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Soil and Base Stabilization and Associated Drainage Considerations

Volume I, Pavement Design and Construction Considerations

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16. Abstract

This report consists of two volumes: Volume I, Pavement Design and Construction Considerations; Volume II, Mixture Design Considerations. These two volumes represent the revisions to the original manuals prepared in 1979 by Terrel, Epps, Barenberg, Mitchell, and Thompson. These manuals include new information and procedures incorporated into the pavement field since that time. A significant portion of the information prepared for the original manuals has been retained. The primary purpose of these manuals is to provide background information for those engineers responsible for using soil stabilization as an integral part of a pavement structure. information is included to assist the engineer in evaluating the drainage problems of a pavement structure. Information is included to assist the engineer in evaluating the drainage problems of a pavement. Sufficient information is included to allow the pavement design engineer to determine layer thicknesses of stabilized layers for a pavement using the 1989 American Association of state Highway and Transportation Officials Guide procedures. material properties are presented with the use of this design procedure, which the materials engineer will find useful in selecting the type and amount of a stabilizer to sue with specific soil types. Construction details are presented with elements of quality control and specifications. The manuals are presented to allow an engineer to recommend where, when, and how soil stabilization should be used, and to assist the engineer in evaluating problems which may occur on current stabilization projects.

This volume presents the specific details of drainage considerations and the use of open graded permeable bases, construction consideration for each stabilizer type, and pavement design considerations. These chapters will provide the engineer with the necessary information to evaluate the pavement for drainage, and thickness adequacy, and to evaluate the construction process. Volume II presents the mixture design and laboratory testing procedures.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

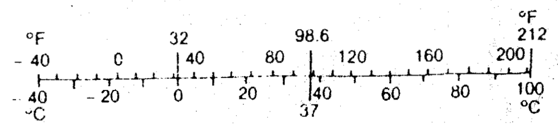
Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F



(Revised April 1989)

* SI is the symbol for the International System of Measurement

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Determining the proper application of stabilization as well as the selection of the appropriate stabilizer is often made without the benefit of adequate field and laboratory testing. The exact characteristics of the soils being used must be made before any determination of their suitability for stabilization can be made. Laboratory tests to determine the engineering properties of stabilized soils and borrow materials must be conducted to show the suitability of the particular stabilization technique and to determine the amount of stabilizer required.

2. STABILIZER SELECTION

Several general guides have been published which assist the engineer in properly selecting a stabilizer for a soil. An in-depth review of this literature is provided in Volume II, Chapter 2 of this user's manual. This chapter summarizes the fundamental concepts contained in these pertinent guides.

While each additive has a specific capability to stabilize, it is necessary to examine the soil that is to be stabilized to determine if its properties are compatible with one or more of the additives available for the project. General soil properties to be considered include:

- Gradation.
  - Maximum particle size.
  - Fines content (passing #200 sieve).
- Plasticity.
  - Liquid limit.
  - Plasticity Index.

Knowledge of the soil to be stabilized in terms of these properties can provide a good indication to the engineer which stabilizer will be most cost-effective.

Several entities have developed guides to assist the engineer in the selection process. A majority of these guides are based on a knowledge of the fundamental properties of the soil. The Soil Stabilization Index System (SSIS) selection method, for instance, provides a step-by-step procedure for determining the type of stabilizer to use.[10] This process is illustrated in figure 1. Note that this system is designed to indicate the best additive for each soil type. All soils can be stabilized with one or more of the additives discussed. It is the engineer's responsibility to make the decision that stabilization is required for any particular project wherein these soils may be encountered.

Additional criteria for stabilizer selection are available in literature pertaining to particular types of stabilizers. The following sections provide brief overviews concerning the types of soils suitable for stabilization by the particular additive.

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CHAPTER 2 SELECTION OF STABILIZER

1. INTRODUCTION

When considering stabilizer additives, it is necessary for the user to keep in mind the purpose of the stabilization process. The intended use of the stabilizer must be directed toward a solution to one or more problems in the pavement under consideration. The mechanics of the stabilization process can indicate whether one technique is more advantageous to the pavement than another. Hence, it may be necessary to employ one additive over another even though the latter may provide better engineering properties.

Individual stabilizer additives do not react equally well with the different soil classifications. Because of the nature of the additives, there is a considerable overlap in the ability of each stabilizer to react with specific soils. A few soils can be stabilized with any of the agents, while other soils are best suited to one or two specific additives. When more than one option exists, equipment availability and material and construction costs must be considered in determining which method is most feasible and cost-effective, assuming the engineering properties of the stabilized materials are similar. To make this judgement, the objectives of a stabilization project must be clearly understood before an additive can be selected.

Some of the primary objectives of stabilization include:

- Provide a stable construction platform
- Improve poor subgrade conditions.
- Provide dust control.
- Improve long term strength and durability.
- Provide moisture control.
- Upgrade marginal base materials.
- Improve workability.
- Increase pavement performance by providing uniform long term support

Each of these objectives provides a valid reason for considering the use of a particular additive. While a number of these objectives are often achieved with the use of an additive, it is not always necessary to satisfy more than one objective.

Although many benefits may be realized with stabilization, it must be emphasized that stabilization is not a panacea for the problems that may exist in a particular pavement. Great care must be exercised in evaluating the pavement system and its components for items such as drainage, durability, and strength.

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Asphalt cement

A fluxed or unfluxed asphalt specially prepared as to quality and consistency for direct use in such construction industries as highways and structures.

Cutback asphalt

Asphalt cement that has been made liquid with the addition of petroleum diluents such as naptha or kerosene.

Emulsified asphalt

Asphalt cement that has been mechanically liquified with the addition of emulsifying agents and water.

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Definitions Associated with Cement Stabilization

Portland Cement

A hydraulic cement produced by pulverizing clinker consisting essentially of hydraulic calcium silicates, and usually containing one or more of the forms of calcium sulfate as an inter-ground addition (ASTM C-1).

Cement Stabilized Soil

A mixture of soil and measured amounts of portland cement and water which is thoroughly mixed, compacted to a high density, and protected against moisture loss during a specific curing period.

Soil-Cement

A hardened material formed by curing a mechanically compacted, intimate mixture of pulverized soil, portland cement, and water. Soil-cement contains sufficient cement to pass specified durability tests.

Cement-Modified Soil

An unhardened or semi-hardened intimate mixture of pulverized soil, portland cement, and water. Significantly smaller cement contents are used in cement-modified soil than in soil-cement.

Plastic Soil-Cement

A hardened material formed by curing an intimate mixture of pulverized soil, portland cement, and enough water to produce a material with a mortar-like consistency at the time of mixing and placing. Plastic soil-cement is not in common use today.

Definitions Associated with Asphalt Stabilization

Bitumen

A class of black or dark-colored (solid, semisolid, or viscous) cementitious substances, natural or manufactured, composed principally of high molecular weight hydrocarbons. Asphalts, tars, pitches, and asphaltites are all examples of bitumens.

Asphalt

A dark brown to black cementitious material in which the predominant constituents are bitumens which occur in nature or are obtained in petroleum processing.[9]

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Drainage Coefficient

A factor used to modify layer coefficients in flexible pavements or strengths in rigid pavements. It is a function of how well the pavement structure can handle the adverse effect of water, and is indicated by the relative time to drain water from the pavement, and the percent of time during a year the pavement is exposed to water levels approaching saturation.[8]

Pavement Serviceability

An evaluation of how well the pavement satisfies the design function for that pavement.

Pavement Performance

The trend of pavement serviceability over a period of time.

Open-Graded Base

The portion of the pavement structure beneath the surface course designed to provide free movement of water under all conditions. A minimum coefficient of permeability of 1000 ft per day should always be provided if positive drainage is to be achieved.

Floating Aggregate Matrix

The physical action when finer particles (filler) force aggregate particles apart producing a loss of aggregate interlock and strength.

Sand Equivalency

Test to determine the relative proportions of plastic fines and dust in fine aggregates.

Definitions Associated with Lime Stabilization

Lime

All classes of quicklime and hydrated lime, both calcitic (high calcium) and dolomitic (ASTM C593).

Definitions Associated with Lime-Fly Ash Stabilization

LFA

A mixture of lime and fly ash with aggregate.

LCFA

A mixture of lime, cement, and fly ash with aggregate.

LFS

A mixture of lime and fly ash with soil.

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Mechanical Stabilization

The alteration of soil properties accomplished through one of two means: (1) changing the gradation of the soil by the addition or removal of particles, and (2) densification by compaction.

Aggregate

A granular material of mineral composition used either in its natural state as a base course or railroad ballast or with a cementing medium to form mortars or cement.

ASTM

The American Society for Testing and Materials.

Resilient Modulus

A measure of the elastic property of a treated or untreated soil recognizing certain nonlinear stress-related characteristics in response to a dynamic loading condition.[8]

Resilient Modulus Test

A test similar to that described in AASHTO T274-82, which is not approved, or the SHRP Protocal, which applies a repeated load pulse of a fixed magnitude and fixed time duration to a cylindrical soil sample, similar to an unconfined compression sample, and monitors the deformation in the sample produced by these repeated loads.

Mechanistic-Empirical Design Procedures

Pavement thickness design procedures based on an analytical/ theoretical study of pavement responses (stress, strain, and deflections) through pavement modeling techniques. These theoretical pavement responses are empirically related to the performance of the pavement through laboratory studies and field distress surveys to produce design procedures that are termed mechanistic-empirical approaches.

Reliability

The probability that a pavement section designed using the pavement design-performance process will perform satisfactorily over the traffic and environmental conditions for the design period.[8]

Layer Coefficient (a_i)

The empirical relationship between structural number (SN) and layer thickness which expresses the relative ability of a material to function as a structural component of the pavement.[8]

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- Construct superior bases.
- Improve strength, reducing thickness requirements.
- Improve durability.
- Control volume change of soils.
- Dry wet soils.
- Improve workability.
- Conserve aggregate materials.
- Reduce overall costs.
- Conserve energy.
- Provide a temporary or permanent wearing surface for low volume roads.
- Provide a stable working platform for construction activities.

4. DEFINITIONS

A discussion of soil and aggregate stabilization requires the use of a common terminology. Brief definitions are provided for the following terms which will appear intermittently throughout the user manuals.

General Definitions

AASHO

An abbreviation used to designate the American Association of State Highway Officials. The name of the group was recently changed to the American Association of State Highway and Transportation Officials, and the current abbreviation AASHTO is also used.

Soil

Sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter (ASTM D-18).[7]

Soil Stabilization

Chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise improve its engineering properties (ASTM D-18).

Chemical Stabilization

The altering of soil properties by use of certain chemical additives which, when mixed into a soil, often change the surface molecular properties of the soil grains and, in some cases, cement the grains together, resulting in strength increases.

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Soil stabilization may be defined as the improvement of pertinent soil engineering properties by the addition of various additives so that the soil can effectively serve its function in the construction and life of a pavement. As in all engineering problems, the additional costs associated with soil stabilization must be considered in light of the benefits derived from the stabilization process to determine if stabilization is warranted.

One of the major concerns in recent years has been localized shortages of conventional aggregates. The highway construction industry consumes over half of the annual production of aggregates.[6] However, this traditional use of aggregates in pavement construction has resulted in acute shortages in those areas that normally have adequate supplies. Other areas of the country have never had good quality aggregates available locally. Metropolitan areas have experienced shortages as land use planning has not recognized the need for material availability to support continued growth.

The combinations of regulations which prohibit mining and production of aggregates and land use patterns that make aggregate deposit inaccessible, have combined to produce an escalation of aggregate costs. The result is an increase in highway construction and maintenance costs. Consequently, there is a great need to find more economical replacements for conventional aggregates. Stabilization techniques for substitute materials and for improving marginal materials is a natural focus resulting from this problem.

The energy crisis brought on by the temporary shortage of petroleum experienced in the early and late 1970's is another concern. Although energy costs have decreased today, the need to consider the impact of energy usage has not diminished. A considerable percentage of the energy needed to construct pavements goes into producing highway construction materials. Since relatively small quantities of binders (i.e., lime, cement, fly ash, and asphalt) can be used effectively in upgrading pavement layers, total energy demands as well as costs may be reduced .

In summary, existing literature suggests that soil stabilization is a desired design alternative. It is necessary for the user to keep in mind the purpose of the stabilization process. The intended use of stabilizer, coupled with the mechanics of the stabilization process, form the basis for selecting the type and quantity of stabilizer to be used. Listed below are several reasons and advantages for considering soil stabilization:

- Improve poor subgrade conditions.
- Upgrade marginal base materials.
- Provide dust control.
- Water-proof the soil.
- Salvage old roads with marginal materials.

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Every attempt has been made to present information that is technically correct. Both conventional and state-of-the-art construction and testing technologies are presented. However, the engineer must take into consideration local economic factors, climatic conditions, and other local aspects of a project in order to make prudent decisions with regard to the designs and applications of the technology contained herein.

2. SCOPE

Volume I will provide the engineer with sufficient information to perform the following design activities:

- Select the type or types of stabilizers suitable for a specific soil.
- Identify stabilized material requirements needed to ensure adequate performance, given certain drainage conditions.
- Identify construction sequences and methods suitable for soil stabilization operations.
- Identify construction equipment suitable for soil stabilization operations.
- Design pavement structures containing stabilized layers using AASHTO and/or Mechanistic procedures.

3. BACKGROUND

A problem which engineers continually face is the identification and successful implementation of the procedures and techniques by which otherwise unsuitable soils may be sufficiently improved so that they may be successfully used in construction projects. The concept of soil improvement or modification through stabilization with additives has been around for several thousand years. At least 5000 years ago, soils were stabilized with lime or pozzolans for the same economic reasons that soils are stabilized today. This unique contribution to roadway construction is as beneficial today as it was then.

Soil stabilization is a tool for economical road-building, conservation of materials, investment protection, and roadway upgrading.[5] In many instances, soils that are unsatisfactory in their natural state can be made suitable for subsequent construction by treatment with admixtures, by the addition of aggregate, or by proper compaction.

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CHAPTER 1 INTRODUCTION

1. PURPOSE

This report presents revisions to the two-volume user's manual prepared in 1979. The two manuals are:

- "Soil Stabilization in Pavement Structures, A User's Manual," Volume I, Pavement Design and Construction Considerations, FHWA-IP-80-2.[1]

- "Soil Stabilization in Pavement Structures, A User's Manual," Volume II, Mixture Design Considerations, FHWA-IP-80-2.[2]

There have been significant changes in the pavement industry since these reports were first published. These include the use of new materials, the development of new equipment, and improved construction and design procedures. The 1986 AASHTO Guide for Design of Pavement Structures presents a significant departure from the 1972 Interim Guide for pavement structural design. Drainage considerations have also received increased attention, as it is increasingly obvious that greater material strengths alone cannot alleviate the performance problems of some pavements.

This two-volume user's manual was developed to provide guidance for pavement design, construction, and materials engineers responsible for soil stabilization operations related to the transportation field. Volume I is primarily intended for the use of engineers involved in design and construction. It serves as a guide for the selection of an appropriate stabilizer on a project and provides important information with regard to assessing drainage conditions and understanding construction procedures.

Volume II, on the other hand, tackles the concerns and issues faced by pavement design and materials engineers. This volume contains the information required to determine the type and amount of stabilizer to be used on a project. An in-depth discussion of the tests used to characterize stabilized materials is presented, as well as the manner in which testing is utilized in pavement design processes.

Revisions to the original user's manual are based on several inputs. An extensive review of relevant literature published since 1979 was conducted. In addition, visits to construction sites and discussion and review by experts in the soil stabilization field provided pertinent information which was incorporated into this manual.

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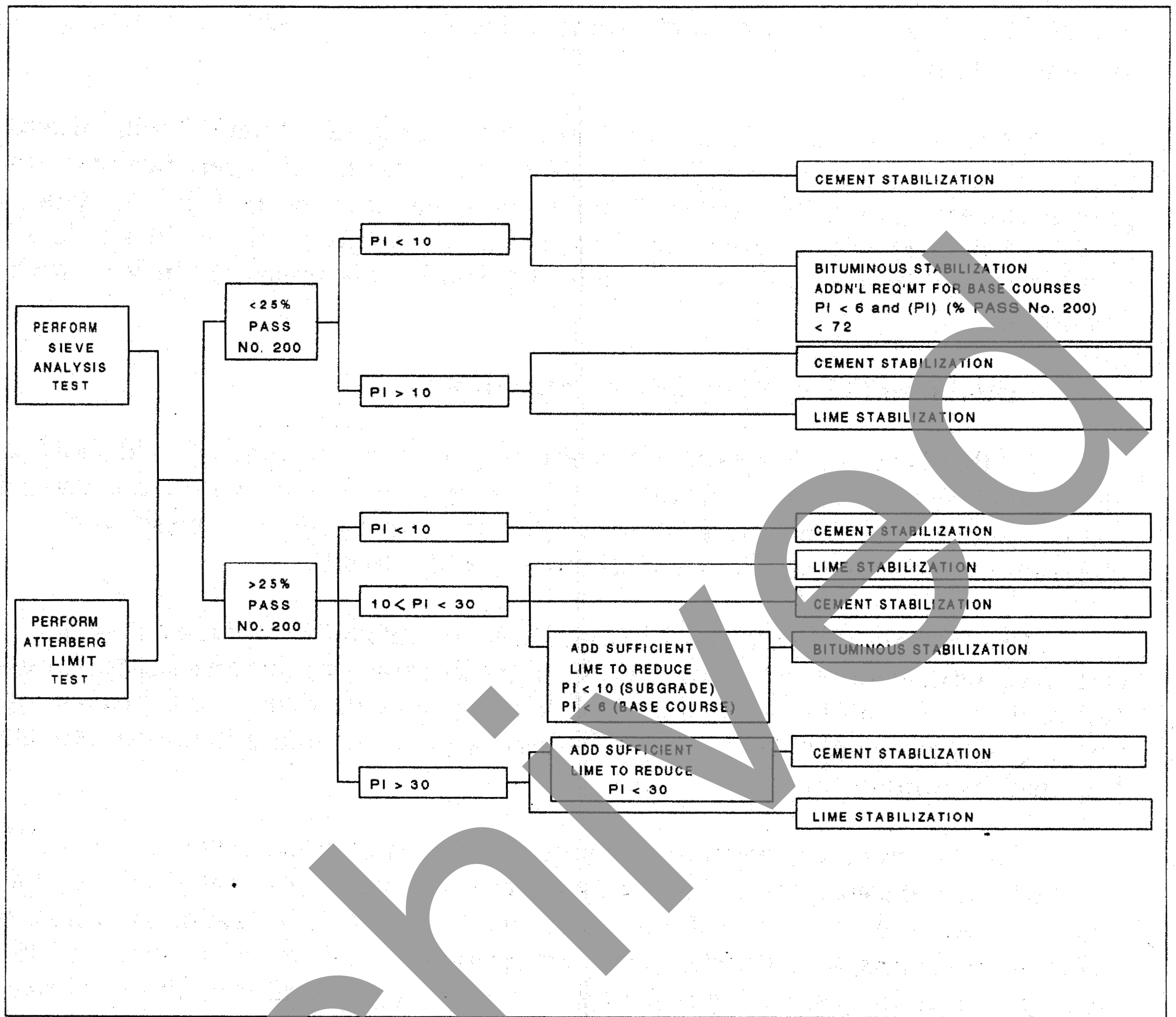

Figure 1. The soil stabilization index system (SSIS) selection procedure.[10]

Criteria for Selection of Lime Stabilization

A general guideline for lime stabilization is that it should be considered as the primary stabilizer, or at least as a pre-stabilizer, for soils with PI's greater than 10 or greater than 25 percent passing the No. 200 sieve.

Experience has shown that lime will react with medium, moderately fine, and fine-grained soils to produce decreased plasticity, increased workability, reduced swell, and increased strength.[11] Soils classified according to the Unified System as CH, CL, MH, ML, SC, SM, GC, GM, SW-SC, SP-SC, SM-SC, GW-GC, CP-GC, or GM-GC should be considered as capable of being stabilized with lime. Soils classified by

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AASHTO as A-4, A-5, A-6, A-7, and some of the A-2-7 and A-2-6 soils are candidates for lime stabilization.

Air Force criteria indicate that the PI should be greater than 12 with at least 12 percent of the material passing the No. 200 sieve.[12] Based on experience with fine grained cohesive soils, Robnett and Thompson, have indicated that lime may be an effective stabilizer of soils with clay contents as low as 7 percent and PI's as low as 8.[11,13] The specifics for the use of lime in stabilization are presented in Volume II, Chapter 4.

Criteria for Selection of Cement Stabilization

Portland cement is suitable for stabilizing a wide range of soils with low to moderately high plasticity.[14] It can be used to modify or improve the quality of the soil (cement modification) or to transform the soil into a cemented mass with significantly increased strength and durability (soil-cement).

The Portland Cement Association (PCA) indicates that all types of soils can be stabilized with cement.[15,16] However, well-graded granular materials that possess sufficient fines to fill the voids and push the aggregate particles apart, producing a floating aggregate matrix have given the best results. Normally the maximum size aggregate is limited to 2 in (5.1 cm).

The Air Force has established limits on the PI for different types of soils.[17] The PI should be less than 30 for sandy materials while the PI should be less than 20 and the liquid limit less than 40 for fine-grained soils. This limitation is necessary to ensure proper mixing of the stabilizer. For granular materials, a minimum of 45 percent by weight passing the No. 4 sieve is desirable. In addition, the PI of the soil should not exceed the number indicated from the following equation:

$$P.I. \geq 20 + [(50\text{-Fines Content})/4]$$

The amount of cement additive required for a particular soil depends upon whether the soil is being modified or if full strength stabilization is desired. For example, if the intent is merely to reduce the PI of the soil, small amounts (3 percent or less) of cement can be incorporated. Larger percentages, as determined from laboratory testing can be added if the objective is to produce a solid material capable of achieving high strengths. Proper testing must be done to avoid extensive problems with uncontrolled cracking at higher additive amounts. The effects various cement contents will be discussed in detail in volume II, chapter 5.

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Criteria for Selection of Asphalt Stabilization

The American Road and Transportation Builders Association (ARTBA) recommends asphalt stabilization with sands having less than 25 percent passing the No. 200 sieve and a maximum PI of 6. In addition, coarse aggregates having less than 15 percent passing the No. 200 sieve and a PI less than 6 are considered suitable for asphalt stabilization.[18]

Several investigators have proposed suitable materials for asphalt stabilization.[19,20,21,22,23] The general consensus of their work indicates the maximum percent passing the No. 200 sieve should be less than 25, the PI less than 6, sand equivalent less than 30, and the product of the plasticity index and the percent passing the No. 200 sieve less than 60. This corresponds roughly to figure 1 which indicates a value of 72 would be acceptable.

In general, materials that are suitable for asphalt treatment include:

- AASHTO
  A-2-4, A-2-6, A-3, A-4, and low plasticity A-6 soils.
- Unified
  SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, SC, SM-SC, GW, GP, GW-GM, GP-GM, GW-GC, GM, GC, and GM-GC with additional requirements.

The specifics for the use of asphalt in stabilization ore provided in Volume II, chapter 6.

Criteria for Selection of Fly-Ash Stabilization

Fly ash is normally used in stabilization operations to act as a pozzolan and/or filler. Flyash is a pozzolan, siliceous and aluminous in nature, that reacts with calcium constituents to produce cementitious products, resulting in a substantial strength increase. While calcium may be present in the material to be stabilized, lime or cement is often introduced to provide additional amounts of calcium for reaction purposes. The glassy phase of a fly ash is the component that reacts with hydrated lime or portland cement in aqueous systems.

Since the particle size of the fly ash is normally larger than the voids in fine-grained soils, the role as a filler is not appropriate for use in fine-grained soils. The major role for fly ash in stabilization of fine-grained soils is that of a pozzolan. Most clays are pozzolanic in nature and thus do not require additional pozzolans. Thus, silts are generally considered the most suitable fine-grained soil type for treatment with lime-fly ash or cement-fly ash mixtures.

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Aggregates which have been successfully utilized in lime-fly ash mixtures include a wide range of types and gradations. These include sands, gravels, crushed stones, and several types of slag. Lime-fly ash is often more economical for use with aggregates than with fine-grained soils. In addition, the coarser aggregates present have greater resistance to frost action and deformation under loads.

Lime-cement-fly ash stabilization is typically used on coarse-grained soils having no more than 12 percent of the material passing the No. 200 sieve. In addition, it is recommended that the PI of the minus #40 sieve fraction not exceed 25. This combination has not been extensively used to date, and care must be exercised in its use. Details are provided in Volume II, Chapter 7 on the use of Lime Fly-Ash in stabilization.

Criteria for the Selection of Combination and Other Stabilizers

Combination stabilizers discussed here primarily include lime-cement, lime-asphalt, lime-emulsified asphalt, and cement-emulsified asphalt. The main purpose for using combination lime stabilizers is to reduce plasticity and increase workability so the soil can be intimately mixed and effectively stabilized. In most applications, lime is the pretreatment stabilizer followed by cement or asphalt.

The advantage of using lime in certain asphalt stabilization operations is to reduce the potential of stripping in the presence of water. In addition, lime and cement can be used to promote curing of the emulsified asphalt-treated materials.

There are a number of exotic additives which are being used in other countries in an effort to use locally available materials. This includes rice ash, slags, etc. The use of salt as a stabilizer has been performed for a long time to control dust and maintain the structural integrity of untreated aggregates used for surfaces of low volume roads. The specifics for the testing and use of these stabilizers is presented in Volume II, Chapter 8.

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3. SUMMARY

The criteria presented in this chapter provide a broad background of information with regard to the selection of a stabilizer additive. A more detailed approach to stabilizer selection is presented in volume II, chapter 2.

Once a stabilizer is selected, detailed laboratory tests should be performed to determine desirable additive quantities. These tests are outlined in volume II, chapter 3 and further discussion is found in each of the chapters associated with the individual stabilizers. Major considerations which are also brought out in these chapters include environmental and safety aspects. General climatic and construction safety precautions are given in table 1.

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Table 1. Climatic Limitations and Construction Safety Precautions.

Type of Stabilizer	Climatic Limitations	Construction Safety Precautions
Lime and Lime-Fly Ash	Do not use with frozen soils. Air temperature should be 40 °F (5 °C) and rising Complete stabilized base construction one month before first hard freeze. Two weeks of warm to hot weather are desirable prior to fall and winter temperatures.	Quicklime should not come in contact with moist skin. Hydrated lime [$Ca(OH)_2$] should not come in contact with moist skin for prolonged periods of time. Safety glasses and proper protective clothing should be worn all times.
Cement and Cement-Fly Ash	Do not use with frozen soils. Air temperature should be 40 °F (5 °C) and rising. Complete stabilized layer one week before first hard freeze.	Cement should not come in contact with moist skin for prolonged periods of time. Safety glasses and proper protective clothing should be worn all times.
Asphalt	Air temperature should be above 50 °F (10 °C) when using emulsions. Air temperatures should be 40 °F (5 °C) and rising when placing thin lifts of hot mixed asphalt concrete. Hot, dry weather is preferred for all types of asphalt stabilization.	Some cutbacks have flash and fire points below 100 °F (40 °C) Hot mixed asphalt concrete temperatures may be as high as 325 °F (175 °C).

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CHAPTER 3 DRAINAGE CONSIDERATIONS

1. INTRODUCTION

The deterioration of a pavement structure as evidenced by individual distresses appearing on the pavement can be directly related to particular moisture properties of the materials in the pavement, and the ability of the designed structure to resist the effects of moisture.[24] Effects of moisture have been reported which compare design, function, and benefits of an effective subdrainage system.[Refs 25-32] The importance of considering adequate drainage and the effect on the structural integrity of a pavement structure is evidenced by the inclusion of drainage parameters in the 1986 AASHTO Pavement Design Procedure for both flexible and rigid pavements.[31] States have determined that drainage can be effective for their use, as evidenced by the study in California which found that drainage could be cost effective through the life extension provided the pavement.[33] When improved materials are obtained by stabilization, it is not cost effective to ignore the principles of moisture control to ensure that the improved materials retain their quality as moisture can deteriorate even the highest quality materials.

Determining the need for subdrainage requires a careful evaluation of the materials to be used in the pavement to assess their susceptibility to moisture damage. The geometry of the pavement must be considered to determine if drains can be effectively placed to remove the water. The potential source of water to the pavement system must be carefully evaluated before a particular subdrainage system is selected, to ensure the drain actually collects the water entering the pavement system. Detailed discussions of the specifics of drainage design mentioned here can be found in the literature.[32,34,35]

There are two general sources of water which must be considered, groundwater and surface infiltration. The initial concerns with groundwater moisture in a pavement system can be broken into two general categories:

- Those which take place when soil particles migrate to an escape exit, causing piping or erosional failures.
- Those which are caused by uncontrolled seepage patterns and lead to saturation, internal flooding, excessive uplift, or excessive seepage forces.

Moisture related failures attributed primarily to surface infiltration of moisture generally result from continual exposure to moisture and can be placed in two categories:

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

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- Degradation of material quality from the interaction of an increased moisture content with the environment, eg. stripping, erosion, and D-Cracking.
- Loss of slab support, or non uniform support developing from pumping action of traffic causing trapped water to relocate base materials through erosion.

A principle in the design of any pavement is to construct a structure that will keep the pavement materials from becoming saturated or even exposed to constant high moisture levels which may be below saturation. There are three general approaches which can be followed to accomplish this:

- Seal the pavement properly and do not allow the water to enter the pavement layers.
- Use stabilized materials that are more moisture resistant and will not contribute to moisture-related distress. This may not be possible in heavy traffic pavements.
- Provide adequate drainage to effectively remove moisture entering the pavement from the materials before damage can be initiated.

To accomplish these objectives it is first necessary to ensure adequate surface control of water to prevent ponding and other circumstances that would increase the availability of mater to enter the pavement structure. The purpose of these two companion volumes on stabilization is to provide information relative to the use of moisture resistant materials, which will be discussed at length. Because maintenance of an impermeable surface is a circumstance which is not always fulfilled, material properties become more important. Recent studies have indicated that permeable granular bases perform better than cement treated or lean concrete bases. This generally requires the installation of a subdrainage system to provide the assurance that material performance can be maintained in the pavement over its useful life.

Stabilization can play an effective role in the improvement of pavement performance with drainage. Subgrade modification can improve the load carrying capacity of the pavement. It provides a stable platform for improved construction of drainage layers such as open graded subbases. It reduces the capillary action, reducing frost heave and ice lensing in the stabilized material. Stabilization of aggregate materials improves their erosion resistance when they become exposed to moisture. However, stabilization by itself is no substitute, in the long term, for adequate drainage which controls the moisture in the pavement.

2. MOISTURE

Damage inducing moisture can be found in all layers of a pavement structure, and it can come from a variety of sources as shown in figure 2. It is commonly

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assumed that groundwater and high water tables are the primary sources of moisture, but there is a growing amount of evidence that surface infiltration can be just as damaging, and contribute a significant amount of damage inducing moisture into the pavement system.[28,29] The degree of moisture damage depends on the surface characteristics, and the internal drainage conditions of the pavement materials and structure. An enlightening example of this can be found in a recent survey of 31 flexible pavement sections in Wisconsin.[36] In a specific number of these sections, the deflections were much higher during the Fall survey than they were during the deflection testing performed during the Spring thaw. This is a situation not normally expected, given typical seasonal conditions, which were found to exist. These pavements with the higher Fall deflections all contained a dense granular subbase, and the subgrades were typically sandy. These sandy subgrades could drain the frost melt water from the spring thaw quite easily, and thus did not show an increased Spring deflection. During the fall the major source of water changes to surface infiltration, and it is surmised that these slow draining subbase materials acted as a holder of the water, producing higher deflections, while in the Spring the water was well below the subbase level.

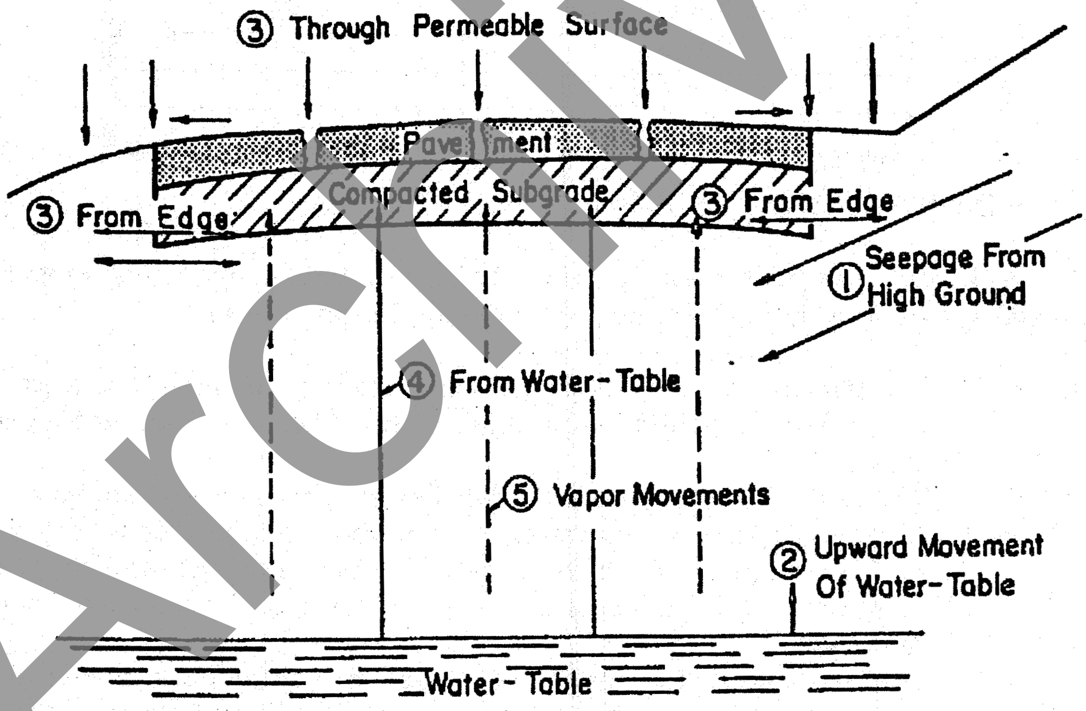

Figure 2. Sources of moisture in pavement systems.

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The amount of infiltration of surface water depends on the permeability characteristics of the pavement surface. Table 2 contains typical ranges of permeability of various old and new flexible pavements, illustrating the sealing effect of traffic on the surface.[32] Cedergren recommends that the surface permeability of rigid pavements be set at 0.20 in/hour, and at 0.50 in/hour for flexible pavements. The permeability of individual layers will affect how readily the entering water can exit the pavement, reducing the potential for damage.

Table 2. Permeabilities of old and new flexible pavements.

