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Soil and Base Stabilization and Associated Drainage Considerations

Volume II, Mixture Design Considerations

Office of Technology Applications

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Innovation Through Partnerships

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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 laboratory testing for the different stabilizer additives. Typical properties are given and the test procedures to select optimum amounts are presented. These chapters provide the engineer with detailed information required from the laboratory to ensure the necessary material properties are obtained in the paving project. Volume I presents the details of drainage and pavement design and construction considerations for stabilization of pavement materials.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
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

Symbol	When You Know	Multiply By	To Find	Symbol
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

Symbol	When You Know	Multiply By	To Find	Symbol
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.

MASS

Symbol	When You Know	Multiply By	To Find	Symbol
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

APPROXIMATE CONVERSIONS FROM SI UNITS

LENGTH

Symbol	When You Know	Multiply By	To Find	Symbol
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

Symbol	When You Know	Multiply By	To Find	Symbol
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

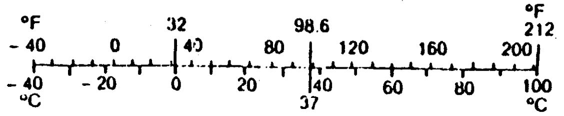
Symbol	When You Know	Multiply By	To Find	Symbol
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3

MASS

Symbol	When You Know	Multiply By	To Find	Symbol
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T

TEMPERATURE (exact)

Symbol	When You Know	Multiply By	To Find	Symbol
°C	Celsius temperature	1.8C + 32	Fahrenheit temperature	°F



* SI is the symbol for the International System of Measurement

(Revised April 1989)

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

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

This two-volume user's manual provides guidance to pavement design, construction, and materials engineers responsible for soil stabilization operations related to the transportation field. Volume I relates primarily to the design and construction of stabilized pavements. It serves as a guide for selecting an appropriate stabilizer on a project and provides important information with regard to drainage, thickness design, and construction procedures, and thickness design.

Volume II, presented here, contains information necessary in determining the type and amount of chemical stabilizer to be used on a project. An in-depth discussion of the tests used to characterize stabilized materials is presented, as well as how testing is utilized in pavement design processes.

Revisions to the original user's manuals were made in several ways. An extensive review of literature published since 1979 was conducted which provided the up-to-date information discussed previously. In addition, the authors' knowledge and experience, coupled with correspondence and project sites visits, have provided a background of pertinent information into which pavement design and construction engineers can tap. Additional input was provided by an array of experts in the soil stabilization field.

Every attempt has been made to present information that is technically correct and that can be applied with reasonable confidence. Both conventional and state-of-the-art construction and testing technologies are presented and appropriately referenced. However, it will be necessary for the engineer to 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

Stabilization of subgrade soils and aggregates by chemical or mechanical means is quite common. In chemical stabilization, chemical additives are incorporated into a soil and serve as a modifying or cementing agent. Mechanical stabilization is achieved by mixing or blending soils of two or more gradations to obtain a material meeting a specific requirement.(1)

The decision as to the appropriate method of stabilization (mechanical or chemical) is one of economy and requires an appreciation of the engineering advantages and disadvantages of each stabilization type. Although chemical stabilization is the primary focus of this manual, certain comparisons can be made between mechanical and chemical stabilization using this manual.

Volume II concentrates on the selection and usage of the various chemical stabilizers and provides the engineer with information to perform the following:

- Select the type or the types of stabilizer suitable for a specific soil.
- Determine individual stabilizer (i.e., lime, cement) or combination stabilizer (i.e., lime-cement, lime-asphalt) contents for particular soils based on results from laboratory tests, typical property values, and an understanding of the physical-chemical reactions of each additive.
- Identify stabilized material requirements needed to ensure adequate performance in a pavement system.
- Develop pavement design alternatives which contain both unstabilized and stabilized layers.
- Compare life cycle costs and energy requirements of alternate pavement designs using stabilized materials and drainage installations.

