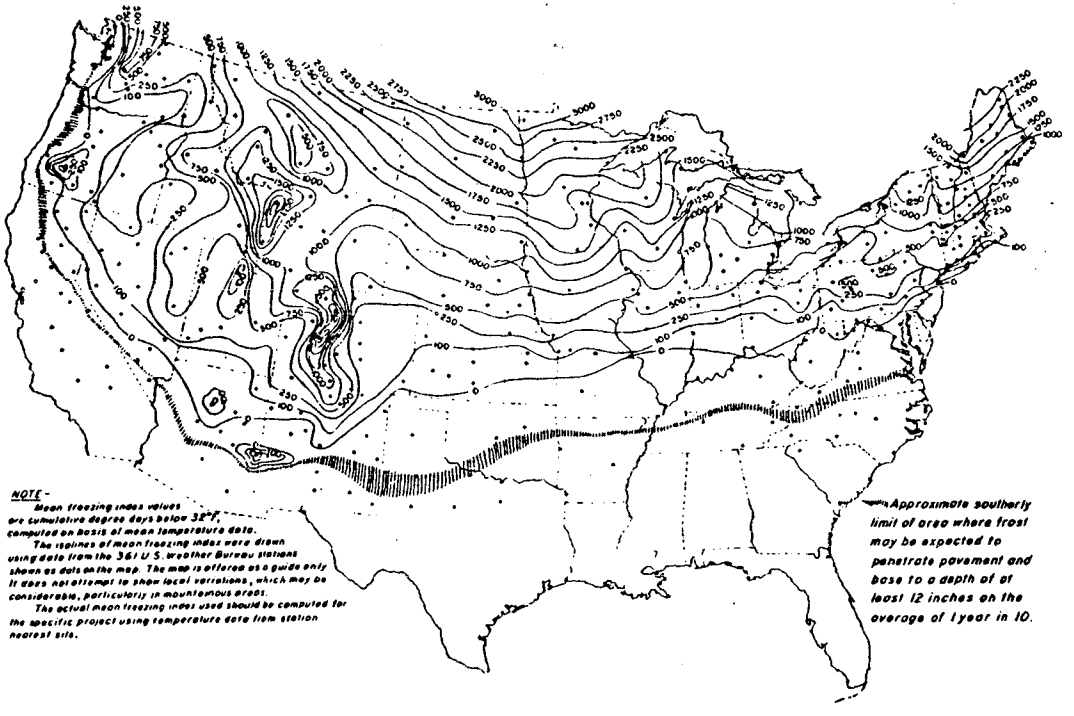


Figure E.5. Distribution of design air freezing index values in the Continental United States (Ref. 30).

4. Determine the frost susceptibility of all soils encountered. Table E.1 and Figure E.6 can be used as an aid in assigning a soil to a frost group using the Corp of Engineers procedure (Ref. 30).

5. Examine the boring logs, soil profiles, and classification tests and select representative soil layers for design testing. Using the procedures outlined in Volume II (Ref. 5) determine a resilient modulus for each type of subgrade soil encountered.

6. Use the soil profile to relate resilient modulus to each type of subgrade soil encountered along the alignment. Select a design subgrade resilient modulus that is representative of each boring. For design purposes it is recommended that the weakest subgrade layer be selected as the design resilient modulus, unless the material is removed, improved or stabilized.

Record the selected soil properties on Worksheet A. Using the above information from classification tests, determine if the existing subgrade soil should be stabilized as illustrated by Figure E.7. If stabilization is required, use Figure E.8 to determine the type of stabilizer. The amount of stabilizer to be used should be substantiated by approved testing methods.

Traffic

Traffic is usually expressed in 18-kip (80 kN) equivalent single-axle loads as presented in the AASHTO Interim Guide (Ref. 106) and as developed at the AASHO Road Test (Ref. 109). In this design manual, total traffic is also expressed as the number of 18-kip (80 kN) equivalent axle loads (EAL) which occur during the minimum 20 year maintenance-free design life. The design lane traffic may be computed using Traffic Worksheet B contained in Figure E.9. The projected number of axles for each load category is listed on Worksheet B. The number of axles in each axle load category is then multiplied by a load conversion factor to obtain the equivalent number of 18-kip (80 kN) single axle loads. For simplicity the AASHTO equivalency factor for a terminal serviceability of 3.0 can be used. The number of 18-kip (80 kN) single axle loads are summed to obtain the total axles per lane per day. The cumulative number of axles during the design life is calculated by multiplying the number per day for the design lane by a summation factor from Table E.2 and recorded on Worksheets A and B.

ASPHALT CEMENT GRADE

As stated previously, Figures E.3 and E.4 can be used to estimate the minimum pavement temperature of the asphalt concrete. This minimum pavement temperature is used in Figure E.10 to select an asphalt cement grade that should not exceed the low-temperature cracking criteria.

TABLE E.1. FROST DESIGN SOIL CLASSIFICATION (REF. 97)

Frost Group	Kind of Soil	Percentage Finer than 0.02 mm by Weight	Typical Soil Types Under Unified Soil Classification System
F1	Gravelly soils	3 to 10	GW, GP, GW-GM, GP-GM
F2	(a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	3 to 15	SW, SP, SM, SW-SM, SP-SM
F3	(a) Gravelly soils	Over 20	GM, GC
	(b) Sands, except very fine silty sands	Over 15	SM, SC
	(c) Clays, PI 12	--	CL, CH
F4	(a) All silts	--	ML, MH
	(b) Very fine silty sands	Over 15	SM
	(c) Clays, PI < 12	--	CL, CL-ML
	(d) Varved clays and other fine-grained banded sediments	--	CL and ML; CL, ML, and SM; CL, CH, and ML; CL, CH, ML, and SM

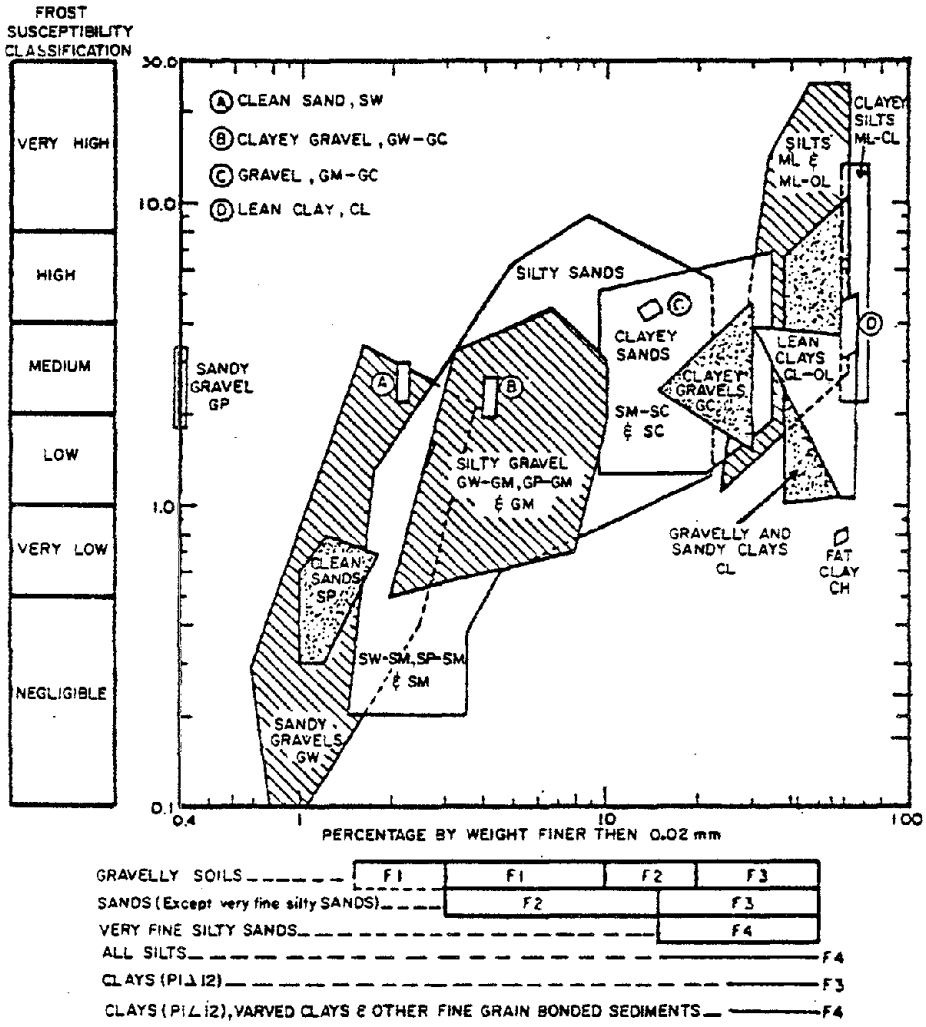


Figure E.6. Average Rate of heave vs percentage finer than 0.02 mm for natural soil gradations.

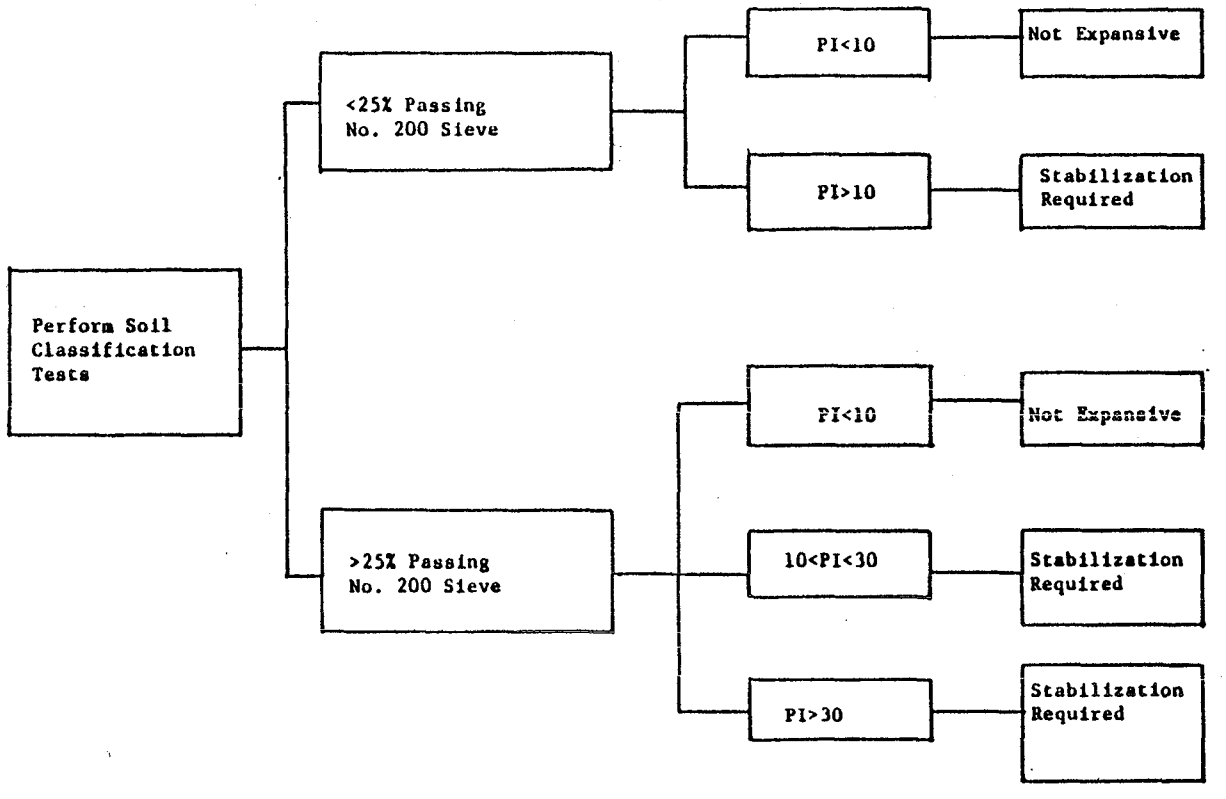


Figure E.7. General Flow Diagram for Determining if a Soil can be Stabilized to Reduce the Swelling Potential.

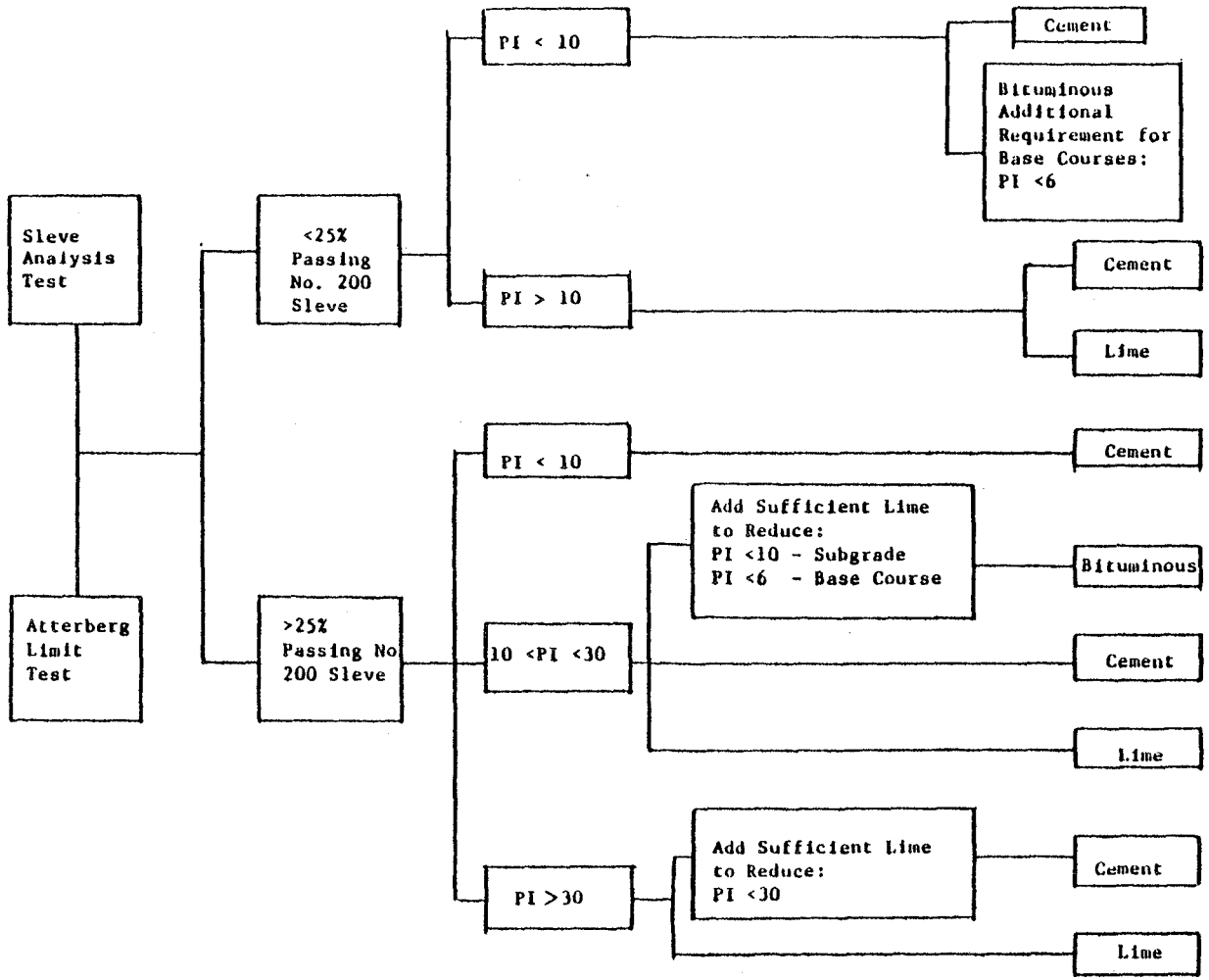


Figure E.8. Flow Diagram to Determine the Type of Stabilizer for a Specific Soil Condition.

"ZERO-MAINTENANCE" DESIGN

TRAFFIC-WORKSHEET B

Project:_____ Date:_____

Location:_____

(1) Axle Load L, kips	(2) Projected Number of Axles per day (Design Lane), N	(3) Equivalency Factor, E (Appendix B)	(4) Projected Equivalent Number of 18 kip (80 kN) axles per day (Design Lane), Ne

(5) Total Projected Number of Equivalent 18-kip (80 kN) axles per day (Design Lane) _____		
(6) Total Projected Number of Equivalent 18 kip (80 kN) per year (Design Lane), (5)x365= _____		
(7) Growth Rate Projected for Design Period _____		
(8) Design Life of Pavement for Zero-Maintenance, Years_____		
(9) Traffic Summation Factor (Table 2.5)_____		
(10) Total Projected Number of 18-kip (80 kN) axles in Design Lane, (6)x(9) = _____		

Figure E.9. Worksheet for Obtaining the Design Number of 18-kip (80kN) Equivalent Axle Loads.

TABLE E.2. TRAFFIC SUMMATION FACTORS*

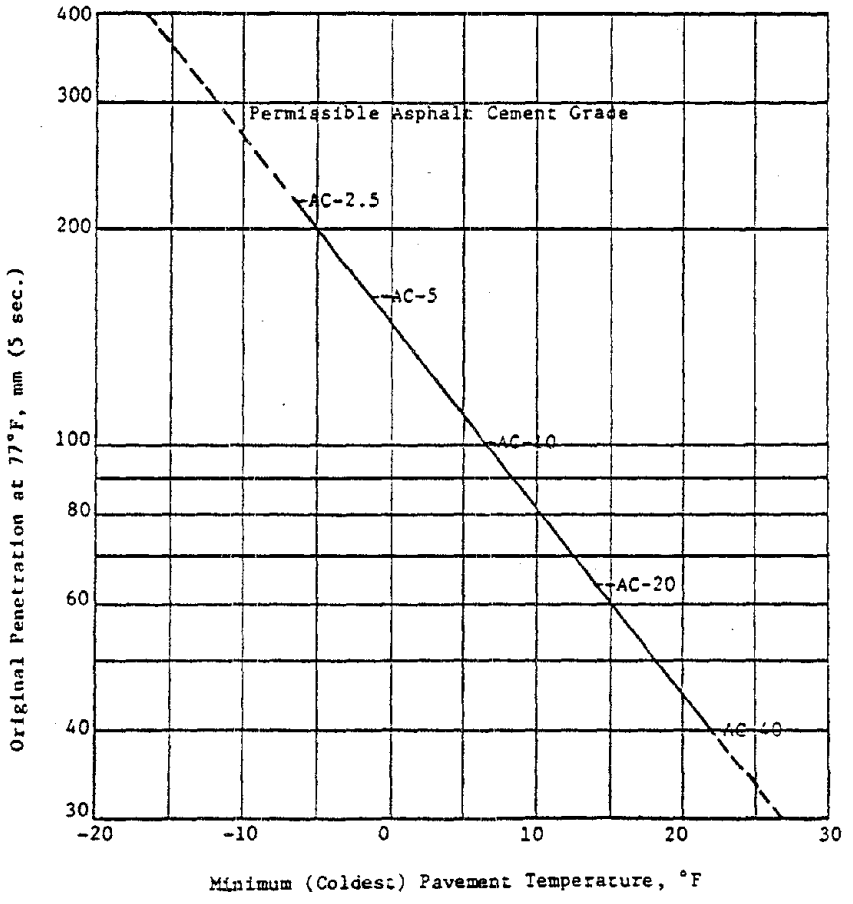
Growth Rate, % \ Design Life, Years	15	20	25	30	35	40
1	16	22	28	35	42	49
2	18	25	33	41	51	62
3	19	28	38	49	62	78
4	21	31	43	58	76	99
5	23	35	50	70	93	127
6	25	39	58	84	118	164
7	27	44	68	101	148	214
8	29	49	79	122	186	280
9	32	56	92	149	235	365
10	35	63	108	181	298	487

$$* \; F_Y = \sum_{n=1}^{Y} \left(1 + \frac{r}{100}\right)^n$$

r = Rate of Growth, %

n = Time Interval (1 Year)

Y = Design Life, years



°C = (°F - 32)/1.8

Figure E.10. Determination of the Asphalt Cement Grade or the Allowable Original Penetration for Asphalts with a Penetration Index of -1.0.