NEW PAVEMENT	PERMEABILITY, K, FT/DAY
US 101, air permeability	150
US 101, left wheel path	46
US 101, between wheel paths	90
California Spec.	40
OLD PAVEMENTS	**PERMEABILITY, K, FT/DAY**
South Africa, cracked	2.0
Belgium	7.0
Connecticut, traffic lane	4.4
Connecticut, shoulder	7.0

3. DRAINAGE REQUIREMENTS

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

- The pavement system and adjacent areas must be maintained as impervious as possible to minimize the infiltration of water into critical areas.
- The drainage facility should be designed with a water-removing capability such that infiltrating water can be removed in a very short period of time to minimize moisture-induced damage, and the water must not be allowed to back up and return into the pavement once removed
- The drainage system must be designed as a structural member of the pavement structure. It must not decrease the performance of the pavement or require exceptional measures to compensate for material problems.
- The installation of the drainage system must be coordinated in such a manner that it is not adversely impacted by any highway appurtenance

such as guard rails, sign posts, or delineators which may puncture and block the drain pipes.

- Maintenance must be planned for in the design, and must be performed on a routine basis.

The time for moisture to be removed from the pavement is a main element in the 1986 AASHTO procedure for assigning drainage coefficients. This time is a function of the characteristics of the materials, and the external factors of the climate. If the drainage system is to function as an integral element of the pavement, material selection is important. Often drainage materials require special handling and construction techniques, and the structural adequacy of the final structure should not be reduced by the addition of these materials. These two factors must be evaluated in view of the actual amount of moisture entering the pavement structure.

4. AMOUNTS OF WATER

The sources of water having the potential to enter the pavement structure and accelerate moisture damage include:

- Groundwater
- Melt water from ice lenses
- Vertical outflow

The determination of the amounts of each water type is precisely outlined in the Highway Subdrainage Manual, and in the recently released microcomputer program, DAMP, and the User's manual.[32,34,35] Application of these procedures requires a knowledge of water sources and material properties to derive the total amount of water entering the pavement, the net inflow. The net inflow quantity is required to evaluate material suitability for drainage, and to size the subdrainage system, should such a system be required. The knowledge of the sources of water are used to assist in the selection of a specific drainage system. All sources should be identified, irrigation activity is a source that has not been adequately considered.

5. TYPES AND USES OF SUBDRAINAGE

Subdrainage can be classified as to the source of the subsurface water they are designed to control, the function they perform, or their location and geometry. Often a subdrainage system is required to control water from the groundwater and/or from infiltration of water seeping into the pavement system. In doing such, the drainage system may intercept or cutoff the seepage above an impervious boundary, draw down or lower the water table, or collect the flow from other drainage systems. The most common subdrainage systems are best differentiated by their geometry as:

- Longitudinal drains

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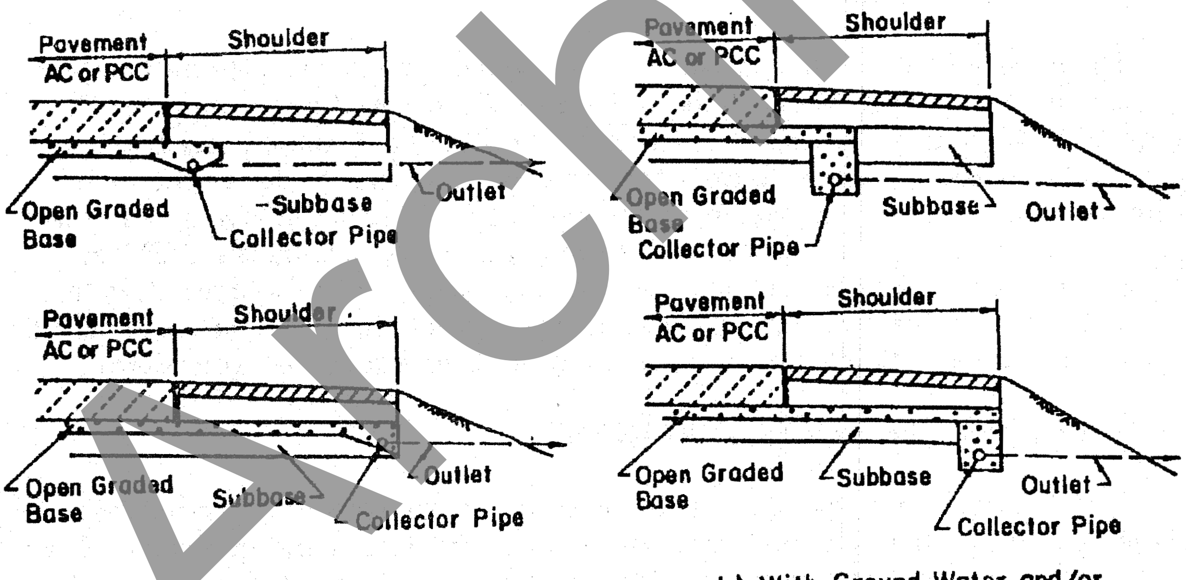
- Transverse and horizontal drains
- Drainage blankets
- Well systems

Each system may be designed and constructed to control water entering a pavement system, and complete the functions described above.

Longitudinal Drains

A longitudinal drain is located parallel to the roadway centerline as shown in figure 3. It can require a trench of specified depth, a collector pipe, and some form of protective filter. The trench may be of any size, but it must be sufficient to address the net inflow into the drainage system. The trench must be deep enough to intercept the groundwater, if that is the main source of water. The trench must be placed in a manner that it collects the water as close to the point of entry as possible, which normally requires placement as near the longitudinal lane-shoulder joint as possible without producing structural problems with paving operations destroying the installation before paving is completed. Specific details for sizes can be obtained elsewhere.



a) No Ground Water

b) With Ground Water and/or Frost Penetration

Figure 3. Typical cross section for longitudinal edge drain.

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The collector system in the edge drain is typically a four inch diameter rigid plastic pipe. The purpose of this component is to ensure rapid collection of the moisture that has been intercepted by the drainage system. Often the pipe is eliminated and a one sized gravel is used as the collector system. While this is not recommended, when a pipe is not used, the material must have exceptionally high permeabilities, and requires protection from the surrounding soil to prevent piping and clogging. This filter protection is typically provided by matching gradations of the trench backfill, or by using filter fabrics to wrap the trench. There are specific requirements which the granular backfill or fabric must meet to ensure performance of the system. The use of a stabilized permeable material can minimize backfill problems.

A newer type of installation that functions as a longitudinal drain is the geocomposite fin drain. This system consists of a plastic core wrapped with filter fabric. The plastic core provides the collection and removal area, and the fabric provides the filter protection. Precast plastic parts are used for outlets and end pieces which can be attached as needed. There is no trench excavation for this device as it is typically less than 2 inches thick, and some 18 to 24 inches deep, and can be "plowed" or placed into a groove formed with a wheel saw. These devices do not allow for cleaning after installation, which may limit their usefulness if they clog from migrating fines in the pavement system. They are currently not recommended for use in new construction or reconstruction.

Transverse Drains

These installations are commonly located at right angles to the roadway centerline, as illustrated in figure 4, although in some instances they may be skewed, forming a "herringbone" pattern. These drains have been used to drain surface infiltration and groundwater from bases or joints, and have been particularly useful where the longitudinal grade is such that flow tends to be primarily in that direction, and not transversely. The components of this system are similar to longitudinal drains, and require interception of the water, a collector system, and a filter protection system. The same requirements as outlined for the longitudinal systems apply here.

In areas of seasonal frost there may be problems with transverse drains. There have been instances where the sections of pavement without drains have developed frost heave while the drained areas have been stable. This leads to a rough ride during the winter months.

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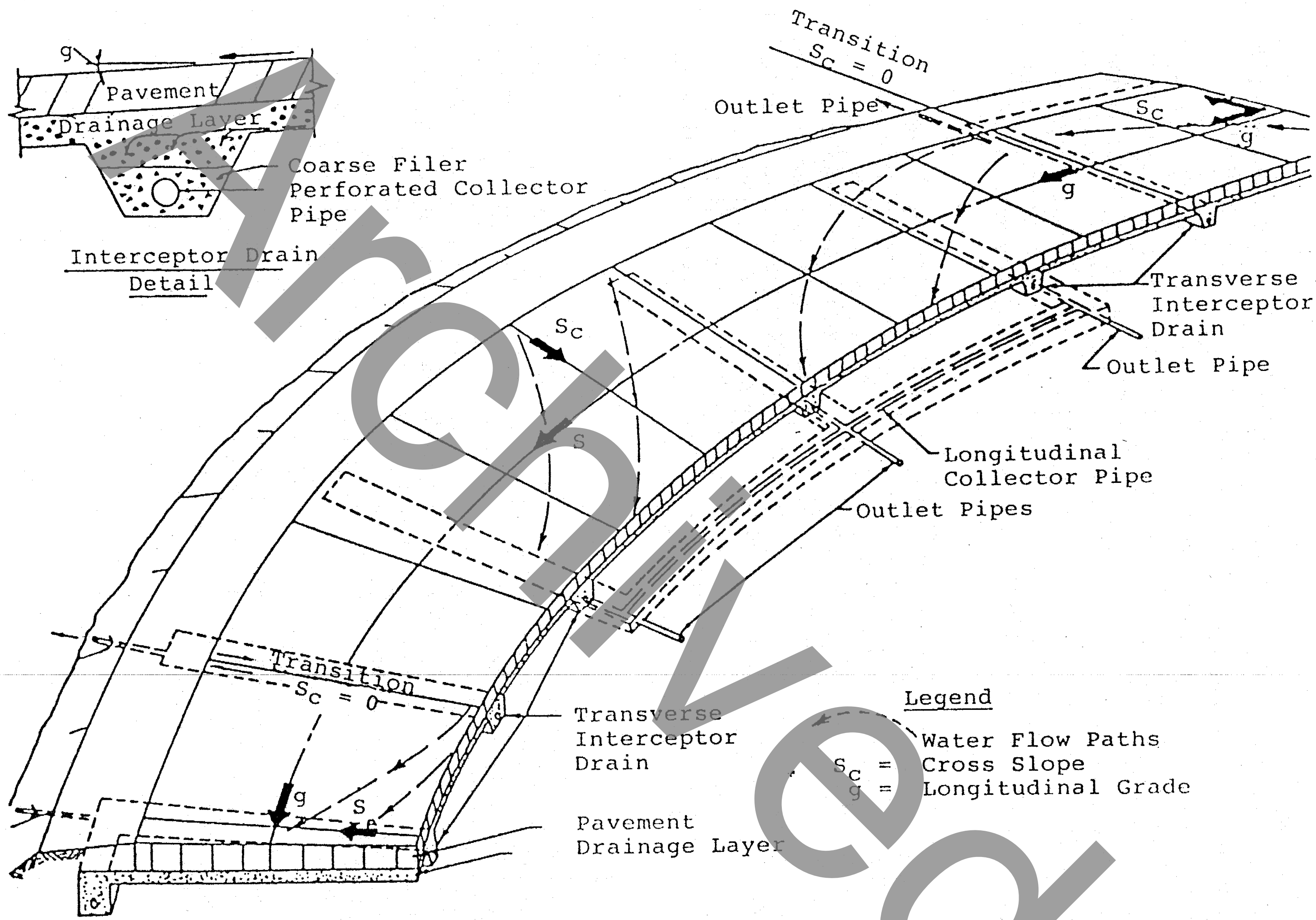

Figure 4. Transverse drains on superelevated curve.

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Drainage Blankets

A drainage blanket is a very permeable layer whose width and length (in the direction of flow) is large relative to its thickness as illustrated in figure 5. Properly designed blankets can be useful in controlling both groundwater and infiltration, depending on the existing conditions. There has been a renewed interest in drainage blankets and the material used in these systems in recent years.

The drainage blanket can be used beneath, or as an integral part of the pavement structure to remove infiltration or to remove groundwater from both gravity and artesian sources. Although relatively pervious granular materials are often utilized for base and subbase courses, these layers will not function as drainage

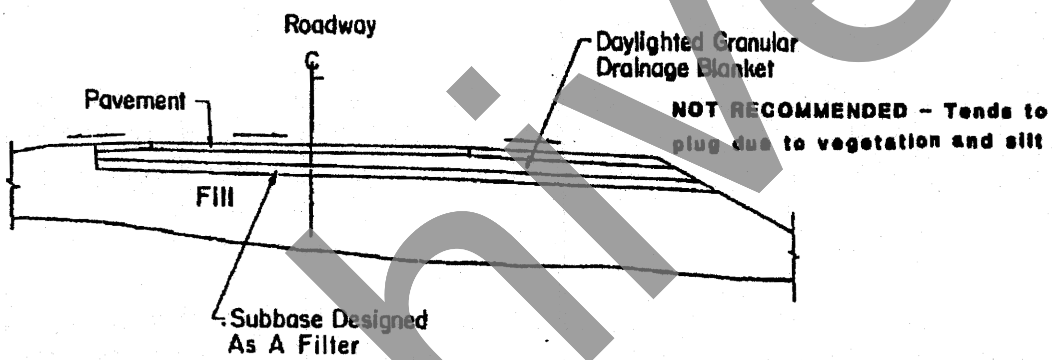

Figure 5. Drainage blanket.

blankets unless they are specifically designed and constructed to do so. This requires an adequate thickness of material with a high coefficient of permeability, a positive outlet for the water collected, and in most instances, the use of one or more protective filter layers. Specific material requirements and construction considerations will be discussed in a latter section.

A positive outlet for the collected water must be provided. Edge drains are recommended as an integral part of a drainage blanket installation, acting as the collector and outlet system for the installation. Although "daylighting" is sometimes done on drainage blankets, it is not a recommended method of controlling the water being collected. While better than a bathtub construction a more positive, maintainable outlet should be provided.

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Well Systems

Vertical well system are used to control the flow of groundwater and relieve porewater pressures in potentially troublesome highway slopes. They are often pumped to lower water tables during construction, or simply left for overflow of artesian pressures. Often a collector system is constructed to remove water from the base of the wells, maintaining a dry condition. This aspect of water removal is more detailed than typical subdrainage considerations, and will not be pursued further here.

5. MATERIAL CONSIDERATIONS

As part of a subdrainage system for a pavement, the following materials must be evaluated before being used in the drainage installation:

- Drainage pipe
- Drainage medium
- Envelope materials

Each of these materials has special concerns when used in various parts of the drainage system. If these concerns are not addressed, the total drainage installation will not function as an integrated unit and drainage will not be ensured.

Drainage Pipe

While there are several types of pipe material available for use in longitudinal edge drain installations, the most popular is the corrugated plastic tubing. This material is manufactured in rolls of about 200 to 300 ft (61 - 91 m). For subsurface drains, the diameter is typically between 4 to 6 in (10 to 15 cm). An analysis of flow characteristics will provide the appropriate diameter required for water removal, which should be less than the diameter specified.

The flexible nature of the corrugated tubing requires special handling when compacting trench material as they can be collapsed from overloading. The flexibility of these tubings makes them susceptible to installation problems from the bending in the pipe. Care must be taken during installation to ensure the pipe lies flat in the trench. This extra consideration for positioning the pipe typically makes flexible pipes unsuitable for outlets. It is recommended that a solid pipe be used to ensure proper grade tolerances are achieved. The flexible pipes curl up at the ends making proper installation problematic., and adding to the possibility of damage from construction equipment.

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Drainage Medium

The drainage medium provides the material through which the water must flow before it reaches the collection system of pipes. This material is placed in the trench, or in the pavement if an open graded drainage layer is used as a permeable base or subbase. The actual gradation of these materials is not critical once the permeability has been established. For a permeable base layer, the permeability should be above 1,000 ft/day. Ths can easily be obtained with stabilized open graded materials

If a longitudinal edge drain is used with a permeable base, the edge trench medium should be the same as the permeable base layer. If a material with a lower permeability is used in the edge trench, the incompatibility between the two materials could cause water to back up if flow is restricted in the edge trench. Depending on the amount of water being handled, this backup could produce adverse affects by holding water in the pavement for a longer time than normal.

Edge drains placed without permeable base layers must have a permeability capable of handling the predicted amount of infiltration water for that pavement structure. Excessively high permeabilities are not normally required, and several states are using concrete sand with permeabilities in the range of 200 plus ft/day. If adequate permeability can be obtained to handle the amount of infiltration, these materials are satisfactory, and the next consideration is the envelope material.

Envelope Material

The function of the envelope material is to protect both the drainage medium and the surrounding soil. Water movement, and repeated stresses from the traffic, provide the means for fines to migrate from one material to another. This may result in loss of pavement support and premature distress of the pavement structure. With a permeable base the subgrade fines must not be allowed to be pumped upward into the base, clogging the drainage, and reducing the strength. In longitudinal edge drains, the surrounding soils must not be allowed to be carried into the drainage medium by the groundwater moving through the drain. If a layer is not added to protect the drainage medium, the drain will eventually cease to function, and the structural adequacy of the pavement may be compromised.

Aggregate

The envelope material may be either an aggregate, or a filter fabric. When an aggregate envelope is used, the gradation of the envelope material must be matched to the gradation of the drainage medium and the surrounding soil. The relations which must be satisfied come from the Terzaghi gradation matching criteria, which establish limits for grain sizes present in each material as indicated by the D_{xx} criteria where the subscript xx represents the percent finer.[28] The relationships are:

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$D_{15(DRAIN)}/D_{85(ENVELOPE)} \leq 5$
$D_{15(ENVELOPE)}/D_{85(SUBGRADE)} \leq 5$
$D_{50}/D_{50} \leq 25$ for both combinations

The drain and envelope must be checked individually to prevent material in the envelope from migrating into the drain material. If this is not checked, the drain will become clogged. There are other criteria to check to prevent clogging of the drain, but this criteria represents protection from infiltration of fines, which is the major problem in pavement subdrain installations.

Fabric

Filter fabric are either woven or non-woven mats constructed of polypropylene or nylon fibers. These fabrics take the place of the aggregate envelope material and serve the same function as the aggregate envelope material. Because there are no aggregate particles in the fabric to migrate into the drain material, there is no requirement to match the fabric to the drain material. The subgrade soil must be checked and the gradation evaluated to determine if it is compatible with the fabric. Each fabric has openings which are either woven into the fabric, or pressed into the fabric between the individual threads. These openings are classified by the "Apparent Opening Size" (AOS) of the fabric, which represents the opening size in the fabric in millimeters that has 95 percent of the openings smaller.[38] The following criteria apply for the fabrics to ensure retention of material:

- Coarse Grained Subgrade, Steady flow, $AOS = B*D_{85}$
  - $B = 1$ for: $2 < C_U > 8$
  - $B = 0.5$ for: $2 > C_U < 4$
  - $B = C_U/8$ for : $4 > C_U < 8$
- Coarse Grained Subgrade, Dynamic flow, $AOS < D_{15}$
- Fine Grained Subgrade, Steady flow
  - $AOS < D_{85}$ for woven fabric
  - $AOS < 1.8D_{85}$ for non woven fabric
  - $AOS < 0.3$ mm
- Fine Grained Subgrade, Dynamic flow, 50 percent opening in fabric < $0.5\ D_{85}$
- Fabric permeability must be at least 10 times greater than the subgrade soil for severe installations.

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The following criteria apply to prevent clogging of the fabric:

- The 95 and 15 percent opening size in the fabric must be greater than three times the D_{15} of the subgrade.

If the 95 percent opening size for clogging is greater than the AOS for retention, the gradient ratio test should be performed, and the gradient-ratio should be less than or equal to three.

Some agencies make use of filter fabric to directly wrap the pipes in the collector trenches, eliminating or reducing the need for the aggregate drain material. This use of fabric alone increases the importance of carefully evaluating the fabric and its match to the gradation of the subgrade soil. The aggregate drain material provided a measure of safety, presenting somewhat of a buffer between the pipe and the subgrade. Without the aggregate, the fabric is the only material between the pipe and subgrade capable of preventing migration of fines which results in loss of pavement support. These fines may wash through the fabric,or they may clog it making the drain ineffective.

6. PERMEABLE BASE CONSIDERATIONS

The materials described in the previous section are commonly available, but the application of permeable bases, and the consideration of erosion in the base for rigid pavements necessitates further discussion of particular material composition and construction questions.which must be addressed before these materials can be used in an appropriate manner with a resulting improvement in pavement performance.

Permeable Bases

The open graded permeable base (OGPB) approach to improve drainage performance in pavements has received increased attention in recent years, primarily under rigid pavements. In these installations the OGPB does not function as a structural layer in the pavement structure, it provides material only for water removal from surface infiltration. Although it may possess some structural capcity, the function of the OGPB is to enhance the water removal from pavement reducing the problems that develop when the pavement and underlying foundation material are exposed to moisture for prolonged times. The permeable base is composed of an open-graded, crushed, angular aggregate with very few fines. This aggregate may be untreated, or it may be stabilized with either asphalt cement or portland cement. The final material must have adequate permeability to quickly drain any available water, and maintain adequate stability for construction operations and subsequent repeated applications of heavy traffic loadings.

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A stabilizer may be necessary if there is a the need to carry some construction traffic on the drainable base. An untreated OGPB can be unstable even under light construction traffic. Even with an aggregate that is extremely angular, only a few agencies have been able to construct a relatively stable working platform. The addition of the cementing agent serves to maintain structural integrity in the compacted aggregate under the traffic, not to impart any increased structural capacity to the material. For this reason, several agencies give little or no structural capacity to the OGPB in their structural pavement design procedure. Research is indicating that these materials, even the untreated materials provide support that is comparable to traditional dense graded bases. The main concerns which must be met to obtain this performance is adequate design of the OGPB (gradation and crushed faces) and construction that seats the aggregates and locks the structure together.

Untreated permeable base

States which have recently tried untreated permeable bases include: Iowa, Kentucky, Minnesota, New Jersey, Pennsylvania, and Wisconsin.[39] Kentucky uses AASHTO No. 57 stone for their base. New Jersey uses an equal part blend of AASHTO No. 57 and No. 9 stone. This New Jersey gradation is considered to be the most stable untreated permeable base with high permeability. The remainder of the States mentioned here essentially used the gradation for their dense graded base materials, and removed some of the fines to increase permeability. Typical permeabilities and gradations are shown in Table 3.

Table 3. Untreated permeable base gradations.

Sieve Size	Percent Passing					
	IA	KY	WI	MN	NJ	PA
2 in						100
1.5 in		100			100	
1 in	100	95-100	100	100	95-100	
3/4 in			90-100	65-100		52-100
1/2 in		25-60			60-80	
3/8 in			20-35	35-70		35-65
No. 4		0-10	0-10	20-45	40-55	8-40
No. 8	10-35	0-5	0-5		5-25	
No. 10				8-25		
No. 16					0-8	0-12
No. 30						0-8
No. 40				2-10		
No. 50	0-15				0-5	
No. 200	0-6	0-2		0-3		0-5
K, Permeability ft/day	500	20,000	18,000	200	2,000	1,000

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These materials should essentially contain all crushed material to achieve stability during construction. Construction of these materials should focus on seating and locking the aggregate, not densifying the aggregate. If construction traffic is allowed on the permeable base, a roller should be used ahead of the paver to reseat and smooth the surface, to lock the aggregates and reduce the rutting.

<u>Treated permeable base</u>

States which have recently tried treated permeable bases include: California, North Carolina, West Virginia, Oklahoma, and Wisconsin.[39,45] The predominant gradation for these mixes is AASHTO No. 57 stone. Common permeabilities run from 3,000 to 20,000 feet per day, with some going even higher. Extremely high permeabilities are generally in excess of what is required to handle the surface infiltration into concrete pavements, and stability considerations may arise in these blends.

When asphalt cement is used as the binder, the asphalt content is generally in the 2 to 3 percent range. Most installations have used 2.5 percent with success. Again, this binder is not to impart structure to the mix, but to provide stability to the mix under construction traffic. The aggregate must provide adequate stability without the binder if they are going to be successful. If the aggregate is unstable without the binder, it should be reexamined and altered before use in an OGPB installation.

Portland cement binder provides a very stable permeable base material. The amounts of cement are quite small, typically in the 2 - 4 bag per cubic yard range, which may still be far in excess of what is required. A recent study in Wisconsin investigated the effect of cement content on stability of the permeable bases, and their ability to carry construction traffic.[40] The findings indicate that rolling the mixes to achieve higher compaction actually destroys the structural integrity of the material, and have recommended that these stabilized materials not be over-rolled for density.[41] The study of cement contents indicated that cement contents of 150 pounds per cubic yard can be used for short hauls over a stable foundation. A cement content of 200 pounds is probably suitable for general use. An OGPB with a cement content of 250 pounds per cubic yard is recommended where questionable support conditions exist or where heavy trucking will take place on the permeable base. These recommendations are in line with the low end of the 2 to 4 bag mix recommendations typically used. Caution must be advised when using the higher cement contents. Stiffer bases can produce non-uniform support under a slab that is curling when compared to an untreated base. This non-uniform support can shorten the life of the pavement, negating the benefit of carrying construction traffic. An additional factor for the design of the pavement is the increased friction which may develop when the concrete of the pavement penetrates into the base. This is particularly important during the initial curing period.

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Construction Concerns

In all cases, these OGPB installations must be protected with a filter layer to prevent intrusion of subgrade fines which can destroy the drainage capabilities of the OGPB, and reduce load carrying capacity significantly. Recent installations have used a six inch thick OGPB layer. A typical installation is shown in figure 6. This may be a filter fabric, or a preferred granular layer that is typically four inches thick. Several States use their standard dense graded base material for this layer. Care should be taken to ensure that the gradations are compatible with the criteria given previously. Some research indicates that filter fabrics may act as a small source of water, holding some moisture directly below the fabric.[42] There is no documented long term performance for filter fabrics which are more difficult to construct over without damaging the fabric and compromising the filtration requirements.

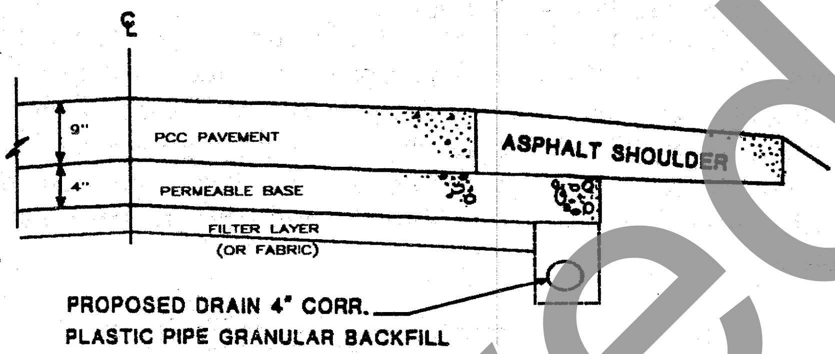

Figure 6. Typical permeable base pavement section.

The permeable base is typically drained with a longitudinal edge trench. This trench is typically constructed using the same material as the permeable base, and this presents some construction concerns. It is desirable to avoid having construction traffic on the trench which requires moving the trench away from the edge of the pavement. Because of the high permeability in the base and trench, it is not necessary to have the drain directly at the lane shoulder joint, as is typically the norm for retrofit installations with dense graded bases. The edge drain may be placed some distance away from the lane shoulder joint, or it may be placed at the outer edge of the shoulder. When this choice for placement is used, the permeable base must be extended out to the edge drain. There must be a continuous connection of permeable base material with the trench.

For cost reasons, placement of the trench under the shoulder close to the pavement would be preferred, to avoid use of OGPB material under the entire shoulder. The permeable base should not be extended beyond the edge drain unless extra care is taken to ensure that the slope of the extra material is such that it will drain back into the edge drain. Typical slopes would trap and pond water at the outer edge of the OGPB layer, acting as a reservoir under the shoulder accelerating

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shoulder deterioration. In areas of frost heave, differential frost heave may develop with non-uniform material usage.[44]

Erosion Potential

A new design consideration in the 1986 AASHTO pavement design guide addresses the erosion of base material from pumping actions under jointed concrete pavements. There is a loss of support (LOS) parameter in the design procedure that models the distance erosion has developed under the edge of a slab. This reduced support increases the stress in the slab and reduces the life of the pavement. The open graded permeable base materials eliminate the problem of erosion and loss of support because they remove the presence of free water which is necessary for erosion to develop. Dense graded base materials, on the other hand, are highly susceptible to erosion, whether stabilized or not. Typical design values were given in the pavement design chapter (Volume II, Chapter 3).

Extensive studies in France have shown the erosion potential of various materials and they have developed testing procedures and guidelines for material evaluation.[44] The PIARC studies have developed a series of recommendations relative to pumping and its control through drainage structure and use of low-erodability materials. There are five classes of erodible materials:

- **A** "Extremely erosion resistant" - Example: lean concrete with 7 or 8 percent cement; bituminous concrete with at least 6 percent bitumen.
- **B** "Erosion resistant" - Example: cement-treated granular material with 5 percent cement, treated in the plant.
- **C** "Erosion resistant under certain conditions" - Examples: cement-treated granular materials with 3.5 percent cement, treated in the plant; bitumen-treated granular material with 3 percent bitumen.
- **D** "Fairly erodible" - Examples: granular material treated in place with 2.5 percent cement; fine soils treated in place; untreated granular materials.
- **E** "Very erodible" - Example: contaminated untreated granular material; untreated fine soils.

Granular materials stabilized with either cement or asphalt would qualify as erosion resistant depending on the amount of stabilizer and method of mixing. The PIARC study investigated the economics of drainage installations and low-erodability material on the performance of the pavement. Among their findings they stress the need to prevent water entry into the pavement through sealing, provide adequate maintenance of the drainage system, and increased dimensions of the drainage

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structures to handle the water more efficiently. The increased size for drainage includes increased voids in aggregates (30 vs 12 percent typical), and general increases in sizes of drainage pipes, and shorter distances between outlets.

7. THE NEED FOR DRAINAGE

Not all pavements require the installation of subdrainage systems to perform adequately over their life. Low traffic volumes mitigate the effect of moisture. Areas with low rainfall and/or light traffic will not benefit economically from drainage, particularly if they use materials which are moisture resistant, and have good maintenance programs. Areas with high rainfall will undoubtedly require drainage to handle water entering the pavement, but adequate maintenance routines for sealing and the use of highly moisture resistant materials can reduce the reliance on drainage for improved performance if positive load transfer is provided for PCC pavements. With medium or high traffic the increased performance anticipated justifies improved drainage features.

The use of stabilized materials reduces the impact of moisture on the pavement design process. Drainage coefficients are used for untreated granular materials in both flexible and rigid pavements. The stabilized materials are assumed to be moisture resistant in the AASHTO procedure, and are not assigned a reduced structural adequacy, representing loss of strength due to water. Because this degree of moisture insensitivity is not universally achieved with all forms of stabilization, this increases the importance of adequate mix design, stabilizer content selection, and construction on the development of the final properties that are developed in the stabilized material. Inadequate compaction, or curing, or selection will produce a material that does not possess moisture resistance. This will result in premature failure which may relate to moisture effects. For rigid pavements, short slabs and positive load transfer are recommended for all but low volume truck routes.

A thorough review of the pavement, climatic area, geology, and materials is required to determine if drainage is required. A thorough treatment of all aspects of evaluating a pavement for drainage needs is contained in the FHWA training course "Pavement Design - Principles and Practice." In areas of high annual rainfall, or areas with high seasonal rainfall, drainage systems, even with stabilized materials provides an increased measure of safety for the pavement structure. The total pavement must be designed as a total system and no one feature can be emphasized without considering the interaction of other factors if satisfactory long term pavement performance is to be assured. Together, stabilization, drainage technology, and pavement design can provide for an excellent pavement.

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8. EXAMPLE DRAINAGE APPLICATIONS

A drainage system is to be designed for a reinforced jointed concrete pavement consisting of two 12-foot lanes constructed on a 6-inch granular base, having dense-graded asphalt shoulders in an area without frost problems or high groundwater. The joint spacing is 45 ft. The permeability of the subgrade was determined to be 0.002 fpd, and the base is 0.8 fpd.

The precise details for this analysis can be found in the FHWA Subdrainage Manual, and the microcomputer program DAMP.[32,34] The figures and nomographs necessary for all solutions are presented in these two manuals, several figures will be reproduced here for illustrative purposes.

Determine Net Inflow

Because there is no groundwater or frost melt, the sole source of water will be surface infiltration which can be determined by the following:

$$\mathbf{I_c[N_c/(W + W_c/(W\ C_s))] + k_p}$$

Where:

	I_c	is a standard value of 2.4 cfd/ft
	N_c	is the number of longitudinal joints, 2
	W	is the width of drainable base, 12 ft
	W_c	is the length of the crack or joint in the surface, 12 ft
	C_s	is the spacing of joints or cracks, assumed to eventually be 12 ft
	k_p	is the permeability of pavement surface, 0.0

This calculates a net inflow of **0.56 cfd/sq ft**. Multiplying this by the width of the pavement , 12 ft produces an inflow to the drain of **6.7 cfd/ft** of pavement length. This is the amount of water entering through all cracks, which must be trensmitted by the base to the edge drain. The next step is to determine if the base can drain this much water to the drain.

The drainage capacity of the existing base is determined from the forumla:

$$\mathbf{q_b = K_b \times t_b}$$

where:

	K_b	is the permeability of the base, 0.8 fpd
	t_b	is the thickness of the base, 0.5 ft

This calculated a drainage capacity of this base material of **0.4 cfd/ft**. This means that the infiltration through cracks, etc. will not be handled by the base, and will runoff the surface. It will potentially enter the longitudinal joint which is typically directly over the edge drain. Therefore the infiltration through the longitudinal joint should

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be calculated. This is q_d = **2.4 cfd/ft**, which should be used as it is more than will reach the drain through the base.

Adequacy of Trench Width

The 12 inch width must be checked using the formula:

$$\mathbf{q_d < 2b(k_t)}$$

where b is the trench width, 12 in

k_t is the permeability of the trench backfill material, estimated from the gradation, shown in figure 7, and P0. = 0, D_{10} = 0.4mm,, and density = 105 pcf, produces a permeability of 1000 fpd.

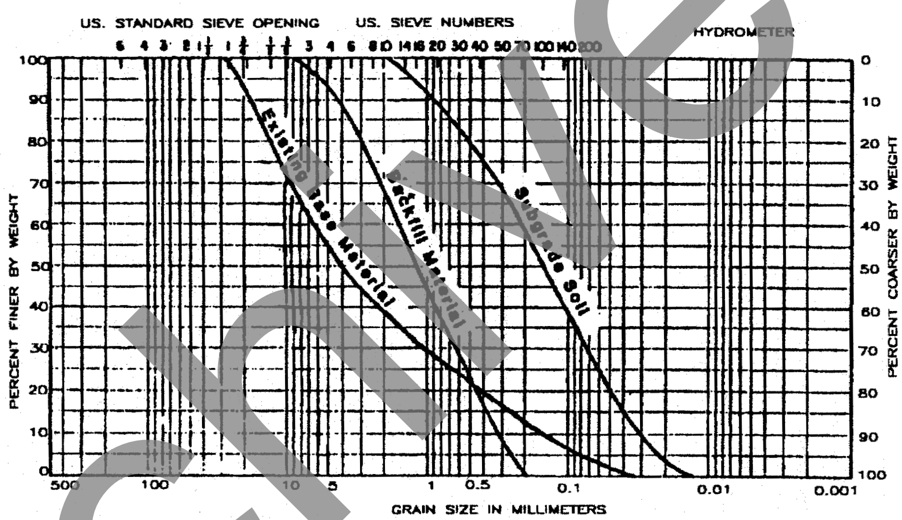

Figure 7. Gradation curves for soils in example problem.