The chemical stabilizers to be discussed in detail pertain to soil and base stabilization and include:

- Lime
- Lime-fly ash
- Portland cement

- Asphalt
- Combinations and other additives

The use of other chemical stabilizers, such as calcium chloride and sodium chloride, are included in the sections on Combinations and Others.

3. BACKGROUND

The problem of improving unsuitable soils has continually forced engineers to evaluate new procedures and techniques to accomplish this improvement. The concept of soil modification through stabilization with additives has been around for at least 5000 years, when soil was stabilized with lime or pozzolans for the same economic reasons soils are stabilized today. This unique contribution to roadway construction is as beneficial today as it was then. Soil stabilization is a tool for economic road-building, conservation of materials, investment protection, and roadway upgrading.(2) 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.

A major concern in recent years has been localized shortages of conventional high quality aggregates. The highway construction industry consumes over half of the annual production of aggregates.(1) 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. As in all engineering problems, the economics must be considered in light of the benefits derived from the stabilization process to determine if stabilization is warranted.

The cost-effectiveness of each stabilizer additive is generally the final criteria in selecting one or more of the additives for use. However, this criteria may be overruled by the availability of a particular stabilizer. No matter how beneficial or cost-effective a stabilizer is, if it is not available, it will not be used.

The combinations of regulations which prohibit mining and production of aggregates, and land use patterns that make aggregate deposit inaccessible, combine 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 may be reduced as well as costs.

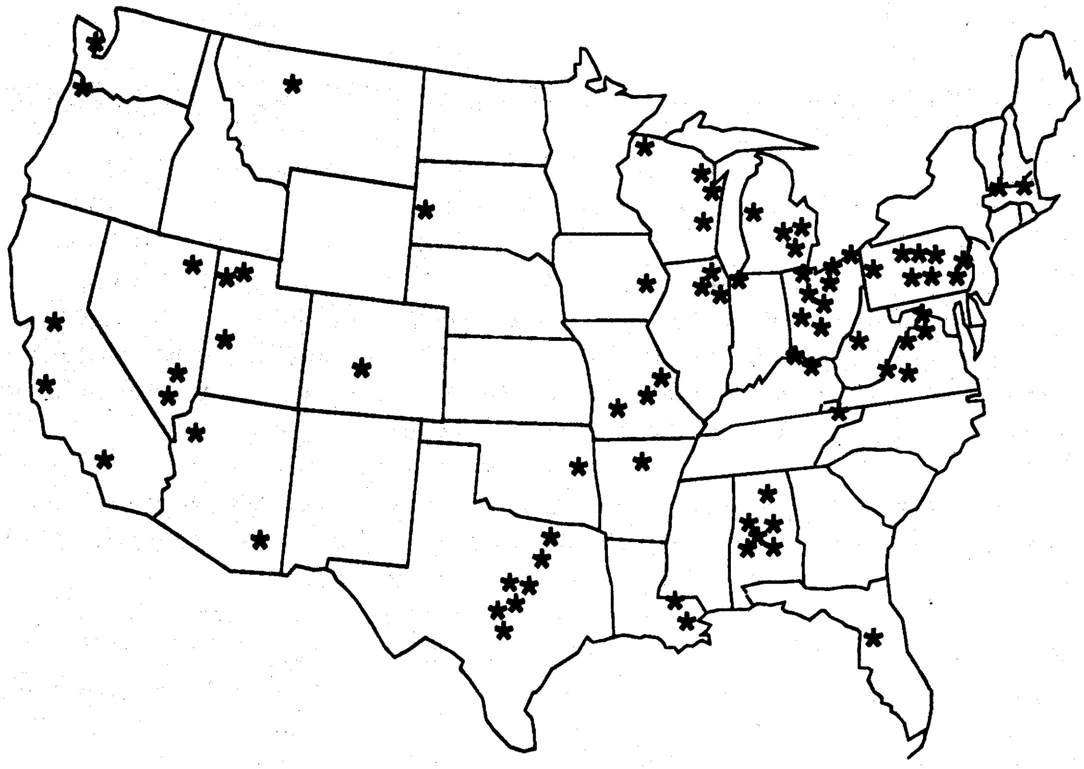

Figure 1. Commercial lime plants in the United States, 1990.[(3)]