EFFECTIVE ASPHALT CONCRETE MODULUS

Variations in ambient temperature produce significant changes in the modulus of asphalt concrete materials. A procedure was developed that includes a weighted mean asphalt concrete modulus that is determined for each asphalt cement type and climate. The term effective or weighted mean asphalt concrete modulus was developed to consider the effects of temperature variations on both rutting and fatigue cracking. These effects cannot be adequately determined simply by averaging the asphalt concrete modulus over all seasons because of the interaction of temperature and modulus with the fatigue damage produced in the pavement. The effective asphalt concrete modulus for a particular asphalt mix and climate is computed using Worksheet D (Fig. E.11). First the seasonal pavement temperature values are determined using one of Figures E.12 through E.16. Using each seasonal pavement temperature and the viscosity of the asphalt cement, asphalt concrete modulus is determined from Figure E.17. These modulus values for each temperature are entered in Worksheet D. For each asphalt concrete modulus a fatigue factor is then determined from Figure E.18. The asphalt concrete modulus is then multiplied by the appropriate fatigue factor and recorded on Worksheet D. These products are then summed and divided by the sum of the fatigue factors to obtain an effective asphalt concrete modulus that is representative of a climate and mix.

THICKNESS DETERMINATION

Using the total projected traffic, effective asphalt concrete modulus or low-modulus portland cement concrete flexural strength (28-days), and the subgrade modulus, the thicknesses can be determined for each pavement type. For all pavement cross-sections which require a stabilized base layer, an improved subgrade layer thickness of 24 inches (61 cm) is required. The minimum thickness of the stabilized base layer is recommended to be 6 inches (15.2 cm). For conditions when the base thickness is less than 6 inches (15.2 cm), a reduction in the improved subgrade layer can be made using a layer equivalency value for the following materials:

Cement Treated Base - 5
Asphalt Treated Base - 2

FULL-DEPTH ASPHALT CONCRETE

Figures E.19 through E.22 are used to determine the thickness of full-depth asphalt concrete required to resist fatigue cracking and rutting while retaining an acceptable level of PSI. If traffic levels other than those shown are needed, interpolation to the desired level is satisfactory. Figures E.23-E.26 are used to determine the thickness required of asphalt stabilized base material, as a function of subgrade modulus, traffic and asphalt concrete summer modulus. This total thickness requirement satisfies the surface deflection and subgrade vertical

PREMIUM PAVEMENT DESIGN

EFFECTIVE MODULUS WORKSHEET D

Project:_____ Date:_____

Location:_____

(1) Mean Annual Air Temperature Worksheet A, °F . . . . . _____

(2) Mean Yearly Temperature Change, °F . . . . . _____

(3) Environmental Region (Figure E.2) . . . . . _____

(4) Minimum Pavement Temperature, (Figure E.3, E.4), °F . . . . . _____

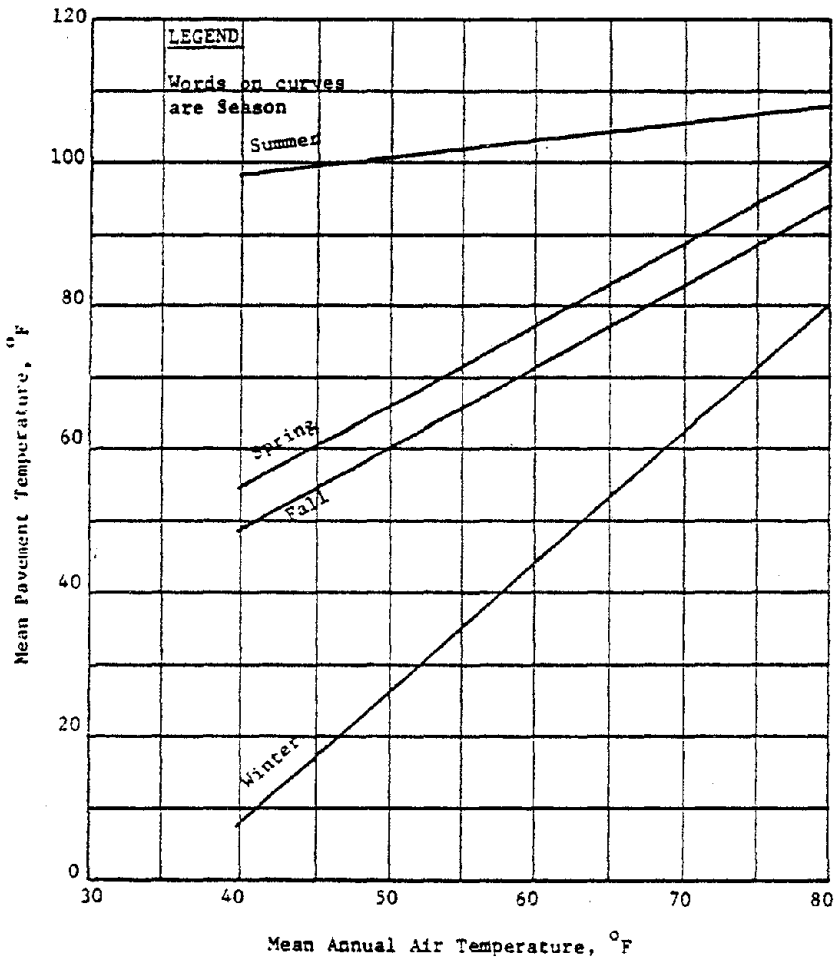
(5) Allowable Minimum Original (Figure E.10) Penetration . . . . . _____

(6) Asphalt Cement Grade . . . . . _____

(7) Season	(8) Mean Pavement Temperature (Figure E.12 - E.16) °F	(9) Asphalt Concrete Modulus (Figure E.17) psi	(10) Fatigue Factor (Figure E.18)	(11) (9) x (10)
Fall				
Winter				
Spring				
Summer				
(12) Total				
(13) Asphalt Concrete Effective Modulus, psi ($\sum$(11)+$\sum$(10))				

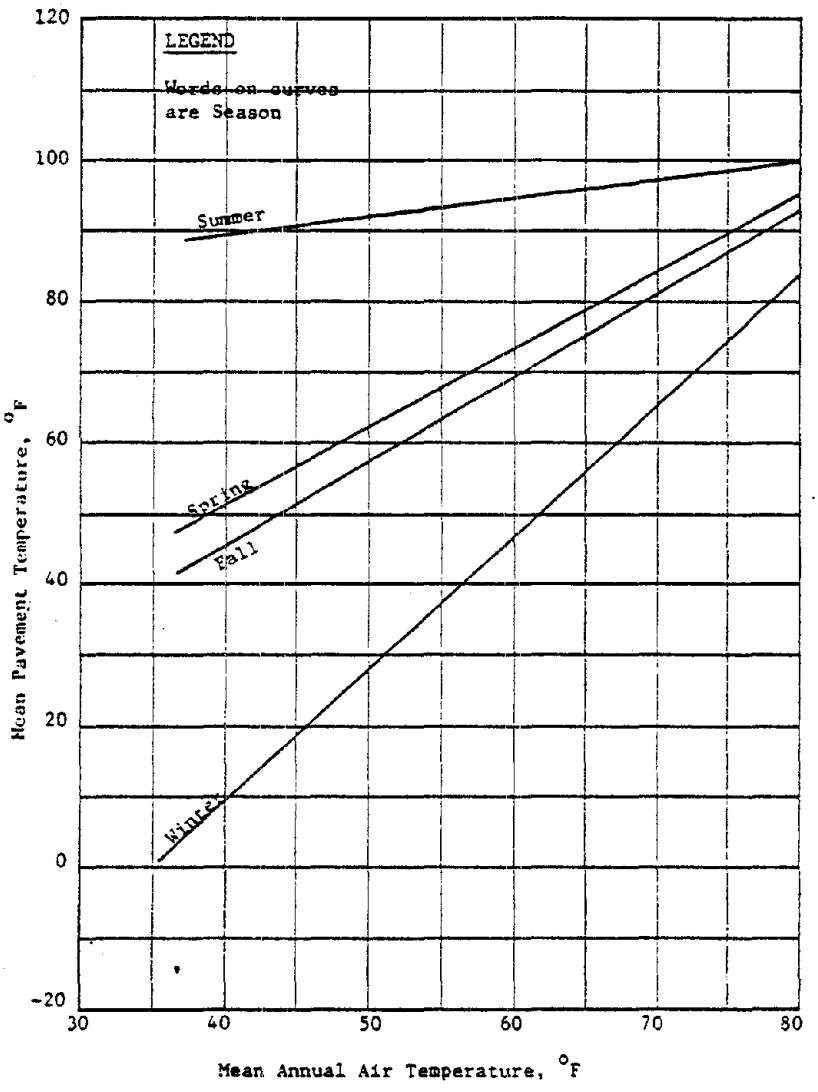
°C = (°F - 32)/1.8 1 psi = 6.89 kPa

Figure E.11. Worksheet for Computing the Asphalt Concrete Effective Modulus.



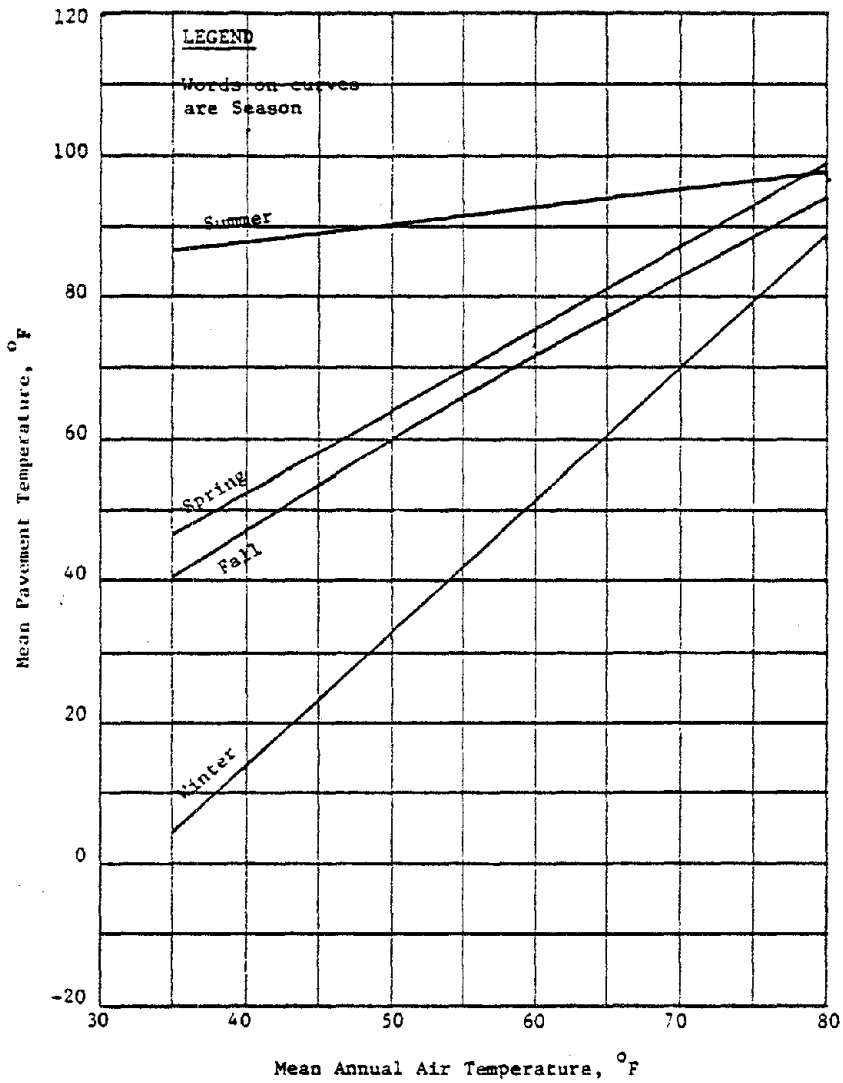
°C = (°F - 32)/1.8

Figure E.12.Estimation of the Mean Pavement Seasonal Temperature for Environmental Region No. 1.



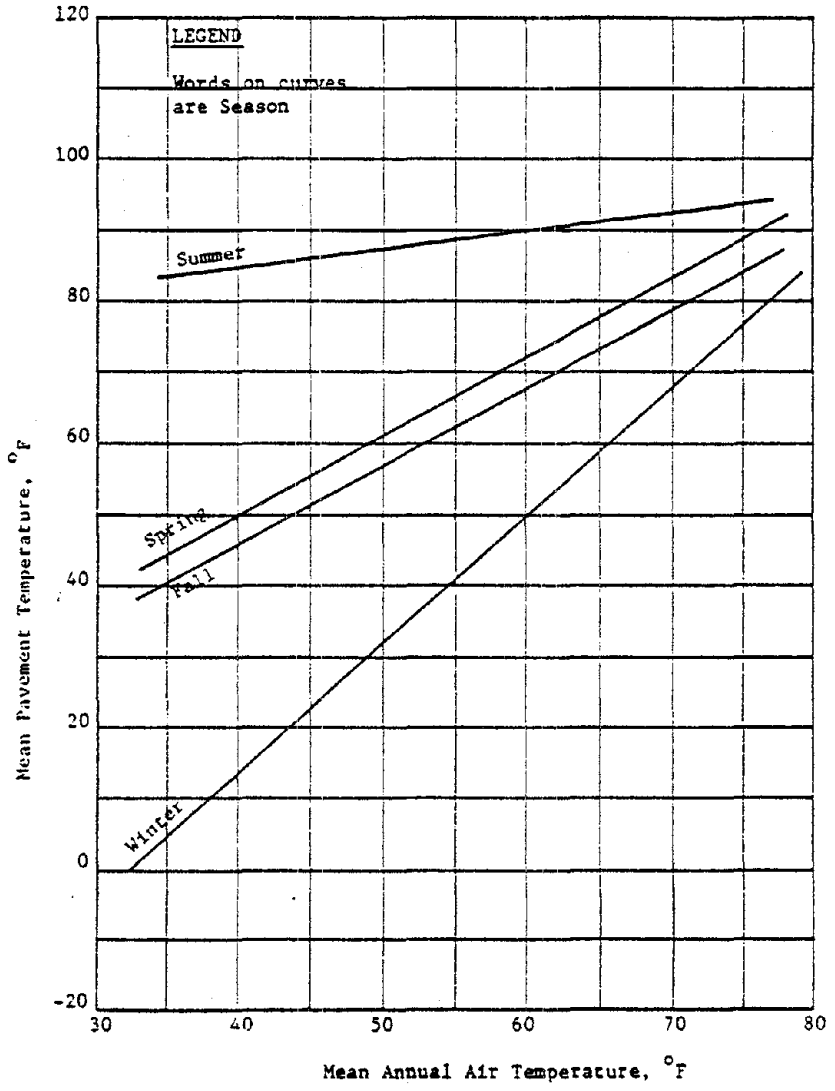
°C = (°F - 32)/1.8

Figure E.13. Estimation of the Mean Pavement Seasonal Temperature for Environmental Region No. 2.



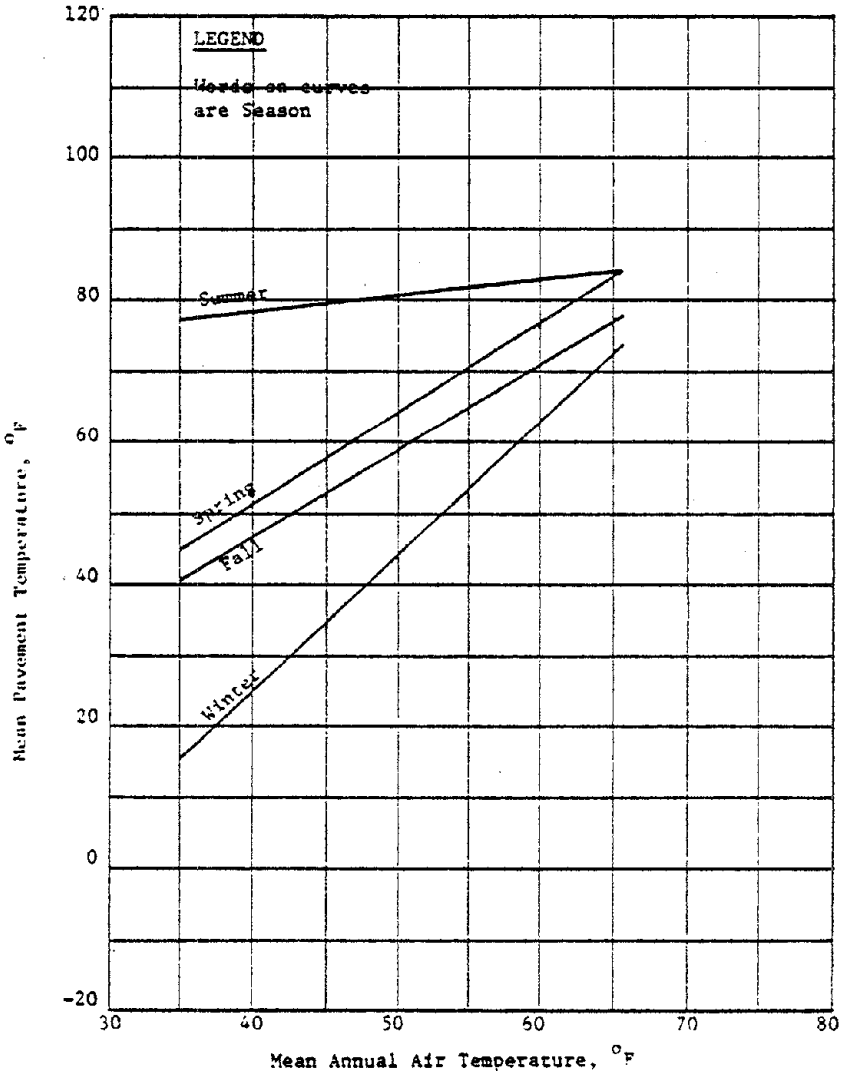
°C = (°F - 32)/1.8

Figure E.14. Estimation of the Mean Pavement Seasonal Temperature for Environmental Region No. 3 Under Dry Conditions (Average Yearly Rainfall < 12 in).