Since 2.4 is less than 1 times 1000, the trench width is adequate. It is not advisable to decrease the trench width below specified minimums due to construction equipment requirements.

Filter Adequacy

First the backfill/base combination must be checked using criteria shown earlier. the D_{15}/D_{85} is 0.4/15 = 0.03 which is less than 5, and is satisfactory. The D_{50}/D_{50} ratio is 1.2/4 = 0.3 which is less than 25, and is satisfactory. These checks will satisfy the plugging criteria.

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Next the trench backfill/subgrade combination must be checked. The ratios for this combination are 0.4/0.7 = .57 which is less than 5 and is satisfactory, and 1.2/0.13 = 9.2 which is less than 25 and is satisfactory.

The backfill is a satisfactory material. If the base course was permeable, which it is not, the combination of base/subgrade would have to be checked. If an open graded base material was used, and a filter aggregate layer was placed beneath the open graded base, the base/filter, and the filter/subgrade combinations would need to be checked.

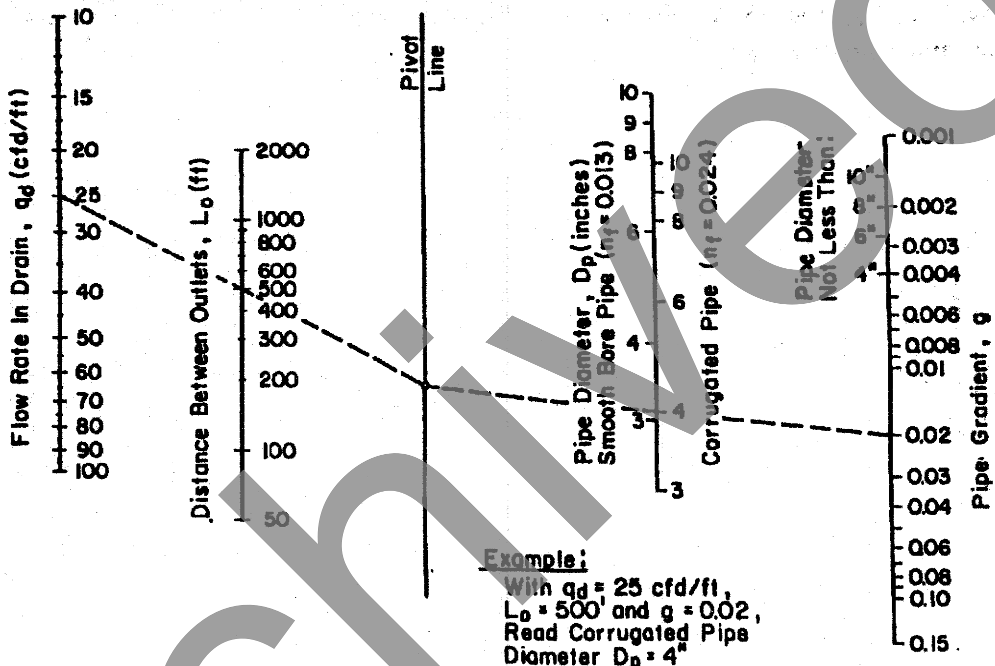

Figure 8. Nomograph for pipe sizing and outlet spacing.

Adequacy of Pipe Size

Figure 8 may be used to design and/or check pipe sizing and outlet spacing. The lowest flow rate on this nomograph (10 cfd/ft) exceeds the design flow rate for this example. Using the nonograph minimum flow rate it can be demonstrated that the proposed 4-inch pipe size will be adequate. Outlets should be spaced at no greater interval than 600 feet to allow for cleaning, and 150 to 200 ft is more common for normal operations.

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Evaluation of Performance

It must be recognized that even when all design parameters are properly evaluated and included in the design, the performance of the subdrainage may not be as expected, and the benefits discussed earlier may not be attainable, and the integrity of the stabilization, and the pavement will be comprimised. It is necessary to implement an evaluation program that will provide data the engineer can use to determine if there are any areas that may be detrimental to long-term performance. These programs cannot be short-term evaluations because distress takes time to develop. Measurements of outflow cam be made, and use of video cameras similar to those used to log sewer pipe can be used to document the condition of the pipe system quite effectively.

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CHAPTER 4 CONSTRUCTION PROCEDURES AND EQUIPMENT

1. INTRODUCTION

The key to successful construction of a stabilized soil system is obtaining a thorough mixture of pulverized soil or aggregate with the correct amount of stabilizer and sufficient moisture to permit maximum compaction. The curing process must include favorable temperature and moisture conditions for strength development. In addition, the stabilized soil must be protected from traffic to prevent abrasion and allow for proper curing.

There are two construction methods associated with soil stabilization: Mixed-in-place and central plant mixed. The choice of method is dependent on the project economics and equipment availability. If in-place materials can be effectively and economically stabilized, then mixed-in-place would be the preferred method. However, if borrow material is to be used on a large project, it may be more economical to use central plant mixing techniques.

Whether in-place mixing or central plant mixing is performed, there are several different pieces of equipment available today which can successfully mix the stabilizer and soil. Techniques vary among equipment but the general construction principles and objectives are the same. Figure 9 illustrates the various techniques employed by in-place and central plant mixing equipment.

2. MIXED-IN-PLACE

Although mixture uniformity in a mixed-in-place operation is typically less than that obtained using central plant mixing operations, satisfactory results can be obtained with road mixing equipment for all of the major chemical stabilizers. In-place mixing is done in one of three ways:

- Mixing with existing subgrade materials.
- Mixing with a borrow source at the construction site.
- Mixing with a borrow source at an off-pavement site and transporting to the pavement site.

Mixing operations with existing subgrade materials are often performed with single- and multiple-shaft flat type mixers or motor graders. Mixing with borrow materials is often performed with windrow or hopper type mixers if base or subbase

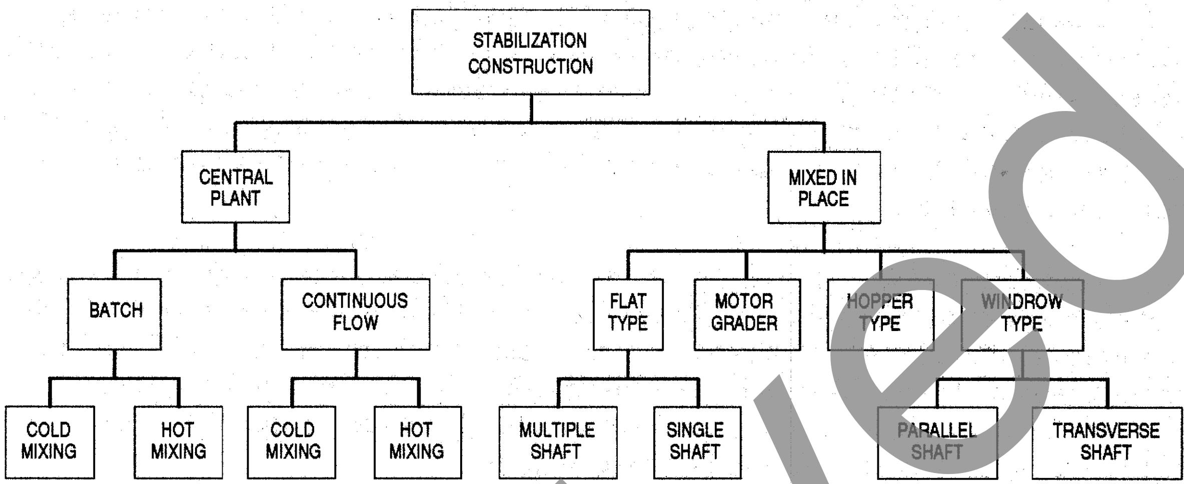

Figure 9. Soil stabilization construction equipment.

course materials are to be produced. In either case, the following five construction steps are employed:

1. Soil preparation
2. Stabilizer application
3. Pulverization and mixing
4. Compaction
5. Curing

Of the three mixed-in-place procedures, stabilization of in-situ subgrades is most common. The following sections discuss the procedures associated with each in-place mixing method.

Subgrade Stabilization

The five construction steps mentioned above are basic to the use of lime, cement, asphalt, and fly ash stabilization of existing subgrade materials.

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Soil Preparation

The existing subgrade soil should be brought to the grade and alignment as shown on the construction plans. Next, initial scarification and partial pulverization should be performed to the specified depth and width of stabilization. During and after scarification and pulverization, all deleterious materials such as stumps, roots, and stones greater than 3 in (76 mm) should be removed. In situations where the soil is too dry, water should be added to aid in pulverization. This also supplies a portion of the water required for the mixing stage of stabilization. If the soil is extremely wet, the scarifier-pulverizer operation can aerate and dry the soil.

Initial scarification and partial pulverization is commonly achieved by use of a grader-scarifier or rotary mixer (figures 10 and 11). However, various types of harrows, plows, cultivators, and other agricultural equipment have been successfully substituted for the normal highway construction equipment in the soil preparation phase.

Figure 10. Grader-scarifier used in soil preparation.

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Figure 11. Rotary mixer used in soil preparation.

Stabilizer Application

In asphalt stabilization, the asphalt can be applied to the soil by one of two means. It may be sprayed or distributed from an asphalt distributor or it may be injected into the mixing chamber of a travelling rotor mixer. This latter method of asphalt application is preferred because the asphalt is distributed more evenly. It is important in both methods that the soil be at the proper moisture content (less than 3 percent) prior to asphalt application.

The rate of asphalt application can be determined based on the proposed stabilizer content, stabilization widths and depths, and the forward speed of the applicator. In the distribution method, a distributor traverses the prepared subgrade distributing proportioned asphalt through spray bars. The soil and asphalt is then uniformly mixed and laid back down in preparation for further mixing or compaction.

The injection method employs a rotary mixer equipped with internal spray bars. The mixer traverses the prepared subgrade, picking up the soil, injecting and mixing the proportioned asphalt, and discharging the material in preparation for further mixing or compaction.

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Distribution of lime and cement can be performed by either spotting bags on the roadway or by applying bulk stabilizer directly from transport trucks. In addition to these two dry methods, lime can be applied in a slurry form should dusting present a problem during dry application. Slurry application of cement is not practical due to rapid hydration.

The use of bagged lime or cement is generally the simplest method but it entails greater labor costs and slower production. This method is most practical on small projects such as streets or on projects where there is difficulty in utilizing large equipment. Bag placement consists of spotting bags (typically 50 lb) of stabilizer in a pre-determined grid pattern such that the required stabilizer content can be met. The bags are then slit and the stabilizer is dumped into piles or transverse windrows. The stabilizer is then levelled either manually with rakes or mechanically with spike-toothed harrows or tractors equipped with drags.

On large projects where dusting is not a problem, bulk application is much more economical. Bulk spreading can be achieved directly using a self-unloading transport with a mechanical spreader or a bulk haul unit equipped with a pneumatic spreader as shown in figure 12. A variation of this type unit is shown in figure 15, in which a pneumatic device is used to pump the stabilizer into a mechanical spreader for application. The bulk application method is the least costly method of spreading stabilizer because there is no rehandling of material and large payloads can be transported and spread quickly. However, even though application is rapid, it requires field control to ensure proper spread rates.

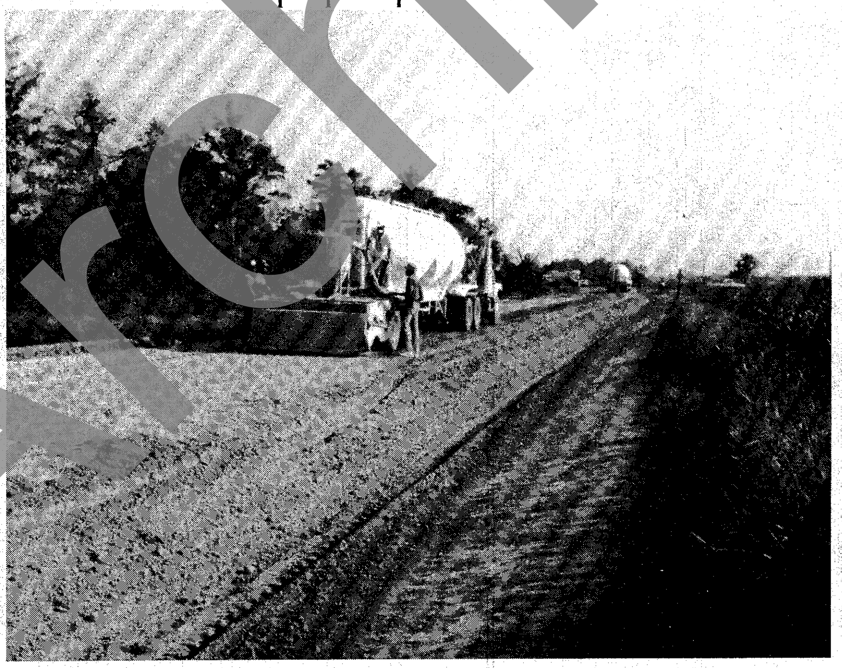

Figure 12. Bulk application using transport with pneumatic pump and mechanical spreader.

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Two common methods are used for ensuring proper spread rates. One is to establish that the applicator is consistently covering a predetermined area based on the known weight of stabilizer in the transport unit before and after application. The other method requires the use of a large flat receptacle (i.e., a pan or canvas 1 to 2 yd^2 in area) and a scale. A forward truck speed is selected and the mechanical spreaders are calibrated by weighing the amount of stabilizer caught on the pan or canvas.

In most lime-fly ash stabilization projects, lime and fly ash are spread separately. However, it is possible to preblend these two components before spreading. When the lime and fly ash are preblended, it is necessary that they be stored in a dry state. The preblend is normally spread in the dry condition.

If lime and fly ash are spread separately, standard lime spreading techniques are utilized. Although fly ash is occasionally spread in a dry state, it is generally conditioned with water (i.e., residual moisture content of 15 to 25 percent) prior to spreading. In dry form, fly ash is very dusty and flowable, which makes spreading somewhat difficult and time-consuming. Upon delivery of fly ash in dump trucks, the stabilizer may be dumped and spread using a spreader box, motor-grader shown in figure 13, or other type of spreader.

Figure 13. Spreading of dry fly ash using motor grader.

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Dry application of lime and cement can occasionally create dusting. In order to limit the amount of dusting, water may be sprayed onto the lime which has been spread. However, in cases where dusting is a severe problem with dry lime application, water can be mixed with either hydrated lime or quicklime to create a slurry. Hydrated lime and water are mixed in a central mixing tank, jet mixer, or in a tank truck. Typical mixing proportions are 1 ton of lime and 500 gallons of water. Once the components have been sufficiently mixed, the slurry is spread over the scarified roadbed through tank truck spray bars or is added to the soil during the mixing operation. Cement is normally not slurried because of its rapid hydration process.

Quicklime can be made into a slurry, which is preferred because of the potential for caustic reactions when used dry which can injure workers. Creation of a quicklime slurry requires the lime to be slaked first and then excess water can be added to produce the slurry. Portable batch slaking units have been developed to allow on-site quicklime slurry production. Processing the slurry generally takes about 1 to 1.5 hr and the exothermic action of the quicklime in water creates temperatures of about 185 °F.

Not only is slurry application dust-free, but better distribution is achieved. In addition, the spreading and sprinkling operations are combined, thereby reducing job costs. On the other hand, lime slurry requires more equipment and is not practical for use with wet soils, which could make the operation two to three times slower than dry lime application.

Double application of lime is often required when extremely plastic clays are encountered (P.I. ≥ 50). Lime is added in two increments to facilitate adequate pulverization and obtain uniform mixing. Typically 2 to 3 percent lime is added, partially mixed, and the layer is lightly rolled to seal the surface. After a 24- to 48-hour period, further pulverization is attempted, the final lime application is made, and the mixing of the lime and soil completed. The first lime application mellows the clay and helps in achieving final pulverization and the second lime application completes the lime-treatment process.

The primary objective of stabilizer application is to uniformly distribute the proper proportion of the stabilizer material. Field experience has indicated that mixing by itself will not greatly improve uniformity of distribution. Therefore, an important part of quality control is stabilizer application.

Pulverization and Mixing

Although motor graders and certain agricultural equipment can be used in the mixing stage of stabilization, the desired uniformity of mixing is not always obtained. Single- and multiple-shaft rotary (flat type) mixers shown in figures 14 and 15

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respectively are commonly utilized to pulverize and mix stabilizers with subgrade soils.

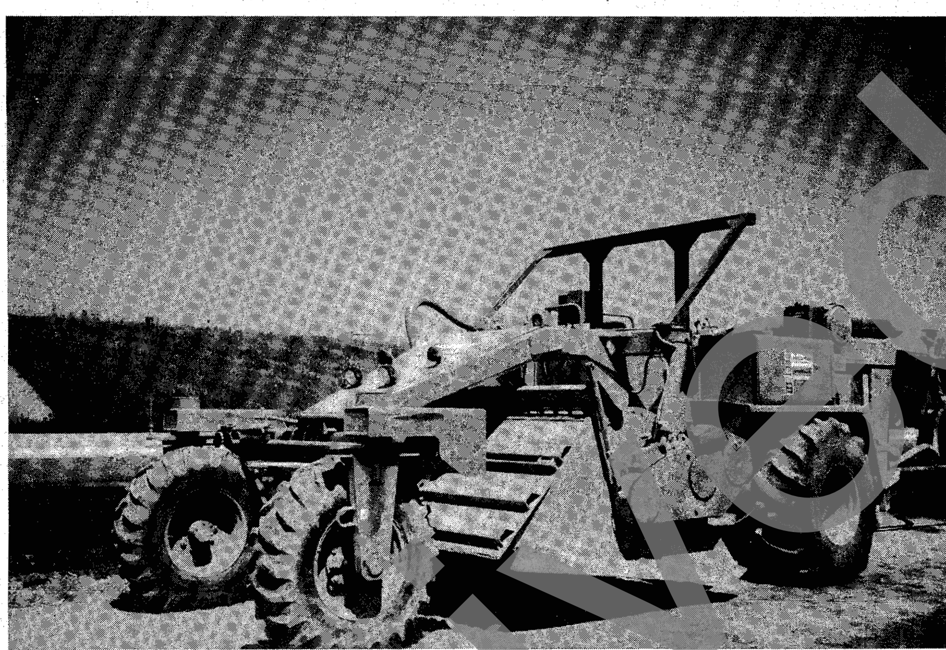

Figure 14. Single-shaft rotary mixer.

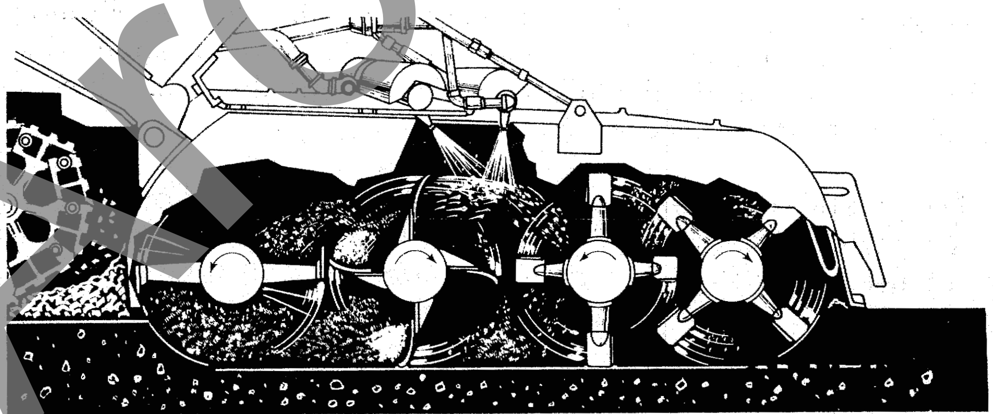

Figure 15. Multiple-shaft rotary mixer.[37]

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The use of single-shaft rotary mixers often consists of an initial pass, whereby the soil and stabilizer are mixed together prior to watering. Water is then added and a second pass is made. This process is repeated two or three times until a uniform soil-additive mixture at the desired moisture content is reached. The more recently developed rotary mixers are capable of picking up soil, injecting proper proportions of water for desired water content, and mixing and pulverizing the stabilized material, all in one pass. Rotary mixers are preferred for application of the asphalt because they allow direct metering of the additive during mixing, giving increased uniformity.

Mixing difficulty increases with increasing fineness and plasticity of the soils being treated. In-place mixing efficiency, as measured by the strength of the treated soil, may be only 60 to 80 percent of that obtained in the laboratory. Occasionally, this reduced efficiency is overcome by increasing the stabilizer content one or two percent over the laboratory determined value.

For lime stabilization, pulverization and mixing should continue until 100 percent of the soil binder passes a 1-in screen and at least 60 percent passes the No. 4 sieve. Most specifications for soil-cement mixtures require that fine-grained soils be pulverized such that at the time of compaction 100 percent of the mixture will pass a 1-in sieve and a minimum of 80 percent will pass the No. 4 sieve.

Mixing and pulverization requirements for lime-fly ash and cement-fly ash mixtures are typically those for lime and cement stabilization, respectively. It is crucial that uniform mixing be achieved because two stabilizers are being utilized and both must be mixed uniformly to achieve the desired results. Similarly, the mixing of asphalt with soil and water should be continued until a uniform mixture is obtained.

Compaction

Compaction should commence as soon as possible after uniform mixing of water and stabilizer when lime-fly ash, cement-fly ash, and cement are used as stabilizers. Most specifications require that materials be compacted within four hours of mixing and always be completed on the same day the soil is mixed with the stabilizers. However, less compaction effort is necessary to reach the desired density if the material is compacted within an hour of mixing and pulverizing.

Since lime-fly ash and cement-fly ash materials often behave as if they are basically granular in nature, with little or no cohesion at the time of compaction, pneumatic and static and vibratory steel-drum rollers depicted in figures 16 through 18 are usually most effective in providing initial densification. Lift thicknesses of 6 in

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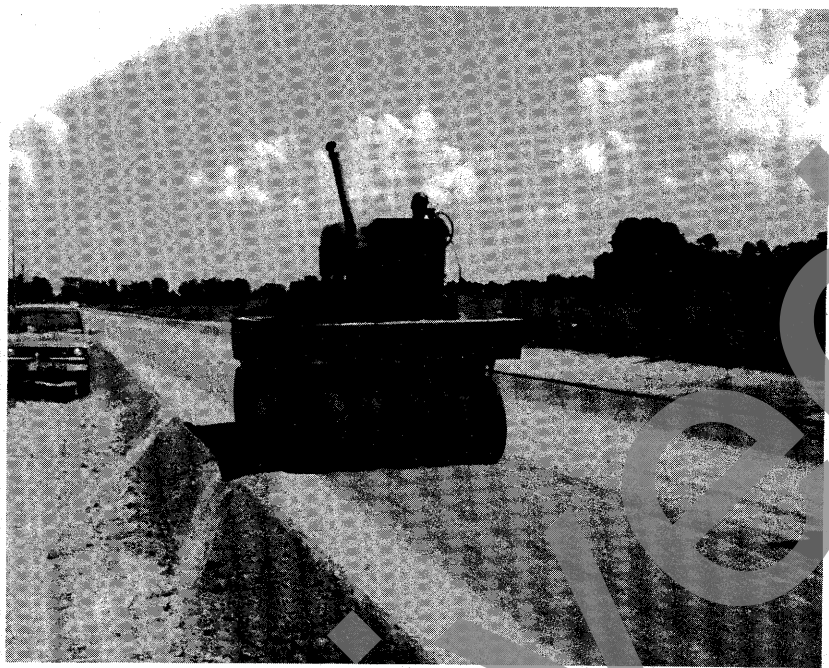

Figure 16. Pneumatic roller.

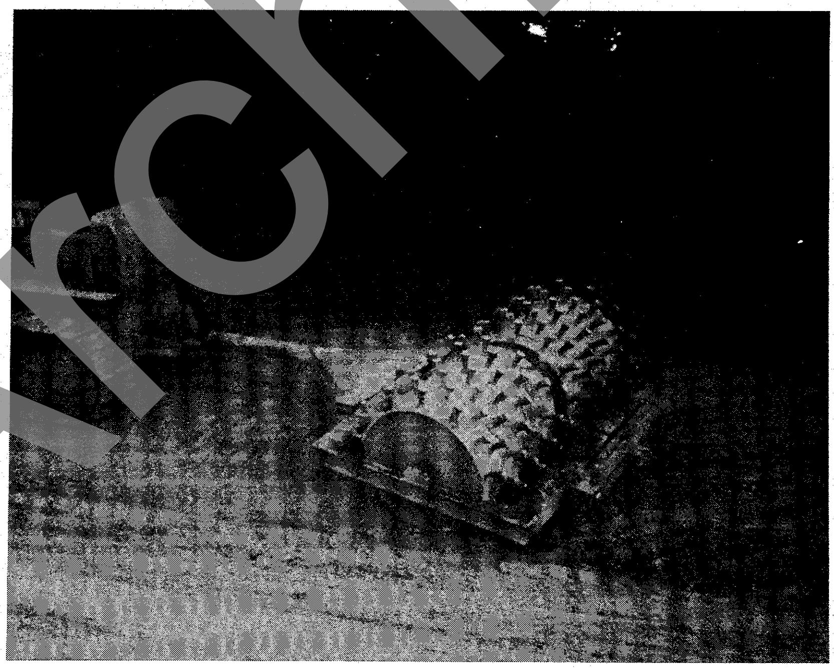

Figure 17. Static sheepsfoot roller.

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Figure 18. Vibratory steel-drum roller.

150 mm) are quite common. Care should be taken to maintain a reasonable template and cross slope for each lift.

Compaction equipment for cement stabilized materials should be the same equipment as would be selected for compacting the unstabilized soil. For fine-grained soils, vibratory sheepsfoot rollers (figure 19) are frequently used. Whereas, for cement stabilized granular materials, pneumatic and static and vibratory steel-drum rollers are most appropriate.

For maximum strength, lime stabilized soils should be compacted shortly after uniform mixing is achieved. Because the reaction associated with lime stabilization is long term compared to cement, additional time is available for mixing and pulverizing lime stabilized soils. In situations where pulverization is difficult, the mixture can be lightly rolled, allowed to mellow for one or two days, and repulverized and remixed without harm.

The most common practice for compacting fine-grained soils stabilized with lime is to compact in one lift with a vibratory sheepsfoot roller until it "walks out" and follow with a pneumatic roller. In some cases, a steel drum roller is used for finishing. Single lift compaction can also be accomplished on some of the more

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granular soils using a vibratory drum roller or heavy pneumatic roller. Finishing is done in these situations with pneumatic or steel-drum rollers.

Figure 19. Vibratory sheepsfoot roller.

When light pneumatic rollers (less than 8 tons) are used, compaction is generally done in thin lifts of 1.5 to 2 in (40 to 50 mm). Slush rolling of granular soil-lime mixtures with steel-drum rollers is not recommended. During compaction, light sprinkling may be required, particularly during hot, dry weather, to compensate for evaporative losses.

Emulsified asphalt mixes should be compacted immediately before or as the emulsion starts to break. At this time, moisture is sufficiently present to act as a lubricant between the aggregate particles, but is reduced to the point where it does not fill the void spaces, thus allowing air void reduction under compactive forces. Also, by this time, the mixture should be able to support the roller without undue displacement. The breaking of the emulsion can be detected by a color change from brown to black.

Cutback asphalt mixtures should be properly aerated prior to compaction. Correct aeration is achieved when the volatile content is reduced to about 50 percent of that contained in the original asphaltic material and the moisture content does not exceed 2 percent by weight of the total mixture.

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Asphalt stabilized materials are typically granular. Therefore, pneumatic and static and vibratory steel-drum rollers should be utilized.

If the stabilized layer requires placement of multiple lifts, partial surface scarification of the bottom lift is often required for lime, lime-fly ash, cement-fly ash, and cement stabilization. This not only promotes bonding between lifts, but removes any negative effects of carbonation which may develop. Carbonation at the top of lime stabilized layers often results when sprinkling is used for curing. The carbonation creates a weak interlayer.

When liquid asphalts are utilized it is important that the lifts have sufficient time to cure prior to placement of the next layer. One week delays in hot, dry weather normally result in the desired curing.

Curing

Proper curing of lime, lime-fly ash, cement, and cement-fly ash stabilization is important because strength gain is dependent upon time, temperature, and the presence of water. Generally, a 3- to 7-day curing period is required, during which time equipment heavier than pneumatic rollers is kept off. In cases where an overlying pavement layer is to be placed shortly after construction of the stabilized layer, curing may be limited or eliminated completely.

Two types of curing can be employed to ensure that moisture is retained in the stabilized layer: moist curing and membrane curing. Moist curing involves sprinkling with water to keep the surface damp and light rolling to keep the surface knitted together. Although this method has proven successful, the preferred method is membrane curing. In membrane curing, the stabilized soil can be sealed in two ways. One way is to apply a single coat (0.10 to 0.25 gal/sq yd [0.5 to 1.2 liters/sq m]) of cutback asphalt within one day after final rolling, where allowed under EPA restriction. The other way is to prime with increments of asphalt emulsion during the curing period.

If asphalt stabilized materials are to be opened to traffic, it is desirable to place a sand or fine-aggregate protective cover to prevent pickup. Protective covers should not be used after construction if traffic will not immediately use the facility. Strength gain of emulsion and cutback stabilized materials develops with the loss of volatiles and a protective seal reduces the rate of loss of volatiles. For this reason, the final asphalt seal or wearing course should not be placed for at least seven days or more, depending on local requirements.

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Subbase and Base Course Stabilization

Stabilization of subbase and base course materials in-place is similar to subgrade stabilization. The major difference is the usual use of borrow material which allows for accurate in stabilizer application and material mixing. The same five steps are employed for subbase and base stabilization:

1. Soil Preparation
2. Stabilizer Application
3. Pulverization and mixing
4. Compaction
5. Curing

Soil Preparation

The most important element in soil preparation is to ensure that the underlying subgrade is compacted and trimmed to the proper grade and cross slope. If a soft or undercompacted subgrade is present the desired density of the stabilized subbase or base material cannot be obtained. Therefore, a compacted platform, free of soft spots, must be created.

Since most borrow materials are granular, pulverization prior to the addition of the stabilizer is not normally required. However, if borrow materials contain considerable portions of clay, partial pulverization may be required prior to the addition of stabilizer.

Stabilizer Application

The most common form of stabilizers for use in these procedures is bulk. Lime and cement can be distributed conveniently with bulk trucks, spreaders, or with specially designed trucks. Although fly ash can be applied in dry form, it normally is conditioned with moisture prior to spreading.

As in the case with subgrade stabilization, asphalt can be distributed through the mixer or by distributors. Lime slurries can be distributed to the soil using rotary mixers with suitable pumps for accurate metering.

The addition of water prior to the introduction of asphalt into the material is often necessary in asphalt stabilization to aid in mixing. Dry soil and lime or cement should be premixed prior to the addition of water for best uniformity. The higher plasticity index soils require an increase in water.

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Pulverization and Mixing

Stabilization of granular materials can be successfully accomplished with either of two types of equipment: travelling pugmill mixers or hopper travel plants. Best performance is provided with the travelling pugmill mixers which move along the pavement, picking up material, mixing it with stabilizer and water in the pugmill, and depositing the mixture ready for spreading. Additional mixing may be necessary and can be performed with a motor grader prior to spreading and compacting.

Hopper-type travel plants are often used for subbase and base course stabilization. Aggregate is deposited in the hopper and mixed with the proper amount of stabilizer in the mixing chamber. Good stabilizer distribution is normally obtained if the operation is carefully controlled.

Compaction and Curing

These operations are identical to those utilized in subgrade stabilization. It is important, however, to recognize that additional aeration of emulsion and cutback stabilized materials may be required if pugmill or hopper-type mixers are utilized because the mixing operation affords only limited opportunities for the volatiles to escape.

3. CENTRAL PLANT MIXED

Central plant mixing operations provide the best opportunity to produce uniform stabilized materials. High mixing efficiency (as measured by strength of mixture in field versus strength of mixture in laboratory) can be achieved with this method.

The two major central plants are the batch plant shown in figure 20, and the drum plant shown in figure 21. Production at continuous plants is higher, although better uniformity and control is associated with batch plants. Both plant types are capable of performing hot and cold mixing operations. Asphalt cements normally require hot central plants for mixing, although soft asphalt cements and foamed asphalt cements have been utilized on mixed-in-place operations. Emulsified and cutback asphalts have been used in hot processes where temperatures are typically in the range of 150 to 220 °F. Both batch and drum mix type plants are commonly used.

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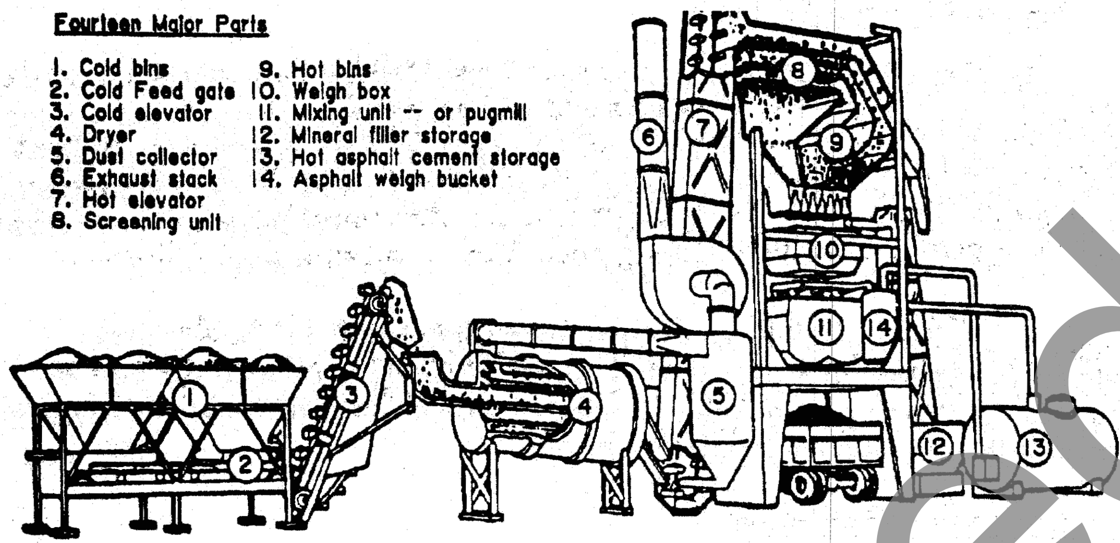

Figure 20. Asphalt batch mix plant.