Analysis of stabilizer production is somewhat difficult since figures are not generally available in detail. Although a given highway agency will normally be aware of nearby sources, several maps showing general distribution are somewhat illustrative of the wide availability. Figure 1 shows the location of commercial lime plants in 1990.[(3)] Fly ash, as produced from the burning of coal, is not produced in quantities shown in figure 2, but the actual locations are noted.[(4)] Figures from 1984 indicated that approximately 51.3 million tons of ash were produced, of which 10.4 million tons were reclaimed.[(5)] With more recent changes to coal as a fuel, these figures are likely to change over the next few years.

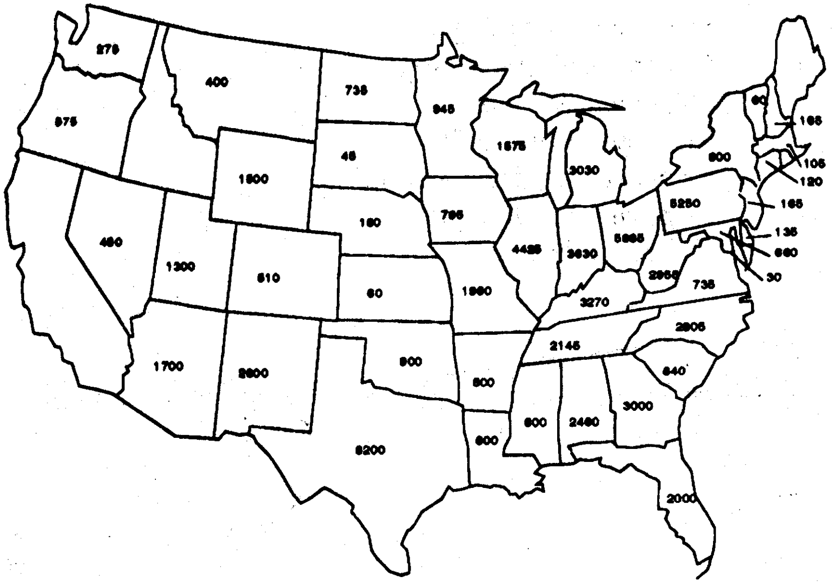

Figure 2. Approximate ash production (in 1,000's of tons) by major electric utilities.[4]

Portland cement plant sites, as of 1989, are shown in figure 3.[6] Asphalt production by location is not as important as the location of terminal distribution points, which are distributed uniformly nationwide, and no map is provided. During recent years, as sources of crude oil have changed, refineries have changed their product and production accordingly. A given agency will be able to readily determine the availability of desired products by contacting local suppliers.

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 here are several reasons and advantages for using stabilization:

- Provide a temporary or permanent wearing surface for low volume roads.
- Provide a stable working platform for construction activities.
- Improve poor subgrade conditions.
- Upgrade marginal base materials.
- Provide dust control.
- Water-proof the soil.
- Salvage old roads with marginal materials.
- Construct superior bases.

- Improve strength, reducing thickness requirements
- Improve durability.
- Control volume change of soils.
- Dry back wet soils.
- Improve workability.
- Conserve aggregate materials.
- Reduce overall costs.
- Conserve energy.