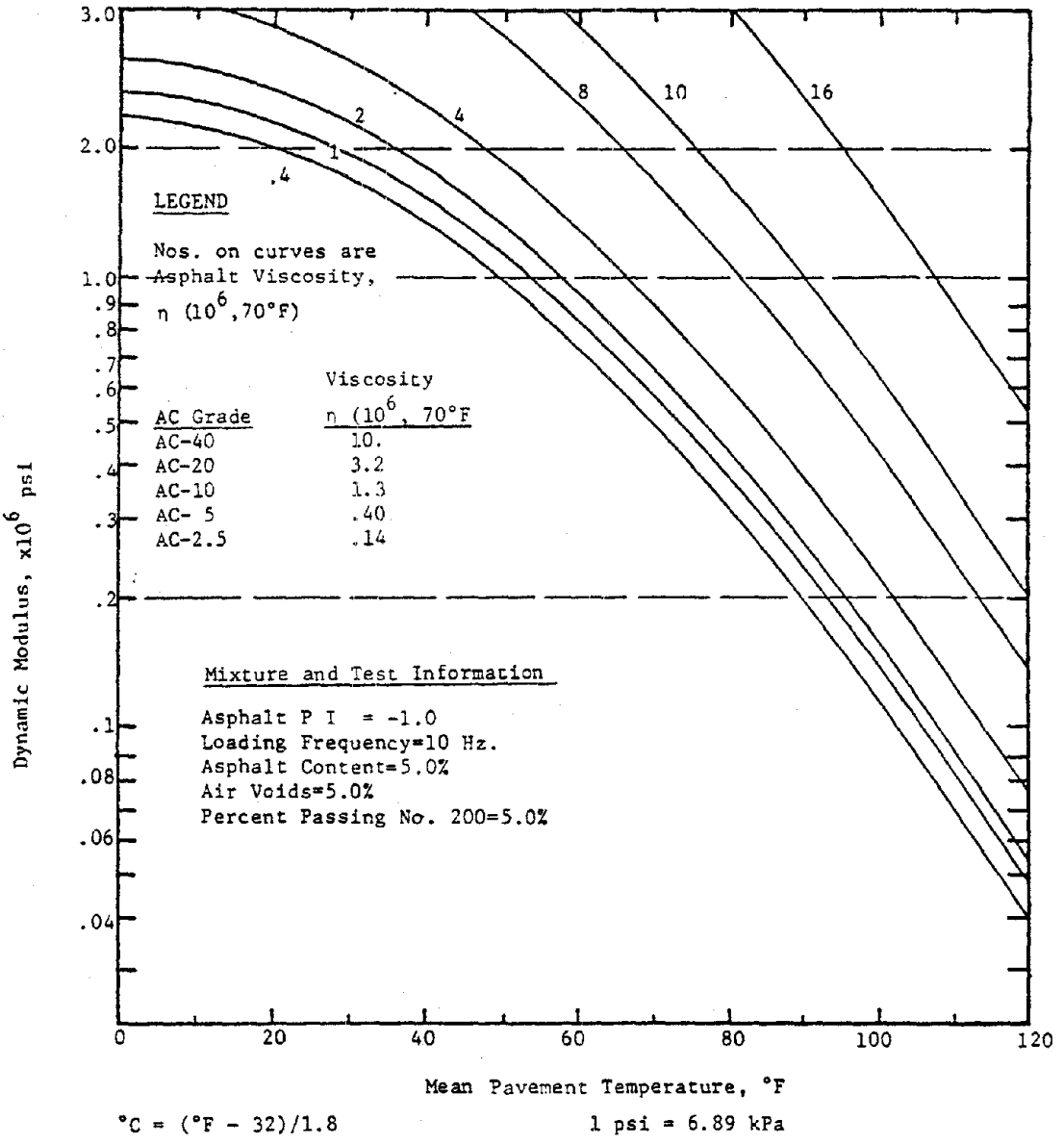
°C = (°F - 32)/1.8

Figure E.15. Estimation of the Mean Pavement Seasonal Temperature for Environmental Region No. 3 Under Wet Conditions (Average Yearly Rainfall > 12 in).



°C = (°F - 32)/1.8

Figure E.16. Estimation of the Mean Pavement Seasonal Temperature for Environmental Region No. 4.



°C = (°F - 32)/1.8 1 psi = 6.89 kPa

Figure E.17. Estimation of the asphalt concrete dynamic modulus as a function of mean pavement temperature.

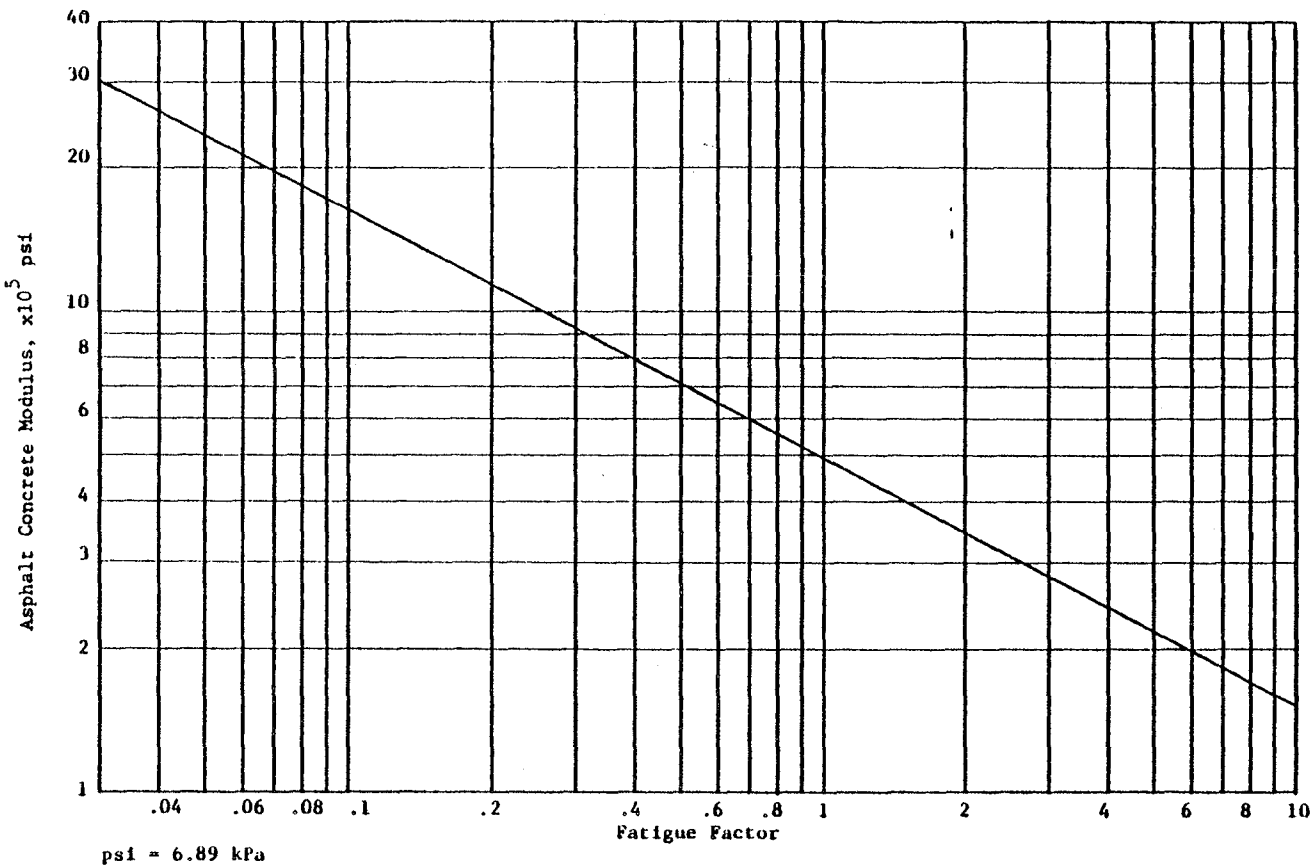
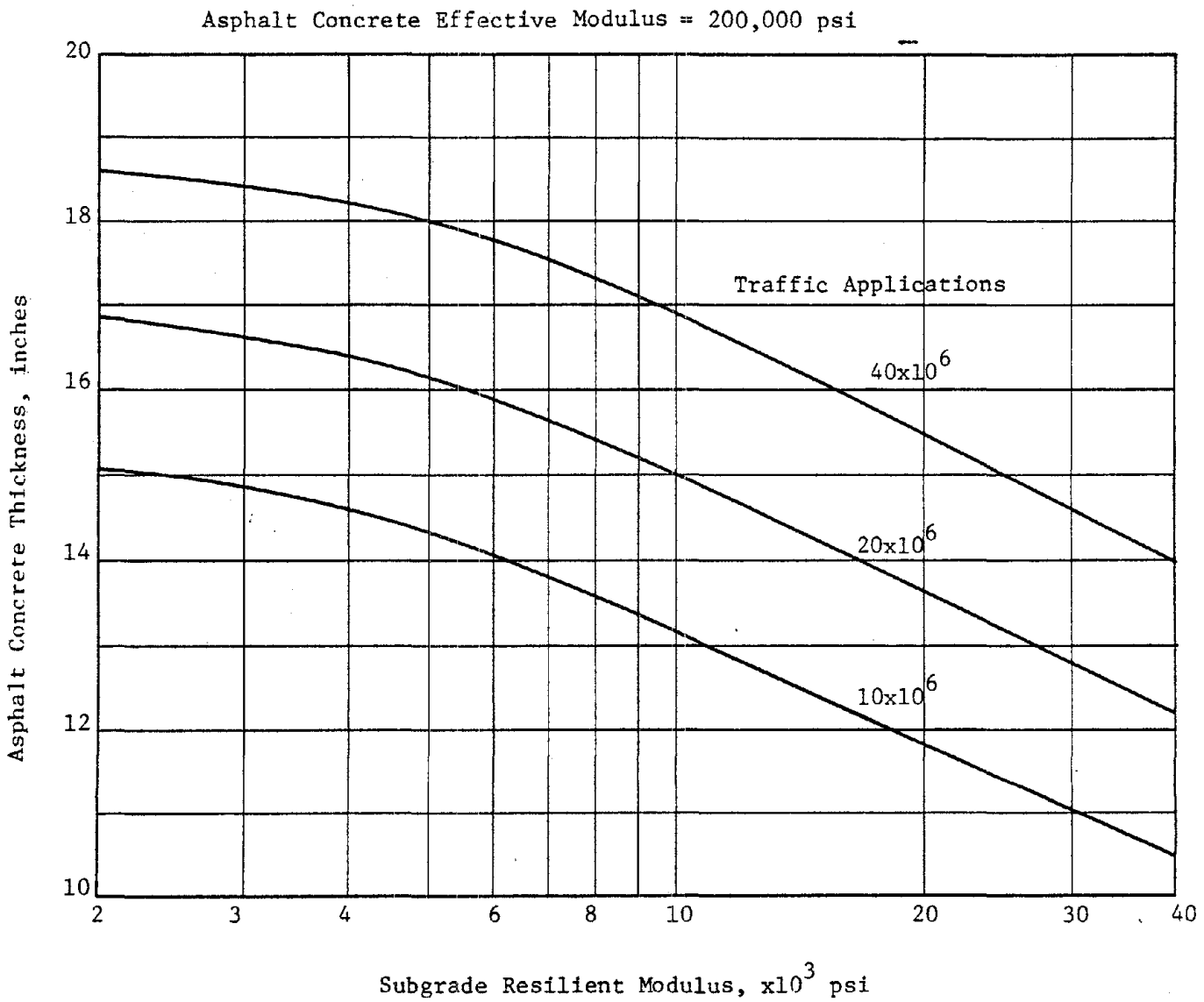
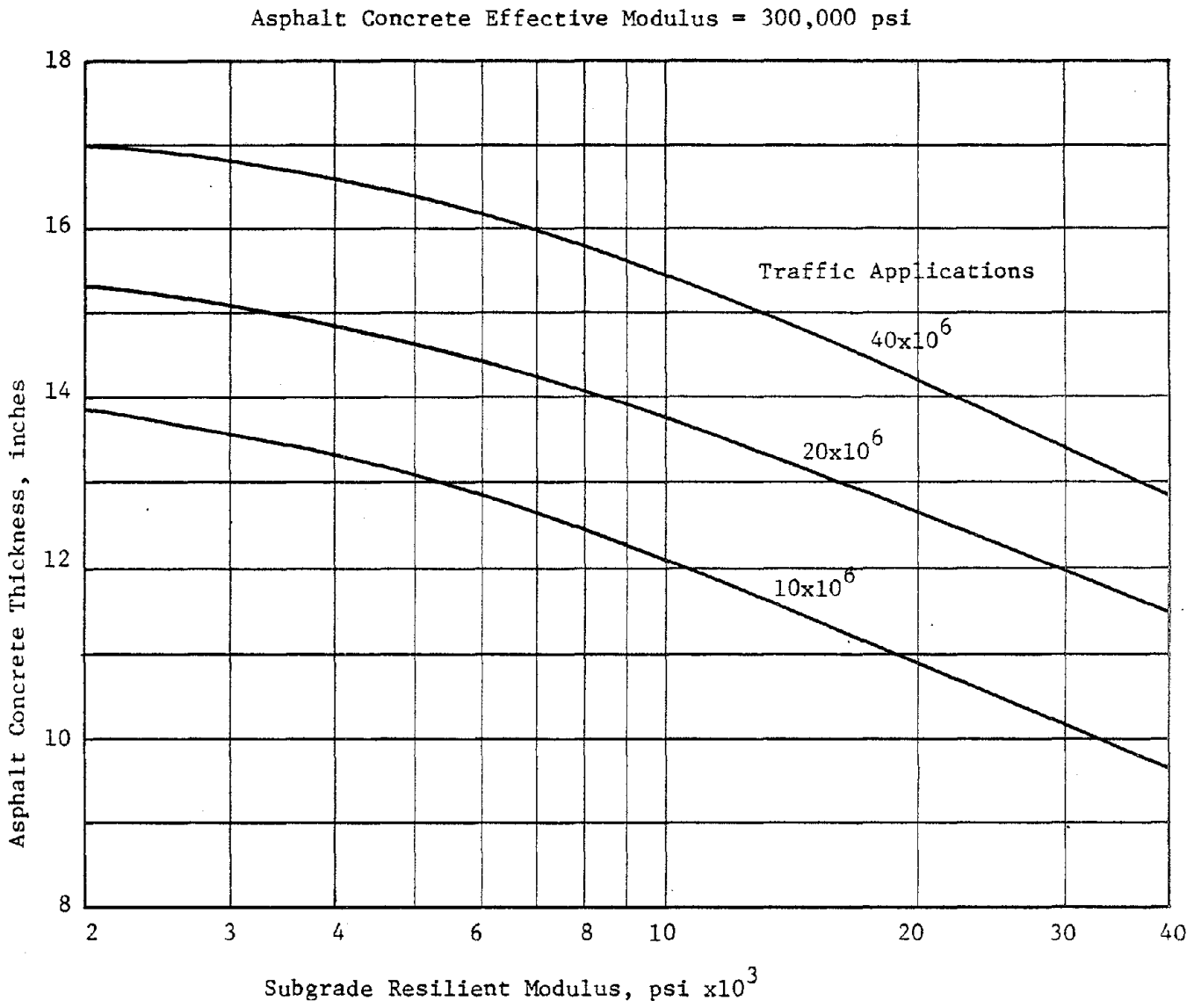


Figure E.18. Estimation of the fatigue factor.



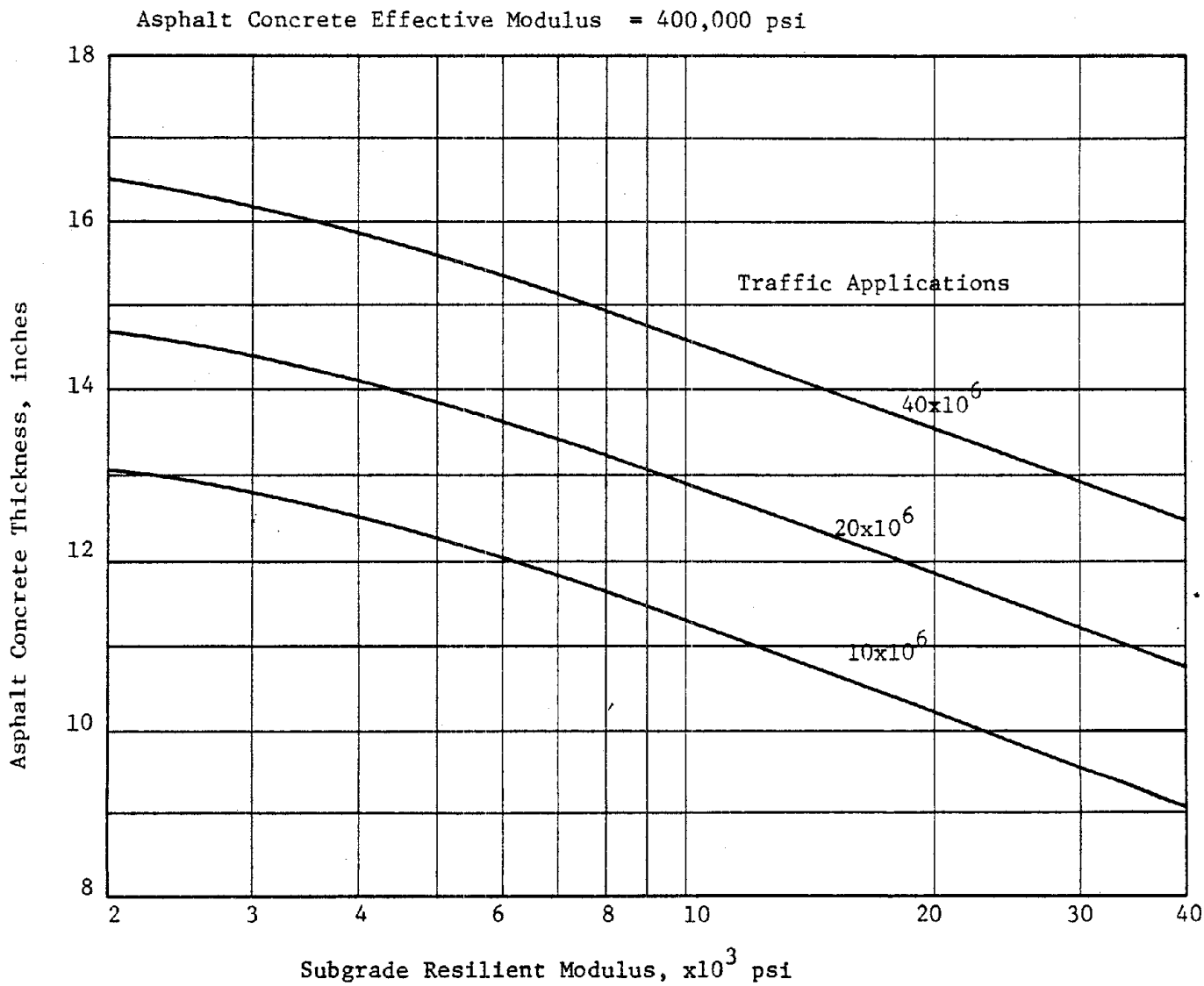
1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.19. Asphalt concrete surface thickness as required by fatigue cracking, rutting and roughness criteria.