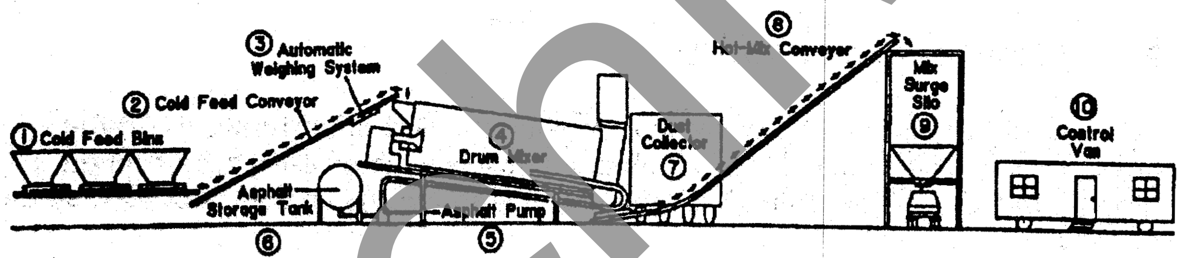

Figure 21. Asphalt drum mix plant.

Cold central plant mixing operations have been used for lime, lime-fly ash, cement-fly ash, and cement stabilization. Continuous pugmill plants similar to the schematic shown in figure 22 are used more often than batch type plants due to their high production capabilities. Pugmill type mixing chambers on the continuous and batch plants are most popular, although central plant portland cement concrete plants have been used for cement and lime-fly ash stabilization projects.

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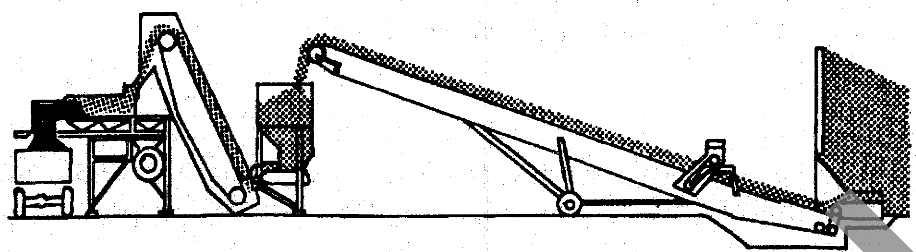

Figure 22. Flow diagram of a typical cold mix continuous plant.[37]

Operations

Typical central plant mixing operations consist of the following:

- Receiving and storage of materials
- Mixing
- Hauling
- Spreading
- Compaction

Receiving and Storage of Materials

Stabilizer and borrow materials (aggregate) must be stored at the plant site. Typically, lime and cement are stored in vertical silos and delivered to the plant by gravity and compressed air. For continuous plants where lime and cement are metered in volumetrically, the stabilizer is usually transferred from the large storage silos to small feed trucks capable of supplying a continuous, calibrated feed.

Asphalt materials are normally stored in heated storage tanks. The temperatures of these tanks are adjusted to provide the correct asphalt viscosity for pumping and mixing.

Fly ash is normally stored in open stockpiles which have been conditioned with sufficient water to prevent dusting (usually 15 to 25 percent). During dry weather, the stockpile surfaces must be kept moist or the stockpile covered to prevent dusting. Conditioned fly ash is normally charged into a feed hopper prior to mixing.

Aggregate materials are normally stockpiled and fed through a belt feed system. Sufficient stockpiles to provide the desired gradations should be utilized. They may vary from one to four in number. Variable speed feeder belts are desirable at the cold feed.

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A water storage tank or a well with pressure system can be utilized to handle the water required for mixing and compaction.

Mixing

Mixing must be accomplished in such a way that the proper amount of stabilizer is uniformly distributed. Plants suitable for this purpose have been discussed previously.

Hauling

Lime, lime-fly ash, cement-fly ash, and cement stabilized mixtures which are blended in a central plant location can be hauled to the road site in conventional, open-bed dump and bottom dump trucks. If haul distances are long or drying of the material enroute poses a problem, then provisions should be made to cover the trucks with tarpaulins or other suitable covers to prevent loss of moisture and scattering of environmentally objectionable dust along the haul routes.

Dusting is rarely a problem with asphalt stabilization operations. However, tarpaulins or other suitable covers are used to prevent heat loss when long hauls are required on cold days.

Sufficient trucks should be made available so that all equipment such as the mixing plant, spreaders, rollers, etc., can operated at a steady, continuous pace rather than on a stop-and-go basis.

Spreading

Spreading should be accomplished as uniformly as possible and with a minimum of segregation. Spreader boxes, laydown machines, and other equipment with automated grade control are recommended. An alternate method of spreading is to place the stabilized material in windrows from trucks and spread with road graders. When using graders care should be taken to place the final lift to sufficient elevation to allow the treated material to be properly trimmed. With the windrow operation, care must be taken not to over-manipulate the stabilized material, which may cause segregation and drying.

Layers of stabilized mixtures are normally spread to a thickness of between 15 and 35 percent greater than the desired final thickness to attain the required compacted thickness. The amount of excess thickness is a function of the aggregate type and source, as well as the method of spreading. Some experimentation may be necessary to determine the proper spread thickness for each operation, because some types of spreading operations provide different degrees of initial consolidation.

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The maximum recommended thickness for a single stabilized layer after compaction is 8 to 10 in, with some agencies specifying a 4-in minimum. If thicknesses of lime, lime-fly ash, cement-fly ash, and cement layers greater than the specified maximum are needed to develop an adequate pavement system, the material should be spread and compacted in lifts. If the material is placed in lifts, the time between lifts should be kept as short as possible so that the lower layer has not "set up" before the next layer is placed. If the stabilized material in the lower layer is fresh and the surface free of loose material, the next layer can be spread without scarifying the lower layer.

During the spreading operation, the moisture content should be monitored, and generally maintained at or slightly above optimum moisture content. It can be maintained below optimum during spreading, particularly for LFA mixtures, but when compaction is ready to begin, the final moisture content should be as near optimum as possible.

As a general rule, subsequent layers should be placed the same day, but with multiple-layered pavements, such as airport and marine terminal pavements, this is not always possible. If the stabilized mixture in the lower layer has taken on an initial set, steps should be taken to ensure the development of a bond between the two layers. Specifically, steps should be taken to ensure that there is no loose material on the lower layer and that the surface is moist before placing the material for the subsequent layer.

If multiple layers of emulsion or cutback stabilized layers are required to satisfy pavement thickness requirements, a time delay between layers is beneficial to allow for the escape of volatiles and thus for a gain in strength. If multiple layers must be placed with little delay, a longer curing period should be considered for thickness design considerations.

Compaction

The necessity of this operation is identical to that utilized for mixed-in-place operations with the exception of the urgency of compaction where hot asphalt stabilization is utilized. Breakdown rolling of asphalt cement stabilized mixtures should be complete before the temperature reaches 175 °F. Different rolling trains may be required for cold mix stabilization, with finish rolling not being as important.

Permeable Bases and Subbases

The selection, stabilization, construction, and compaction of permeable bases or subbases is a very sensitive and specialized area. The discussion in this chapter does not present the details for these materials because of these reasons. Chapter 3, Drainage Considerations, provides some material selection considerations.

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4. CONSTRUCTION EQUIPMENT

A vast array of equipment exists for the various stages comprising soil and base stabilization operations. Table 4 lists equipment generally associated with mixed-in-place subgrade stabilization operations. While preparation equipment such as disk harrows, plows, cultivators, and grader-scarifiers is certainly important in stabilization, it is considered to be beyond the scope of this chapter. Hence, discussion pertaining to equipment will center on mixing and compaction equipment.

Although there are numerous manufacturers of these types of equipment, information was solicited and obtained from a few national and regional manufacturers with regard to their latest line of stabilizers and/or roller compactors. A synopsis of the brochures received is presented in the following sections.

In-Place Stabilization

<u>Rotary Mixers</u>

Single-shaft rotary mixers are quite popular at present. These mixers are generally equipped with automated depth controls capable of mixing and pulverizing to a depth of 18 in (457 mm) and a width of 8 ft (2.44 m). Multiple-shaft rotor speeds are standard to provide mixing action appropriate for various soils. In addition, some models have rotor options to further assist in matching the machine to the material.

Some single-shaft rotary mixers come with standard water spray injection systems. Optional spray injection systems are available on most models for use in injecting certain stabilizers.

Stabilizer Spreaders

<u>Bulk Application</u>

Auger-type Units

Spreading is handled by mechanical-type spreading unit, or metal downspout chutes, or flexible rubber boots extending from screw conveyors. Mechanical spreaders incorporate belt, screw, rotary vane, or drag-chain conveyors to distribute the stabilizer uniformly across the spreader width.

The use of boots or chutes creates windrow deposits of stabilizer. In the case of lime, the material's lightness and flowability causes it to be more uniformly distributed than can be obtained with windrows.

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Pneumatic Units

Lime is blown from the tanker compartments through a pipe or hose to a cyclone spreader or to a pipe spreader bar mounted at the rear. The cyclone spreader distributes the stabilizer through a split chute or with a spreader bar equipped with several large downspout pipes. Air pressure can be adjusted such that the spreader width is automatically controlled.

<u>Slurry Application</u>

Hydrated Lime

A continuous slurry preparation process is used featuring the Halliburton jet slurry maker. Lime is pneumatically pumped to the jet mixer where water is fed under pressure. The resulting slurry is continuously pumped into a slurry truck for spreading. The jet slurry maker can process a 20-ton load of hydrated lime in one hour.[46]

Quicklime

The Portabatch portable slaker unit has been developed in recent years for the production of quicklime slurries. The unit consists of a 10-ft diameter by 40-ft tank that incorporates a 5-ft diameter single shaft agitator turned by a 100-hp diesel engine.[47] The batch slaker can handle 20 to 25 tons of quicklime and about 25,000 gal of water. Approximately 1 to 1.5 hr are required for the production of the slurry.

Compaction Equipment

Major manufacturers of soil compaction equipment provide a full line of rollers varying by type, size (weight and dimension), and compaction force. These include self-propelled or towed vibratory steel-drum rollers, vibratory and static sheepsfoot rollers, and pneumatic rollers.

Vibratory steel-drum and vibratory sheepsfoot rollers can be acquired with weights ranging from 4,700 to 40,500 lb. Dynamic forces of up to 50,000 lb can be achieved with the heavy duty models. While all of the self-propelled sheepsfoot rollers are drum-driven, only some of the self-propelled steel-drum rollers are drum driven. Drum widths between 47 in and 84 in are available on both roller types.

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Table 4. Equipment typically associated with mixed-in-place subgrade stabilization operations.

Stabilizer	CONSTRUCTION OPERATION: Soil Preparation	Stabilizer Application	Pulverization and Mixing	Compaction	Curing
Lime[46] *1	• Single-shaft rotary mixer (flat type) • Motor grader • Disc Harrow • Other agricultural-type equipment	• Dry-bagged • Dry bulk • Slurry • Slurry through mixer	• Single- and multi-shaft rotary mixers • Motor graders • Other agricultural-type equipment	• Sheep's foot • Pneumatic • Steel Wheel	• Membrane (asphalt) • Moist (water)
Lime-fly ash, cement-fly ash[47] *2	• Single-shaft rotary mixer (flat type) • Motor grader • Disc Harrow • Other agricultural-type equipment	Separate Application • Lime: dry or slurry • Fly ash: conditioned Combined Application • Dry-bagged • Dry bulk	• Same as lime	• Steel Wheel • Pneumatic • Vibratory	• Membrane • Moist
Cement[48] *3	• Single-shaft rotary mixer (flat type) • Motor grader • Disc harrow • Other agricultural-type equipment	• Dry-bagged • Dry bulk	• Same as lime	• Sheep's foot • Pneumatic (clay soils) • Vibratory (granular soils)	• Membrane • Moist
Asphalt[37] *4	• Motor grader • Single-shaft rotary mixer (flat type)	• Asphalt spray distributor • During mixing process	• Single- and multi-shaft rotary mixer (flat type) • Motor grader	• Pneumatic • Steel Wheel • Vibratory	• Volatiles should be allowed to escape and/or the pavement to cool

	COMMENTS	SAFETY PROCEDURES
*1	Double application of lime may be required to facilitate mixing. The soil and air temperature should be greater than 40 to 50 °F to insure adequate strength gain. Construction should be completed early enough in summer or fall so that sufficient durability will be gained to resist freeze-thaw action.	Lime spreading should be avoided on windy days. Proper clothing should be worn so that workmen can avoid skin contact with quicklime. Workmen should avoid prolonged contact with lime and breathing lime dust.
*2	Fly ash must be conditioned with moisture prior to distribution to prevent dusting. Mixing and compaction should be completed shortly after stabilizer application. The soil and air temperature should be greater than 40 to 50 °F to insure adequate strength gain. Construction should be completed early enough in summer or fall so that sufficient durability will be gained to resist freeze-thaw action.	Fly ash, lime and cement spreading should be avoided on windy days. Workmen should avoid prolonged contact with the stabilizers and breathing the stabilizers.
*3	Mixing and compaction must be completed shortly after stabilizer application. The soil and air temperatures should be greater than 40 °F to insure adequate strength gain. Construction should be completed early enough in summer or fall so that sufficient durability will be gained to resist freeze-thaw action.	Cement spreading should be avoided on windy days. Workmen should avoid prolonged contact with cement and breathing the cement dust.
*4	Proper soil moisture content must be achieved to aid distribution and mixing. Stabilized material should be properly aerated prior to compaction. The soil and air temperature should be above 40 °F to allow for proper curing and sufficient time for compaction if hot mix processes are utilized. Thick lifts of hot, asphalt cement stabilized materials can be placed below 32 °F.	Proper clothing should be worn so that workmen can avoid skin contact with hot asphalt.

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CHAPTER 5 PAVEMENT THICKNESS DESIGN

1. INTRODUCTION

The field of pavement design is continually changing in response to the development and implementation of new analysis tools and more comprehensive design methods. There have been many efforts over the past 40 years to develop procedures for pavements thickness design that are based on rational approaches. Some of these procedures are widely used and a few have become firmly established.

The AASHTO method and mechanistic-empirical procedures have been adopted for a wide range of conditions and materials and are used by many agencies. The AASHTO approach was originally developed from the AASHO Road Test and has been modified several times, most recently in the 1986 AASHTO Guide for Design of Pavement Structures. Recently, agencies have utilized the versatility of the layered system to develop design systems based on suitable criteria for stresses and strains for various pavement layers. The American Coal Ash Association has a new manual presenting several design approaches.[76] The design method developed by Shell has seen increasing use and has been updated.[49,50] Chevron has also developed a useful design approach that includes specific materials such as cement-modified, asphalt emulsion mixtures.[51]

The Asphalt Institute design procedure is an example of a mechanistic-based analysis procedure, modified by calibration.[52] The PCA program for rigid pavement design includes procedures to account for loss of support under concrete pavements, and they have a procedure for cement treated bases under flexible surfaces.[58,75] The basis for these mechanistic programs is a computer program that calculates stresses or strains, such as a elastic layer program for asphalt concrete pavements, or a finite-element procedure for portland cement concrete pavements. Many researchers have improved the usefulness of these programs by refining inputs for the modulus and Poisson's ratio, as well as limiting criteria to reduce rutting and fatigue damage.

This chapter includes a summary of available design methods. These procedures can be used with stabilized layers in the design process. Design input requirements are also discussed. Where appropriate, suggestions are made for typical values of such inputs for lime, lime fly-ash, cement, and asphalt, as well as untreated materials Volume II can be used to assist in developing an understanding of design values and typical values..

2. CONSIDERATIONS IN THE PAVEMENT DESIGN PROCESS

There are several important factors to consider in a pavement design, regardless of the procedure used. Many of these variables are new, having been added into the 1986 AASHTO design procedure.[3] As such, they represent factors that require extra consideration by agencies implementing the 1986 design procedure. The variables to consider include:

- Material Properties.
  Resilient (elastic) modulus.
- Vehicles and Traffic.
- Variability and Reliability.
- Drainage.

A detailed description of these variables, their development, and use in the design of pavements can be found in the FHWA/NHI training course, "Pavement Design, Principles and Practice."[53]

Pavement design is the determination of the thickness and vertical position of pavement material elements which can best be combined to provide a serviceable roadway for predicted traffic over the selected pavement design life. These elements include the various subbase and base courses as well as the pavement surface and a suitable recognition of the subgrade soils. Each layer of the pavement structure can be designed and located to take advantage of the particular properties of that material. The goal in design should be to use the most economical arrangement and minimum thickness of each material necessary to protect the underlying layers and the subgrade from distresses caused by imposed traffic loads.

Two basic approaches are being actively developed to determine the required layer thickness for pavement structures:

- Empirical Procedures are derived from experience or field observations alone, often without due regard for system behavior or pavement theory. The basis for many design methods are empirically derived relationships between performance, load, and pavement thickness for a given geographic location and climatic condition. These models are generally used to determine the pavement thickness, the number of load applications to failure, or the occurrence of distresses as a function of pavement materials properties, subgrade type, climate, and traffic. The AASHTO and U.S. Army Corps of Engineers methods are among a large family of pavement design techniques which were developed primarily on the basis of observed field performance.

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- Mechanistic-empirical procedures are those which calculate pavement responses, stresses, strains, or deflections, and couple these with distress or performance prediction models to predict load repetitions to failure. These models are developed from laboratory data and are normally calibrated using observed performance of in-service pavements and are used to estimate the maximum number of repetitions of a given level of stress, strain, or deflection a pavement can withstand before reaching an unacceptable state of serviceability.

3. AASHTO DESIGN METHOD

One of the major objectives of the AASHO Road Test was to provide information that could be used to develop pavement design criteria and procedures. This objective was met with the development and circulation of the "AASHO Interim Guide for the Design of Rigid and Flexible Pavements" in 1961, which contained design procedures based on empirical models derived from data collected at the AASHO Road Rest. After the Guide had been used for several years, the AASHTO Design Committee evaluated and revised the Interim Guide in 1972 and again in 1981 for rigid pavements applications.(3)

Further evaluations of the Guide were undertaken in 1983, and it was determined that although the Guide was still serving its main objectives, some improvements could be made to incorporate advances in pavement design and analysis technology that had been made since 1972. Thus, between 1984 and 1986, the Subcommittee on Pavement Design and a team of consultants revised the existing guide under NCHRP Project 20-7/24 and issued the current version entitled "AASHTO Guide for the Design of Pavement Structures 1986."(3)

Major changes have been made in several areas, including the following:

- Incorporation of the design reliability factor (based on a shift in the design traffic) to allow the designer to use the concept of risk analysis for various classes of highways.

- Replacement of the soil support number with the resilient modulus (AASHTO T 274).

- Use of the resilient modulus test for assigning layer coefficients to both stabilized and unstabilized material.

- Replacement of the subjective regional factor with a rational approach to adjustment of designs to account for environmental considerations such as moisture and temperature climate considerations, including thaw-weakening and other seasonal variations in material properties.

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- Provision of guidance for the construction of subsurface drainage systems and modifications to the design equations to take advantage of improvements in performance that result from good drainage.

The 1986 Guide also includes recommendations and guidelines for conducting economic analysis of alternatives designs and a summary of the latest concepts concerning the development and use of mechanistic-empirical design procedures.

AASHTO Thickness Design Procedures

Background

The AASHTO procedure is based on providing enough strength in the pavement layers to prevent overloading of the subgrade soil by the applied loads. The pavement performance is measured by a Present Serviceability Index (PSI), which is a function of the mean slope variance in the two wheel paths, the amount of cracking and patching in the pavement surface, with the depths of rutting in the wheel paths being included for the flexible sections.

The test facility consisted of six two-lane test loops. The north tangent of each loop was constructed of flexible pavement sections and the south tangent was constructed of rigid pavement sections. Most of the 234 flexible pavement structural design sections (468 test sections, 160 ft [48.8 m] in length) comprised a complete replicated factorial experiment investigating the effects of varying thickness of surfacing (1, 2, 3, 4, 5, and 6 in), base course (0, 3, 6, and 9 in), and subbase (0, 4, 8, 12, and 16 in). Several additional studies were also conducted to evaluate surface treatments, shoulders and four different types of base course (crushed stone, gravel, cement-treated gravel, and bituminous-treated gravel). The rigid sections included slab thicknesses from 2.5 to 12.5 in (63.5 to 317.5 mm) with untreated gravel/sand subbase from 0 to 9 in (0 to 228.6 mm). The jointed reinforced sections had a joint spacing of 40 ft (12.2 m), and used smooth welded wire fabric. The jointed plain sections had joint spacings of 15 ft (4.6 m). All joints were dowelled.

The sections were constructed in a series of loops and were subjected to 1.1 million single- and tandem-axle load applications that ranged from 2,000-lb (9 kN) to 48,000-lb (30 kN), with each section being exposed only to axle loads of one particular size and configuration. In this way, the effects of different loads could be assessed together with the variations in pavement structure. All load applications were completed over a two-year period.

Performance measurements were taken at regular intervals to provide information concerning the roughness and visible deterioration over time of the surfacing of each section. These measurements included transverse pavement

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profiles (rutting), cracking, patching, deflections, strains, layer thickness and temperatures and numerous other measurements.

The performance data was used to develop an empirical model using regression techniques. The structural number (SN) was determined for the flexible pavement sections by assigning relative structural strength coefficients (a_i) to a unit thickness of each material that would allow the substitution of a certain thickness of one type of material for another (in proportion to their strength coefficients) with the same resulting load carrying capacity. The structural number of a pavement section was defined as the sum of the products of thickness and layer coefficient for each of the pavement layers ($SN = a_1D_1 + a_2D_2 + ...$). The rigid pavement design model uses the slab thickness as the structural design parameter. Serviceability loss was then related to the applied number of 18-kip equivalent single-axle loads (ESALs), and the other important parameters. This model could be used to predict serviceability loss for a given set of design inputs (layer thickness, material properties, traffic, etc.) or produce a required structural number given traffic and serviceability.

General Design Variables

General design variables are those that must be considered in the design and the construction of any pavement surface. Included in this category are the time constraints for the design, traffic considerations, design reliability, drainage coefficient selection, roadbed soil evaluation for each pavement type, and loss of support effects for the rigid pavements.

Time Constraints

The selection of performance and analysis periods forces the pavement designer to examine various strategies that may range from a low maintenance structure that lasts for the entire analysis to staged construction alternatives that require an initial structure and planned maintenance or overlays.

In the past, pavements were designed typically and analyzed for a 20-year performance period. It is now recommended that consideration be given to longer periods, since these may be better suited for the evaluation of alternative long-term strategies based on life-cycle costs. In any event, it is recommended that the analysis period should be selected to include at least one rehabilitation of the pavement.

Traffic

The AASHTO thickness design procedures are based on cumulative expected 18-kip ESALs during the analysis period (W_{18}). The process of collecting mixed traffic data and converting it into equivalent 18-kip ESAL is complex. It is important to realize that axle type, dual, tandem, or tridem, and weight are far more critical for

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pavement performance that vehicle gross weight. Much researach isbeing conucted into tire type as it affects AASHTO values. To calculate the total 18-kip ESAL applications for the pavement over its analysis period, the following data are required:

- The daily volume of each vehicle type for the base year.
- Appropriate growth rate for each class of vehicle for the 20-year design period.
- The multiplying the ESAL factor for each vehicle classification type in the traffic stream.
- The lane distribution and directional distribution factors.

These data are converted into the total number of ESALs (W_{18}) using the pavement over the design period.

Reliability

Design reliability refers to the degree of certainty that a given design alternative will last the analysis period. The AASHTO design-performance reliability is controlled through the use of a design reliability factor (F_r) that is multiplied by the traffic prediction (W_{18}) to produce design traffic applications (W_{18}) for use in the design equation. For a given reliability level (R), the reliability factor is a function of the overall standard deviation (S_o) that accounts for standard variation.

The recommended levels of design reliability for pavements with various functional classifications by AASHTO are the same for flexible and rigid pavements. The recommended levels are:

Recommended Level of Reliability, R (%)

Functional Classification	**Urban**	**Rural**
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-90	75-95
Collectors	80-95	75-95
Local	50-80	50-80

The selected standard deviation must be representative of local conditions. The following values are recommended for general use, but should be evaluated for local usage:

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Design Condition	Standard Deviation
Variation in pavement performance prediction without traffic error.	0.35 flexible 0.25 rigid
Total variation in pavement performance prediction and in traffic estimation.	0.45 flexible 0.35 rigid

When staged construction is to be considered, it is important to recognize the need to compound the reliability of each individual stage of the strategy to achieve the desired overall reliability. The design reliability of each stage can be expressed as:

$$R_{stage} = (R_{overall})^{1/n}$$

where n is the number of stages being considered. For example, if three stages are to be constructed and the desired level of overall reliability is 95 percent, the reliability of each individual stage must be $(0.95)^{1/3}$ or 98.3 percent.

Environment Impacts

Temperature and moisture changes have an effect on the strength, durability, and load-carrying capacity of the pavement and subgrade materials though the mechanics of swelling soils, frost heave, and other phenomena.

If a swelling clay or frost heave potential exists and the pavement design does not take steps to prevent adverse effects, the loss of serviceability over the analysis period should be estimated using published AASHTO procedures and added to that resulting from cumulative axle loads.

Performance Criteria

The serviceability of a pavement is defined as its ability to serve the type of traffic which uses the facility. The primary measure of serviceability used by the AASHTO procedures is the PSI, which ranges from 0 (impassible) to 5 (perfect).

Initial and terminal serviceability indexes must be established to compute the total change in serviceability that will be input to the design equations. Initial serviceability index (p_o) is a function of pavement design and construction quality. Typical values from the AASHO Road Test were 4.2 for flexible pavements and 4.5 for rigid pavements. Terminal serviceability index (p_t) is the lowest index that can be tolerated by the travelling public before rehabilitation, resurfacing or reconstruction becomes necessary. This value varies with the importance or functional classification of the pavement. Recommended terminal serviceability indexes are often 2.5 or

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higher for major highways and 2.0 to 2.5 for less important pavements. The required input to the AASHTO flexible pavements thickness design procedure is serviceability loss, equal to p_o - p_t.

Roadbed Soil Evaluation

The AASHTO design procedure requires the input of a roadbed soil support value that is calculated from the resilient modulus for the subgrade, which includes the combined effect of all seasonal effects on the modulus values. For flexible pavements this is the effective resilient modulus, and for the rigid pavements this is an effective modulus of subgrade reaction, derived from the effective resilient modulus. The computation of the effective resilient modulus should be used only for estimating the modulus of soils under pavements that are to be designed using serviceability criteria.

First, separate the year into time intervals during which the different seasonal moduli are effective. All of the "seasons" must be definable in terms of the selected time interval. It is suggested that the one-half month should be the shortest time interval used. The seasonal modulus values may be determined from laboratory testing, field deflection testing with backcalculation, or from estimates of seasonal variability. Next, the relative damage value (u_f) corresponding to each seasonal modulus be estimated using the vertical scale or corresponding equation shown in figure 23. The relative damage values should all be added together and divided by the number of seasonal increments to determine the average relative damage. Finally, the effective subgrade soil resilient modulus (M_R) is estimated as the value corresponding to the average relative damage on the M_R - u_f scale using figure 23 again. This modulus value is the effective modulus value for use in flexible pavement thickness design. The same procedure is followed for the effective modulus of subgrade reaction for rigid pavements using figure 44 and other figures for relative damage in rigid pavements in the same manner as demonstrated here, making adjustments to rigid layers at depth.

Pavement Layer Materials Characteristic

Although the concept of layer coefficients is still central to the AASHTO flexible pavement design procedure, the 1986 AASHTO Design Guide relies more heavily on the determination of materials properties for the estimation of appropriate layer coefficient values. The preferred tests are the resilient modulus (AASHTO T 274 has not been approved at present) for subbase and unbound granular materials and elastic modulus (ASTM D 4123 or ASTM C 469) for asphalt concrete and other stabilized materials.

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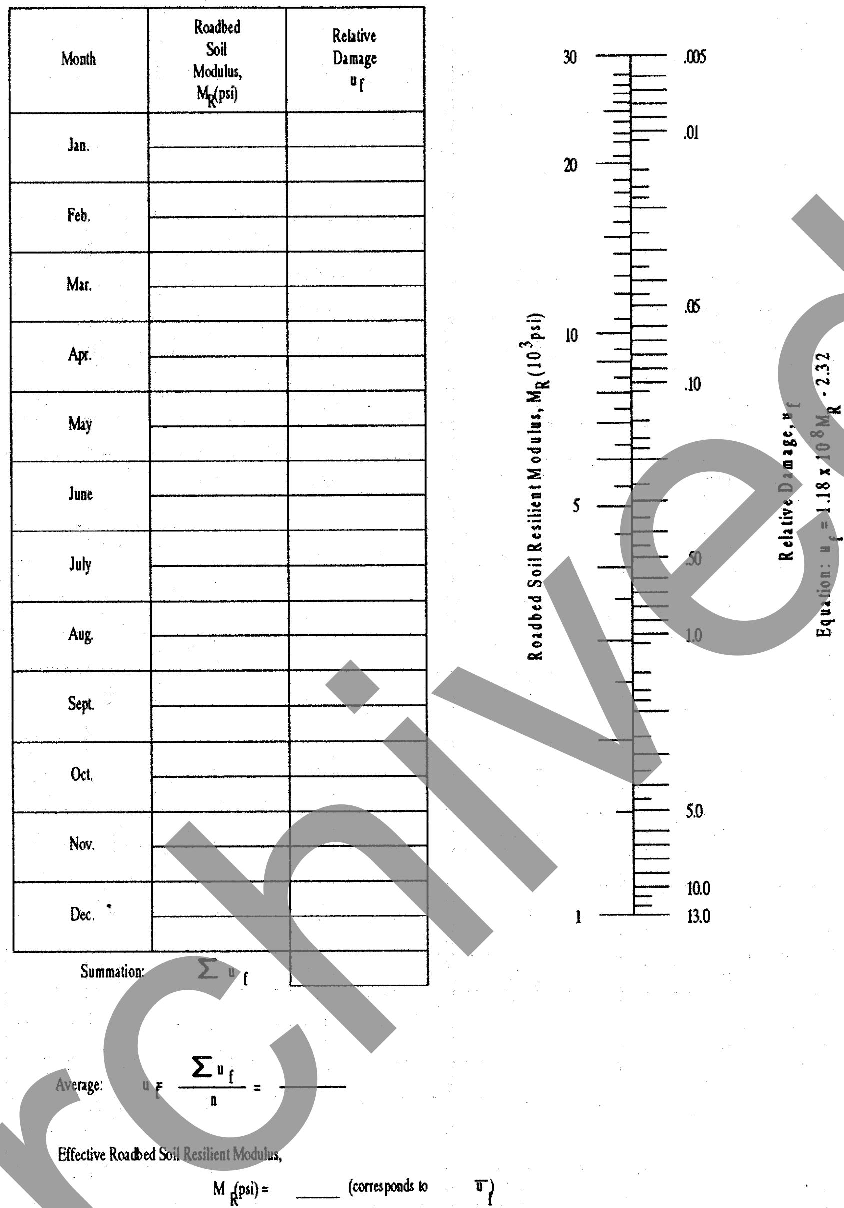

Figure 23. Chart for estimating effective roadbed soil resilient modulus.[3]

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Layer Coefficients

The AASHTO flexible pavement layer coefficient (a_i) is a measure of the relative ability of a unit thickness of a given material to function as a structural component of the pavement. For example, 2 in (50.8 mm) of a material with a layer coefficient of 0.20 is assumed to provide the same structural contribution as 1 in (25.4 mm) of a material with a layer coefficient of 0.40.

SUBBASE: TYPE	GRANULAR
THICKNESS (inches)	6
LOSS OF SUPPORT. LS	1.0
DEPTH TO RIGID FOUNDATION (feet)	5
PROJECTED SLAB THICKNESS (inches)	9

(1) MONTH	(2) ROADBED MODULUS M_R (PSI)	(3) SUBBASE MODULUS, E_{SB} (PSI)	(4) COMPOSITE K-VALUE (PCI)	(5) K-VALUE (PCI) ON RIGID FOUNDATION	(6) RELATIVE DAMAGE u_r
Jan.	20,000	50,000	1100	1350	0.35
Feb.	20,000	50,000	1100	1350	0.35
Mar.	2,500	15,000	160	230	0.86
April	4,000	15,000	230	300	0.78
May	4,000	15,000	230	300	0.78
June	7,000	20,000	410	540	0.60
July	7,000	20,000	410	540	0.60
Aug.	7,000	20,000	410	540	0.60
Sep.	7,000	20,000	410	540	0.60
Oct.	7,000	20,000	410	540	0.60
Nov.	4,000	15,000	230	300	0.78
Dec.	20,000	50,000	1100	1350	0.35
			SUMMATION:	$\sum u_r =$	7.25

AVERAGE: $\bar{u}_r = \frac{\sum u_r}{n} = \frac{7.25}{12} = 0.60$

EFFECTIVE MODULUS OF SUBGRADE REACTION: K (PCI) = 540

CORRECTED FOR LOSS OF SUPPORT: K (PCI) = 170

Figure 24. Sample table for determining effective modulus of subgrade reaction.[3]

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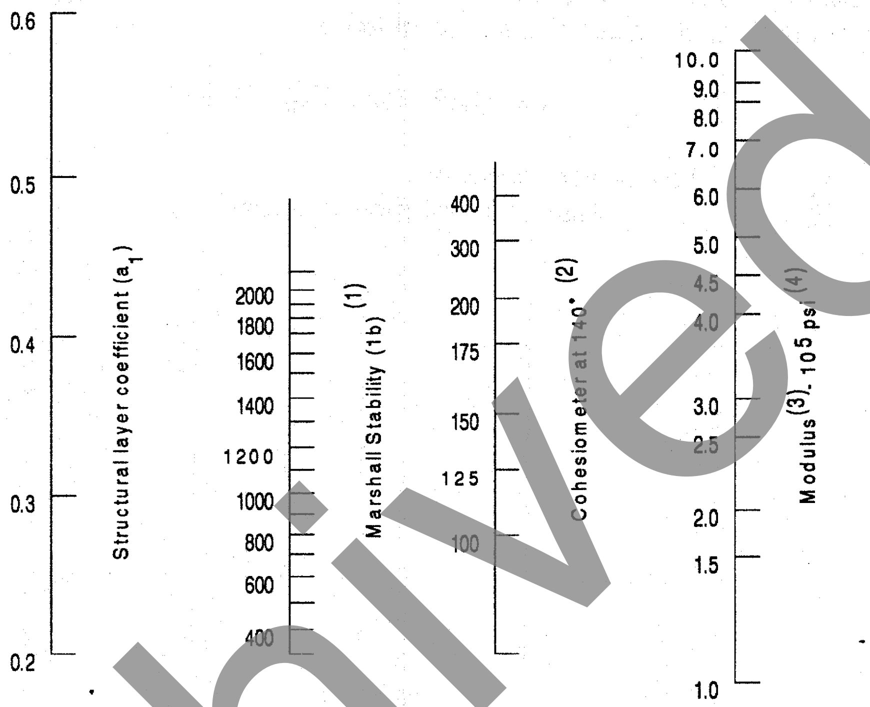
Asphalt Concrete Surface Course

Figure 25 presents the chart that can be used to estimate the structural layer coefficient of a dense-graded asphalt concrete surface course based on its elastic (resilient) modulus (E_{AC}) at 68 °F (20 °C), and several other tests.



(1) Scale derived by averaging correlations obtained from Asphalt Institute, IL, LA, NM, & WY.

(2) Scale derived by averaging correlations from CA & Texas.

(3) Scale derived on this project.

(4) Modulus at 68 °F.

Figure 25. AASHTO structural layer coefficient related to other asphaltic concrete tests.[3]

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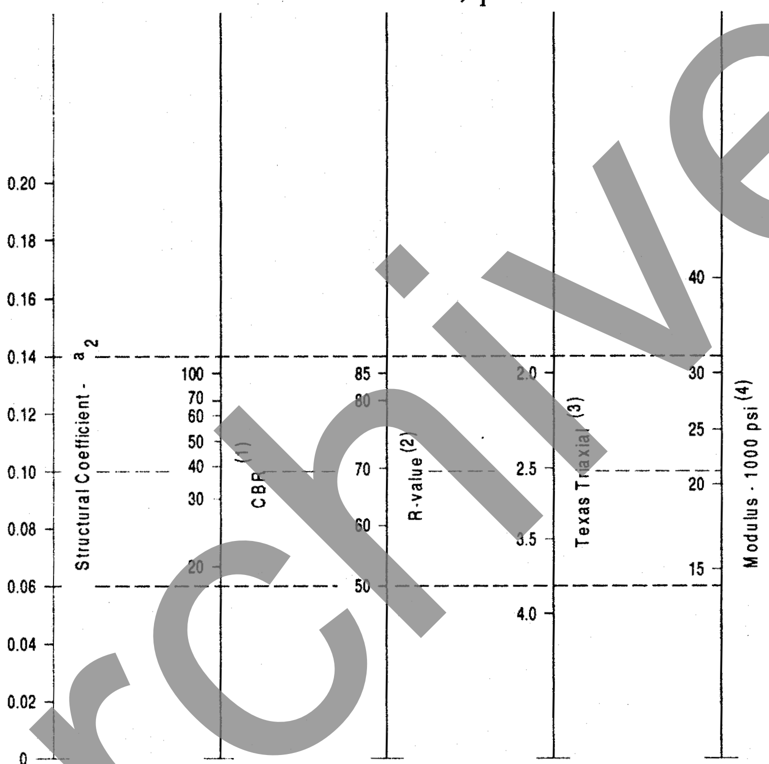
Granular Base Layers

Figure 26 presents the chart to estimate a structural layer coefficient for a granular base material (a_2) based on one of four different laboratory test results, including base resilient modulus. The following relationship may be used to estimate the layer coefficient for a granular base material:

$$a_2 = 0.249 \cdot (\text{Log } E_{BS}) - 0.977$$

where: a_2 = base layer coefficient
E_{BS} = base resilient modulus, psi



(1) Scale derived by averaging correlations obtained from Illinois.
(2) Scale derived by averaging correlations obtained from California, New Mexico and Wyoming.
(3) Scale derived by averaging correlations obtained from Texas.
(4) Scale derived on NCHRP project, (3).

Figure 26. Variation in granular base layer coefficient (a_2) with various base strength parameters.[(3)]

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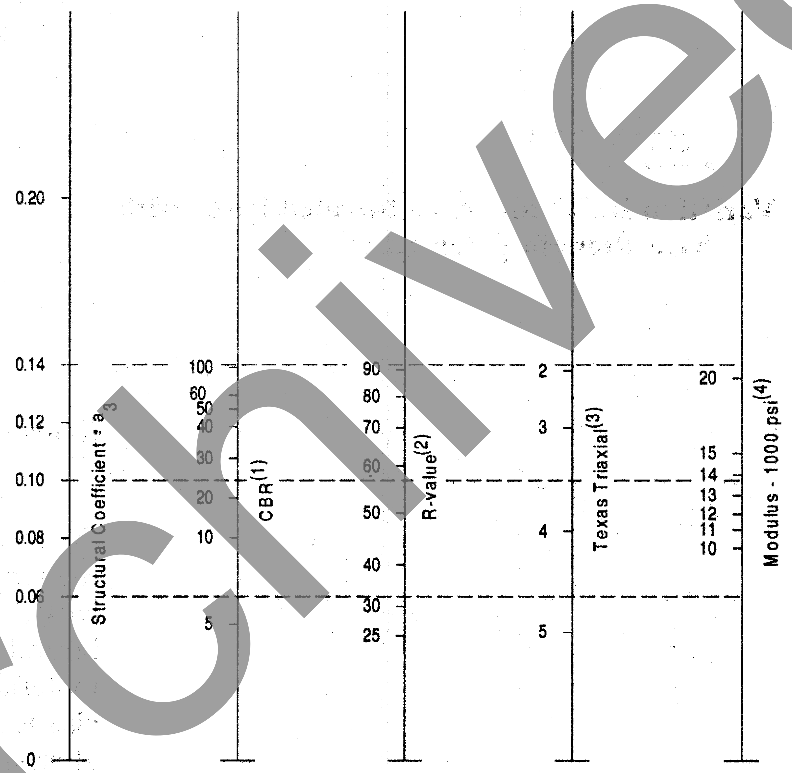
Granular Subbase Layers

Figure 27 presents the chart used to estimate a structural layer coefficient for a granular subbase material (a_3) based on one of four different laboratory test results, including subbase resilient modulus. The following may be used to estimate the layer coefficient for a granular subbase material:

$$a_3 = 0.227 \cdot (\mathrm{Log}\ E_{SB}) - 0.839$$

where: a_3 = subbase layer coefficient
E_{SB} = subbase resilient modulus, psi



(1) Scale derived from correlations obtained from Illinois.
(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.
(3) Scale derived from correlations obtained from Texas.
(4) Scale derived on NCHRP project (3).

Figure 27. Variation in granular subbase layer coefficient (a_3) with various subbase strength parameters.[3]

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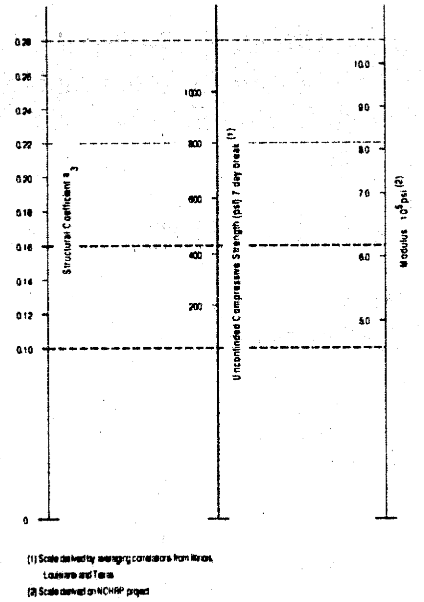

Figure 28. Variation in "a" for cement-treated bases with base strength parameter.[3]

Cement-Treated Bases

Figure 28 provides the chart used to estimate the structural layer coefficient for a cement-treated base material from either its elastic modulus or alternatively, its 7-day unconfined compressive strength (ASTM D 1633).[3]

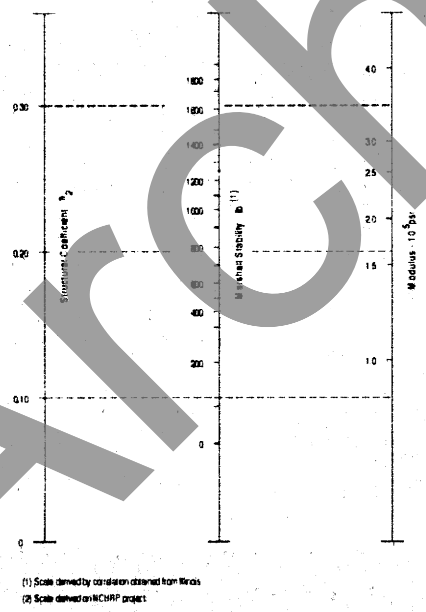

Figure 29. Variation in a_2 for bituminous-treated bases with base strength parameter.[3]

Bituminous-Treated Bases

Figure 29 presents the chart used to estimate the structural layer coefficient for a bituminous-treated base material from either its elastic modulus or alternatively, its Marshall stability (AASHTO T 245, ASTM D 1559).[3]

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Portland Cement Surfaces

The elastic modulus and the flexural modulus of rupture are used in the design. The elastic modulus can be estimated using the following:

$$E_c = 57{,}000(f'_c)^{0.5}$$

where:
- E_c = PCC elastic modulus, psi
- f'_c = compressive strength (AASHTO T 22, T 140, or ASTM C 39), psi

Drainage

The AASHTO pavement design procedure provides a means to adjust layer coefficients to take into account the effects of certain levels of drainage in pavement performance of flexible pavements (m_i) and rigid pavements (C_d). These coefficients are not designed to replace good drainage in the pavement section, and guidance concerning the design or effectiveness of various drainage approaches is provided in chapter 3 of volume I.

The effect of drainage of all untreated layers below the surface is considered by multiplying the layer coefficients (a_i) by the drainage coefficient (m_i), for flexible pavements and the modulus of rupture by C_d for rigid pavements. The drainage coefficient is a function of the quality of drainage, the drainage characteristics of the granular materials and the subgrade, and the amount of time the soil is exposed to moisture levels approaching saturation. The structural number equation modified for drainage becomes:

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

where:
- SN = structural number
- a_i = layer coefficient of layer i
- D_i = thickness of layer i, in
- m_i = drainage modifying factor for layer i

The design engineer must identify the level of quality of drainage that is achieved under a specific set of drainage conditions. The drainage conditions at the AASHO Road Test are assumed to be "fair" and the m_i values there are assumed because the structural models should not require adjustment for the conditions at the Road Test. However, the same materials would probably receive drainage modifying factors of less than 1.0 for a new construction projects, and the designer should select appropriate values to reduce the possibility of a poor design.

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Table 5 and 6 provide recommendations for modifying structural layer coefficients of untreated base and subbase materials in flexible and rigid pavements pavements. The values in table 5 apply only to the effects of drainage on untreated base and subbase materials. Although stabilized layers can be affected by moisture as well, the effects are not as quantifiable. Table 6 contains the recommendations for drainage coefficients for rigid pavements.

Table 5. Recommended m_i values for modifying structural layer coefficients of untreated base and subbase materials in flexible pavements.[3]

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

Table 6. Recommended values of drainage coefficient (C_d) for rigid pavement design.[3]

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

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Loss of Support

The rigid pavement has an added material consideration related to the subbase material used in the system. It is recognized that the subbase material may be subject to erosion loss, commonly resulting in pumping. This erosion produces a loss of support (LS) which reduces the life of the slab. Different materials exhibit more resistance to this erosion, as illustrated in table 7. The development of erosion is tied to the presence of moisture in the system, and different drainage considerations may provide different erosion potentials for the same material. A different loss of support may develop when a very stiff stabilized base is used. In this design, the slab will curl and lose contact with the base, resulting in increased stresses in the slab. These increased stresses can lead to premature cracking of the slab because there is no support under the edges and corners, much the same as erosion. Further testing is required to establish precise criteria for selection of LS factors for all situations.

Table 7. Typical ranges of loss of support (LS) factors for various types of materials.[3]

Type of Material	Loss of Support (LS)
Cement Treated Granular Base (E = 1,000,000 to 2,000,000 psi)	0.0 to 1.0
Cement Aggregate Mixtures (E = 500,000 to 1,000,000 psi)	0.0 to 1.0
Asphalt Treated Base (E = 350,000 to 1,000,000 psi)	0.0 to 1.0
Bituminous Stabilized Mixtures (E = 40,000 to 300,000 psi	0.0 to 1.0
Lime Stabilized (E = 20,000 to 70,000 psi)	1.0 to 3.0
Unbound Granular Materials (E = 15,000 to 45,000 psi)	1.0 to 3.0
Fine Grained or Natural Subgrade Materials(E = 3,000 to 40,000 psi)	2.0 to 3.0

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Computation of Required Pavement Thickness

The design equations can be solved manually using a series of nomographs or recently-developed computer software.[54] The complexity of the design procedure can make the manual solution a tedious process. The computerized approach allows easy consideration of all design factors and provides accurate solutions to the design equations. Figure 30 is the nomographic solution for the flexible pavement. Figures 31 and 32 contain the nomograph for the rigid pavement.

There are many more variables for each pavement type that impact the selection of a final thickness. Each of these variables cannot be described here and the designer should consult the AASHTO guide for further details, or the FHWA/NHI training manual for more in-depth coverage.[53]

NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R * S_0 + 9.36 * \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1004}{(SN+1)^{5.19}}} + 2.32 * \log_{10} M_R - 8.07$$

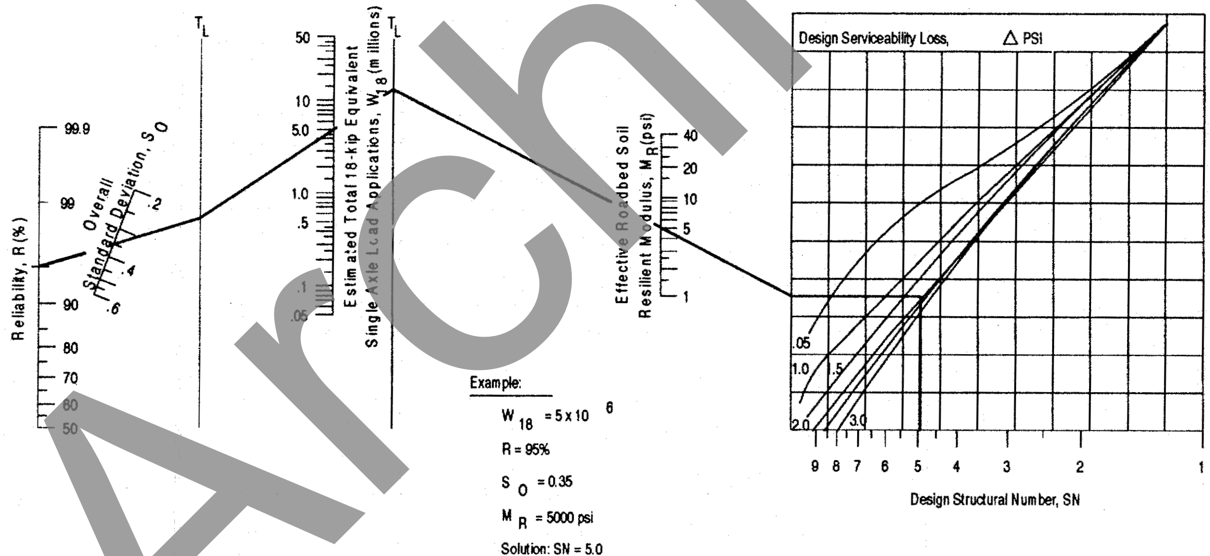

Figure 30. AASHTO flexible pavement thickness design nomograph.[3]

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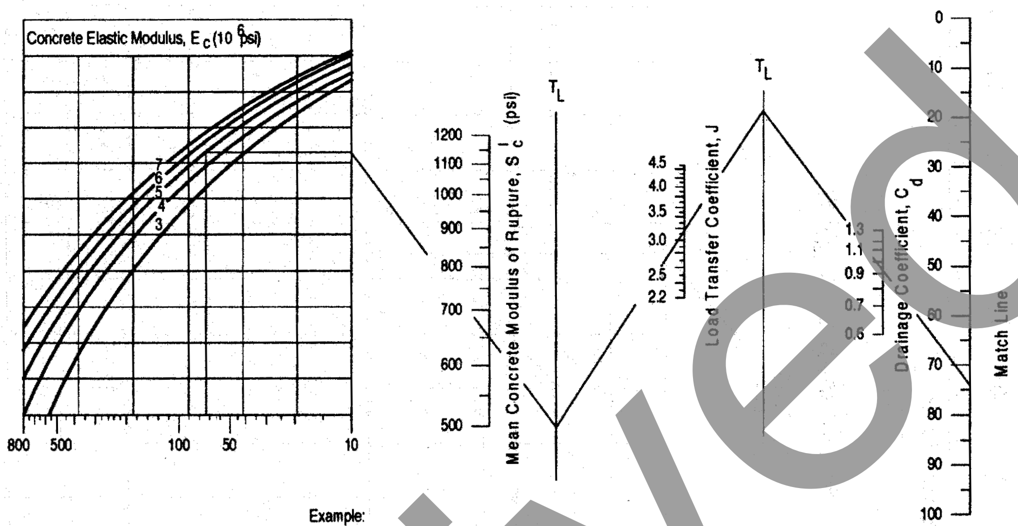
NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R * S_O + 7.35 * \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624*10^7}{(D+1)^{8.46}}} + (4.22 - 0.32p_t) * \log_{10}\left[\frac{S'_c * C_d\left[D^{0.75} - 1.132\right]}{215.63 * J\left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$



Example:

$k = 72$ pci

$E_c = 5 \times 10^6$ psi

$S'_c = 650$ psi

$J = 3.2$

$C_d = 1.0$

$S_o = 0.29$

$R = 95\%$ ($Z_R = -1.645$)

$PSI = 4.2 - 2.5 = 1.7$

$W_{18} = 5.1 \times 10^6$ (18 kip ESAL)

Solution: D = 10.0 inches (nearest half-inch, from segment 2)

Figure 31. Design chart for rigid pavement design based on using mean values for each input variable (Segment 1).[3]

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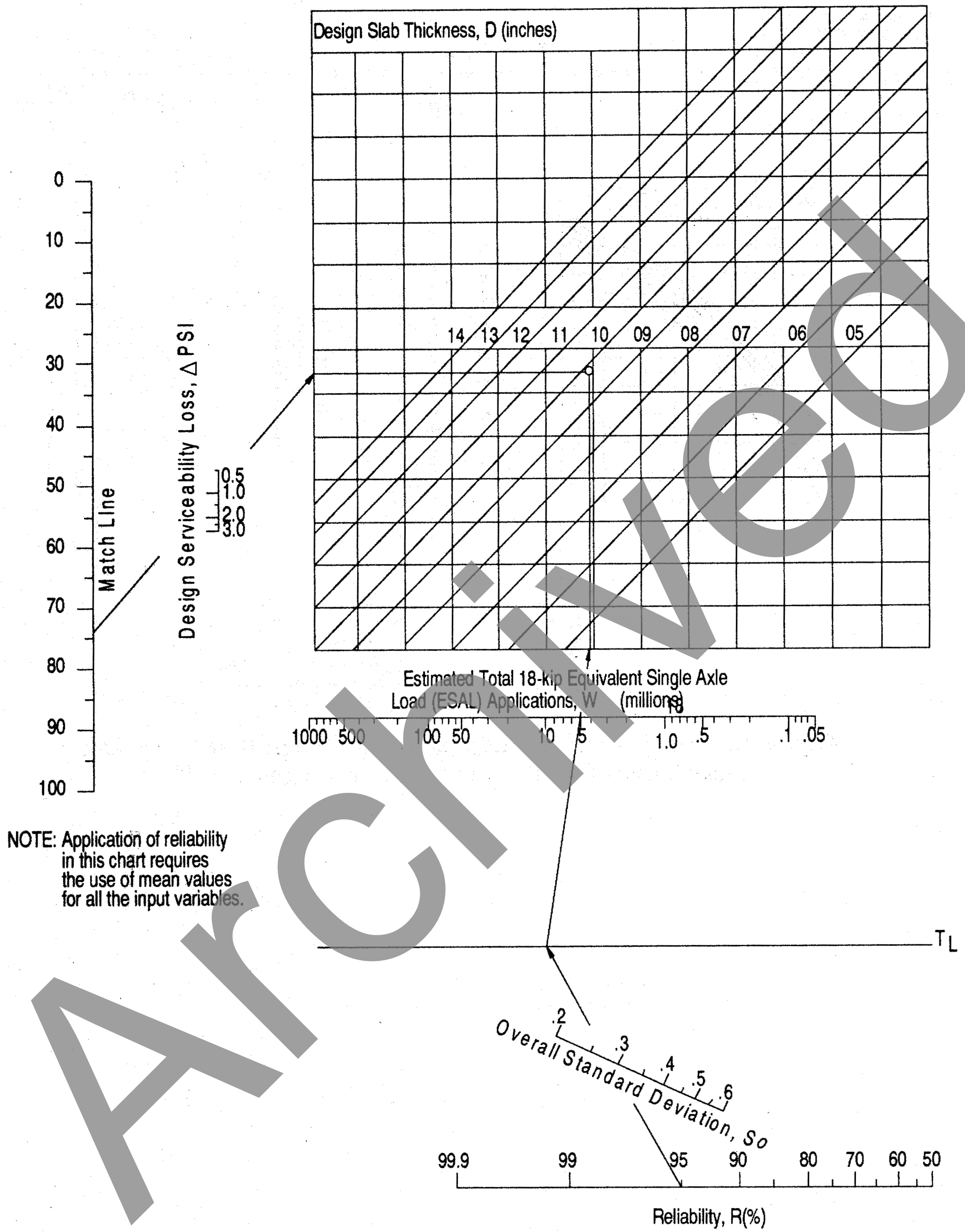

Figure 32. Design chart for rigid pavement design based on using mean values for each input variable (Segment 2).[3]

4. MECHANISTIC-EMPIRICAL DESIGN

Mechanistic pavement design procedures represent an approach to pavement design that uses sophisticated computer programs to calculate the stresses, strains, and deformations that develop in the individual layers of the pavement structure when subjected to a variety of loadings. The mechanistic concept requires selection of the critical pavement responses and the development of suitable transfer functions which convert the pavement responses into a number of load applications causing failure in that pavement structure. The structure is altered to obtain a satisfactory life.

Flexible Pavement Responses

Pavement responses are typically obtained from computer-based solutions to mechanics of deformable bodies. The most common computer programs in use today are the linear layered elastic programs and the finite-element programs. These programs provide for application of various load configurations on a layered pavement structure of 5 to 15 layers and the calculation of stresses, strains, and deflections at selected points within the pavement structure. These stresses, strains, and deflections are the "response" of the pavement structure to the loading imposed on the structure. These responses are responsible for the deterioration of the pavement under repeated applications of the load. Newer mechanistic procedures have allowed stress evaluation for temperature curling which can produce maximum stresses at the top of the slab. Each situation must be investigated thoroughly.

Rigid Pavement Responses

The structural design of a rigid pavement is traditionally controlled by the tensile stress at the bottom of the slab. The three load positions are corner, edge, and center slab, with the edge being the most critical for the performance of the slab. Varying levels of slab support can be modeled to determine their impact on slab life from the fatigue standpoint.

Critical Values of Pavement Responses

The stresses, strains, and deflections are present at all locations within the layered pavement structure. They will be higher at certain locations than at others. The location of the highest stress on the material least resistant to that stress will determine the critical location. The pavement will generally deteriorate the fastest at the locations where the responses are the greatest. For a flexible pavement, the critical locations are as follows:

- Radial stress or strain is maximum at the bottom of the asphalt concrete layer directly under the center of the wheel load. This stress or strain

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produces fatigue or alligator cracking in the surface layer under repeated loadings.

- Vertical compressive stress is a maximum directly under the centerline of the wheel load, in each layer. The maximum vertical stress is in the surface layer, which should be of the highest quality and most capable of resisting the deformations. The compressive stress on the subgrade is the critical value because the subgrade is the weakest material. The compressive stress produces rutting in the pavement.

- The shear stress is a combination of all stresses, and is normally greatest on the base course layer immediately under the surface layer. The shear stress in the base produces unstable behavior in the base, and leads to corrugations and rutting in the base.

- Tensile stress at the bottom of the slab is critical for a non-stressed concrete slab. The location of the load at the edge is most critical. Curling stresses developing from temperature gradients add a stress to the wheel load stress, even producing a maximum stress at the top of the slab. This location depends on slab thickness and temperature profile in the slab during the day.

Changing the pavement structure changes the location of the critical stress. Figure 33 shows the changes produced when a stabilized base layer replaces a granular base in a flexible pavement structure over a subbase. Most notably, the radial tensile stress moves to the bottom of the stabilized layer, completely changing the potential performance of the pavement section and necessitating very different design and construction considerations for pavements with stabilized materials.

Transfer Functions

The design of a pavement requires that the number of loadings to failure be predicted. This is done with the transfer function. The transfer function is a relationship developed either in the laboratory or from field observations that relates the stress or strain at the critical locations in the pavement to the number of load applications that will produce failure. The most common transfer functions are for the tensile stress at the bottom of the asphalt concrete or portland cement concrete layer for fatigue cracking, and the vertical compressive stress on the subgrade for rutting in the flexible pavement. Example curves are shown in figures 34, 35, and 36 for flexible fatigue, rutting, and rigid fatigue, respectively.

When a radial strain is calculated by a structural analysis model such as ELSYM5 or ILLI-SLAB, the strain is converted directly into a number of loadings to

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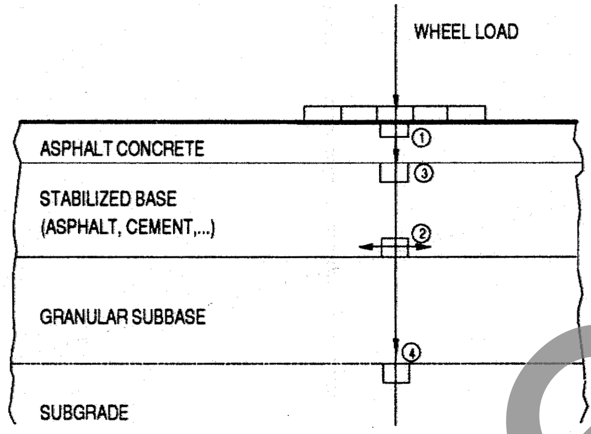
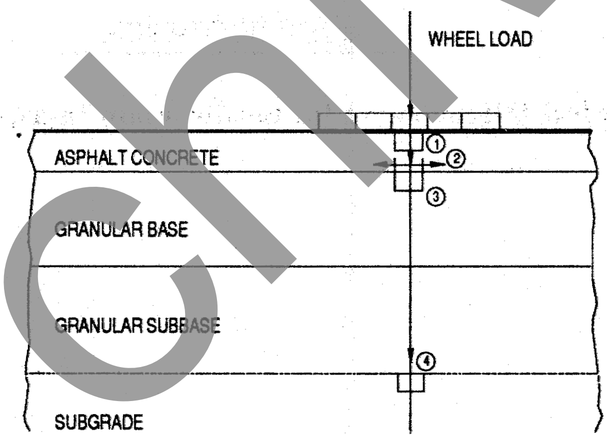
produce failure. Failure can be defined differently in studies which develop different transfer functions, and the exact definition of failure must be established by the design agency before curves such as these can be applied to different pavement



TYPICAL FLEXIBLE PAVEMENT
WITH STABILIZED BASE



TYPICAL FLEXIBLE PAVEMENT
WITH GRANULAR BASE

Figure 33. Typical asphalt pavements with granular and stabilized bases showing the critical stress/strain locations.[53]

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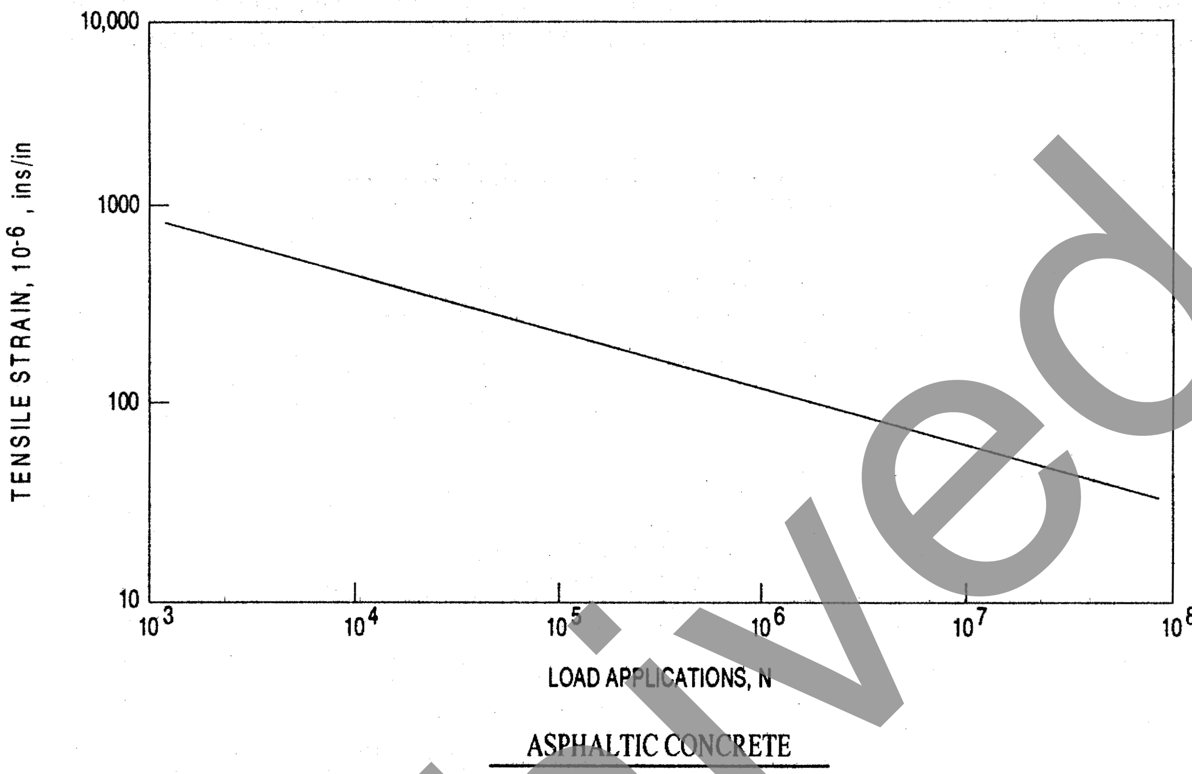

Figure 34. Typical fatigue curve for tensile strain in asphalt concrete.[53]

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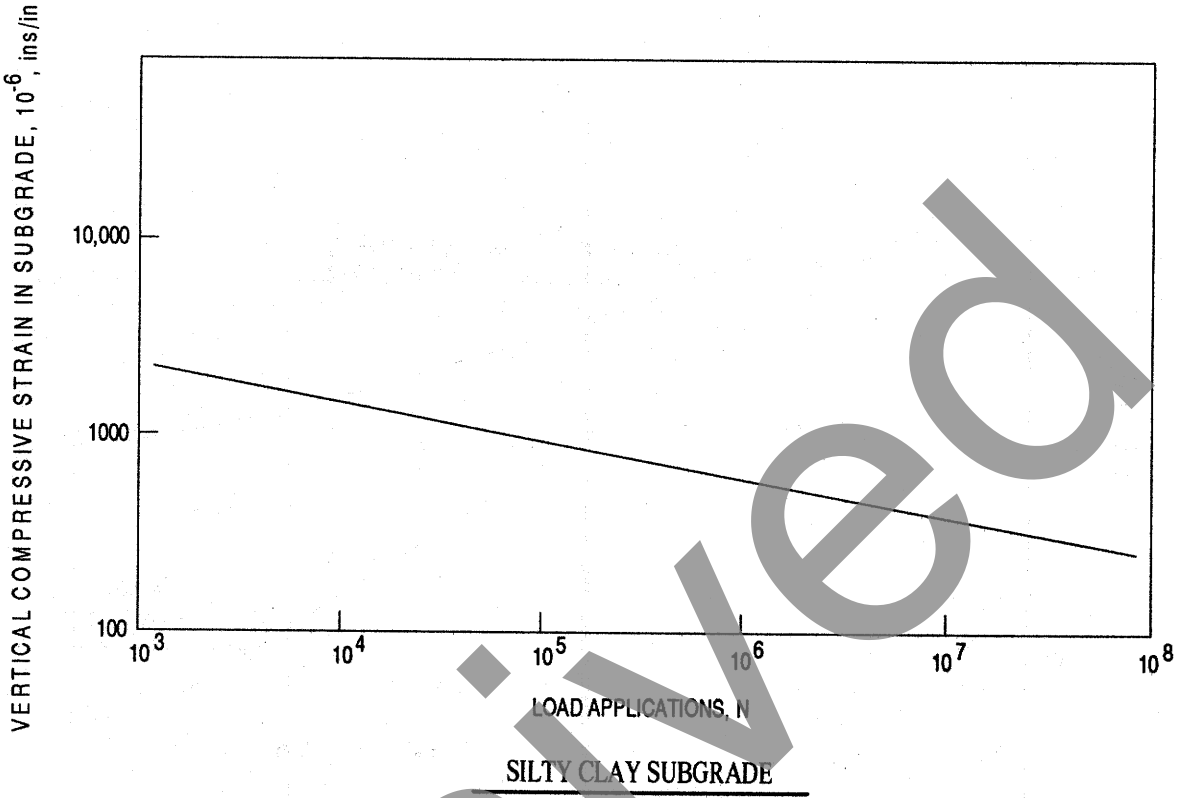

Figure 35. Typical fatigue curve for permanent deformation in a silty-clay subgrade.[53]

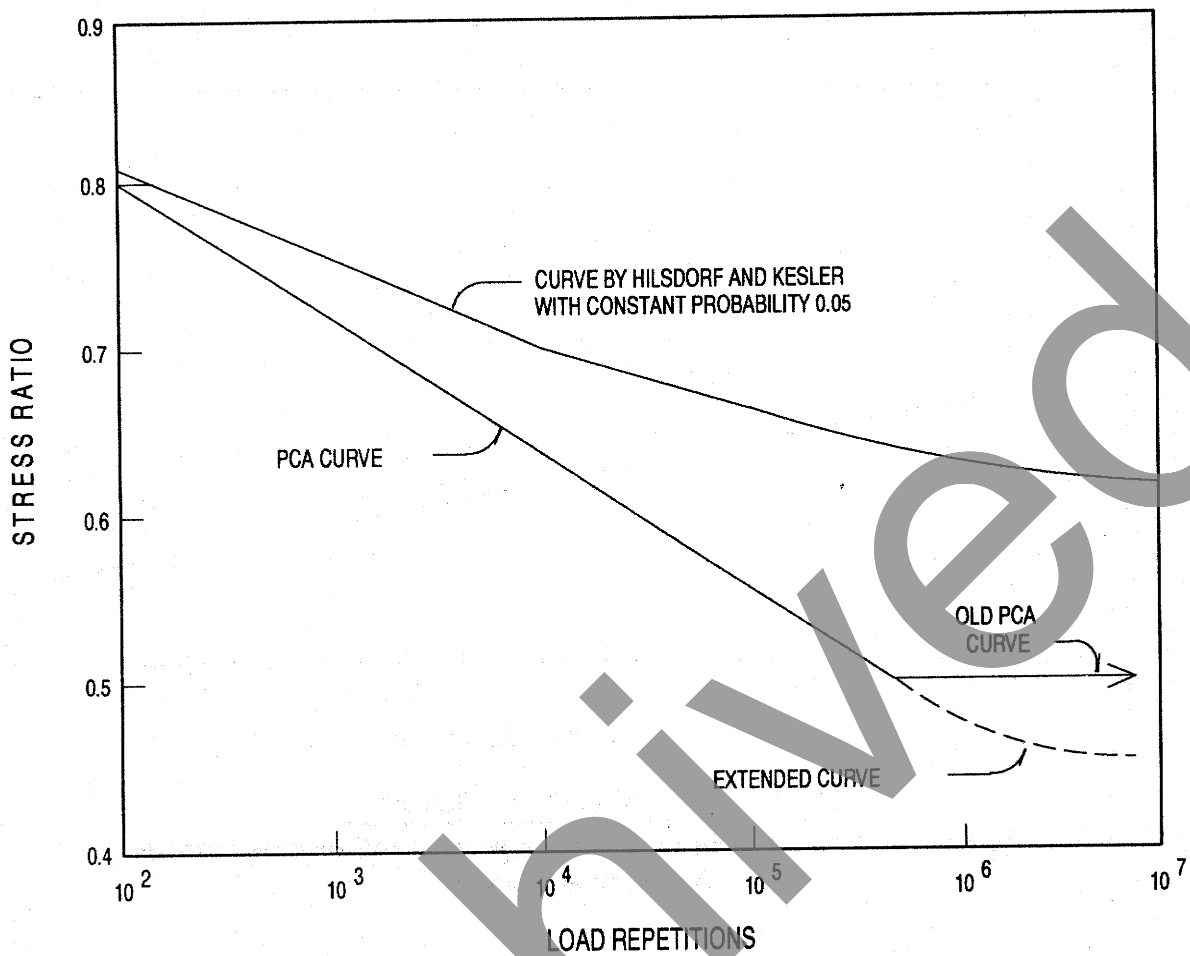

Figure 36. Fatigue curve for portland cement concrete slab.[58]

situations. They require a great deal of laboratory testing and field evaluation to accurately define failure before they can be used for design.