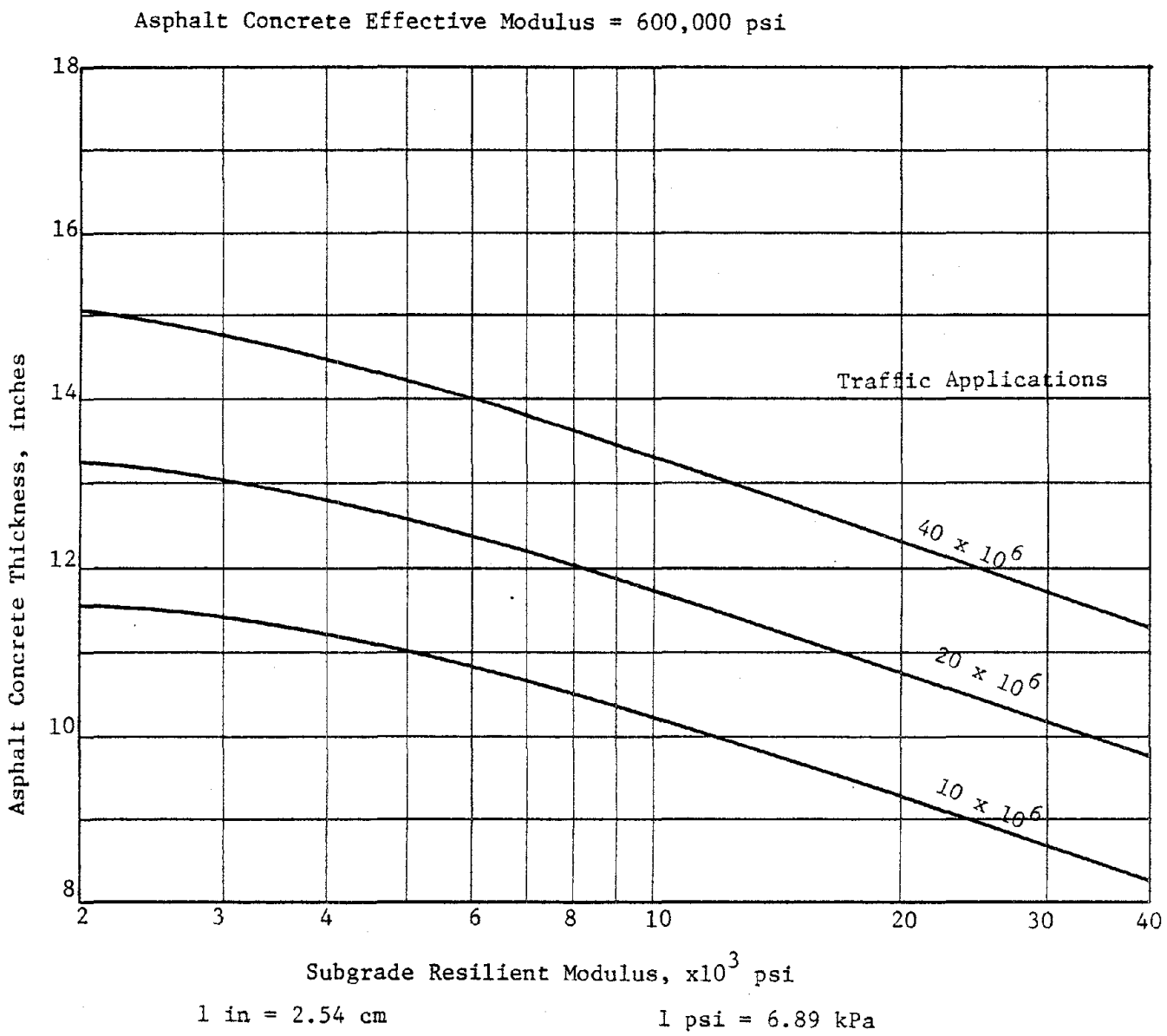
1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.20. Asphalt concrete surface thickness as required by fatigue cracking, rutting and roughness criteria.



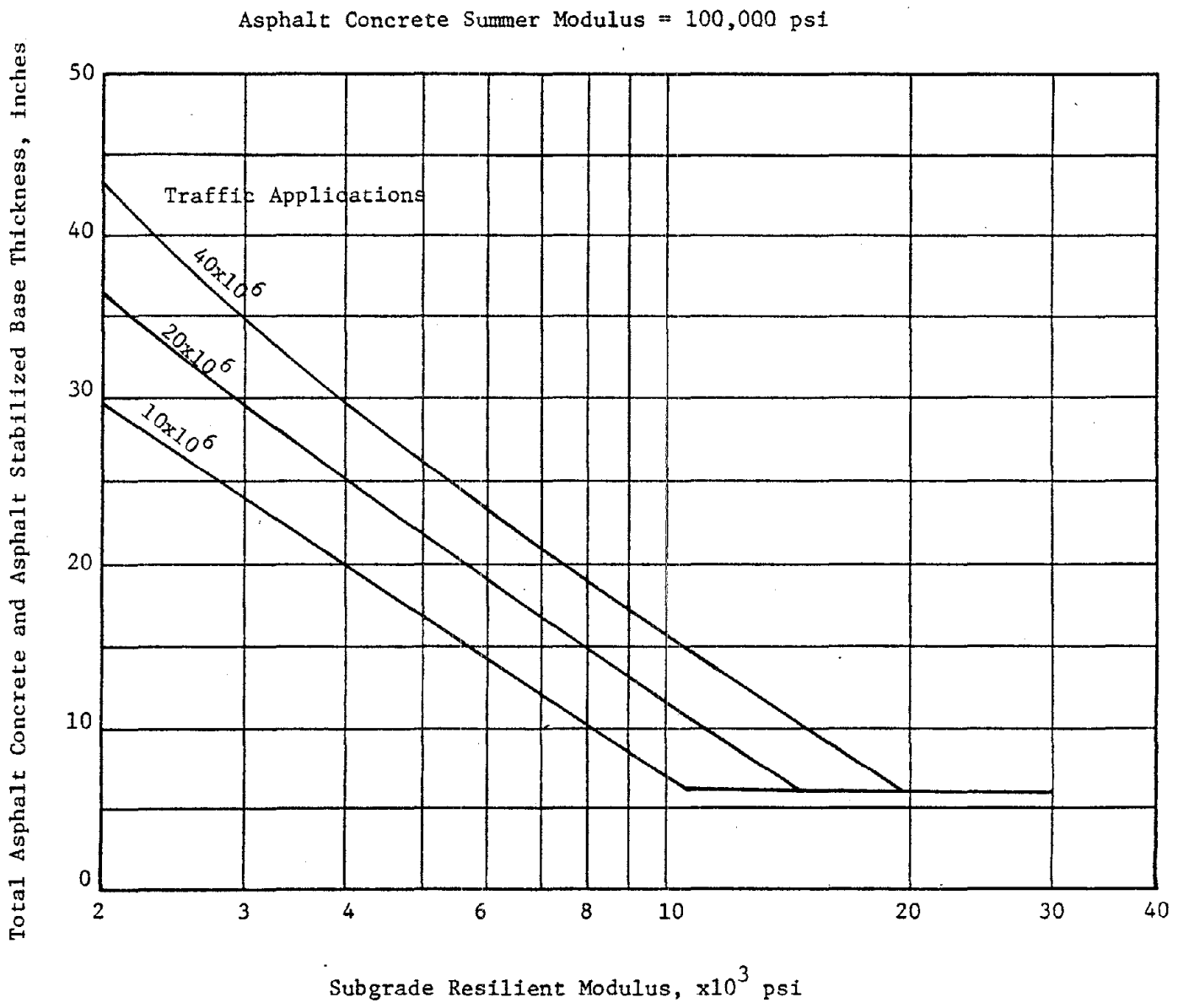
1 in = 2.54 cm 1 psi = 2.89 kPa

Figure E.21. Asphalt concrete surface thickness as required by fatigue cracking, rutting and roughness criteria.



1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.22. Asphalt concrete surface thickness as required by fatigue cracking, rutting and roughness criteria.



1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.23. Total pavement structure thickness required to meet the surface deflection and subgrade vertical compressive strain criteria.

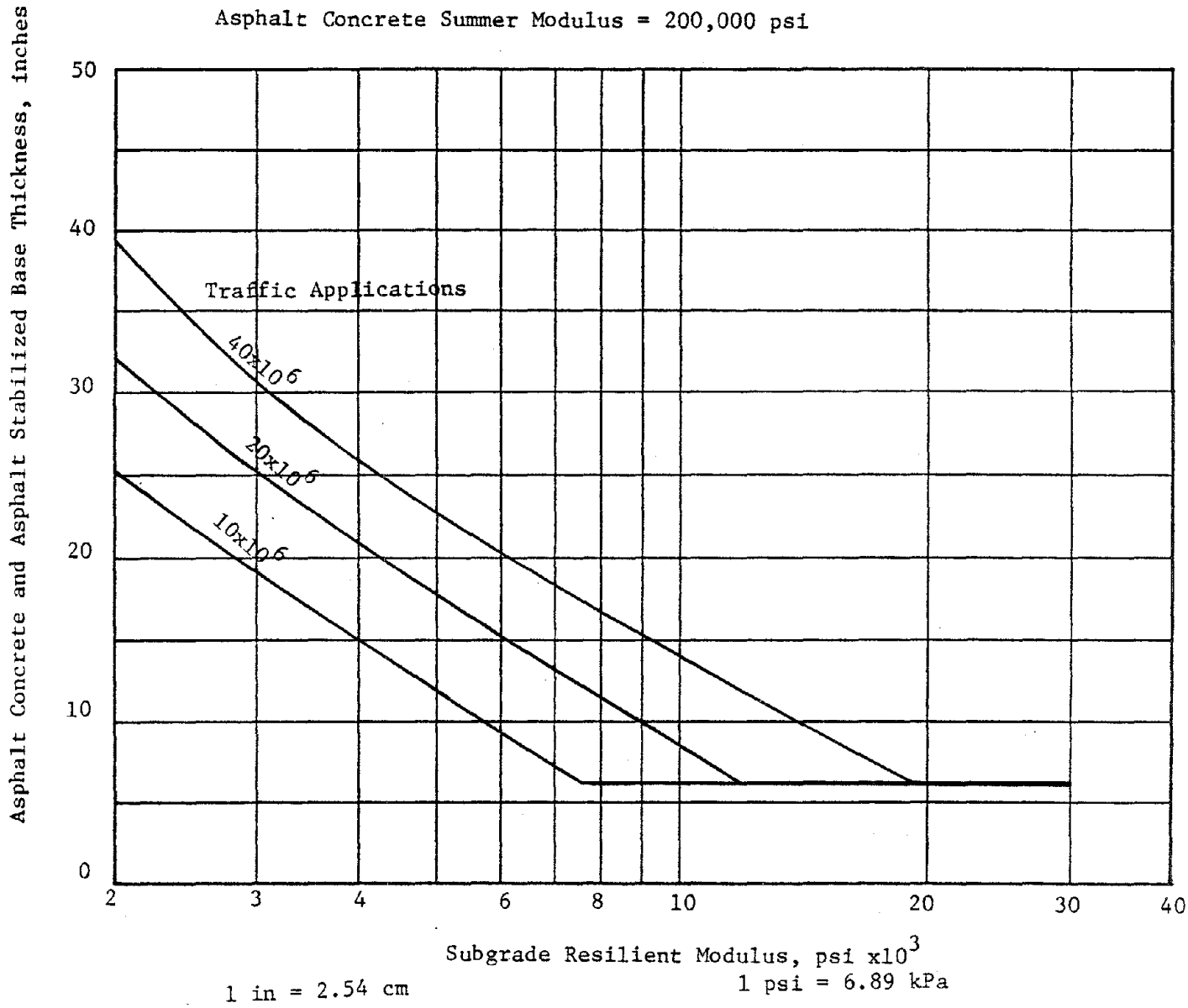
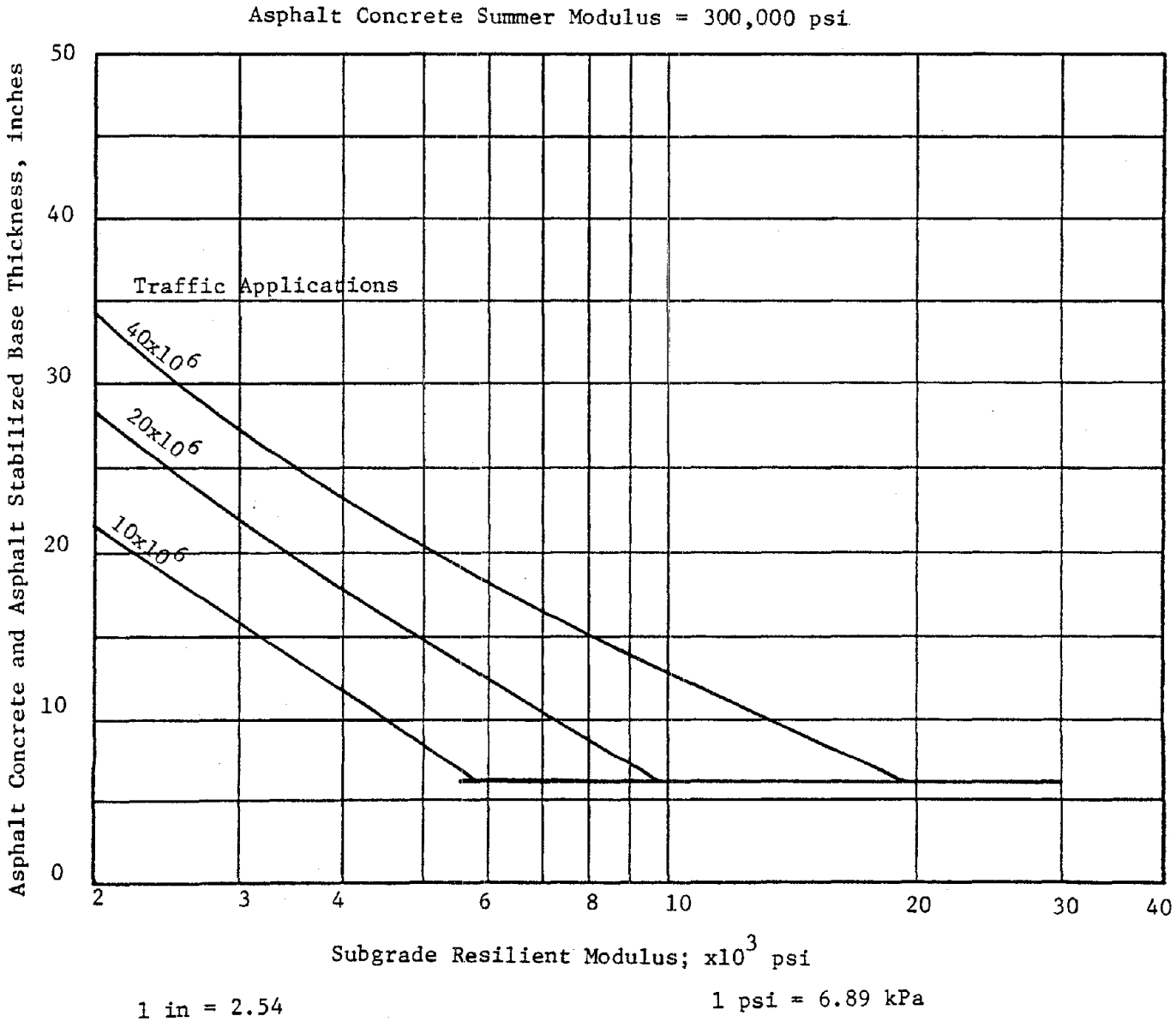
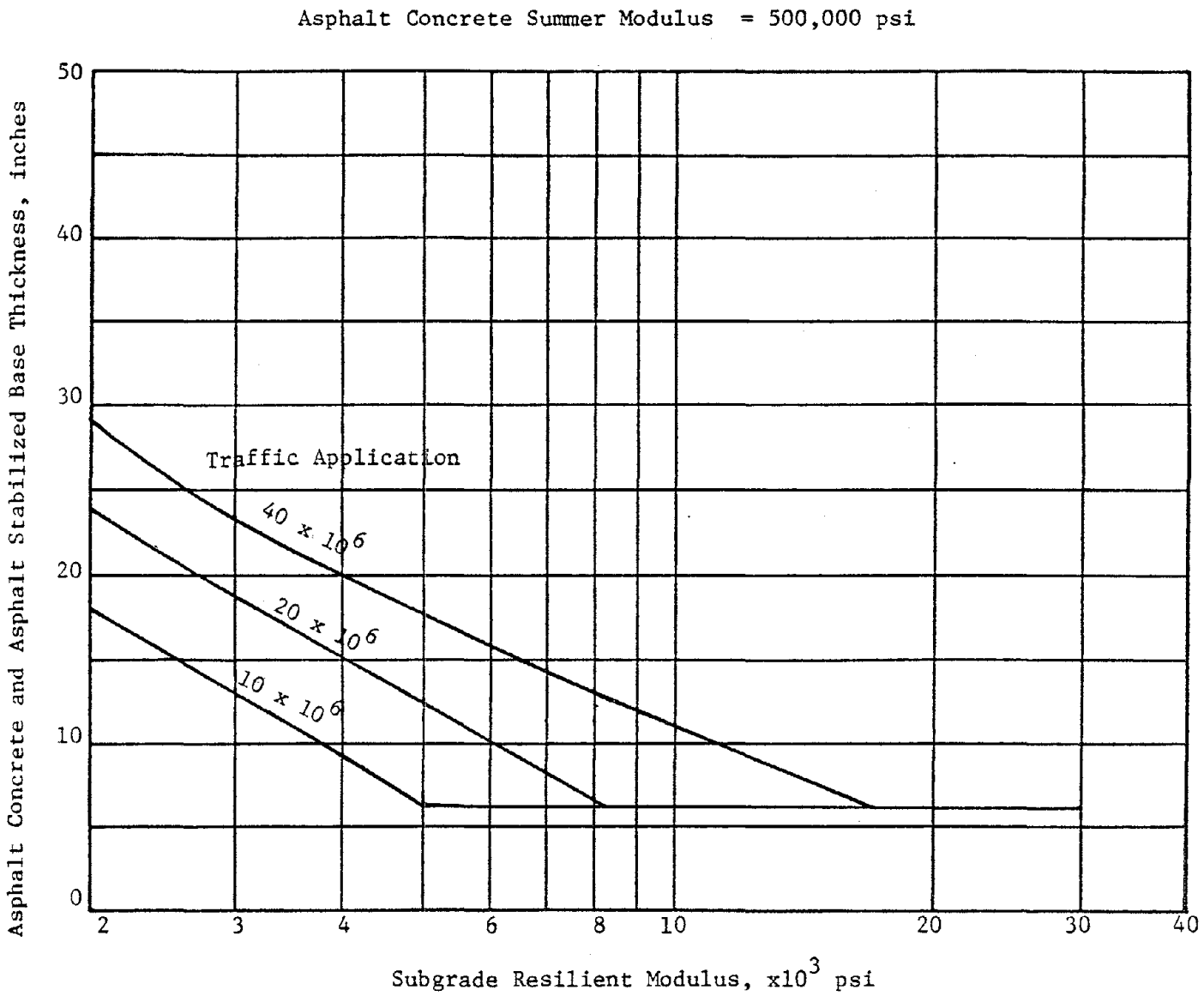


Figure E.24. Total pavement structure thickness required to meet the surface deflection and subgrade vertical compressive strain criteria.