Benefits of Mechanistic-Empirical Design Procedures

The benefits which could result from the successful application of mechanistic-empirical procedures include:

- The ability to more accurately model the behavior of pavement sections.
- The ability to extrapolate general pavement performance from limited field data and laboratory results.
- The ability to predict the occurrence of specific types of distress.
- Improved design reliability.

Several important design factors that cannot be accurately addressed using empirical techniques can be considered using mechanistic techniques. Among those factors are the stress dependency of both the subgrade and base course, the time and temperature dependency of the asphaltic layers, the interface conditions between layer components, and modeling of the major distress modes of failure (rutting and fatigue cracking) by distress functions derived from the laws of mechanics. Mechanistic methods offer the potential to incorporate the numerous variables that influence pavement performance into the design procedure. The use of mechanistic theories offers the possibility of universal designs, which cannot be said of empirical methods. Some examples of design procedures incorporating various levels of mechanistic theory into the design process are described below.

Shell Method

This pavement design procedure has been developed for flexible highway pavements and later adapted to airfield pavement design.[55] The method is applicable to situations with asphalt concrete resting on granular material and in turn on subgrade soils whose strength index can be defined by the CBR procedure (either by measurement or estimation). In addition, the procedure can be used for selecting the thickness of asphalt pavements resting directly on subgrade soils. Although not a part of the original design procedure, the use of the substitution-ratio concept would permit the inclusion of stabilized materials.

<u>Principal Design Considerations.</u>

The pavement structure is represented by a three-layer elastic system (full friction at interfaces of layers) and the critical conditions for design are:

- Horizontal tensile radial strain on the underside of the asphalt-bound layer; if excessive, cracking may occur on the asphalt layer, and

- Vertical compressive strain in the surface of the subgrade; if excessive, permanent deformation may occur in the subgrade, leading, in turn, to permanent deformation on the surface of the pavement.

An 18,000-lb (80 kN) single-axle load (9,000-lb (40 kN) wheel load) is used for traffic estimates. Because of limitations in computer solutions for multi-layer elastic systems at the time the procedure was developed (1962), subgrade strains were determined for a load applied to a single circular area with a radius of 6 in (152.4 mm) and a contact pressure of 80 psi (550 kPa); tensile strains were, on the other hand, determined using a circular area with a radius of 4.2 inches (106.7 mm) and a contact pressure of 80 psi (550 kPa) (equivalent to 4,500-lb (20 kN) on one wheel of dual tires). Repetitions of the 18,000-lb (80 kN) axle load are considered as a part of the design process and the allowable strains associated with various numbers of repetitions are shown in tables 8 and 9.

Table 8. Allowable Tensile Strain in Asphalt-Bound Layer Corresponding to Different Load Applications (55).

WEIGHTED LOAD	TENSILE STRAIN
10^5	2.3×10^{-4}
10^6	1.45×10^{-4}
10^7	9.2×10^{-5}
10^8	5.8×10^{-5}

Table 9. Allowable Subgrade Compressive Strain Values Corresponding to Different Load Applications (55).

WEIGHTED LOAD APPLICATIONS	COMPRESSIVE STRAIN ON SUBGRADE IN. PER IN.
10^5	1.05 X 10-3
10^6	6.5×10^{-4}
10^7	4.2×10^{-4}
10^8	2.6 X 10-4

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Material Properties.

Materials in each of the three layers are assumed to be homogeneous, isotropic, and elastic.

Asphalt concrete.

The time-of-loadings and temperature dependency of asphalt concrete are included as design factors. Tensile strains in the asphalt concrete are determined for an assumed stiffness of 900,000 psi (6,200,000 kPa) (corresponds to a temperature of 50 °F [10 °C] and a time of loading of 0.02 sec.). For the subgrade strain, the air temperature is assumed to be 95 °F (35 °C), and effective stiffness modulus (depending on the thickness of asphalt concrete) is selected from Figure 37.

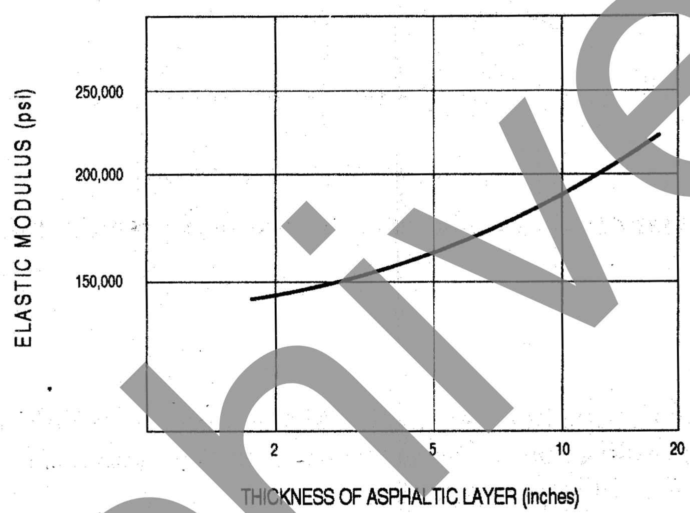

Figure 37. Relation of asphalt layer modulus to thickness of layer (air temperature of 95 °F [35 °C].[55]

Untreated aggregate base.

The modulus of the granular base is expressed in terms of the subgrade modulus and is dependent on the thickness of the base layer (figure 38).

Subgrade soil.

From dynamic (vibratory) tests in-situ, an approximate relationship between subgrade modulus (E_3) and CBR was established as 1500 times the CBR for soils with a CBR less than or equal to 10.[31]

Since the computations were developed in the early 1960's at a time when solutions were available only for a Poisson's ratio of 0.5 in each of the three layers, the design charts are based on this value for all the materials.

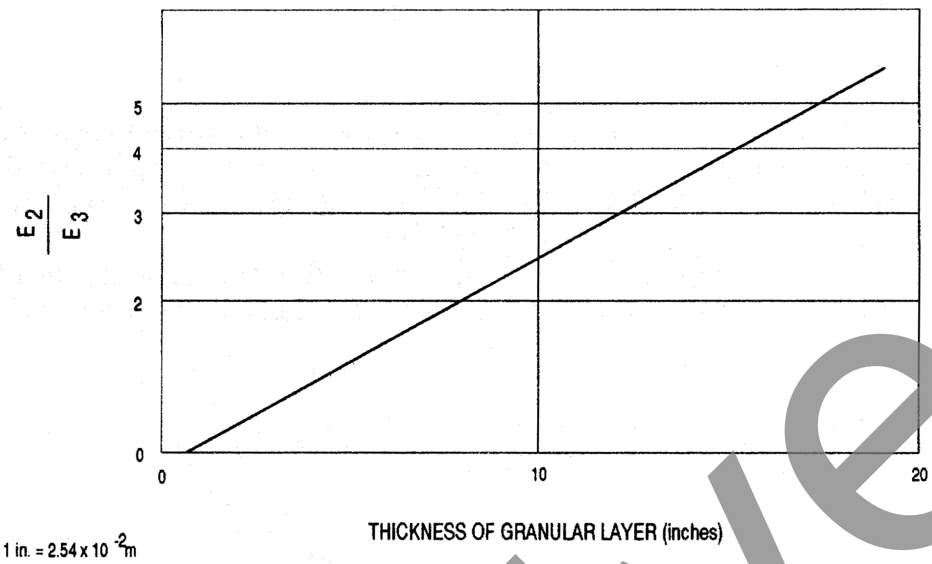

Figure 38. Relation of modular ratio to granular base thickness.[55]

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Materials tests.

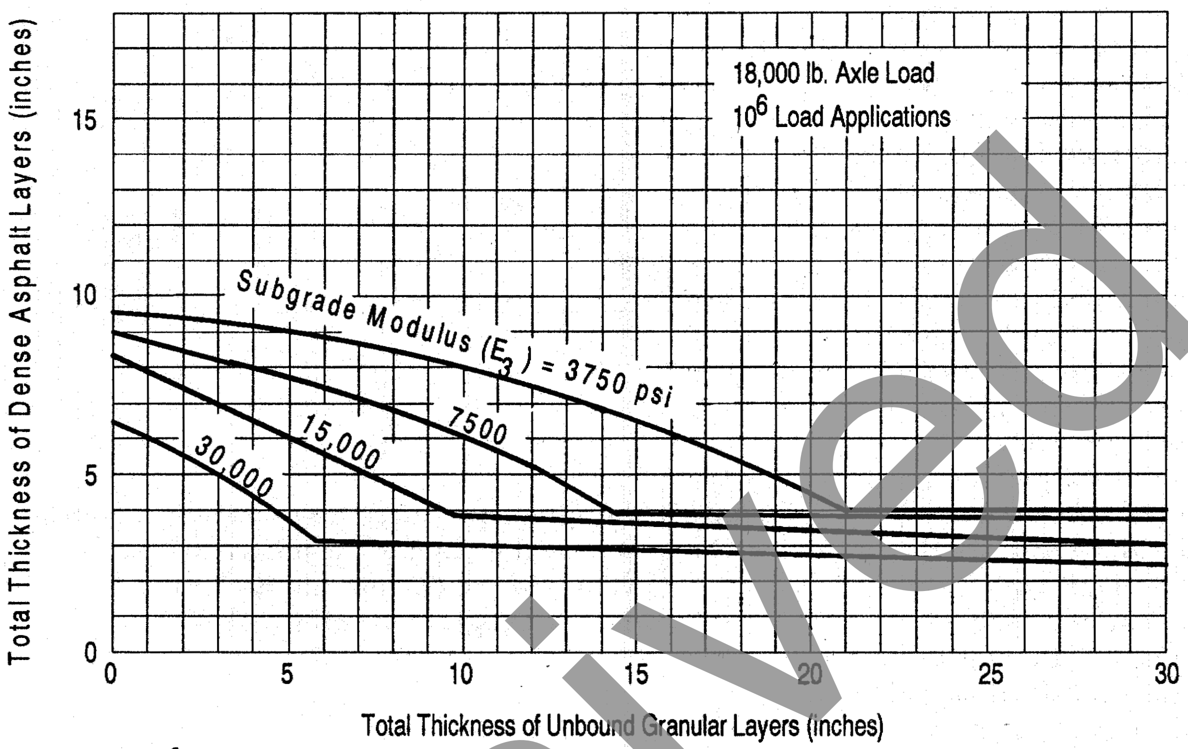
In this procedure the only test potentially required is a CBR test on the subgrade soil to permit estimation of the modulus from the equation E = 1500 CBR (psi), with the limitations noted previously.

Typical design relationship.

Design curves for a range in subgrade moduli are shown in figure 39 for 10^6 repetitions of an 18,000-lb (80 kN) axle load. In this procedure, the design process simply consists of selecting a combination of thicknesses of asphalt concrete and untreated granular material from the appropriate curve.

Thick-lift asphalt concrete sections.

From curves such as those shown in figure 39, it is possible to select, for a specific subgrade modulus, thicknesses of asphalt concrete corresponding to a thickness of the granular layer equal to zero. Alternatively, Heukelom and Klomp have formulated a relationship developed from the design chart which is plotted in figure 40 and has the form:[10]



1 in. = 2.54 x 10^{-2} m

1 psi = 6.89 x 10^{3} Pa

Figure 39. Design curves for 10^6 load applications.[55]

$$h = 10 \cdot (2/3 \text{ Log } N - \text{Log } E_s) + 13$$

where: h = thickness of asphalt-bound layer, cm (h > 6 cm)
N = number of repetitions of axle load
E_s = subgrade modulus, kg/cm^2 (E_3 in previous section)

Cement-stabilized layers.

A substitution ratio could be utilized with a value for stabilized material being selected from available data.

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Chevron Method

The Chevron Research Company has developed a thickness design procedure for pavement structures constructed with asphalt concrete, dense-graded emulsified asphalt mixes, or cement-modified emulsified asphalt mixes.[51] Although this procedure has not had the widespread use of the two methods described previously, it has a number of desirable features that provide it with the potential for more effective use of asphalt and emulsified asphalt stabilized materials.

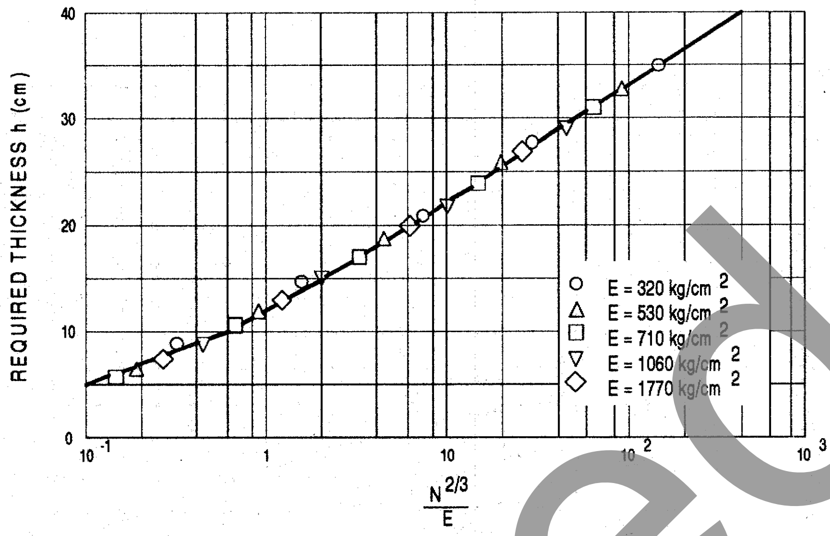

Figure 40. Design of thickness (h) of asphalt concrete layer resting directly on the subgrade as a function of the design number (N) and the subgrade modulus (E).[55]

Two critical strains, estimated by elastic layer theory, are examined in determining proper pavement thickness. These are the horizontal tensile strain (ε_t) at the bottom of the treated layer and the vertical compressive strain (ε_v) at the surface of the subgrade.

Two locations are checked for the critical strains under a standard 9000-lb (40 kN) wheel load (18,000-lb [80 kN] axle load) on dual tires used for design. One location midway between the wheels and the other directly under one of the wheels.

Allowable values for horizontal tensile strain are based on fatigue data developed from laboratory tests on asphalt concrete, emulsified asphalt, and cement-modified emulsified asphalt mixes. Vertical strain criteria for the subgrade have been selected to minimize surface rutting caused by overstressing the subgrade.

The steps in the design procedure are illustrated by the flow diagram of figure 41 will be summarized in the following sections.

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Traffic.

The mixed traffic is reduced to the number of daily equivalent 18,000- lb (80 kN) single-axle load (W_{18}) expected on the design lane during the selected design life of the structure. This can be determined in the same manner as it was for the AASHTO design.

Material Characteristics.

The subgrade modulus, or stiffness, can be determined from repeated load triaxial compression tests, estimated from conventional tests [e.g., E (psi) = 1500 CBR] or predicted from a soil classification. Poisson's ratio is assumed to be 0.45.

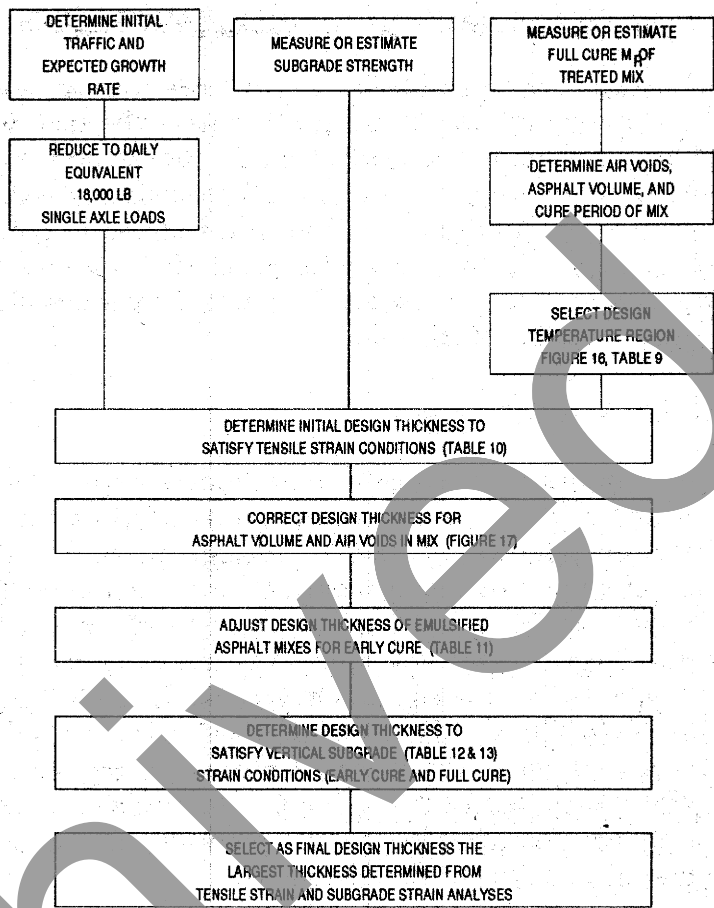

Figure 41. Flow diagram for structural design of emulsified asphalt pavement.[Ref]

The modulus of asphalt or emulsified asphalt mixes can be determined with the diametral resilient modulus (M_R) device. For this simplified thickness design procedure, M_R is measured at 73 $\pm$ 3 °F (23 $\pm$ 1.7 °C) on a fully cured specimen. A 5:1 ratio of M_R at 73 °F (23 °C) to M_R at 100 °F (38 °C) is assumed for all mixes.

It is also necessary to determine the air void and asphalt contents of the design mix. These properties have been shown to significantly influence the fatigue performance of an asphalt mix, and, hence, the thickness requirements for the pavement.[51] The ratio of asphalt volume to air voids plus asphalt volume is used as an indicator of the relative fatigue behavior of the mix.

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Effect of Early Cure of Emulsified Asphalt Mixes.

The time for an emulsified asphalt mix to reach its final M_R in the field is also important in determining its design thickness. Based in part on Chevron's field experience with emulsified asphalt mixes, the evapotranspiration map shown in figure 42 has been selected as a guide for estimating cure periods of emulsified asphalt mixes.[51] Emulsified asphalt mixes placed in parts of the southwest and most of Texas and Florida are expected to reach their ultimate design modulus in six months. A two-year cure period is assumed for emulsified asphalt mixes placed in the northern regions of the map.

Effect of Temperature.

Temperature has a significant influence on the thickness design of an asphalt or emulsified asphalt pavement through its effect on mix modulus. The effect of temperature is taken into account by designing for four different temperature regions. These are identified by average annual air temperatures (AAAT) of <40 °F, 40-55 °F, 55-65 °F, and >65 °F (<4, 4-13, 13-18, and >18 °C). A partial listing of communities falling into these temperature regions is given in table 9. The communities of Juneau, Alaska (<40°F [<4°C]); Portland, Oregon (40-55 °F [4-13 °C]); Sacramento, California (55-65 °F [13-18 °C]); and Bakersfield,, California (>65 °F [>18 °C]) were selected as being representative of the specific temperature regions and are used to develop the design tables included in this manual. The selection of other communities from the appropriate temperature regions will produce approximately the same thickness requirements, as affected by temperature.

Structural Design.

With the above date, a pavement thickness is selected to ensure that the horizontal tensile strain on the underside of the asphalt or emulsified asphalt-treated layer and the vertical strain at the subgrade surface satisfy the established criteria.

A design summary sheet, like that shown in table 11, can be used to determine the final design thickness of a pavement structure. A minimum full-depth design thickness of 4 in (101.6 mm) is recommended. The following steps are taken in the design, beginning with the tensile strain evaluation.

- Determine the initial design thickness (T_i) from table 12. For values of subgrade modulus and mix modulus (M_R) other than those given in table 12, T_i can be estimated by interpolation or extrapolation. Thicknesses of 2 in (50.8 mm) and 24 in (609.6 mm) have been established as practical lower and upper limits.

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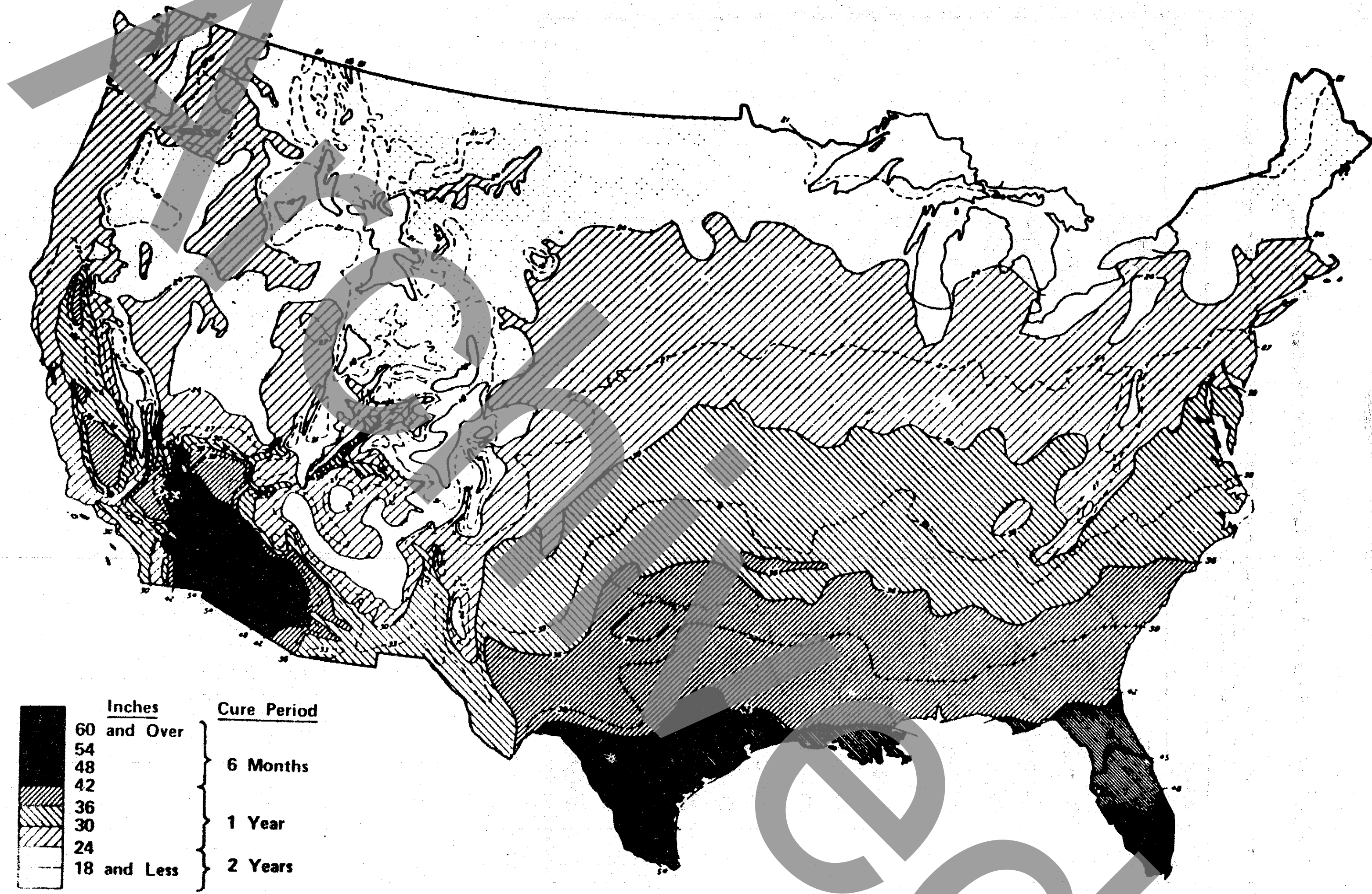

Figure 42. Field cure periods for emulsion-treated mixes based on annual potential evapotranspiration map.[51]

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Table 10. Select Cities in Each Temperature Region.

AVERAGE ANNUAL AIR TEMPERATURE, °F			
<40	40-55	55-65	>65
Anchorage	Flagstaff	Washington, D.C.	Phoenix
Fairbanks	Denver	Louisville	Miami
Juneau	Portland, ME	Oklahoma City	Hilo
Nome	Minneapolis	Richmond, VA	Corpus Christi
	Reno	Mobile	Bakersfield
	Albany	Sacramento	New Orleans
	Fargo	San Diego	Las Vegas
	Spokane	Atlanta	Dallas
	Eureka		
	Chicago		
	Boston		
	Detroit		
	Portland, OR		
	Salt Lake City		
	Boise		
	Omaha		

°C = 5/9 (°F - 32)

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Table 11. Design summary sheet.[51]

Subgrade Modulus, E_s, psi	= 12,000
Design Life, n, Years	= 20
Traffic, W_{18}	= 137
Traffic, EAL	= 1,000,000
Temperature Region, AAAT, °F	= 55 - 65

	Asphalt Mix	Emulsified Asphalt Mix
Modulus, M_R, (73 °F), psi	300,000	600,000
Air Voids, V_a, %	5	10
Asphalt Volume, V_b, %	12	9
Cure Period, Months	--	12
Tensile Strain Evaluation		
Design Thickness, T_t, in.	8.1	10.6
Subgrade Strain Evaluation		
Design Thickness		
Early Cure, T_s, in	--	7.8
Design Thickness		
Early Cure, T_s, in	8.3	8.3
Final Pavement Design		
Thickness, T_A, in.	8.3	10.6

- Correct T_t for the volume of air voids and asphalt residue in the design mix using the variable

$$V_b/(V_a + V_b)$$

where: V_b = volume of asphalt residue, %
V_a = volume of air voids, %

The nomograph in figure 43 can be used to make this correction. The volume of asphalt residue (V_b) in the above expression is obtained from

$$V_b = P_b . G_s . (100 - V_a)/(100 \cdot G_b + P_b \cdot G_s)$$

Table 12. Thickness* (T_i) in inches to satisfy tensile strain requirements.[51]

SUBGRADE MODULUS, PSI	3,000				6,000				12,000				30,000			
AVERAGE ANNUAL AIR TEMPERATURE, °F	< 40	40-55	55-65	> 65	< 40	40-55	55-65	> 65	< 40	40-55	55-65	> 65	< 40	40-55	55-65	> 65
Traffic, EAL = 10^4																
M_R, psi = 50,000	4.9	6.5	7.8	9.1	3.7	5.0	5.8	6.6	2	2.6	2.8	2.9	2	2	2	2
100,000	3.7	5.3	6.2	7.2	2.7	4.0	4.7	5.3	2	2.1	2.5	2.6	2	2	2	2
300,000	2.1	3.4	4.3	5.1	2	2.8	3.1	3.8	2	2	2	2	2	2	2	2
600,000	2	2.4	3.2	3.9	2	2	2.2	2.8	2	2	2	2	2	2	2	2
900,000	2	2	2.6	3.3	2	2	2	2.2	2	2	2	2	2	2	2	2
Traffic, EAL = 10^5																
M_R, psi = 50,000	8.3	11.5	13.9	16.5	7.0	9.7	11.2	12.7	5.4	7.3	8.3	9.3	2.3	2.7	2.8	2.9
100,000	6.3	9.0	10.8	12.9	5.4	7.6	9.1	10.5	4.2	5.8	6.9	7.8	2	2.5	2.7	2.7
300,000	4.0	5.8	7.3	8.7	3.3	5.1	6.0	7.3	2.4	3.9	4.8	5.6	2	2	2.0	2.2
600,000	2.8	4.5	5.5	6.7	2.1	3.7	4.7	5.6	2	2.7	3.6	4.4	2	2	2	2
900,000	2.1	3.7	4.7	5.7	2	3.0	3.9	4.8	2	2.1	2.9	3.7	2	2	2	2
Traffic, EAL = 10^6																
M_R, psi = 50,000	13.0	18.1	21.6	24	11.6	16.1	18.7	21.3	9.8	13.1	15.1	17.2	6.8	8.6	9.3	9.8
100,000	9.8	14.3	17.1	20.3	8.8	12.6	15.2	17.6	7.5	10.6	12.5	14.2	5.2	7.3	8.2	8.7
300,000	6.0	9.1	11.3	13.9	5.5	8.2	10.1	12.2	4.7	6.9	8.6	10.2	3.1	4.9	5.9	6.9
600,000	4.6	6.8	8.6	10.6	4.0	6.0	7.6	9.4	3.3	5.2	6.4	7.9	2	3.5	4.5	5.4
900,00	3.7	5.6	7.2	8.9	3.2	5.1	6.3	8.0	2.4	4.3	5.4	6.7	2	2.8	3.8	4.6
Traffic, EAL = 10^7																
M_R, psi = 50,000	19.2	24	24	24	17.8	23.9	24	24	15.6	20.5	23.5	24	12.1	15.3	16.5	17.3
100,000	14.5	21.0	24	24	13.5	19.3	22.8	24	11.9	16.8	19.5	22.2	9.5	12.9	14.7	15.7
300,000	8.8	13.6	16.7	20.5	8.3	12.5	15.6	18.8	7.4	11.1	13.8	16.4	5.8	8.8	10.8	12.4
600,000	6.4	10.0	12.5	15.6	5.9	9.2	11.6	14.5	5.3	8.2	10.4	12.8	4.1	6.5	8.2	10.0
900,000	5.3	8.2	10.5	13.2	5.0	7.7	9.7	12.1	4.4	6.8	8.6	10.8	3.3	5.3	6.8	8.5

*For asphalt volume, V_b = 11% and air voids, V_a = 5%. Use Figure 17 to correct for other values of V_a and V_b.

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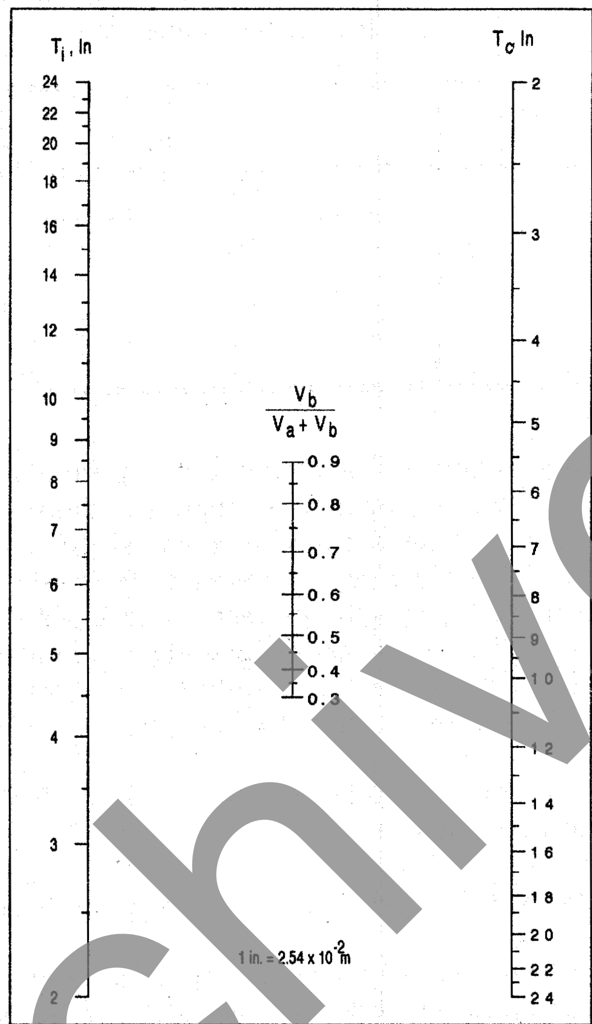

Figure 43. Correction of pavement design thickness for air voids and asphalt content in mix.[51]

where:

P_b	=	percent by weight of asphalt residue (based on dry weight of aggregate)
B_b	=	specific asphalt
G_s	=	average specific gravity of aggregate

- In the case of emulsified asphalt mixes, multiply the thickness determined above by the appropriate early cure adjustment factor in table 13. (This step can be eliminated for asphalt mixes and cement-modified emulsified asphalt mixes.)

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Table 13. Correction factor for early cure period of emulsified asphalt mixes.[51]

CURE PERIOD, MONTHS	DESIGN LIFE, YEARS	
	10	20
6	1.05	1.03
12	1.06	1.03
24	1.15	1.08

The addition of a small amount of portland cement will significantly increase the early strength (modulus) of emulsified asphalt mixes. The use of cement-modified emulsified asphalt mixes eliminates the need for an early cure adjustment to the design thickness. The cement content is normally between 1 and 2 percent by weight of dry aggregate. For a satisfactory fatigue life, the ratio of cement to emulsified asphalt in the mix should not be more than one part cement to five parts emulsified asphalt by weight.[51]

- Record as the design thickness from tensile strain evaluation (T_t) the value determined above.