1 in = 2.54 1 psi = 6.89 kPa

Figure E.25. Total pavement structure thickness required to meet the surface deflection and subgrade vertical compressive strain criteria.



1 in = 2.54 cm 1 psi = kPa

Figure E.26. Total pavement structure thickness required to meet the surface deflection and subgrade vertical compressive strain criteria.

compressive strain in conjunction with limiting the tensile stress and strain in the stabilized base layer.

Asphalt Concrete Over Unbound Granular Base

The modulus of an unbound base material depends on its density, moisture content, thickness, the modulus of the subgrade and other factors. Therefore, in the design of the surfacing layers it was assumed that sufficient base and subbase materials would be placed so that a resilient modulus of 70,000 psi (483 MPa) would be produced at the surface of the base. The required thickness of unbound granular base and subbase layers to achieve the specified resilient modulus of 70,000 psi (483 MPa) at the surface of the base can be determined using Figure E.27. Therefore, special care must be taken in selecting granular base materials to ensure that the material selected is capable of developing this modulus. Figure E.28 is then used to determine the required thickness of asphalt concrete as a function of traffic applications and the effective asphalt concrete modulus calculated on Worksheet D. Figures E.29-E.31 are used to derive the thickness requirements for cement stabilized base layers, based on the surface deflection and subgrade strain criteria. It should be understood that when the cement stabilized base layer exceeds 6 inches, (15.2 cm), the unbound granular base pavement cross-section will not meet the surface deflection criteria established for premium pavements.

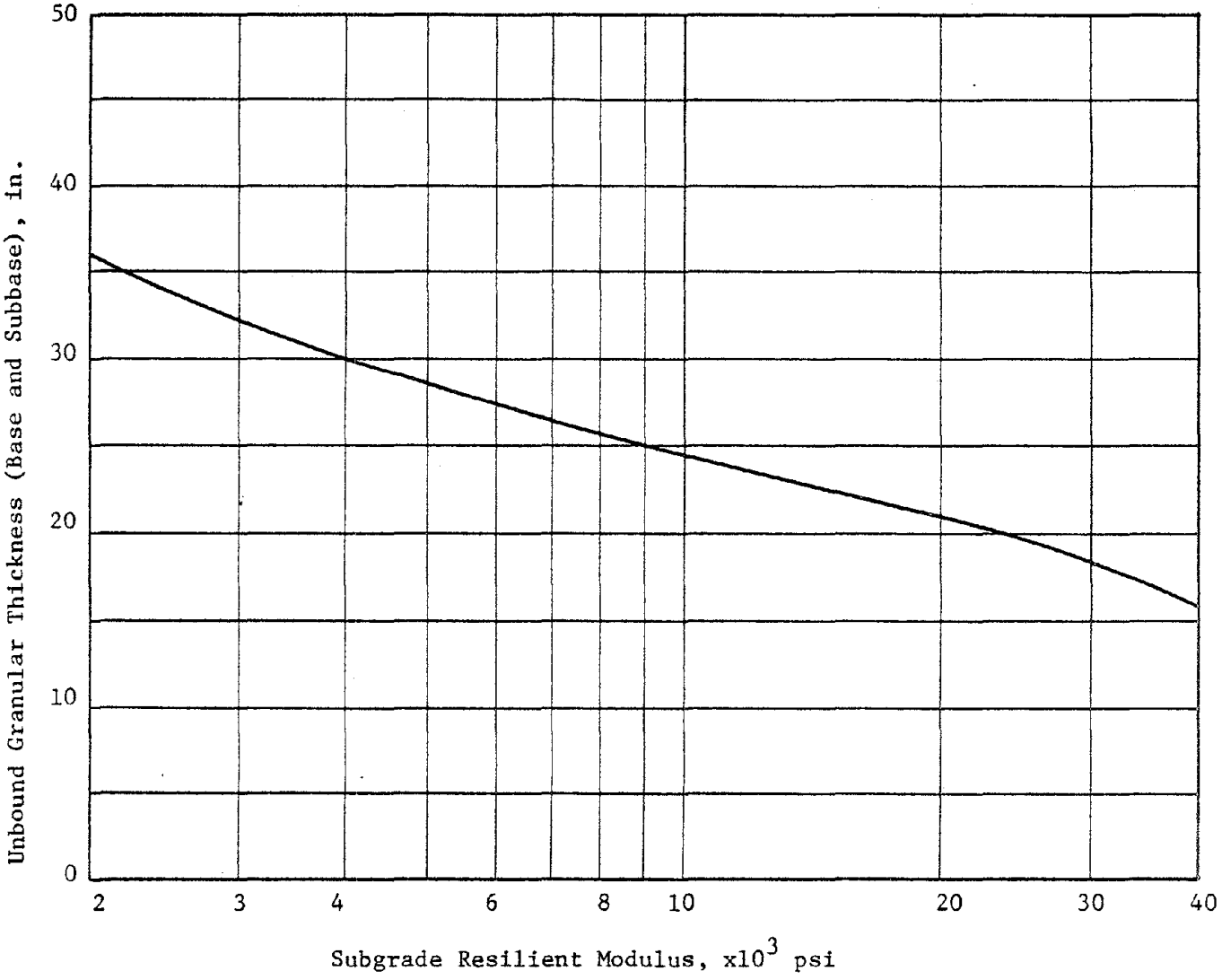
Composite Pavement

The minimum asphalt concrete thickness required over a CRC rigid base is 3 inches (7.6 cm) to minimize curling stresses. The asphalt concrete material placed over a jointed concrete rigid base consists of 3 inches of an asphalt crack relief layer and 2 inches (5.0 cm) of dense graded asphalt concrete. With this surface thickness, Figures E.32-E.37 are used to determine the thickness of low modulus concrete required to resist fatigue cracking and retain an acceptable level of PSI. The selection of a particular design chart (Figs. E.32-E.37) is based on the 28-day concrete flexural strength, determined by ASTM C-78 or AASHTO T-97.

Figures E.38 and E.39 derive the thickness requirements of asphalt and cement stabilized base layers for the low-modulus concrete base of composite pavements, respectively.

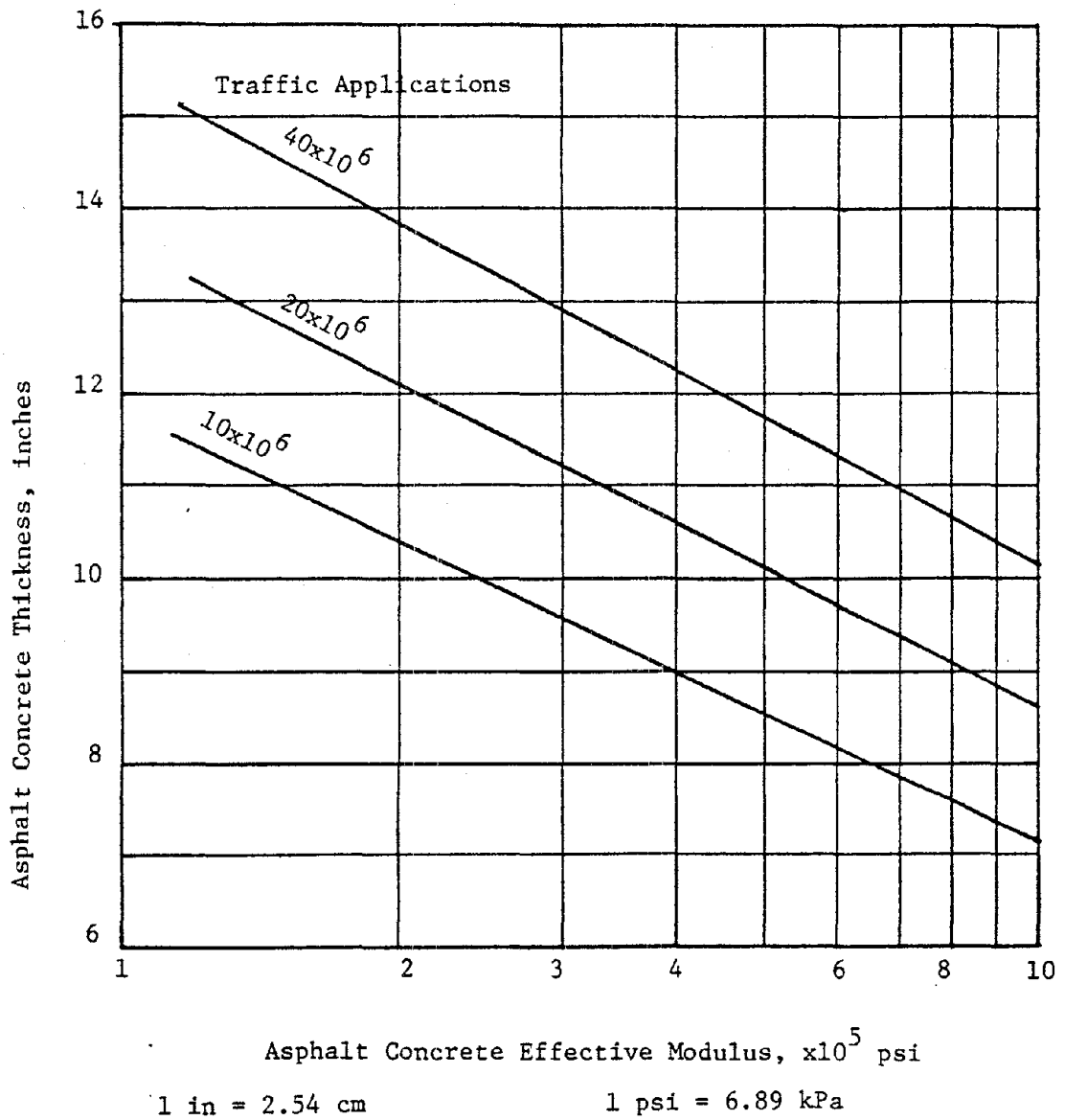
ADJUSTMENT FOR FROST SUSCEPTIBILITY

Using one of Figures E.40 through E.42, the frost penetration below the pavement surface is estimated for the pavement cross-section using the natural soil type and the environmental region. If the subgrade soil is frost susceptible, two design alternates are possible. The engineer may elect to either totally protect the subgrade from frost



1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.27. Unbound granular thickness required in flexible pavements.



1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.28. Asphalt concrete surface thickness as required by fatigue cracking, rutting and roughness criteria.

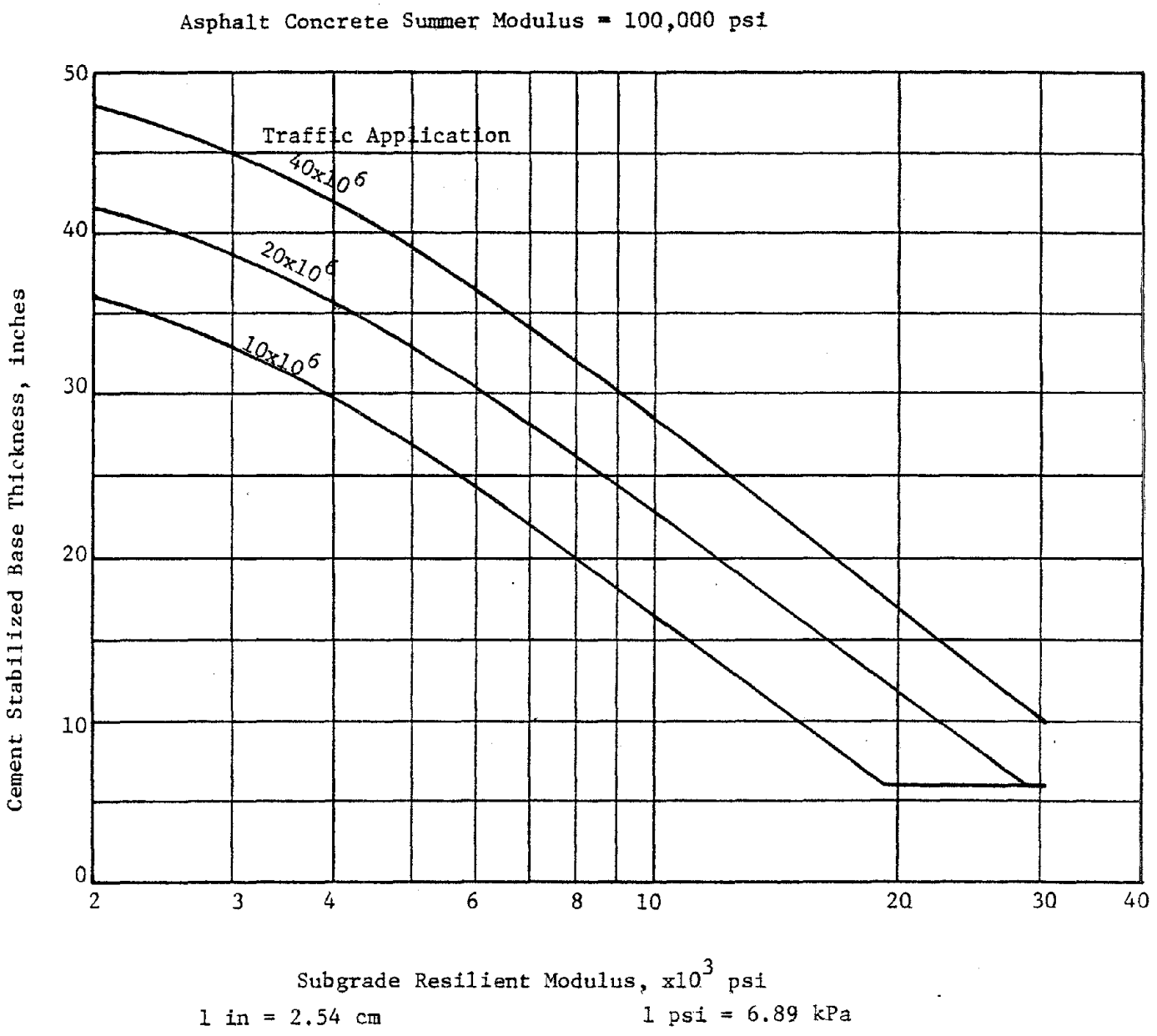
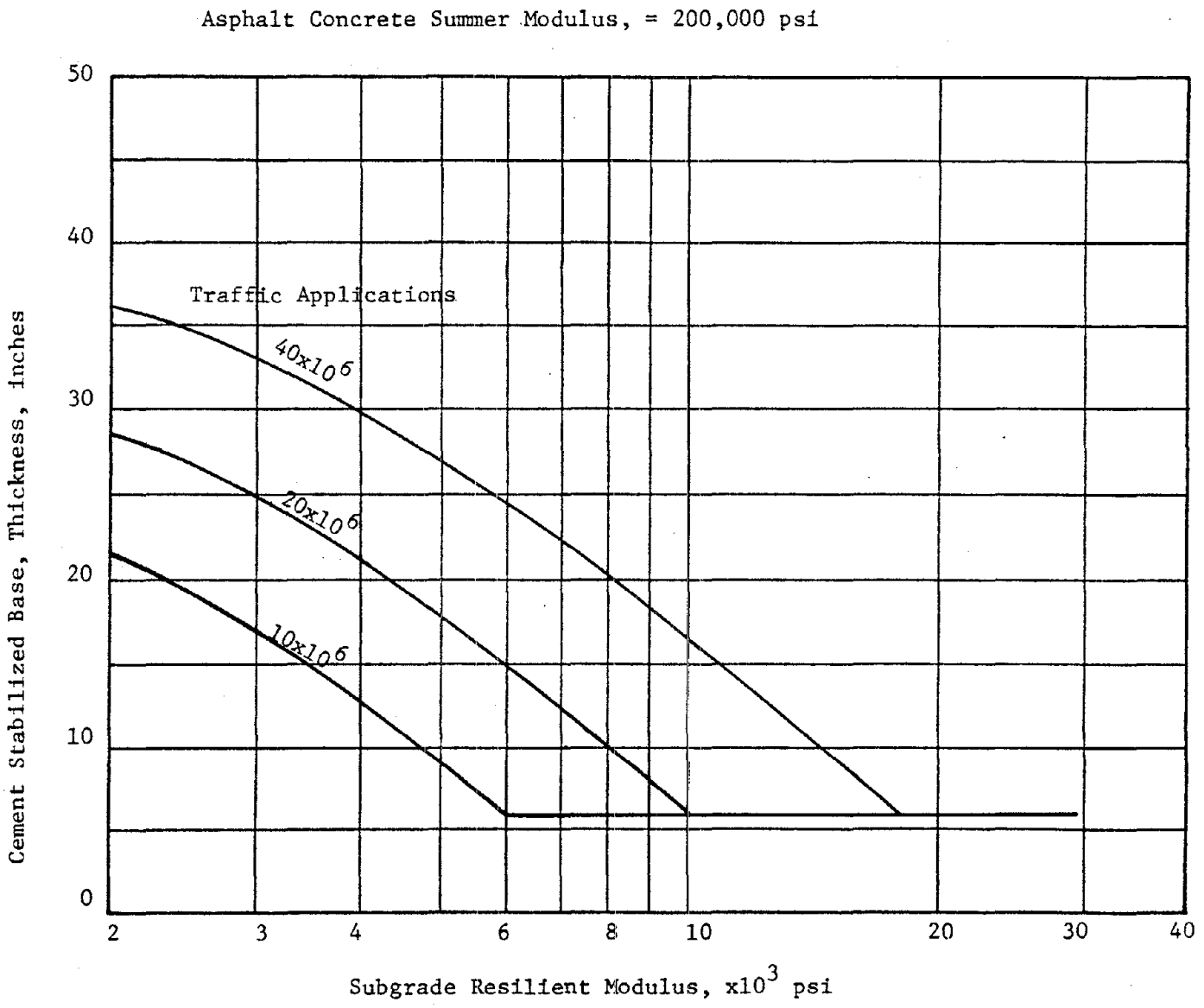
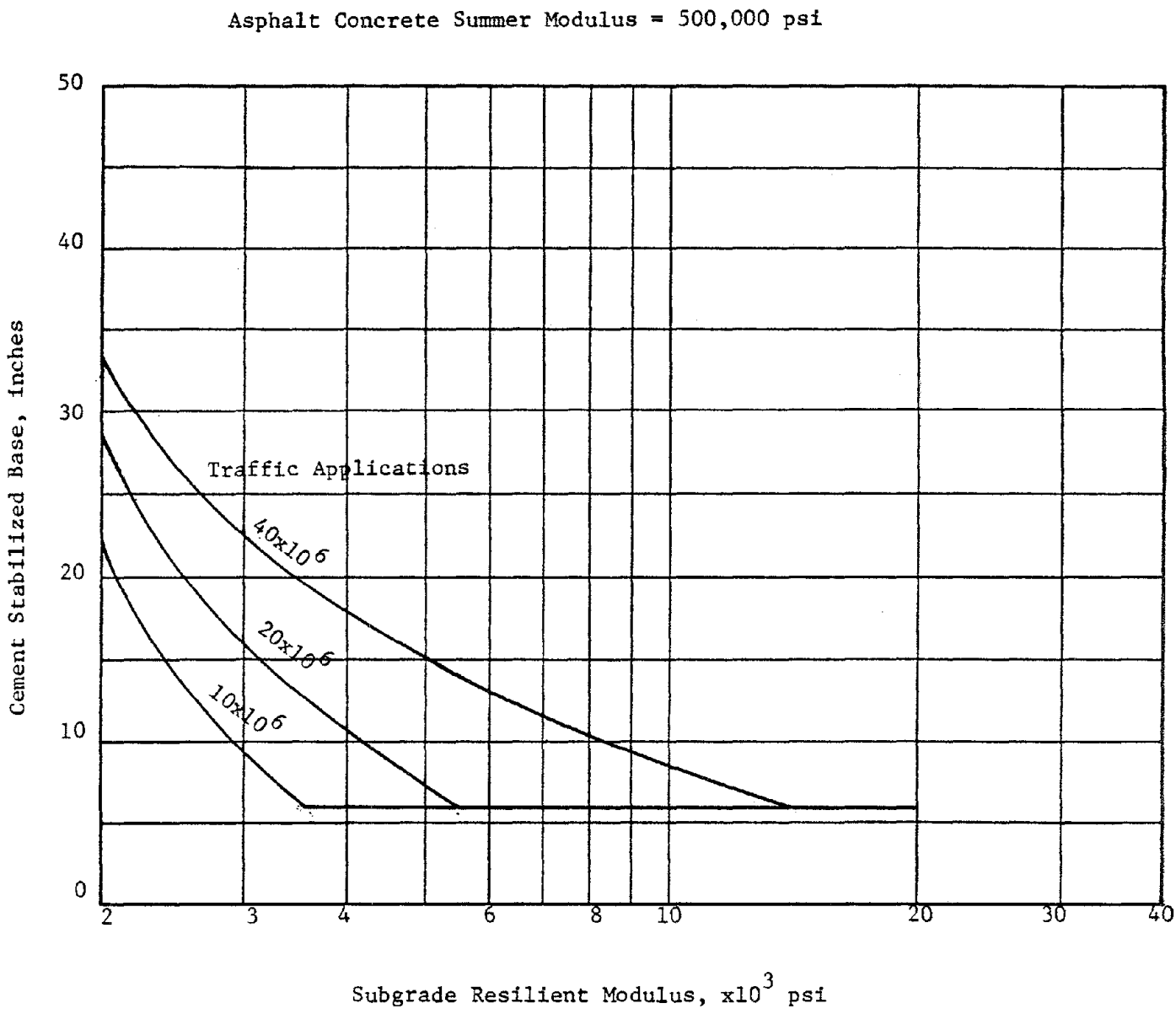


Figure E.29. Total pavement structure thickness required to meet the surface deflection and subgrade vertical compressive strain criteria.



1 in = 2.54 cm 1 psi = 6.89 kPa

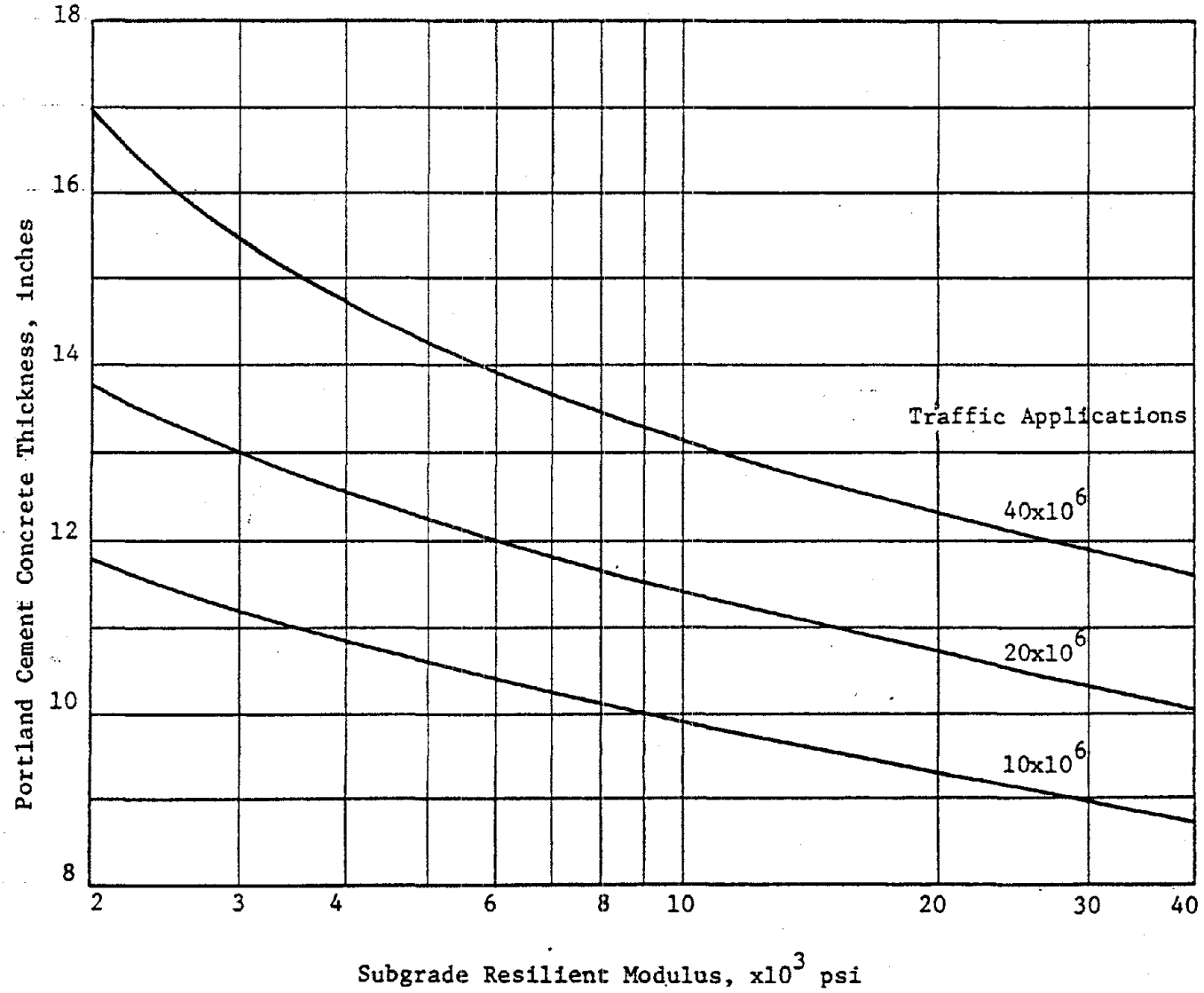
Figure E.30. Total pavement structure thickness required to meet the surface deflection and subgrade vertical compressive strain criteria.



1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.31. Total pavement structure thickness required to meet the surface deflection and subgrade vertical compressive strain criteria.

Concrete Flexural Strength = 450 psi (28 Day Strength)



1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.32. Low-modulus CRC thickness required by fatigue and roughness criteria.

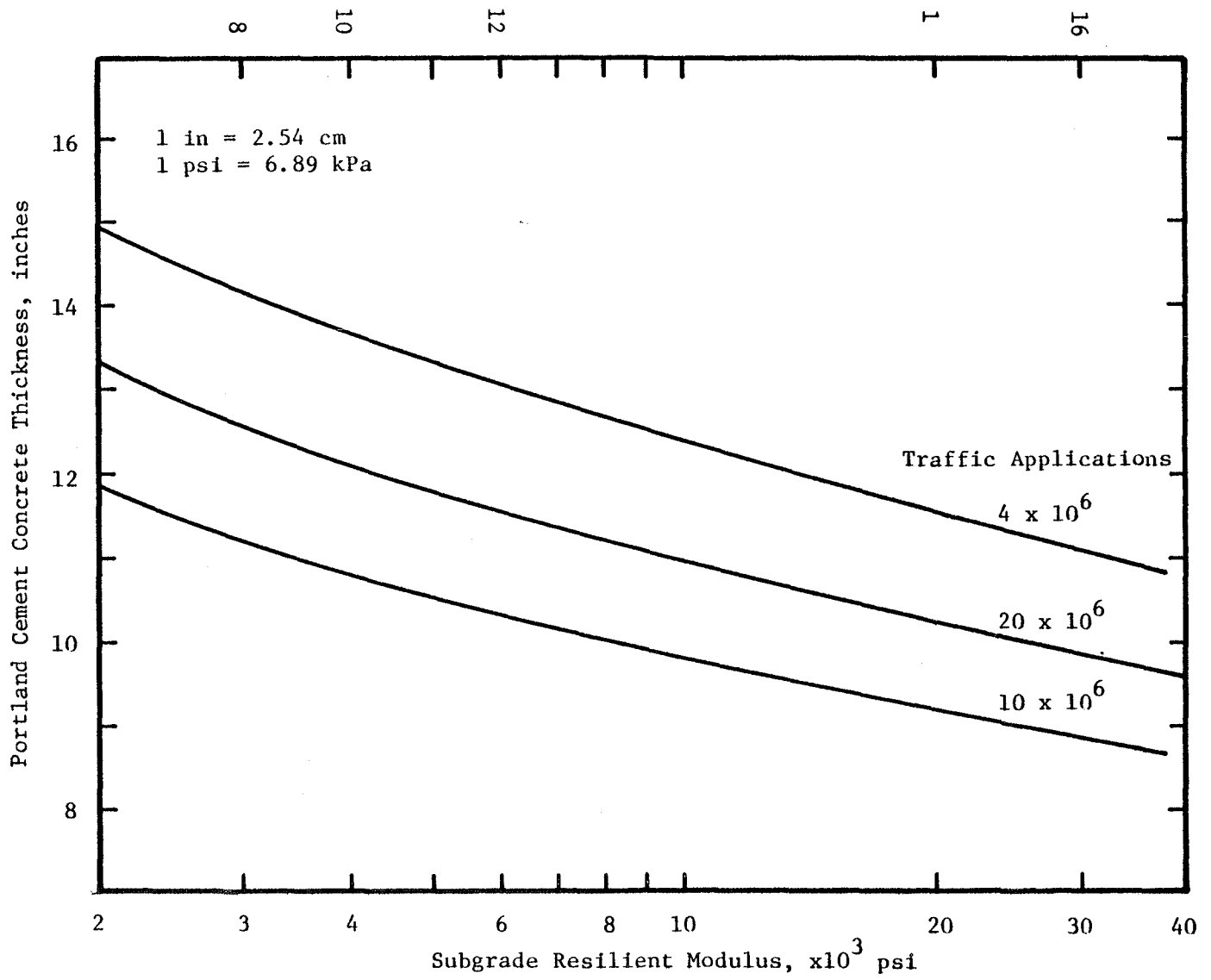


Figure E.33. Low-modulus CRC thickness required by fatigue and roughness criteria for composite pavements with f_r = 500 psi.

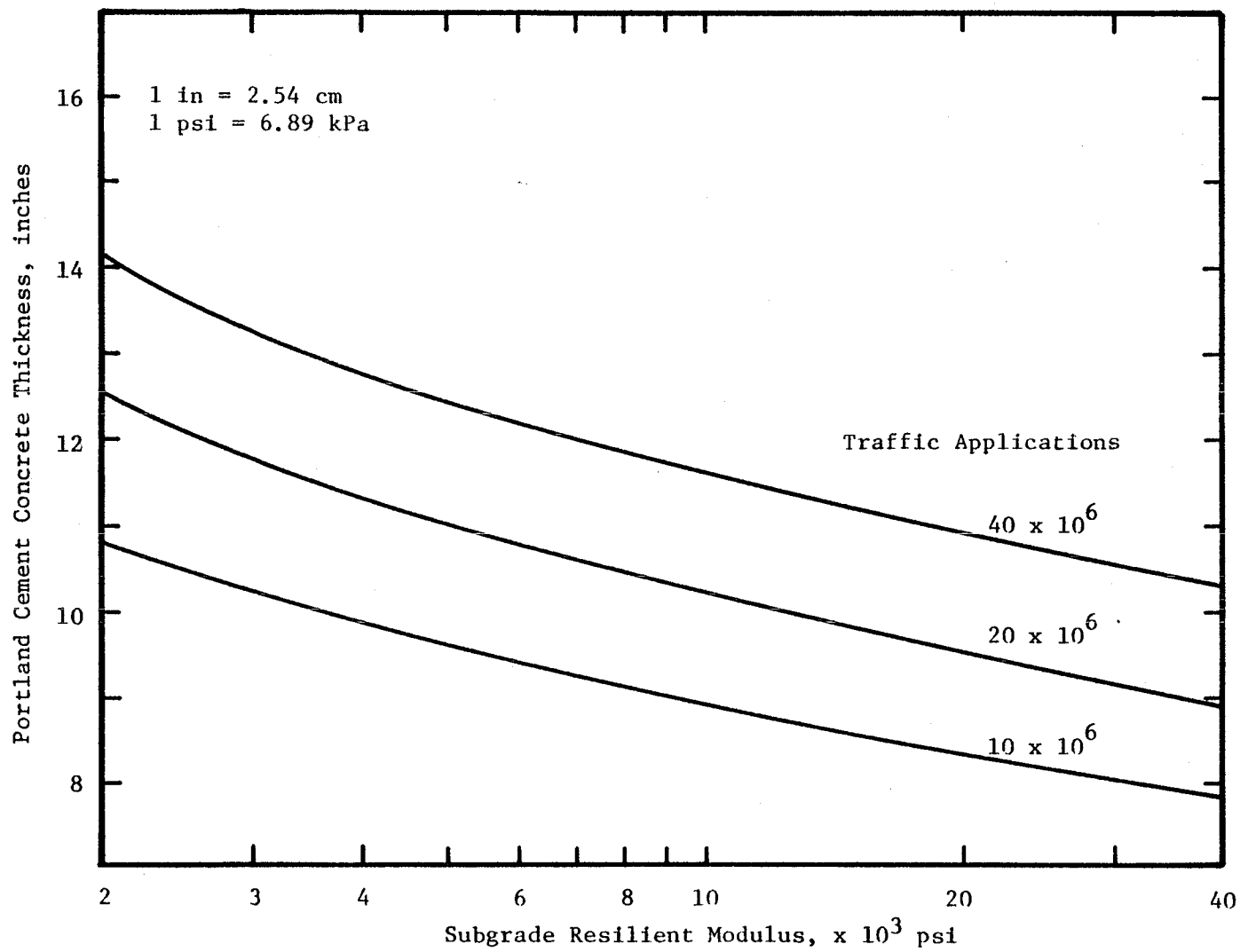


Figure E.34. Low-modulus CRC thickness required by fatigue and roughness criteria for composite pavement with f_r = 550 psi.

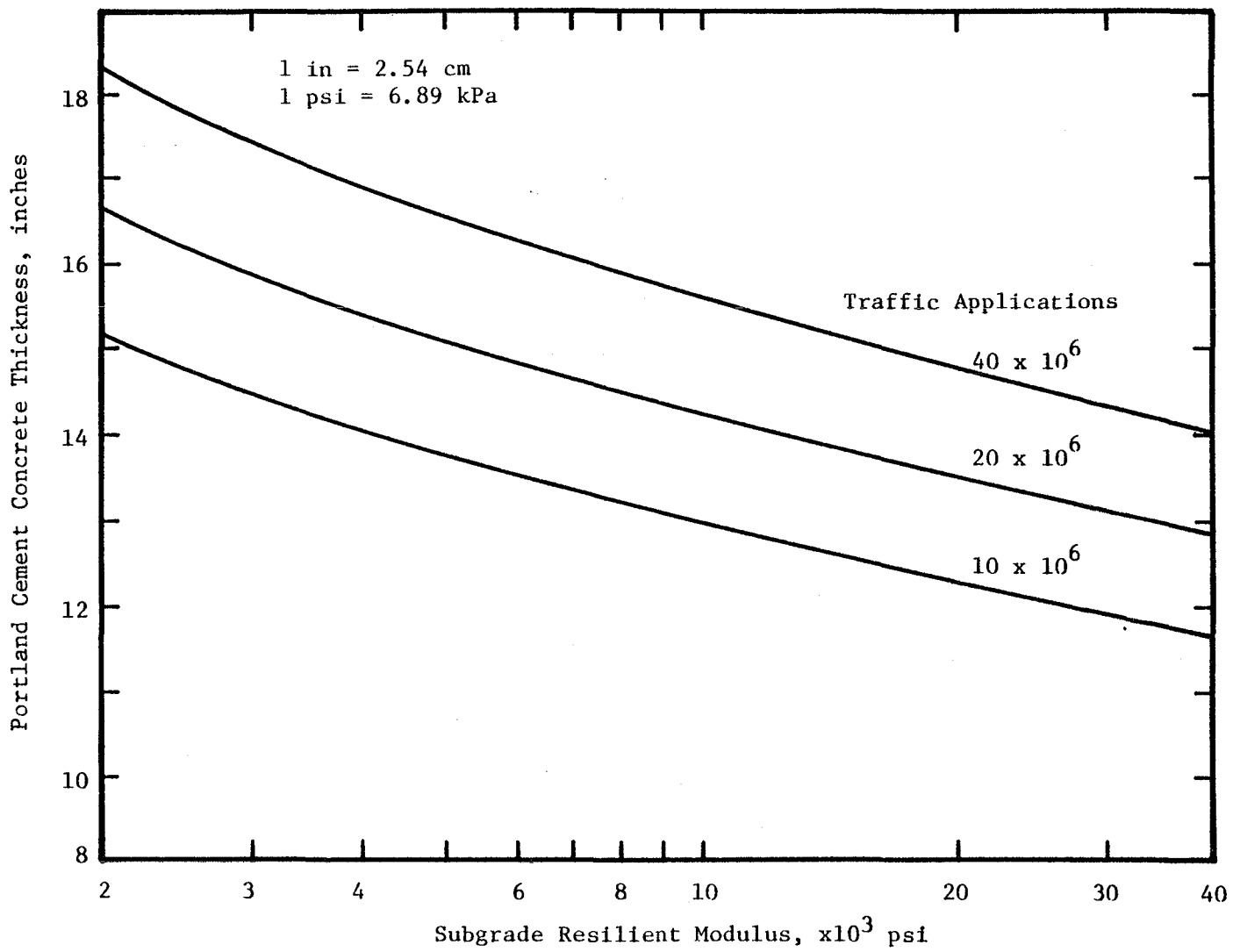


Figure E.35. Low modulus JCP thickness required by fatigue and roughness criteria for composite pavement with f_r = 450 psi.