The next step involves the evaluation of the subgrade strain criteria.

- With emulsified asphalt mixes, examine the early cure condition for subgrade strain using table 14. (This step can be eliminated for asphalt mixes and cement-modified emulsified asphalt mixes.)

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Table 14. Thickness (T_s) in inches to satisfy subgrade strain requirements (early cure condition).[51]

SUBGRADE MODULUS, PSI	3,000				6,000				12,000				30,000			
Average Annual Air Temperature, °F	<40	40-55	55-65	> 65	<40	40-55	55-65	> 65	<40	40-55	55-65	>65	<40	40-55	55-65	>65
Traffic, EAL = 10^2	3.0	7.6	8.7	9.2	2.9	5.4	6.1	6.4	2.6	3.0	3.1	3.1	2.5	2.9	3.0	3.0
Traffic, EAL = 5 x 10^2	5.2	9.2	11.0	11.9	4.4	6.9	7.8	8.2	2.8	4.5	4.9	5.1	2.7	3.1	3.2	3.2
Traffic, EAL = 10^3	6.1	9.9	12.0	13.0	5.0	7.5	8.5	9.0	2.9	5.1	5.7	6.0	2.8	3.2	3.3	3.3
Traffic, EAL = 5 x 10^3	8.1	13.0	15.3	16.4	6.8	10.2	11.6	12.3	4.8	7.3	8.2	8.6	3.1	3.9	4.2	4.4
Traffic, EAL = 10^4	9.0	14.4	16.7	17.9	7.5	11.4	12.9	13.7	5.7	8.3	9.2	9.7	3.2	4.2	4.6	4.8

[1]Early cure period is taken as the most critical first month (normally July) after construction. Mix modulus during this period is assumed to be 50,000 psi at 73°F.

1 in. = 2.54 x 10^{-2} m
1 psi = 6.89 x 10^3 Pa
°C = 5/9 (°F-32)

a. Calculate traffic for critical period (normally first month after construction).

b. For the appropriate subgrade modulus and temperature region, estimate the required design thickness (T_s).

- Examine the fully cured condition for subgrade strain using table 15.

a. Calculate traffic during the fully cured period. No early cure adjustment for traffic is necessary for asphalt mixes and cement-modified emulsified asphalt mixes.

b. For the appropriate subgrade modulus (M_R) and temperature region, estimate the required design thickness (T_s).

- Record as the design thickness from subgrade strain evaluation (T_s) the larger of the values determined above.

- Record as the final pavement thickness (T_A) the larger of the design thicknesses from above.

The composite pavement structures can be analyzed by including the following:

- For composite pavement structures, determine the final design thickness for each mix under consideration. Calculate thickness substitution ratios and use to recommend composite structure.

For example, comparing an asphalt mix and emulsified asphalt mix, the full-depth design might be:

T_A = 12 in (304.8 mm) (emulsified asphalt mix)

T_A = 9 in (228.6 m) (asphalt mix)

The substitution ratio is 12/9 = 1.22. A recommended composite structure might be:

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Table 15. Thickness (T_s) in inches to satisfy subgrade strain requirements (fully cured condition).[51]

SUBGRADE MODULUS, PSI	3,000				6,000				12,000				30,000			
AVERAGE ANNUAL AIR TEMPERATURE, °F	< 40	40-55	55-65	> 65	< 40	40-55	55-65	> 65	< 40	40-55	55-65	> 65	< 40	40-55	55-65	> 65
Traffic, EAL = 10^4																
M_R, psi = 50,000	5.5	8.3	10.8	11.5	4.6	7.0	8.3	9.3	3.0	5.4	6.2	6.9	2.5	3.1	3.3	3.5
100,000	5.5	6.0	7.3	8.2	4.6	5.0	6.1	7.0	3.0	3.4	4.6	5.3	2.5	2.5	2.9	3.0
300,000	5.5	5.5	5.5	5.5	4.6	4.6	4.6	4.6	3.0	3.0	3.0	3.0	2.5	2.5	2.5	2.5
600,000	5.5	5.5	5.5	5.5	4.6	4.6	4.6	4.6	3.0	3.0	3.0	3.0	2.5	2.5	2.5	2.5
900,000	5.5	5.5	5.5	5.5	4.6	4.6	4.6	4.6	3.0	3.0	3.0	3.0	2.5	2.5	2.5	2.5
Traffic, EAL = 10^5																
M_R, psi = 50,000	8.1	12.7	14.5	16.5	6.9	10.3	12.4	13.9	5.5	8.3	9.5	10.6	2.9	5.2	6.1	6.7
100,000	8.1	8.7	10.6	12.0	6.9	7.4	9.1	10.2	5.5	6.0	7.2	8.2	2.9	3.0	4.7	5.2
300,000	8.1	8.1	8.1	8.1	6.9	6.9	6.9	6.9	5.5	5.5	5.5	5.5	2.9	2.9	2.9	2.9
600,000	8.1	8.1	8.1	8.1	6.9	6.9	6.9	6.9	5.5	5.5	5.5	5.5	2.9	2.9	2.9	2.9
900,000	8.1	8.1	8.1	8.1	6.9	6.9	6.9	6.9	5.5	5.5	5.5	5.5	2.9	2.9	2.9	2.9
Traffic, EAL = 10^6																
M_R, psi = 50,000	11.5	16.8	20.1	22.4	9.8	15.0	17.3	19.3	8.3	12.2	14.4	15.8	6.0	8.6	9.5	10.4
100,000	11.5	12.3	14.8	16.5	9.8	10.8	13.0	14.7	8.3	9.0	10.8	12.1	6.0	6.6	7.6	8.5
300,000	11.5	11.5	11.5	11.5	9.8	9.8	9.8	9.8	8.3	8.3	8.3	8.3	6.0	6.0	6.0	6.0
600,000	11.5	11.5	11.5	11.5	9.8	9.8	9.8	9.8	8.3	8.3	8.3	8.3	6.0	6.0	6.0	6.0
900,000	11.5	11.5	11.5	11.5	9.8	9.8	9.8	9.8	8.3	8.3	8.3	8.3	6.0	6.0	6.0	6.0
Traffic, EAL = 10^7																
M_R, psi = 50,000	15.5	22.6	24	24	13.6	20.4	23.9	24	12.0	17.5	20.6	22.6	9.0	12.7	14.3	15.4
100,000	15.5	16.5	19.9	22.2	13.6	15.0	17.9	19.9	12.0	13.0	15.5	17.2	9.0	9.8	11.2	12.6
300,000	15.5	15.5	15.5	15.5	13.6	13.6	13.6	13.6	12.0	12.0	12.0	12.0	9.0	9.0	9.0	9.0
600,000	15.5	15.5	15.5	15.5	13.6	13.6	13.6	13.6	12.0	12.0	12.0	12.0	9.0	9.0	9.0	9.0
900,000	15.5	15.5	15.5	15.5	13.6	13.6	13.6	13.6	12.0	12.0	12.0	12.0	9.0	9.0	9.0	9.0

1 psi = 6.89×10^3 Pa
1 in. = 2.54×10^{-2} m
°C = 5/9 (°F-32)

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Surface layer (asphalt mix) = 3 in (76.2 mm)

Base (emulsified asphalt mix) $(9 - 3) \cdot 1.33^{**}$ = 8 in (203.2 mm). The ratio of 1.33 applies to this example only. Higher or lower values will be obtained for different design situations.

Discussion of Chevron Procedure

The fatigue criteria used in the tensile strain evaluation are for mixes with up to 12.5 percent air voids and an asphalt volume of 11 percent. Very little fatigue information exists on higher void content [lower $V_b/(V_a + V_b)$] mixes. Mixes with extremely high void contents (>20 percent) such as open-graded mixes seldom fail in the field by fatigue. It is conceivable that the primary thickness design consideration for these mixes is vertical subgrade strain. Permanent deformation of the mix itself is also an important design consideration for such materials.

Inspection shows that, for high temperature regions (AAAT >65 °F [18 °C]), relatively large design thicknesses are predicted with low stiffness mixes. This suggests that higher stiffness mixes (equal to or greater than 300,000 psi (2,070,000 kPa) are more appropriate for these regions. One way of obtaining higher stiffness is to use a harder asphalt. Conversely, in low temperature regions, the design thickness obtained from a tensile strain evaluation for moderate-to-high stiffness mixes. This permits a reduction in the stiffness of the mix selected without necessarily increasing the final design thickness. The use of low stiffness mixes in cold climate areas will significantly improve the pavement's resistance to thermal cracking.

This design procedure also permits preliminary examination of the economics of pavement construction, taking into account the interrelationships between asphalt or emulsified asphalt mix characteristics and pavement thickness.

Asphalt Institute Method

The Asphalt Institute procedure can be used to design an asphalt pavement composed of various combinations of asphalt surface and base, emulsified asphalt surface and base, and untreated aggregate base and subbase.

The original Asphalt Institute design methodology was an empirical approach based upon data from the AASHO Road Test, The WASHO Road Test, and other various State and local test sections. This procedure was completely revised in 1981 and the current Asphalt Institute procedure as presented in MS-1, uses multi-layer linear elastic theory for the determination of the required pavement thickness.[52] Full friction is assumed to exist between the layers. Each elastic layer is characterized by a modulus of elasticity (E) and Poisson's ratio.

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A computer program was used in the development of the design procedure to examine two critical stress-strain conditions. The first is the maximum vertical compressive strain induced at he top of the roadbed soil from an applied wheel load; the second is the maximum horizontal tensile strain induced at the bottom of the asphalt concrete layer from applied wheel load. For these stress-strain conditions, two key assumptions are made relative to design considerations:

- If the vertical compressive strain at the top of the roadbed soil is excessive, rutting or permanent deformation will occur in both the roadbed soil and ta the surface of the asphalt concrete layers.
- If the horizontal tensile strain at the underside of the lowest asphalt bound layer is excessive, fatigue (alligator) cracking of the asphalt layers will develop under repeated traffic loading.

The Asphalt Institute flexible pavement design procedure strives to design a pavement structure that will be thick enough to prevent these excessive horizontal tensile and vertical compressive strains from occurring over a predetermined design period. Thickness design charts were developed using the computer program DAMA which modelled these two stress-strain conditions (i.e., for a given set of inputs, the largest strain [either vertical compressive or horizontal tensile] governs the thickness requirements).(11)

Design Considerations

The major design considerations required for the structural design of flexible pavement using The Asphalt Institute procedure include the selection of design input values for traffic, roadbed soil strength, material properties, and environmental conditions.

Traffic

The traffic analysis procedure used by The Asphalt Institute is based on the load equivalency factors developed at the AASHO Road Test. It is assumed that the loads applied to the pavement structure by mixed traffic can be expected in terms of 18-kip ESAL applications. The Asphalt Institute procedure requires that the ESAL factors are selected assuming a terminal serviceability index of 2.5 and structural number of 5 for single and tandem axles.

If traffic data or projections are unavailable, The Asphalt Institute method provides guidelines for estimating input ESAL values from the classification of the highway to be built and from the probable ranges of ESAL factors for the various truck values.(3) The designer must recognize that inadequate designs may result from

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the use of generic traffic estimates. Such estimates are generally acceptable only when a high risk of premature failure is acceptable.

Roadbed Soil Strength

The second major pavement input variable is the strength of the roadbed soil. The roadbed soil is characterized by the resilient modulus (M_R), which was described earlier in this chapter. The resilient modulus used in this design procedure is the "normal" resilient modulus that is not representative of times when the roadbed soil is frozen or when it is undergoing periods of thaw.

The best method to determine a representative roadbed soil resilient modulus is to perform substantial testing on the roadbed soil. This should include all roadbed soil material that is expected to be within 2 ft (0.6 m) of the planned subgrade elevation. If significant roadbed soil variation is present, random sampling should be done to determine the controlling (weakest) soil type, or the limits or boundary of each roadbed soil type. The latter approach allows the project to be subdivided for separate designs if the various soil type areas are large enough.

Material Properties

The properties of the pavement component materials are characterized by a modulus of elasticity and Poisson's ratio. The Poisson's ratios are assigned internally and are based on typical values derived from various research projects.

Other assumed characteristics of the specific materials used in The Asphalt Institute flexible pavement design are discussed below.

<u>Asphalt Concrete</u>

A high quality asphalt concrete was used in producing the charts for this design procedure. Thus, the design assumes that similar quality materials will be used.

<u>Emulsified Asphalt Mixes</u>

In the Asphalt Institute design method, it is permissible to use emulsified asphalt mixtures for base course layers. Three different types of emulsion mixes are allowed, depending primarily on the type of aggregate used in the mixture. The three mixes are:

- Type I, emulsified asphalt mixes with processed dense-graded aggregate.

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- Type II, emulsified asphalt mixes made with semi-processed crusher-run, pit-run aggregate.

- Type III, emulsified asphalt mixes made with sands or silty sands.

Typical material properties were used in the development of the thickness design curves for this particular type.

Untreated Granular Materials

Untreated granular materials must comply with ASTM specifications D 2940, except that the following requirements should apply where appropriate:

Test	**Test Requirements**	
* CBR, minimum or	20	80
* R-Value, minimum	55	78
Liquid Limit, maximum	25	25
Plasticity Index, maximum, or	6	NP
Sand Equivalency, minimum	25	35
% Passing #200 Sieve, maximum	12	7

The roadbed soil resilient modulus relations for CBR do not apply to untreated aggregate base and subbases.

Environmental Conditions

It is assumed in The Asphalt Institute method that environmental conditions can be incorporated through the effects of monthly temperatures changes throughout the year on the asphalt modulus and through consideration of the effects of temperature on the roadbed soil resilient modulus and modulus of the granular materials are not considered directly.

In consideration of the asphalt concrete layers, three sets of environmental conditions were selected to represent the range of conditions to which the design manual should apply. These three sets are shown in table 16.

Mean annual air temperatures were used to characterize the environmental conditions applicable to each region, and the characteristics of the materials were selected accordingly. In cold regions, the asphalt must be less stiff to minimize the potential for thermal cracking; in hot regions, the asphalt must be stiff to increase resistance to rutting and permanent deformation.

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Table 16. Asphalt grades appropriate for various environmental conditions.[57]

Temperature Condition	Frost	Asphalt Grades	
Cold, mean annual temperature < 45 °F	Yes	AC-5 AR-2000 120/150 Pen	AC-10 AR-4000 85/100 Pen
Warm, mean annual temperature between 45 °F and 75 °F	Possible	AC-10 AR-4000 85/100 Pen	AC-20 AR-8000 60/70 Pen
Hot, mean annual temperature > 75 °F	No	AC-20 AR-8000 60/70 Pen	AC-40 AR-16000 40/50 Pen

The Asphalt Institute method (MS-1) provides design charts only for a mean annual air temperature of 60 °F because it is assumed in that manual that if asphalt cement is selected based on the temperature guidelines previously discussed (for mean annual temperatures of 45 °F, 60 °F, and 75 °F), the resulting concrete modulus will remain approximately constant with changes in mean annual air temperature. If this assumption is invalid, the use of MS-1 for climates with mean annual air temperatures significantly different from 60 °F may result in over- or under-designs. Reference 11, which documents the development of and provides the basis for the MS-1 design method, contains a set of graphs for mean annual air temperatures of 45 °F, 60 °F and 75 °F so that variation in asphalt modulus with temperature is incorporated to a certain extent. If the pavement is to be designed for extreme climates, it is recommended that these design charts be utilized.

Structural Design Procedure

The design procedure for the Asphalt Institute method is to restrict the amount of radial tensile strain at the bottom of the asphalt-treated layer (which is a source of fatigue or alligator cracking) and to restrict the amount of vertical strain at the top of the roadbed soil (which is a source of rutting and permanent deformation). The limiting criteria for both fatigue cracking and permanent deformation are based on empirical data. Alligator cracking is limited to 20 to 25 percent of the pavement surface and rutting is limited to 0.5 in (12.7 mm) over the design period of the pavement. The pavement thickness obtained from these design charts satisfy the most critical of these two requirements.

The necessary information to utilize the Asphalt Institute design procedure consists of determining the roadbed resilient modulus, establishing the materials to be used in the pavement, determining the design period traffic, and selecting the thickness. Thickness design charts such as that shown in figures 44, 45, and 46 for

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Full-Depth Asphalt Concrete, Emulsified Asphalt Mix Type II, and 6 in (152.4 mm) Aggregate Base, respectively, are used to determine the required asphalt concrete surfacing thickness. Inputs of design traffic, roadbed soil resilient modulus, and a preselected material documentation (e.g., full-depth asphalt, 4 in (101.6 mm) aggregate base, 6 in (152.4 mm) aggregate base, etc.) are required. Reference 4 contains additional charts not in this manual.

If the pavement structure is to consist of full-depth asphalt concrete, the pavement thickness design charts can be entered directly with the resilient modulus value for the roadbed soil and the design traffic level. For full-depth asphalt concrete pavement structures, a minimum thickness of 4 in (101.6 mm) is suggested by this design method.

The pavement structure can also be designed using an emulsified asphalt base course in place of a portion of the asphalt concrete layer. As discussed previously, three different types of emulsified mixes can be designed depending on the quality of the material used in the mixture. The design charts must be entered with the resilient modulus and design traffic level for a specific emulsified asphalt base type to produce a total pavement thickness. A minimum thickness of asphalt concrete is needed over the emulsion stabilized base course. The minimum asphalt thickness depends on the level of traffic anticipated for the pavement structure. For both Type II and Type III emulsified asphalt bases, the minimum asphalt concrete thickness varies from 2 in (50.8 mm) for traffic levels of less than 10,000 ESALs to 5 in (127.0 mm) for traffic levels greater than 1 million ESALs. The thickness of the emulsified base is then determined by subtracting the minimum asphalt thickness from the total thickness obtained from the charts.

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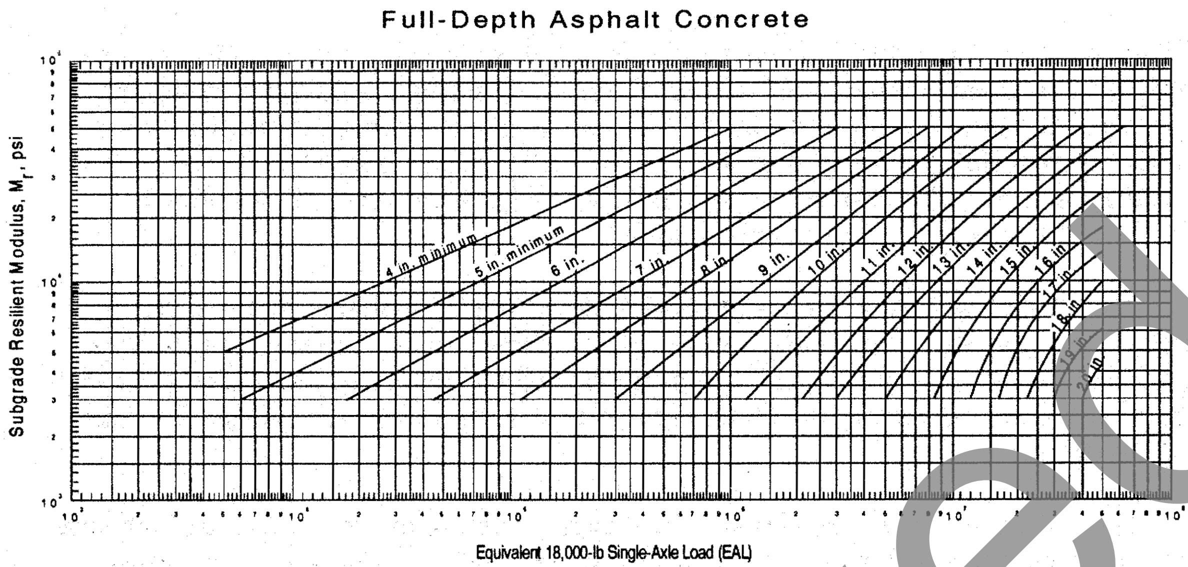

Figure 44. Example Asphalt Institute design chart for full-depth asphalt.(52)

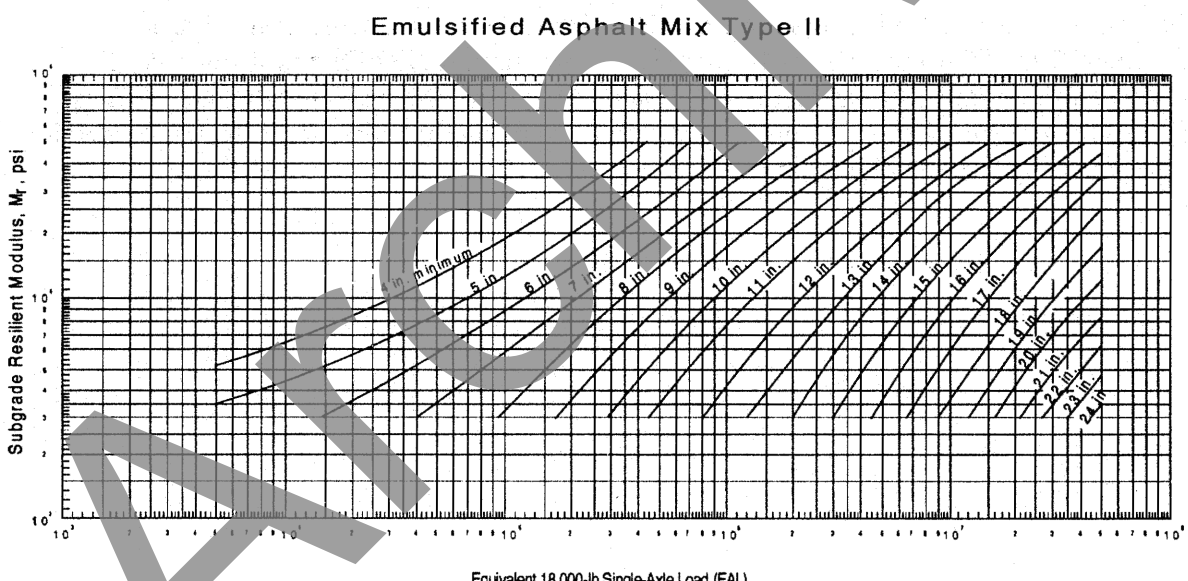

Figure 45. Example Asphalt Institute design chart for emulsified asphalt mix type II.(52)

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Untreated Aggregate Base 6.0 in. Thickness

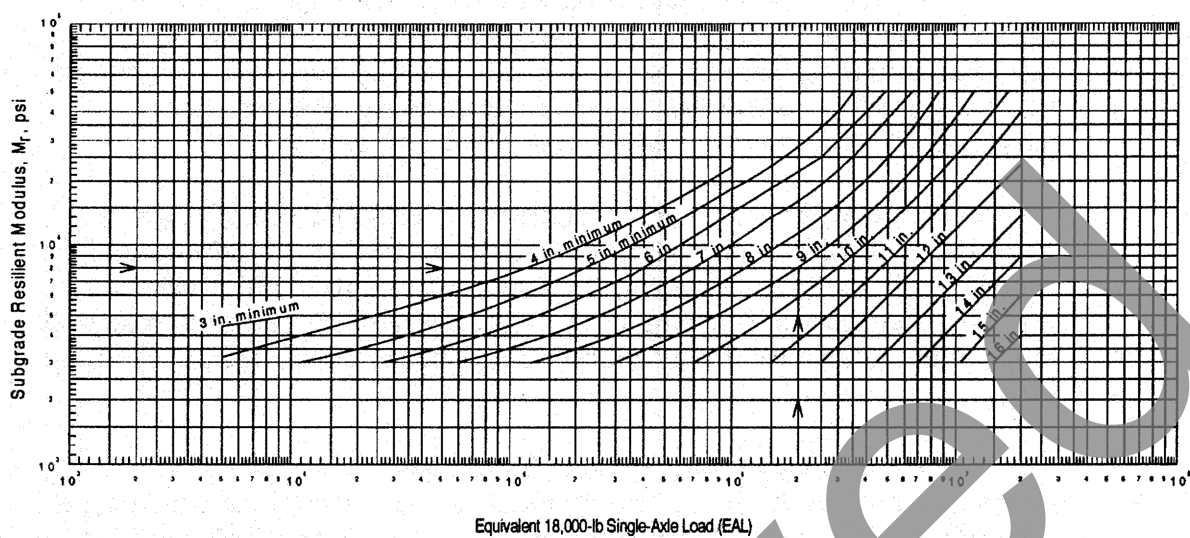

Figure 46. Example Asphalt Institute design chart for 6-in treated aggregate base.[52]

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Granular base courses can also be used as part of the pavement structure. For this material, a given thickness of untreated aggregate base is preselected (based on drainage, frost, or other requirements). The appropriate design chart is then entered (with a roadbed soil modulus and design traffic level) to yield the total thickness of asphalt concrete needed over the preselected granular base course thickness. The design manual provides for six different untreated base thicknesses—4, 6, 8, 10, 12, and 18 in (101.6, 152.4, 203.2, 254.0, 304.8, and 457.2 mm). The minimum thicknesses of asphalt concrete required range from 3 in (76.2 mm) for traffic levels of less than 10,000 ESALS to 5 in (127.0 mm) for traffic levels over 1 million ESALs.[52]

Since there are numerous different layer configurations available when using the Asphalt Institute method (i.e., full-depth asphalt concrete, emulsified asphalt bases, granular bases), there may be some difficulty in determining the pavement system which best addresses a given design situation. Therefore, the following items should be considered when selecting the type of pavement system:

- Full-depth asphalt concrete pavements have the advantages of better resistance to pavement stresses, less total required thickness than pavements with untreated aggregate base courses (meaning reduced excavation costs in some cases), and relative insensitivity to frost or moisture. However, materials for aggregate bases are an abundant resource and are generally inexpensive and readily available. Aggregate bases can perform well when constructed properly, and drained adequately, in the same manner as the full depth sections.

- It is recommended that several designs be determined using different materials and then an economic analysis be performed to determine the preferred alternative. However, there are other factors which should also be considered in selecting the preferred alternative, such as material availability, geometric design problems, utility locations, agency policies, etc.

- Stage construction (the construction of successive layers of asphalt concrete according to design requirements and on a predetermined time schedule) should also be considered in the cost analysis. This approach is beneficial when funds are insufficient for constructing a pavement with a long design life. Stage construction is also desirable when there is a great amount of uncertainty in estimating traffic. The pavement can be designed for an initial traffic level and next stage of construction can be designed using traffic projections based on the in-service traffic data. Finally, stage construction can allow weak spots which develop in the first stage to be repaired in the next stage.

<u>Limitations of the Asphalt Institute Method</u>

A limitation of the published Asphalt Institute procedure is that it does not allow individualized consideration of environmental effects directly in the procedure. While there is an attempt to account for environmental effects in the roadbed soil resilient modulus and in the asphalt grade to be used, it does not accurately account for major climatic considerations such as seasonal variation in moisture.

Another problem lies in the limited environmental applicability of the design charts provided in the MS-1. It contains only design charts for mean annual air temperatures of 60 °F, which accurately represents only a portion of the United States. This limitation can be overcome by using the design charts for mean annual air temperatures of 45 °F and 75 °F presented in reference 4.

Finally, while the Asphalt Institute procedure has a firm basis in mechanistic analysis, it relies heavily on many empirical inputs. These include the computation of traffic equivalencies, the empirical assignment of limiting stress/strain criteria, and the use of empirical correlations between material strength parameters and resilient modulus by agencies without the proper testing equipment.

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PCA Method

The Portland Cement Association's (PCA) thickness design procedure for concrete highways and streets was published in 1984, revising a procedure that has been used since 1966.[58] One aspect of the new procedure is that an erosion analysis is applied in addition to the fatigue analysis. The erosion analysis recognizes that pavements can fail due to excessive pumping, erosion of the foundation and joint faulting. The fatigue analysis recognizes that pavements can fail in fatigue due to excessive load repetitions.

The stress calculations used in development of this procedure were performed using the J-SLAB program, a finite- element analysis program. The design procedure is based on a comprehensive mechanistic analysis of concrete stresses and deflections at pavement joints, corners, and edges using the finite-element program. The finite element formulation can model joint load transfer provided by dowels or by aggregate interlock and the effects of concrete shoulders. Representative slabs are pictured in figure 47.

Truck Load Placement

The PCA procedure recognizes that load placement relative to the free edge produces different stress conditions in the slab which produces different consumption of life. The mechanistic analysis of truck placement allows for a percent of trucks traveling along the edge of the slab to be used in the design. A value of 6 percent is incorporated into the design tables.

Erosion Analysis

A common distress in jointed pavements is the erosion of the base through pumping under the heavy truck loadings. This erosion produces faulting in the pavement, which is often a major factor in deciding when rehabilitation is necessary. This distress is closely associated to the deflection of the slab at the transverse joint when the wheel passes over the joint. The PCA researchers developed the Power concept to explain the impact that the wheel load pounding the slab down into the base material could have in producing faulting. They determined Power to be:

$$\text{Power} = 268.7 \cdot (p^2/h) \cdot k^{(-0.73)}$$

where:

where:	h	=	slab thickness, in
	k	=	modulus of subgrade reaction, pci
	p	=	estimated pressure at the slab-foundation interface, psi

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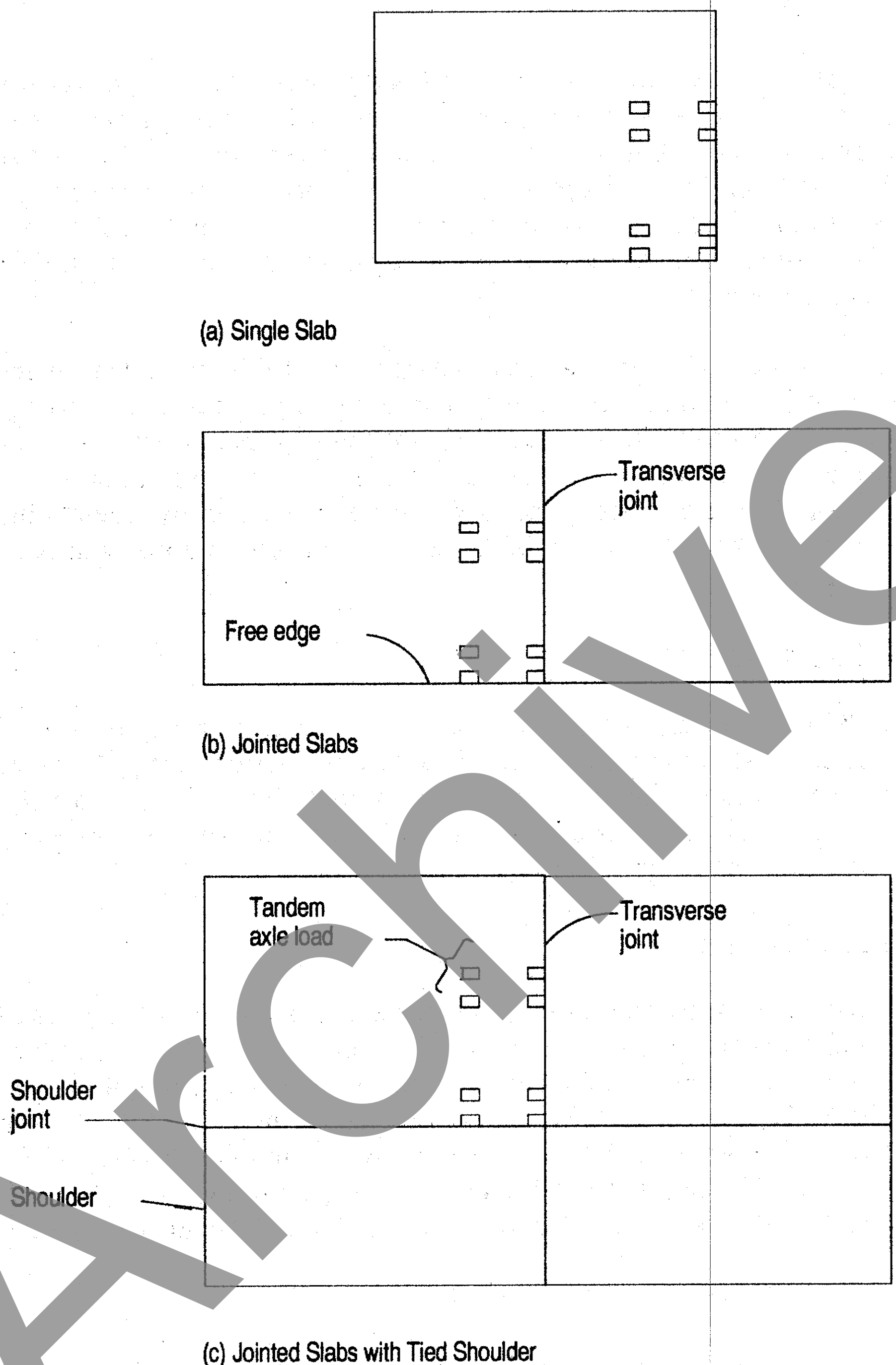

Figure 47. Typical slab systems.[58]

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The allowable loads to produce a failure level of faulting was obtained by matching field data to produce:

$$\text{Log } N = 14.524 - 6.777 \cdot (C_L P - 9.0)^{0.103}$$

where:	N	=	Allowable load repetitions to the end of the design period
	P	=	Power as defined above in terms of h and k
	C_L	=	subbase adjustment factor, varying from 1.0 for normal bases to 0.9 for high strength subbases such as stabilized bases.

The resulting equation for erosion damage is:

$$\text{Erosion Damage Percentage} = 100 \cdot \Sigma n_i \,.(C/N_i)$$

where:	n_i	=	Expected number of axle load repetitions in axle group i
	N_i	=	Allowable number of repetitions in axle group i
	C	=	0.06 for pavements without shoulder, and 0.94 for pavements with concrete shoulder (for 6 percent trucks at edge)

Work is underway to expand the erosion factor to include the effect of drainage conditions in changing the erosion potential in a pavement structure. At present there is no way to include any effect of improved drainage, other than to assume a reduced erosion may possibly occur when drainage is improved. There is no guidance available at present to justify any changes.

Variation in Concrete Strength

Variation in concrete strength is included by reducing the modulus of rupture by one standard deviation. For design purposes, a coefficient of variation of 15 percent is assumed and is incorporated into the design charts and tables. The strength is selected as the 28-day strength. The design procedure incorporates 30 percent strength gain occurring in the field after the 28-day cure period, although the user does not directly apply any corrections.

Fatigue Damage Calculation

The stress ratio fatigue concept for concrete is used to evaluate fatigue damage developing in the slab. The fatigue curve shown earlier in the transfer function section is incorporated into the design process, and the allowable load repetitions for a slab thickness is calculated from this curve.