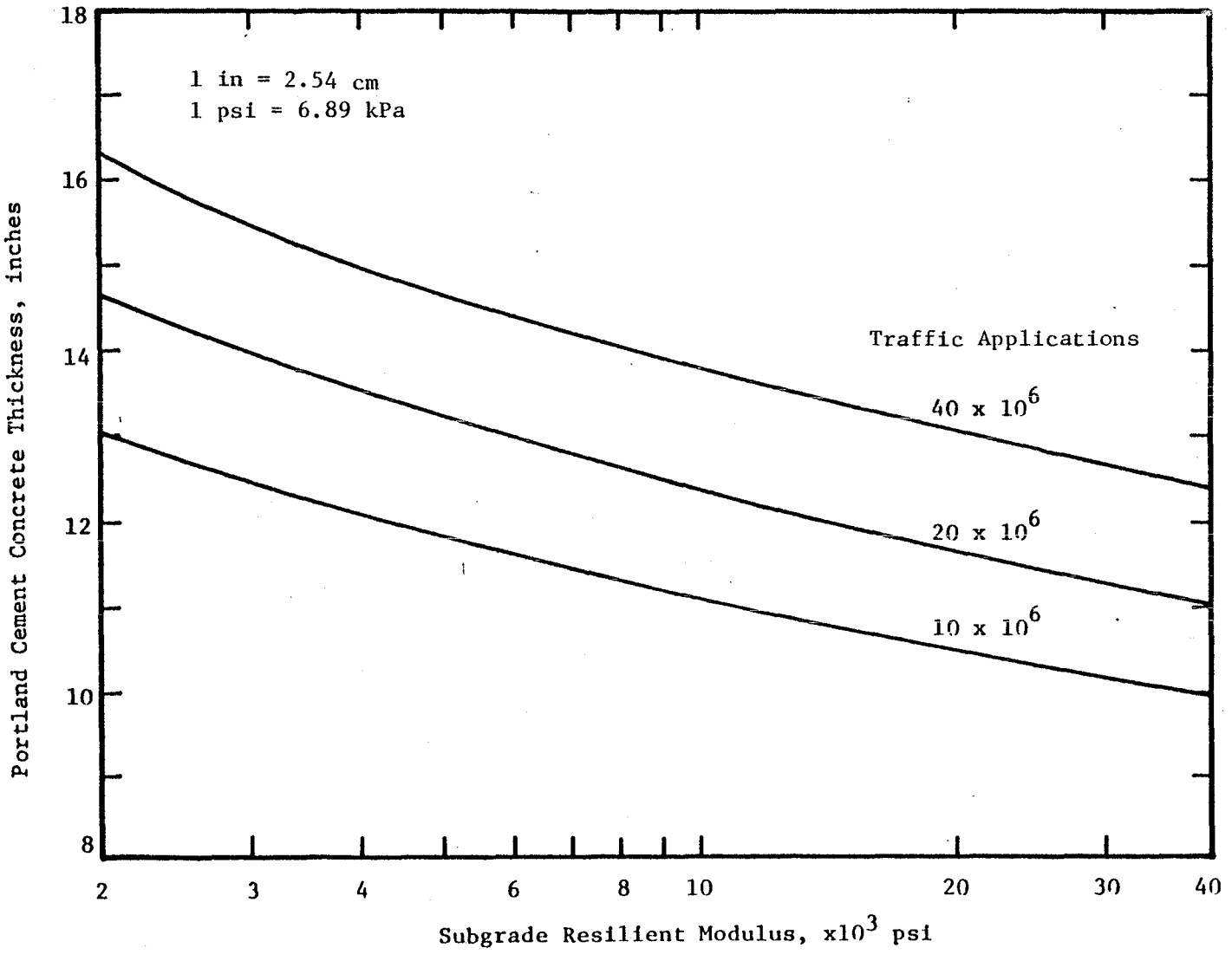


Figure E.36. Low modulus JCP thickness required by fatigue and roughness criteria for composite pavement with f_r = 500 psi.

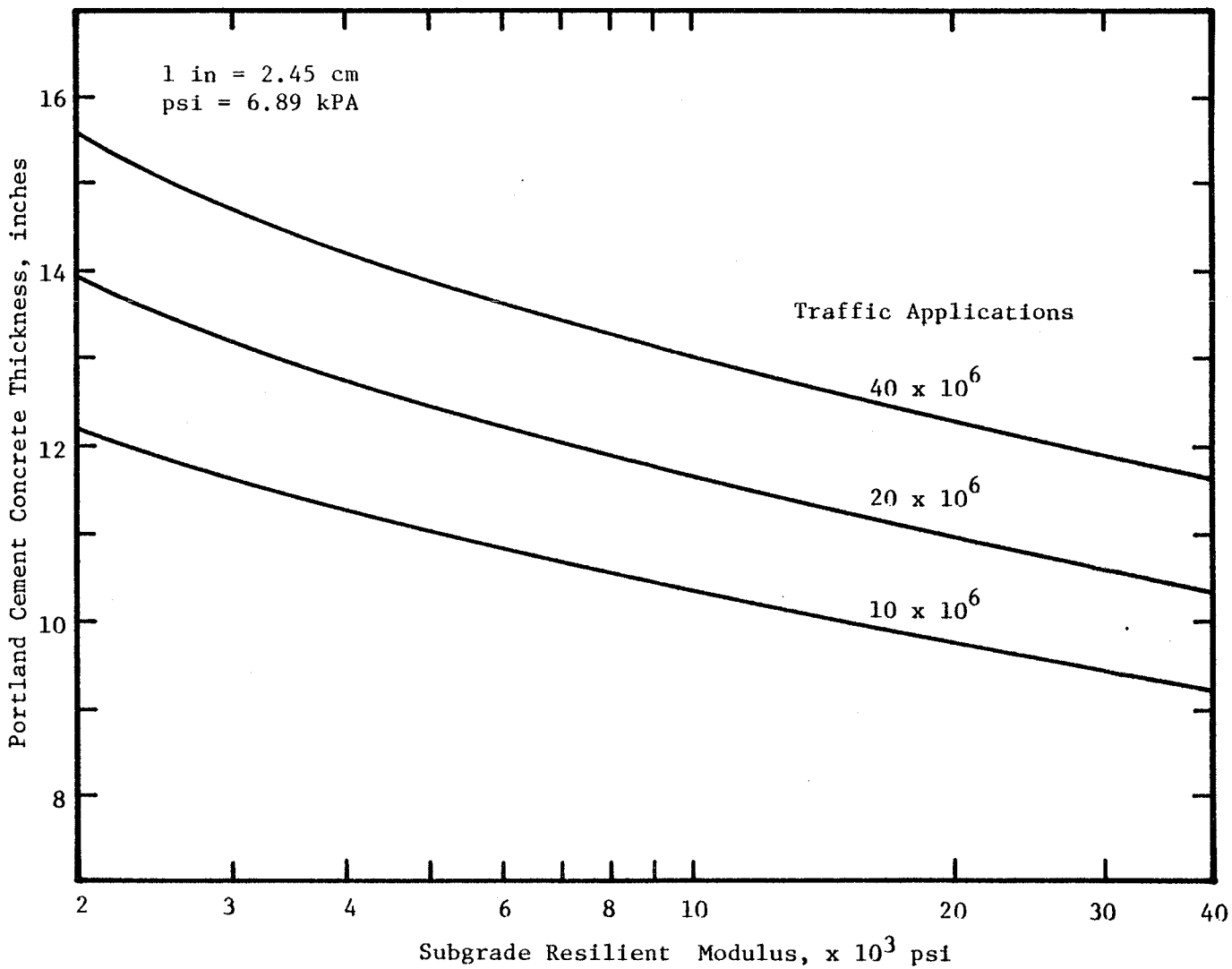
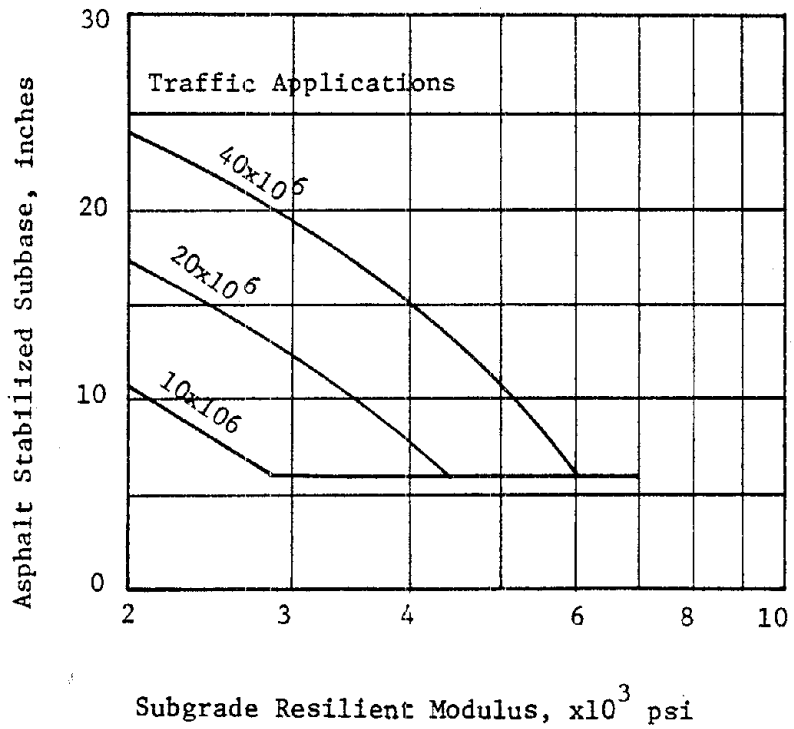
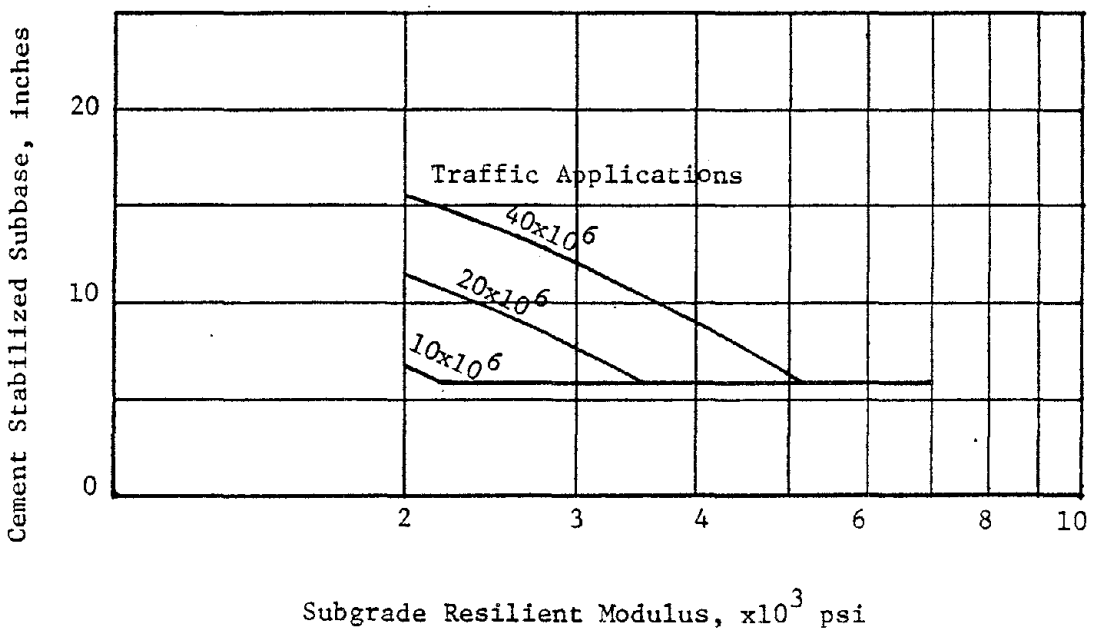


Figure E.37. Low modulus JCP thickness required by fatigue and roughness criteria for composite pavement with f_r = 550 psi.



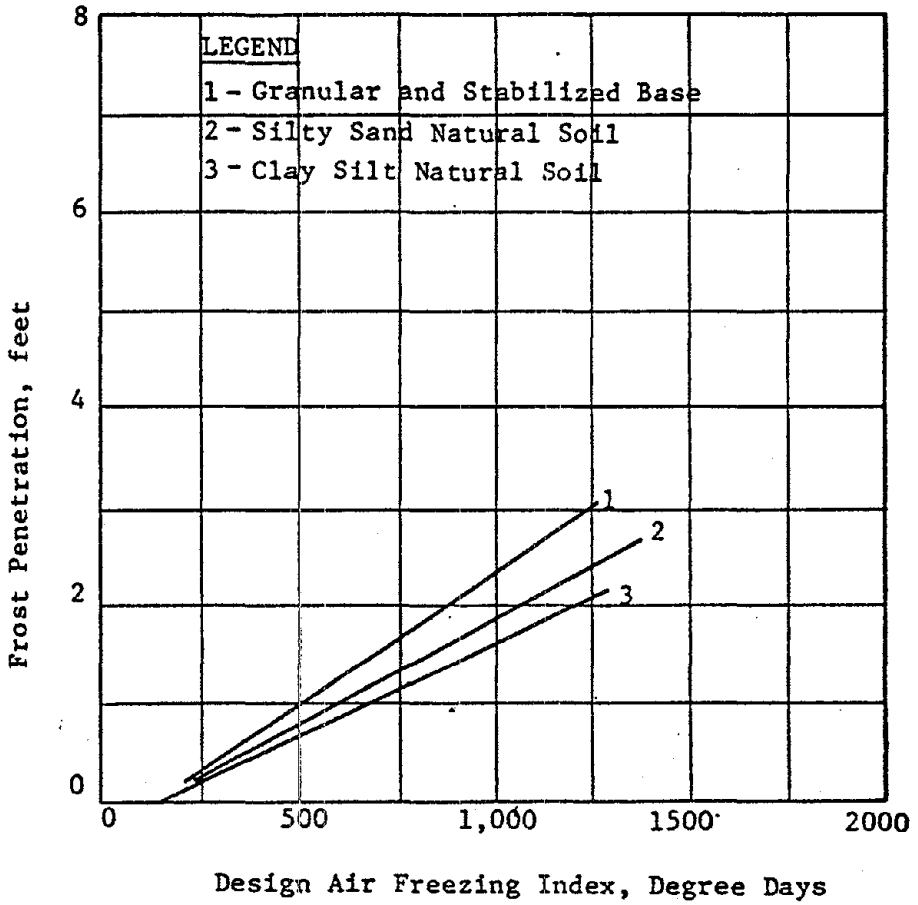
1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.38. Asphalt stabilized subbase thickness required to meet the surface deflection and subgrade compressive strain criteria.



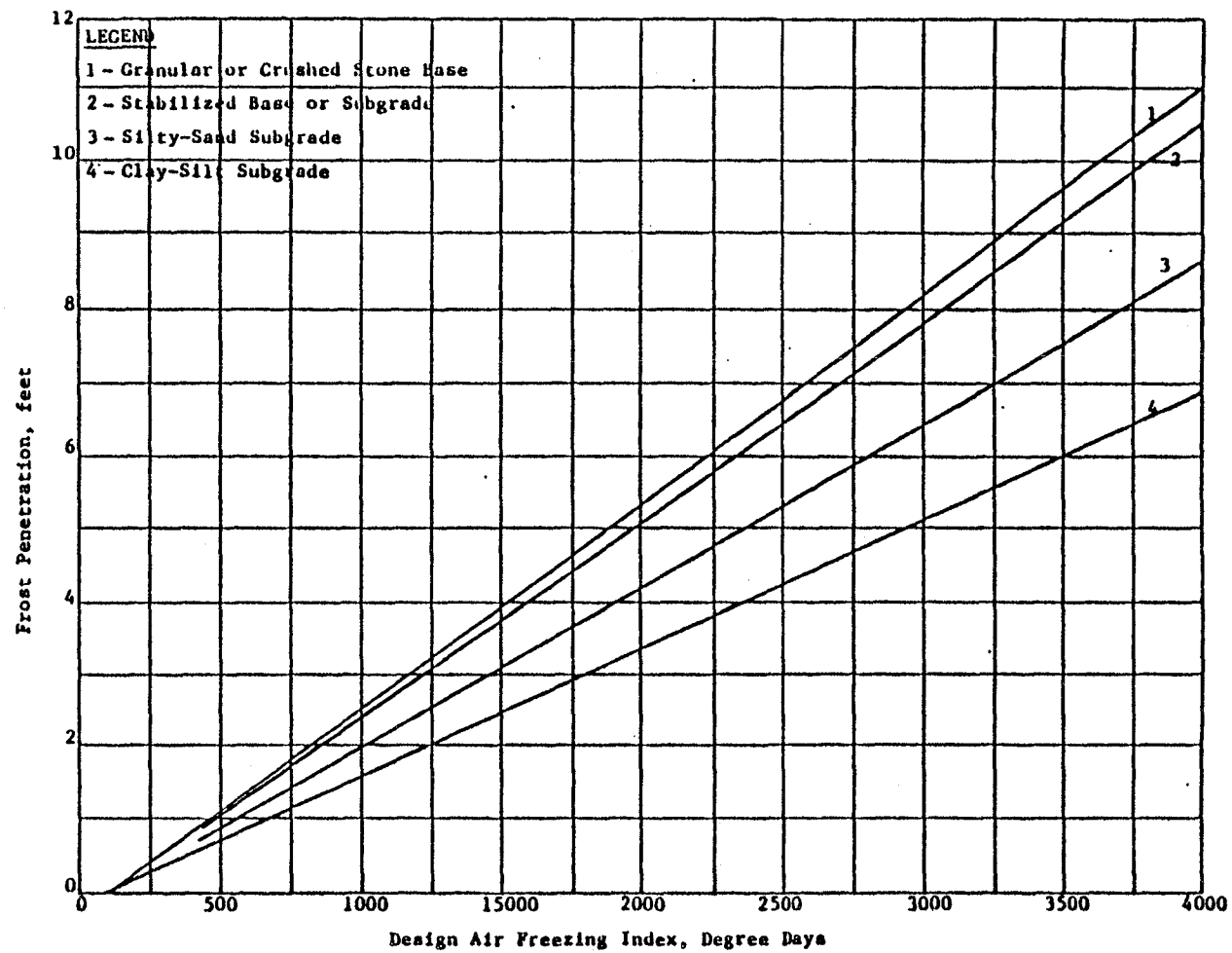
1 in = 2.54 cm 1 psi = 6.89 kPa

Figure E.39. Cement stabilized subbase thickness required of composite pavements to meet the surface deflection and subgrade compressive strain criteria.



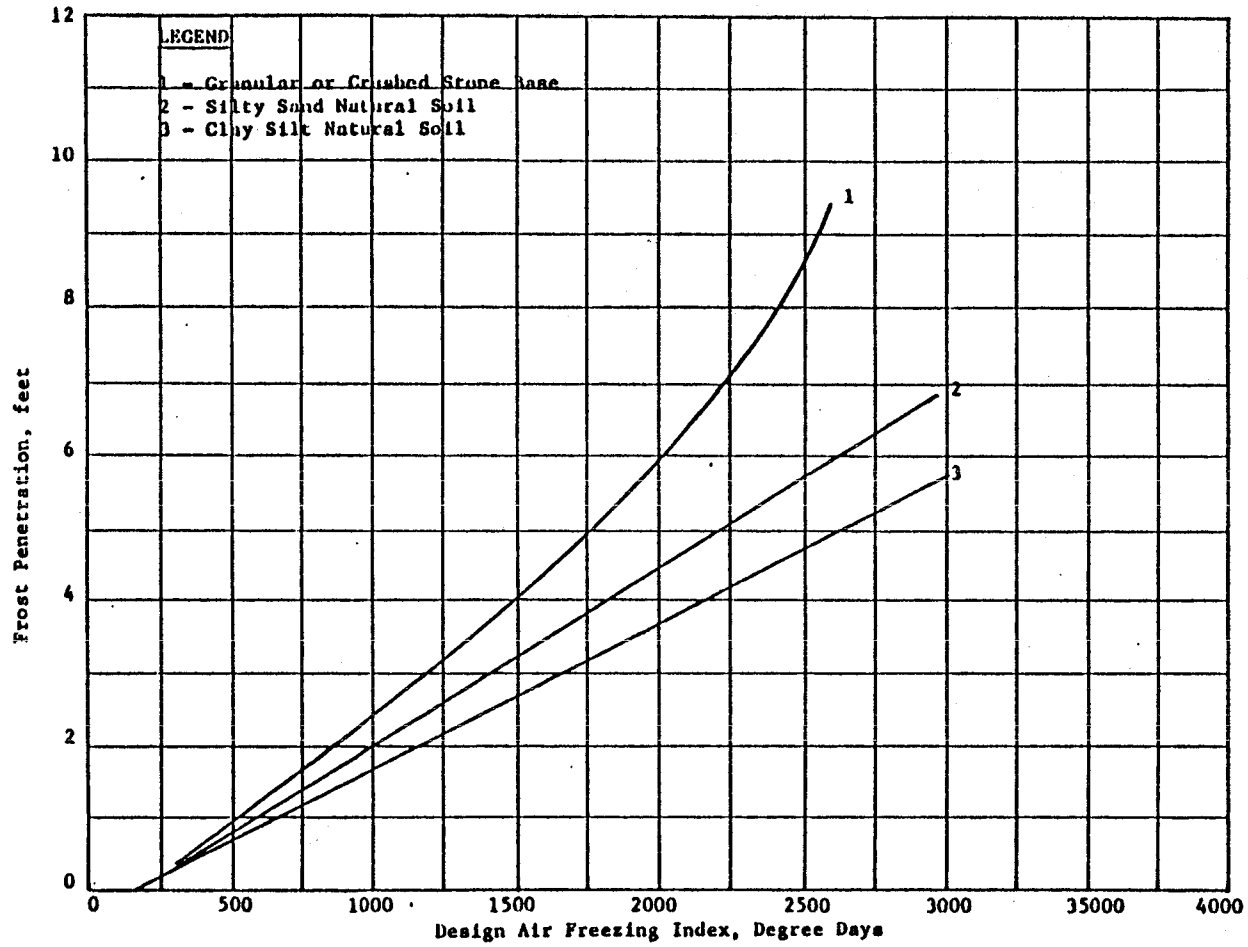
1 ft = 0.3048 m

Figure E.40. Determination of the Frost Penetration into Different Materials below the Pavements Surface for Environmental Region No. 1.



1 ft = 0.3048 m

Figure E.41. Determination of the Frost Penetration into Different Materials below the Pavement Surface for Environmental Region No. 2.



1 ft = 0.3048 m

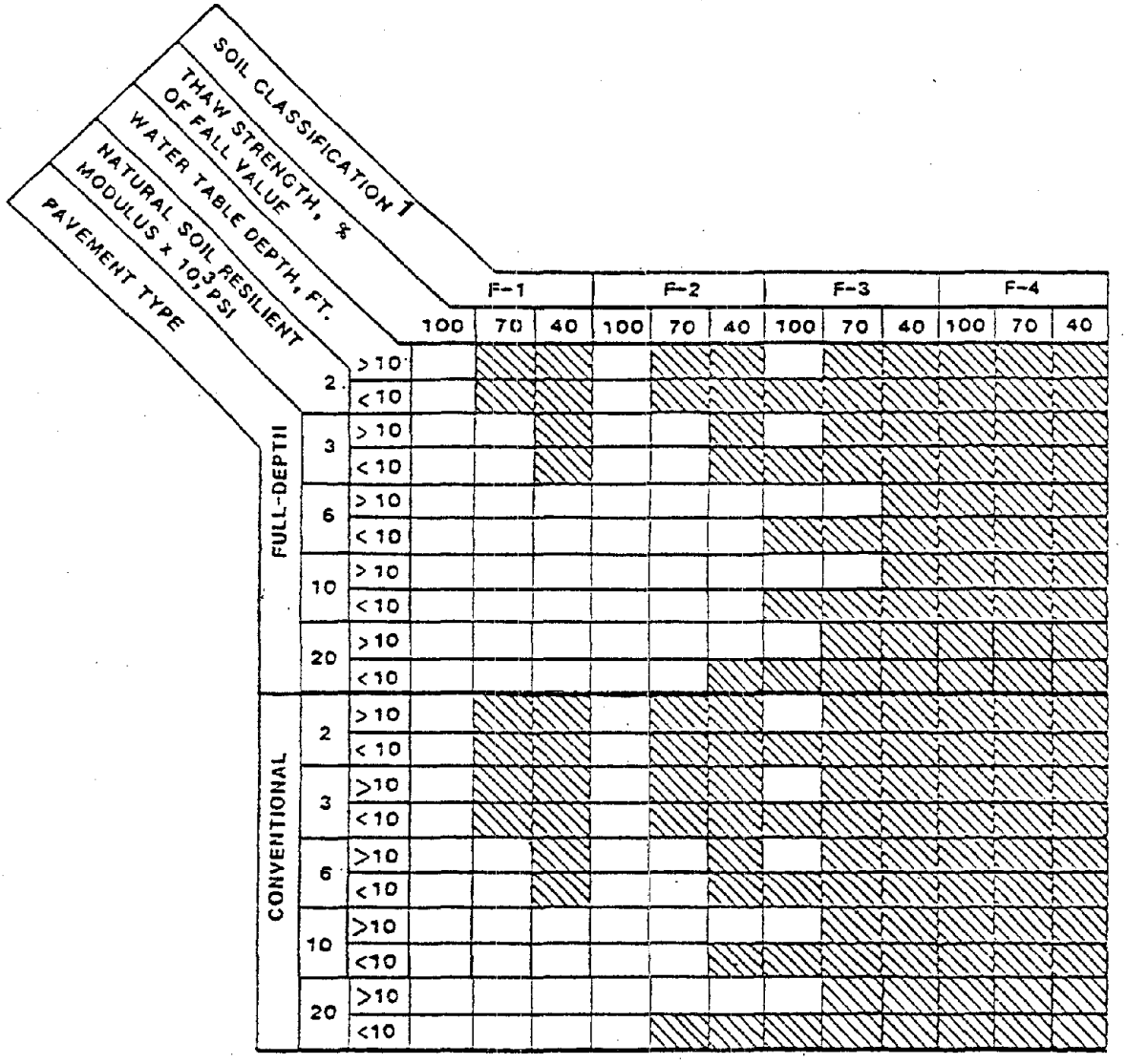
Figure E.42. Determination of the Frost Penetration into Different Materials below the Pavement Surface for Environmental Region No. 3.

penetration or he may elect to increase the structural thickness to account for reduced strength values occurring during spring thaw. Tables E.3 and E.4 provide suggestions to assist the engineer in choosing between full and partial protection from frost penetration. Among the factors that affect this decision are soil classification, strength during the thaw period, water table location, pavement type and soil modulus.

If the engineer elects to fully protect the subgrade from frost penetration, Figures E.40 through E.43 should be used to estimate the additional thickness of non-frost susceptible materials required as a part of the pavement structure. Full protection is especially suggested for the combinations using granular layers in the asphalt pavement. A decrease in the stiffness of subgrade soils due to spring-thaw will result in a stiffness reduction of the granular materials which may not be recovered with time.

If the engineer desires to analyse the partial protection alternate, Figures E.44 and E.45 should be used to estimate the increased damage that occurs as a result of reduced strength during the spring thaw period. After evaluation of both the partial protection and full protection alternatives the engineer can select the most cost effective design.

TABLE E.3 COVER REQUIREMENTS FOR FROST PENETRATION OF PREMIUM FLEXIBLE PAVEMENTS



FULL PROTECTION FROM FROST PENETRATION **REQUIRED**

PARTIAL PROTECTION—INCREASED STRUCTURAL THICKNESS

1 ft = 0.3048 m

1 psi = 6.89 kPa

[1]Refer to Table E.1 and Figure E.6 for soil classification information.

TABLE E.4 COVER REQUIREMENTS FOR FROST PENETRATION OF PREMIUM COMPOSITE PAVEMENTS

PAVEMENT TYPE	MODULUS x 10 PSI	SUBGRADE RESILIENT WATER TABLE DEPTH FT.	SOIL CLASSIFICATION[1] THAW STRENGTH % OF FALL VALUE	F-1			F-2			F-3			F-4		
				100	70	40	100	70	40	100	70	40	100	70	40
LOW MODULUS CRCP	2	>10													
		<10													
	3	>10													
		<10													
	6	>10													
		<10													
	10	>10													
		<10													
	20	>10													
		<10													
LOW MODULUS PLAIN CONC	2	>10													
		<10													
	3	>10													
		<10													
	6	>10													
		<10													
	10	>10													
		<10													
	20	>10													
		<10													

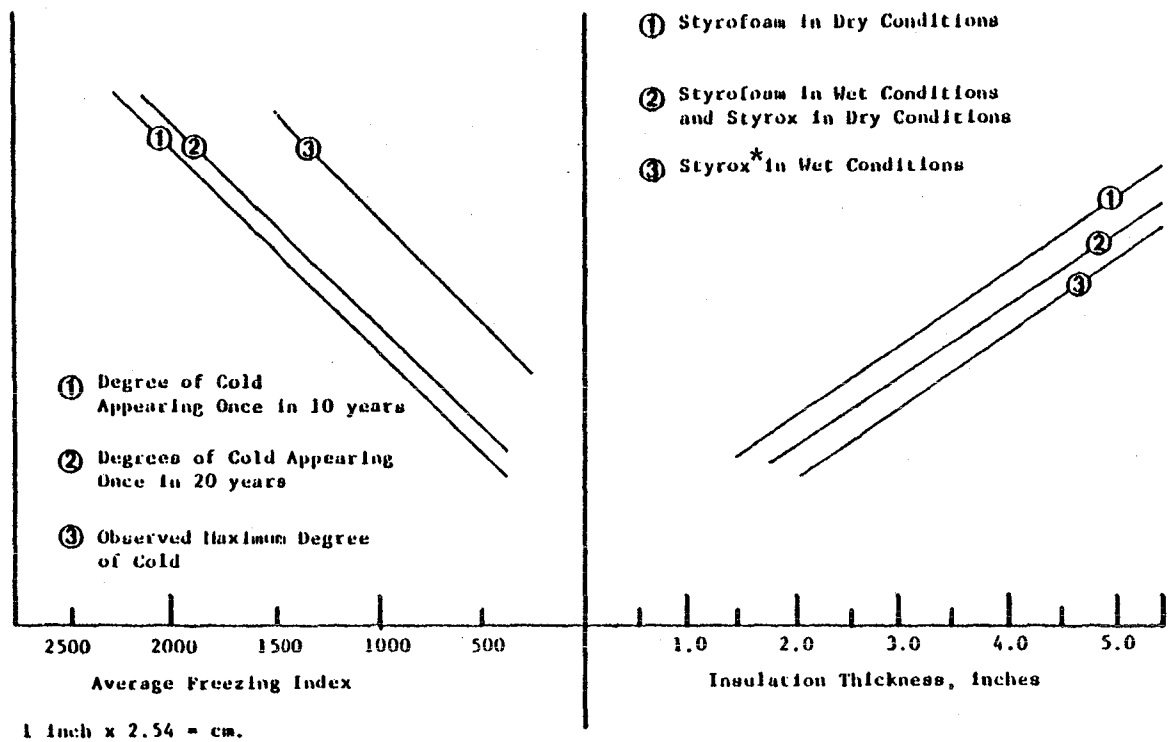
FULL PROTECTION FROM FROST PENETRATION REQUIRED

PARTIAL PROTECTION—INCREASED STRUCTURAL THICKNESS

1 ft = 0.3048 m

1 psi = 6.89 kPa

[1]Refer to Table E.1 and Figure E.6 for soil classification information.



*Styrox is a type of foam insulation used in Finland.

Figure E.43. Design of Insulation Thickness.

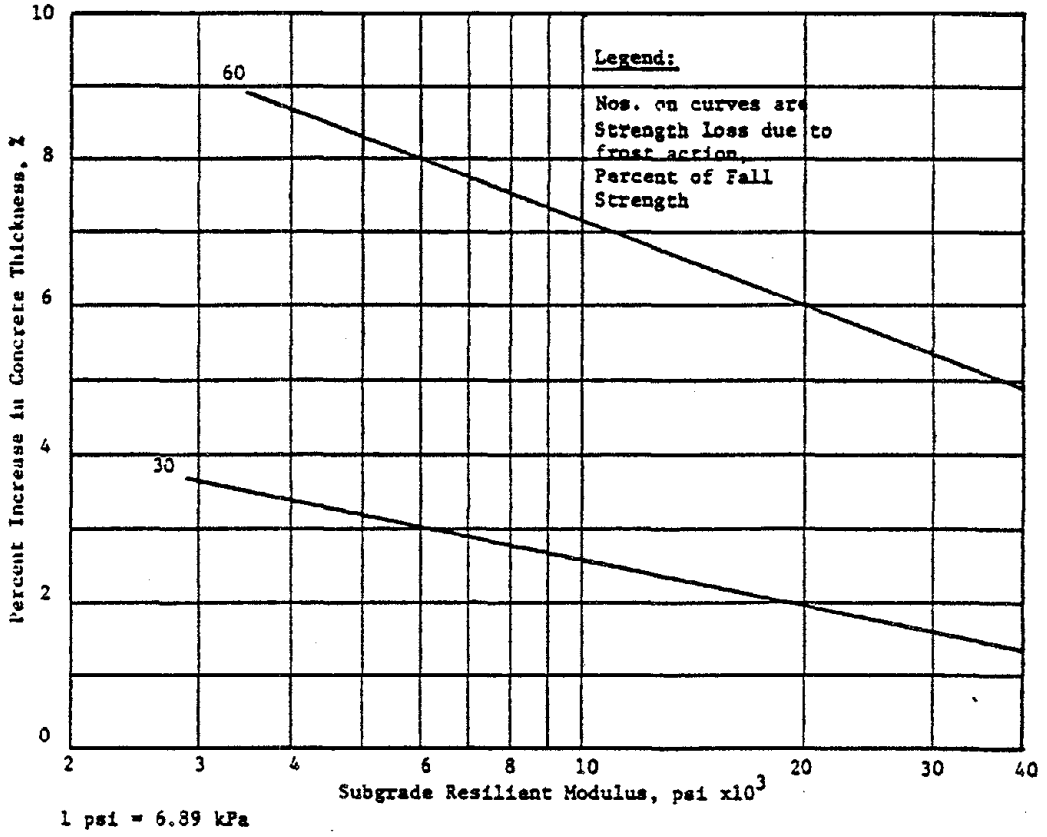
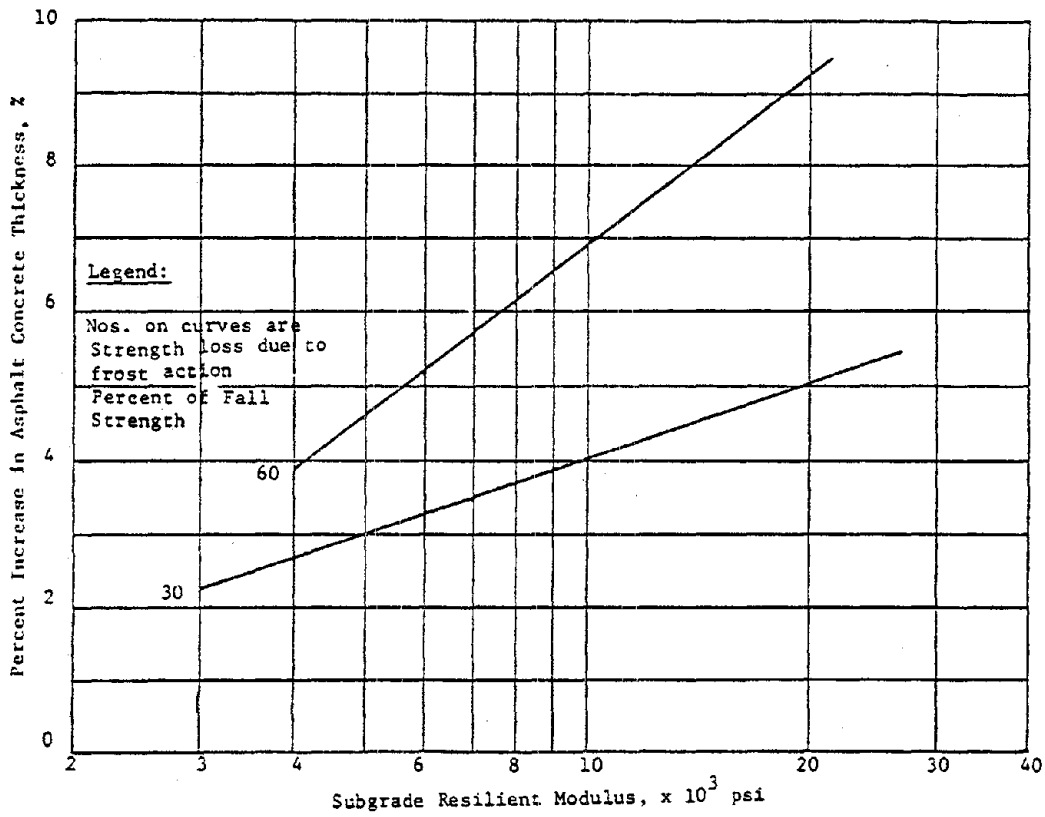


Figure E.44. Increase in structural thickness of composite pavements for different levels of frost damage.



1 psi = 6.89 pKa

Figure E.45. Increase in structural thickness of flexible pavements for different levels of frost damage.

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