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Warping and Curling

Warping and curling thermal stresses are not directly included in the design procedure, which could lead to a non-conservative design as research has shown that thermal curling stresses can add to the stress produced by the wheel load, reducing the life of the slab. This is particularly true when a stiff stabilized slab is used under the rigid surface slab.

Lean Concrete Subbase

The PCA procedure provides some guidance for designing using a lean concrete base in appendix C of the PCA Manual.[58] Such a base is stronger than untreated materials, and is assumed to be non-erodible. Recent studies have indicated that faulting can still develop over these materials.[74]

Design Procedure

Figure 48 shows the format for solving design problems by the PCA procedure. It requires a projection of the weights and volumes of axle loads that will traffic the pavement over a selected design period. This procedure is also included in the computer solution to the PCA design procedure.

Limitations of the PCA Procedure

The most serious deficiency is the assumption that curling and warping stresses are equal and opposite and cancel each other out. This is often an incorrect assumption, and can lead to an underdesigned pavement.

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Calculation of Pavement Thickness

Project Design 1A, four-lane Interstate, rural

Trial thickness 9.5 in.

Subbase-subgrade k 130 pci

Modulus of rupture, MR 650 psi

Load safety factor, LSF 1.2

Doweled joints: yes ✓ no

Concrete shoulder: yes no ✓

Design period 20 years

4-in. untreated subbase

Axle load, kips	Multiplied by LSF 1.2	Expected repetitions	Fatigue analysis: Allowable repetitions	Fatigue analysis: Fatigue, percent	Erosion analysis: Allowable repetitions	Erosion analysis: Damage, percent
1	2	3	4	5	6	7

8. Equivalent stress 206

9. Stress ratio factor 0.317

10. Erosion factor 2.59

Single Axles

30	36.0	6,310	27,000	23.3	1,500,000	0.4
28	33.6	14,690	77,000	19.1	2,200,000	0.7
26	31.2	30,140	230,000	13.1	3,500,000	0.9
24	28.8	64,410	1,200,000	5.4	5,900,000	1.1
22	26.4	106,900	Unlimited	0	11,000,000	1.0
20	24.0	235,800	"	0	23,000,000	1.0
18	21.6	307,200	"	0	64,000,000	0.5
16	19.2	422,500			Unlimited	0
14	16.8	586,900			"	0
12	14.4	1,837,000			"	0

11. Equivalent stress 192

12. Stress ratio factor 0.295

13. Erosion factor 2.79

Tandem Axles

52	62.4	21,320	1,100,000	1.9	920,000	2.3
48	57.6	42,870	Unlimited	0	1,500,000	2.9
44	52.8	124,900	"	0	2,500,000	5.0
40	48.0	372,900	"	0	4,600,000	8.1
36	43.2	885,800			9,500,000	9.3
32	38.4	930,700			24,000,000	3.9
28	33.6	1,656,000			92,000,000	1.8
24	28.8	984,900			Unlimited	0
20	24.0	1,227,000			"	0
16	19.2	1,356,000				
			Total	62.8	Total	38.9

Figure 48. PCA design worksheet.[58]

5. MECHANISTIC DESIGN FOR HIGH STRENGTH STABILIZED BASES

High strength stabilized base course materials (pozzolanic-aggregate-mixture, cement-aggregate-mixture, soil-cement) are utilized in many flexible and rigid pavements. The typical pavement section includes a minimum thickness asphalt concrete surface course over the stabilized base. In some applications, only a surface treatment is utilized. Gomez and Thompson concluded that the AASHTO structural number/layer coefficient design concept is not an adequate procedure for establishing thickness requirements for high strength stabilized base flexible pavements.[59] They developed a mechanistic consideration for structural design of these materials.

Cementitious stabilizers typically increase compressive strength, shear strength, tensile strength, and modulus of elasticity. Freeze-thaw and moisture resistance are significantly enhanced by stabilization. The structural response and performance of stabilized layers (for a given wheel loading) are influenced by the flexural strength, modulus, and thickness of the stabilized layer and subgrade modulus and strength.

Selection of Strength, Modulus, and Fatigue Properties

The mechanistic design of a pavement requires a knowledge of the stress-strain relationships for each of the materials to be used. Ideally, tests would be conducted as a part of the design to identify these relationships. This approach, however, is rarely practical both because the testing is costly and time consuming and because of the exact sources of all materials are frequently not know at the time of the design. Therefore, generalized relationships between the required properties and the properties normally specified for construction control are needed for the mechanistic approach to be useful for routine design.

This section identifies stress-strain relationships for cementitious-stabilized materials. These includes soil-cement, soil-lime mixtures, lime-fly ash aggregate mixtures, cement-aggregate mixtures, and the various stabilization processes that incorporate the combination of cement, fly ash and/or lime. The relationships presented herein are quite general with no distinction being made for the effects of specific test conditions. For example, the compressive-to-tensile strength relationship is discussed without regard to such details as the length-to-diameter (L/D) ratio of the compressive test specimens. This is because the relationships reported are so divergent that the influence of these details become obscured. This, however, does not prevent these generalized relationships from being useful for design purposes, nor does it negate the importance of noting and controlling such details in routine material testing programs.

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Strength Relationships

The strength of the stabilized material is fundamental property required for design, often specified and used for construction control. The types of tests frequently used for control are the flexure (beam) test, the split tension test, and the unconfined compression test with the latter being perhaps the most common because of its relative simplicity. Since each of these provide a different measure of the "true" strength of the material, some relationship between the different measures is required for them to have equal value to the design process.

Numerous investigators have found that the tensile strength obtained for a given material will vary depending upon which type of test is used. Sherwood for example found that the flexural strength was generally about 1.5 times the split tensile strength.[60] Similarly, data reported by Pretorious and Monismith suggest that flexural strength is about twice the direct tensile strength.[61] The direct tension test is generally believed to provide the truest measure of tensile strength.

Raad has demonstrated that these apparent differences are because the modulus of elasticity of these materials is not the same in both compression and tension.[62] He made a detailed analysis of the various tensile strength test using finite-element theory and varying the compression and tension moduli of elasticity. In the analyses, he demonstrated that the tensile strengths from the split tensile test and the direct tension test are about equal, but that the tensile strength from the flexure test can be more than double the direct tension strength.

Relationships between unconfined compressive strength and the various measures of tensile strength have been reported by many investigators. Felt and Abrams found that flexural strength of soil cement was about 20 percent of the compressive strength.[63] Thompson found that the flexural strength of lime soil mixtures was about 25 percent of the unconfined compressive strength.[64] Barenberg reported flexural strengths for lime fly-ash aggregate mixtures that were 18 to 20 percent of the corresponding compressive strengths.[65]

The following conclusions are made regarding the relationships between the various measures of strength for cementitious stabilized materials:

- Both the direct tension test and the split tension test provide a reasonable measure of the "true" tensile strength of a cementitious stabilized material.

- The tensile strength determined from a flexural (beam) test is significantly greater than the "true" tensile strength, often by as much as a factor of two.

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- The unconfined compression tests can be used to approximate the tensile strength of a cementitious stabilized material. An acceptable conservative estimate is that the tensile strength is 10 percent of the compressive strength.

From the conclusions the following relationships can be used in design:

$$TS = ST$$
$$ST = 0.5\ FS$$
$$TS = 0.1\ CS$$
$$FS = 0.2\ CS$$

where:

where:	TS	=	tensile strength (direct tension)
	ST	=	split tension strength
	FS	=	flexural strength
	CS	=	compressive strength

Stress-Strain Relationships

The stress-strain behavior of a pavement material is normally expressed in terms of an elastic or resilient modulus. For cementitious-stabilized materials, the selection of an appropriate modulus value to represent the material for design is complicated not only because of the difficulty in testing but also because of the following reasons:

- Different test methods give different values.[66]
- The relationship is generally nonlinear above 60 percent of the strength of the material.[67]
- The modulus is generally lower in tension than it is in compression.[68]

Because of these difficulties, Packard recommended using a relationship between flexural strength and the modulus of elasticity in place of testing.[69]

Numerous investigators have reported data relating strength and the modulus of elasticity of various cementitious-stabilized materials. They concluded that different relationships exist, dependent upon the quality of the material being stabilized. They classified the material reported as lean concrete, cement-bound granular material, and fine-grained soil cement. For a given strength level, they found the lean concrete to have the highest modulus and the fine-grained soil cement to have the lowest. From examination of these relationships, three strength-modulus relationship curves are recommended for use in design when only the type of material to be stabilized and its specified or expected strength are known. Figure 49 shows these curves where the upper curve is recommended for use with lean

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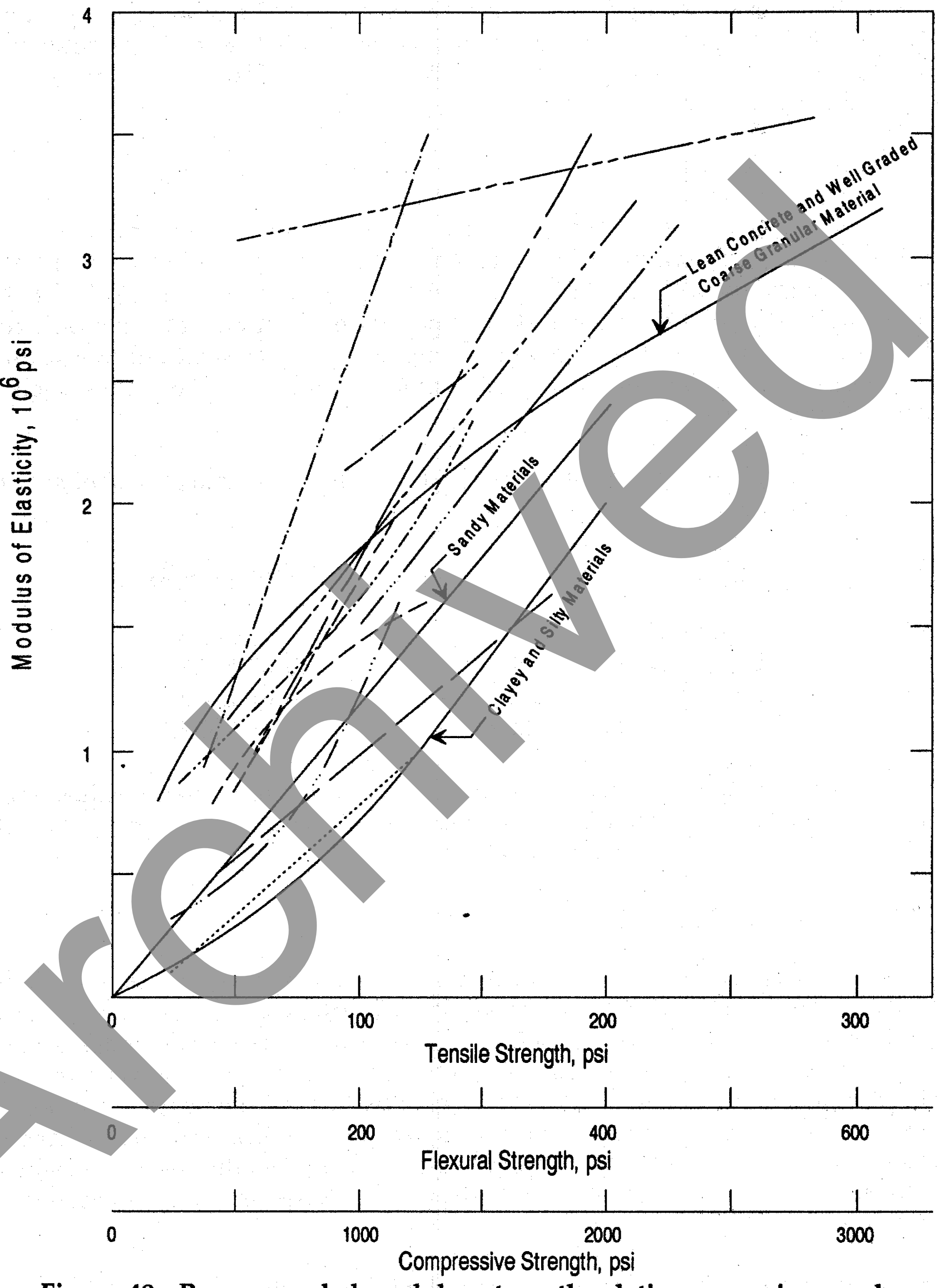

Figure 49. Recommended modulus-strength relations superimposed on reported relations.[69]

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concrete and high quality, well-graded, coarse-grained material and has the equation, $E = 57{,}000 \cdot (CS)$; the middle curve is for sandy material and has the equation, $E = 1{,}200 \cdot (CS)$; and the lower curve is for silty and clayey fine-grained material, and has the equation, $E = 440 \cdot (CS) + 0.28 \cdot (CS)^2$. Where E is the modulus of elasticity, and CS is the compressive strength.

Poisson's Ratio

Numerous investigators have determined Poisson's ratio for various cementitious stabilized materials. Felt and Abrams tested four soil-cement mixtures with varying cement contents.[63] They reported ratios from 0.22 to 0.36 for dynamic tests and 0.08 to 0.24 for static tests. Thompson determined Poisson's ratio for four lime-soil mixtures.[70] He found values ranging from 0.08 to 0.12 at low stress levels (less than 25 percent of ultimate) and from 0.27 to 0.37 at higher stress levels (50 to 75 percent of ultimate).

The following ranges of values for the various cementitious stabilized materials are recommended:

Material	**Poisson's Ratio**
Lime-Soil	0.15 to 0.20
Lime-Fly ash Materials	0.10 to 0.15
Cement Stab. Granular	0.10 to 0.20
Fine Grained Soil Cement	0.15 to 0.35

Based on these recommendations and an examination of the values reported in the literature, a Poisson's ratio of 0.15 has been selected as the appropriate value for use in general pavement design analyses.

Fatigue Characteristics

The fatigue characteristics of cementitious stabilized materials have been studied in terms of radius of curvature, strain levels, stress ratios, and strength reduction. The majority of investigators have used a stress ratio model (applied stress/tensile strength) when studying the fatigue behavior of the various cementitious-stabilized materials. Consequently, the fatigue behavior of cementitious stabilized materials appears at this time to be most comprehensively defined by the stress ratio model. This model is also the one most generally used with Portland cement concrete. A stress ratio model has been selected for use in mechanistic design procedure.

Stress ratio-fatigue relationships reported by various investigators are shown on figure 50. Included in the figure is the fatigue relationship recommended by the

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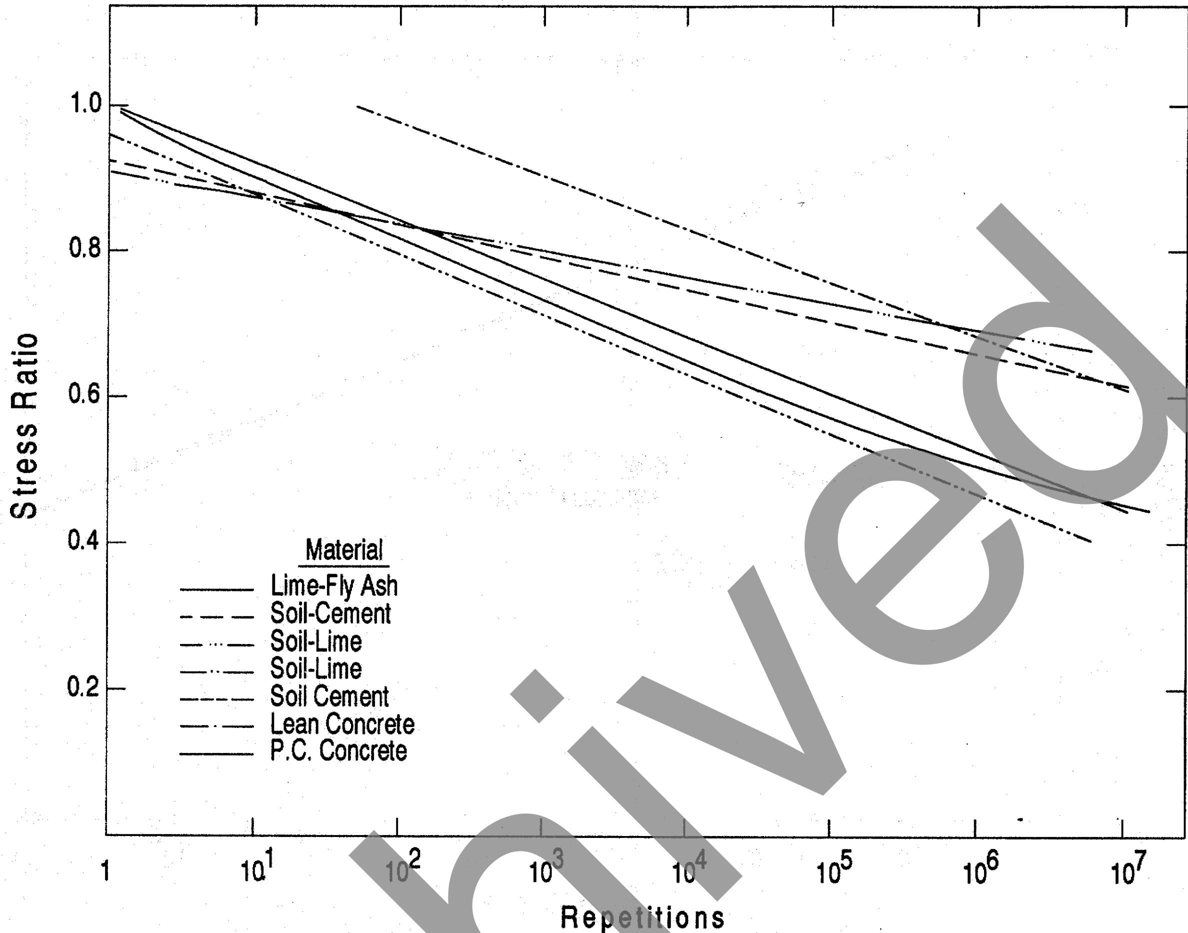

Figure 50. Reported stress ratio-fatigue relations.[58]

Portland Cement Association for use in the design of concrete pavements. This relationship closely approximates several of the relationships reported for the other cementitious materials.[58] Since this curve is also close to the lower boundary of the reported relationships, it has been selected for use in design.

The fatigue relationship recommended for use in design is shown in figure 51. This relationship can also be represented by the equation:

$$\text{Log } N = (0.9722 - S)/0.0825$$

where: N = Allowable number of load repetitions
S = Applied flexural stress/flexural strength ratio

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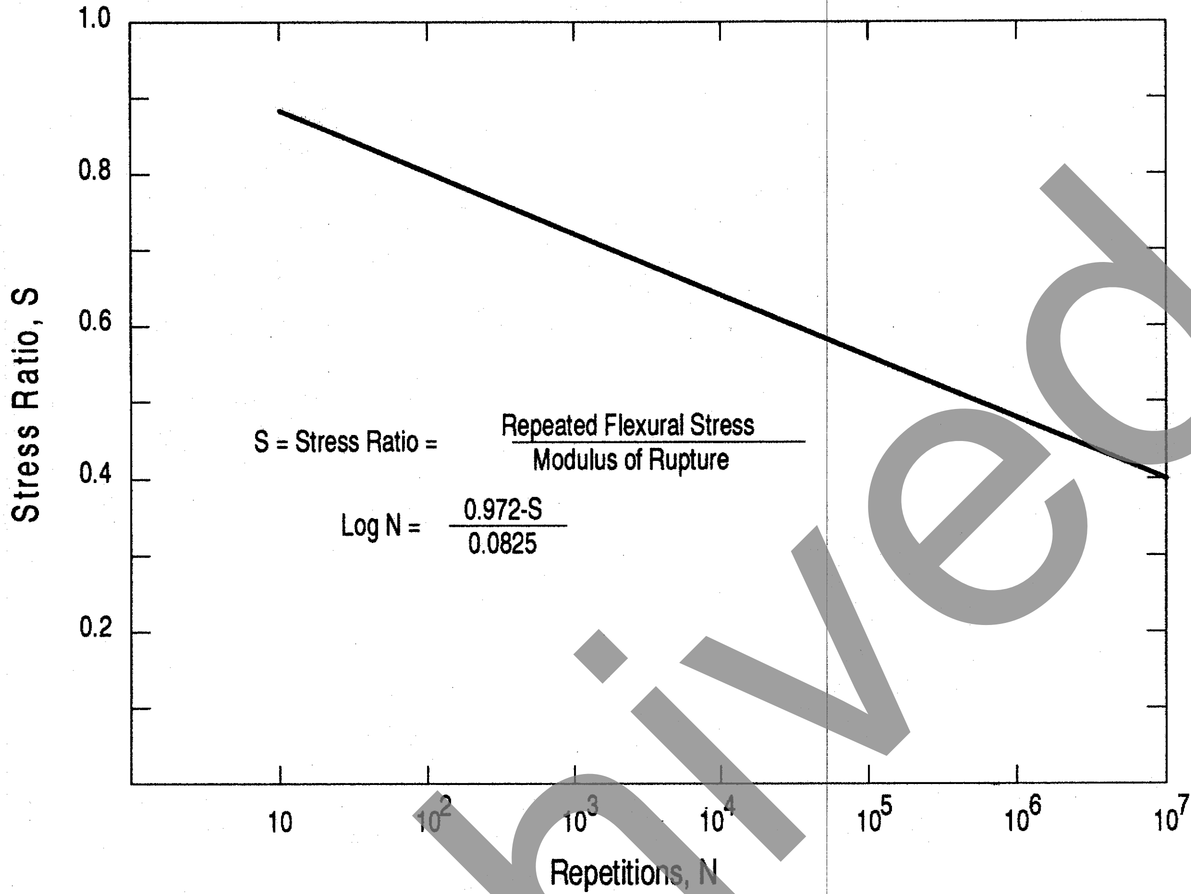

Figure 51. Recommended stress ratio-fatigue relations for cement stabilized materials.[58]

Structural Analysis

For high strength and modulus stabilized base materials, a "fatigue approach" is frequently used to relate stress ratio (S = radial tensile stress/flexural strength) to number of load applications to failure (initial cracking). A conservative S - log N plot is shown in figure 51.

It is generally accepted that the fatigue concept relates to crack initiation in the stabilized material and that additional load repetitions are required to propagate the crack to the surface of the layer.

Extensive structural modeling and pavement nondestructive testing activities have demonstrated that the ILLI-PAVE stress-dependent finite-element program is adequate for characterizing the structural response of flexible pavement systems containing stabilized layers.[70,71] Traditional linear elastic theory models tend to

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predict unrealistically large bending stresses and strains for the stabilized layer and are not recommended.

Design algorithms were developed based on statistical analyses of the ILLI-PAVE data. The major response for thickness design purposes is the flexural tensile stress in the stabilized layer. The algorithm for flexural stress (interior loading) is:

$$\mathbf{Log\ \sigma_r = 2.491 - 0.0698 \cdot T_{eq} + 0.000103 \cdot E_{STAB} - 0.0083 \cdot E_{Ri}}$$

$$R^2 = 0.946,\ SEE = 0.059$$

where:

where:	σ_r	=	Radial tensile stress at the bottom of the stabilized layer (interior loading), psi
	E_{STAB}	=	Resilient modulus of stabilized layer, ksi
	E_{AC}	=	Modulus of AC asphalt concrete, ksi
	T_{eq}	=	Equivalent thickness of pavement section, in

$$T_{eq} = T_{STAB} + T_{AC} \quad E_{AC}/D_{STAB}$$

$$\text{If } E_{AC}/D_{STAB} = 1/3;\ T_{eq} = T_{STAB} + 0.7\ T_{AC}$$

T_{STAB}	=	Stabilized base thickness, in
T_{AC}	=	AC surface layer thickness, in
E_{Ri}	=	Resilient modulus (ksi) of cohesive subgrade soil at a repeated deviator stress of approximately 6 psi.

Examination of the equation indicates that the radial tensile stress is primarily controlled by T_{eq}. Assuming typical values of E_{STAB} = 1500 ksi and E_{Ri} of 3 ksi, the equation is simplified to:

$$\mathbf{Log\ \sigma_r = 2.621 - 0.0698\ T_{eq}}$$

Load placement influences pavement structural response. Flexural stress in the stabilized base course is the controlling design criterion. For given conditions (material strength and modulus, subgrade support, base thickness, loading) flexural stresses are lowest for interior loading and are greater for corner and edge locations.

Stabilized base courses are not continuous slabs. Transverse shrinkage cracks and longitudinal construction joints break the continuity of the stabilized layer. Other than interior loading will thus occur, resulting in increased stabilized base course tensile stresses.

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Cementitious base materials typically develop a transverse shrinkage cracking pattern following construction. The intervals between the cracks and the crack width are related to stabilized base strength. Higher strength materials display long intervals between cracks and the crack widths are wider. Lower-strength materials have shorter intervals between cracks and the crack widths are less. Mitchell et. al. suggested typical values of 10 ft (3 m) crack spacing for cement-treated fine-grained soils and 20 ft (6 m) for cement-treated granular materials.[73]

Corner loading conditions develop at locations where longitudinal and transverse cracks intersect. For joint locations removed from the intersection points, edge-loading conditions prevail. Normal traffic pattern distribution will probably result primarily in interior and edge loading conditions.

Thickness Design Procedure

Pavements with high strength stabilized bases can be designed using an "intact slab" approach. The pavement initially may develop transverse shrinkage cracks, but an adequate stabilized layer design thickness prevents significant additional cracking, particularly longitudinal cracking under traffic loadings. A HSSB pavement layer is susceptible to fatigue failure. For a given magnitude of repeated flexural stress, the pavement can sustain a specific number of applications before HSSB cracking occurs. The number of loads applications increases as the magnitude of the HSSB flexural stress decreases.

The required inputs are traffic data, field strength and subgrade soil, E_{Ri}. The traffic data must be reduced to 18-kip single axle loads.

The field compressive strength is required and should be based on considerations of curing conditions (time and temperature), laboratory versus field conditions (density, mixing efficiency, etc.), construction variability, and cyclic freeze-thaw strength loss. The field strength is considered "the critical factor" in the thickness design procedure and can be used to predict the modulus of elasticity and flexural strength which is approximately 20 percent of the compressive strength.

To estimate HSSB strength at a given time, it is necessary to establish the strength-degree day (DD) relation for the mixture and quantify the expected field curing conditions. To quantify field curing conditions, the available degree days (40° base is recommended) in the pavement location is also required. Some typical strength-DD relations are shown in figure 52. The relations are quite variable and must be characterized for HSSB thickness design calculations. Some mixtures develop strength at a moderate rate over a sustained period of time while others show rapid initial strength increase and then achieve only moderate additional strength increases.

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Subgrade resilient modulus does not have a pronounced effect on the stabilized layer flexural strength. In a "practice oriented" design procedure it may be possible to "estimate" E_{Ri} with sufficient accuracy. There are major factors influencing the resilient modulus, for example, for fine-grained soils are texture, plasticity, and moisture content. Freeze-thaw reduces the resilient modulus drastically. Silty and lower PI soils are more moisture-susceptible and higher clay contents and increased PI type soils suffer a large resilient modulus loss with freeze-thaw action. Subgrade resilient modulus can be established from lab testing, local experience, NDT testing, or estimated from soil classification data.

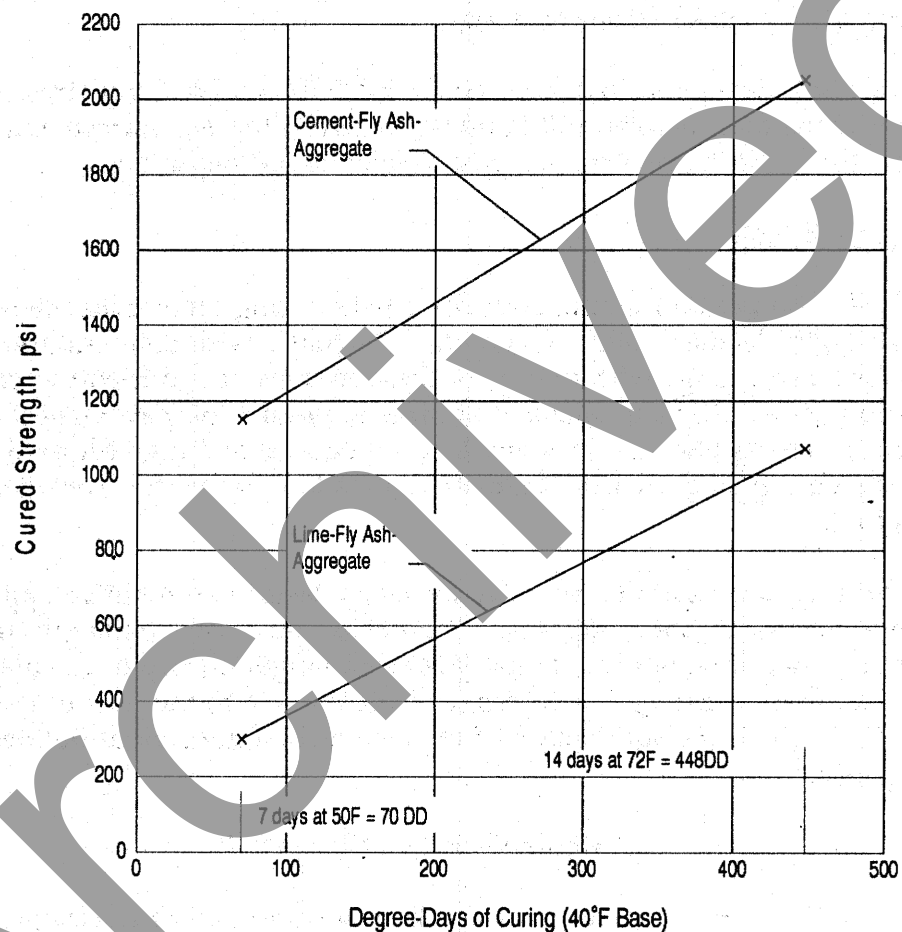

Figure 52. Strength-degree-day relations for pozzolanic stabilized base materials.[73]

Flexural stress for an interior 9 kip wheel load is estimated from the following simplified algorithm (Ref):

$$\text{Log } \sigma = 2.515 + 0.0001\ S - 0.07\ T_{EQ}$$

where: σ = flexural strength, psi
S = compressive strength, psi.

Considering the precision with which HSSB field strength-DD relations can be estimated and the general variability of traffic loading conditions, subgrade E_{Ri}, etc, the equation shown above is acceptable for routine design activities. The interior flexural stress is increased by 50 percent to account for edge loading and HSSB transverse cracking effects. The design flexural stress ($\sigma_F = 1.5\ \sigma$), is used in calculating SR for HSSB thickness design.

The algorithm presented for predicting HSSB layer flexural stress was based on the assumptions that there is "full bonding" between the AC surface and the HSB layer, and the HSSB layer is an intact-homogenous full depth layer.

Curing Time Effects

HSSB strength and modulus increase with curing time; freeze-thaw action may effect a strength decrease; and AC modulus fluctuates with temperature. Thus, the load stresses also change with time. For these reasons, it is difficult to calculate a stress ratio (SR) a particular time and accurately predict the pavement life for several years hence. The application of many load repetitions at a high SR (which may occur in the early curing stages when HSSB strength is low) will effect considerable fatigue consumption.

The time-dependent HSSB behavior lends itself to an iterative approach. For a small time increment, the changes in critical HSSB properties (strength/modulus) during that small time increment, apply Miner's fatigue theory to calculate the "incremental fatigue damage" incurred, and them move to the next time step. Total "Fatigue Damage" is the summation of the incremental damage as defined by the following equation:

$$\textbf{Fatigue Damage (\%)} = \sum_{i=1}^{n} P_i$$

where: P_i = percent fatigue life consumption for the its period
N_i = number of 18-kip ESAL applied during its period
N_{Ti} = number of load applications to failure estimated from figure 51
n = time periods considered

Crack initiation is expected at 100 percent fatigue consumption. Additional load repetitions are required to propagate the crack through the thickness of the HSSB layer.[71,72]

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Design Reliability Considerations

Design reliability for HSSB pavements are very limited. Analysis of recent TRRL research indicates the traffic for "50 percent probability of survival" is approximately 3.5 to 4 times the traffic for "85 percent of survival."(72) The traffic multiplier approach is a number that multiplies the Traffic Factor. There are "multiplier" factors developed for full-depth asphalt concrete pavements and conventional flexible pavements. These multipliers are shown below:

Multiplier	Design Reliability
1	50 (average)
2	80 (intermediate)
3	92 (high)

If the traffic is 1 x 10^6 18-kips ESAL, a pavement designed for 2 x 10^6 ESAL would have an 80 percent probability of sustaining 1 x 10^6 without failure. To achieve 92 percent reliability, the pavement would be designed for 3 x 10^6 ESAL. The traffic Multiplier approach is also utilized in the 1986 AASHTO Guide for Pavement Structures.

Design Criterion Development

The proposed thickness design procedure is predicted on the fatigue failure of an "intact" HSSB layer with a nominal AC surface course (maximum of approximately 4 in (101.6 mm). In this type of pavement structure, the AC radial strains are compressive and subgrade stresses are low. Thus, AC fatigue and subgrade rutting are not significant design criteria. The only thickness design criterion is fatigue consumption in the HSSB layer for considering longitudinal crack formation.

The fatigue relation, shown in figure 51, and Miner's approach to considering cumulative fatigue damage were previously presented. To limit early life fatigue consumption, HSSB thickness/strength must be adequate to effect a stress ratio (SR) less than 0.65 or 0.60 prior to traffic loading. If the section is "overloaded" or "fatigued" or at an early age, the "intact slab" type structural behavior of the HSSB layer is significantly reduced.

Once cracking starts (other than initial transverse shrinkage cracking), the effective modulus decreases. As cracking progresses to the small blocks and then the granular state, further large effective modulus decreases are noted. Initial (precracked state) moduli range from approximately 2000 ksi to 500 ksi. At the small block/granular state the effective modulus is reduced to the 70 ksi- 20 ksi range.

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HSSB fatigue consumption is calculated for the early life (first 56 days/8 weeks of traffic loading) of the pavement and also checked for total load (design life) applications. Since the fatigue life consumption criterion is based on "crack initiation", it is a conservative approach. Additional load repetitions would be required to achieve "crack propagation" through the HSSB layer.

6 SUMMARY

The AASHTO 1986 design procedure and several mechanistic design procedures have been presented in this chapter. Extensive documentation of material properties required for the design of stabilized layered pavements have been presented which provide the necessary design inputs for the thickness design procedures. While this is not an exhaustive treatment of pavement design, the fundamental relationships for successful utilization of stabilized materials in the thickness design process have been presented.

The new developments of mechanistic design, and the expansion of the AASHTO procedure to include some mechanistic inputs provides the necessary push for the development of improved material property relationships in the future as the design procedures become more sophisticated, requiring more sophisticated material properties. The data presented in this chapter provide an indication of where this development will be directed in the future.

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