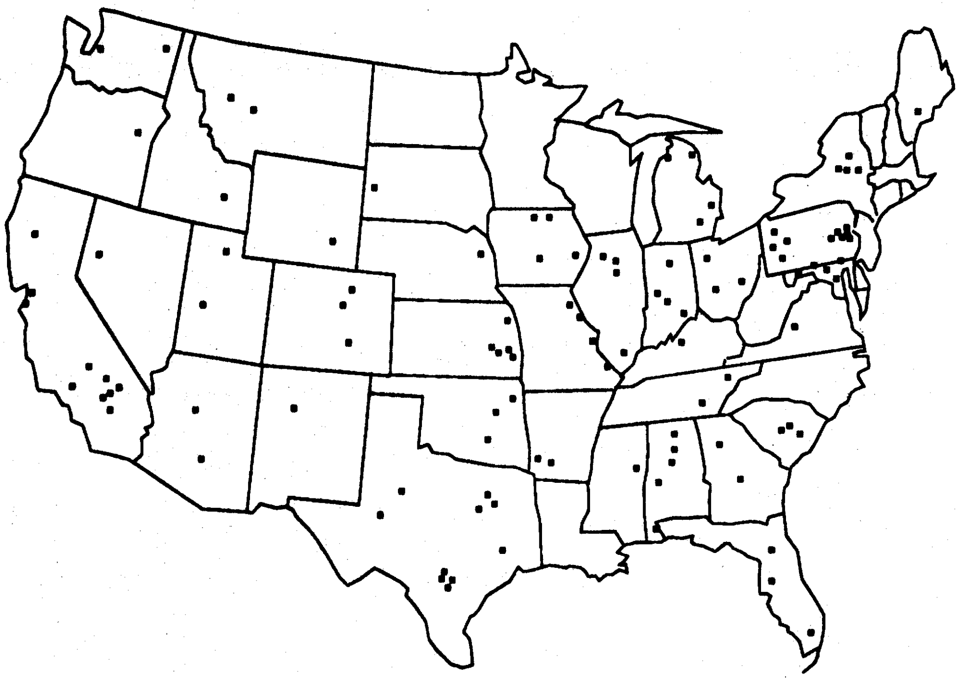

Figure 3. Portland cement plant sites, 1990.(6)

4. DEFINITIONS

Discussion of soil and aggregate stabilization requires the use of terminology which may not be familiar or needs to be defined for clarification. Brief definitions are provided for the following terms which will appear intermittently throughout the user manuals.

General Definitions

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).(4)

Soil Stabilization

Chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise to 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.

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.

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.

ASTM

An abbreviation used to designate 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.(7)

Resilient Modulus Test

The test as adopted by AASHTO (AASHTO T274-82) 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.(7)

Layer Coefficient (a_1, a_2, a_3)

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.(7)

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 as indicated by the time to drain water out of the pavement, and the percent of time during a year the pavement is exposed to moisture levels approaching saturation.(7)

Pavement Serviceability

An evaluation of how well the pavement is fulfilling the design function established 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 feet per day should always be provided if positive drainage is to be achieved.

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

An abbreviation used to designate a mixture of lime and fly ash with aggregate.

LCFA

An abbreviation used to designate a mixture of lime, cement, and fly ash with aggregate.

LFS

An abbreviation used to designate a mixture of lime and fly ash with soil.

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 interground 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 pulverize 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 mortar-like consistency at the time of mixing and placing. Plastic soil-cement is primarily used in highway ditch linings. 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 types of bitumen.

Asphalt

A dark brown to black cementitious material in which the predominating constituents are bitumens which occur in nature or are obtained in petroleum processing.(8)

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 and kerosene.

Emulsified asphalt

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

CHAPTER 2 SELECTION OF STABILIZER

1. INTRODUCTION

This chapter presents criteria that can be used as a guide in selecting the proper type of stabilizer for a given soil. Since no one stabilizer works best with all soils, the stabilization objectives must be defined and all of the factors which might influence the stabilization process should be carefully considered.

There are several reasons to consider the stabilization of soils and base materials. Subgrade soils, for instance, are typically stabilized for one or more of the following reasons:

- To provide adequate strength to support construction equipment.
- To improve workability.
- To reduce frost heave and volume change characteristics.
- To improve performance and increase long term structural strength.

Any of these factors will increase a road's durability and improve performance under traffic. The result is that the life-cycle costs of a pavement can be substantially reduced through effective stabilization.

While there are many benefits of 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 factors such as drainage, durability, and strength.

Identifying a proper application for stabilization as well as selecting the appropriate stabilizer are often done without the benefit of adequate field and laboratory testing. The exact characteristics of the materials being used must be known 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.

Individual stabilizer additives do not react equally well with different soil types. 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.

2. STABILIZATION OBJECTIVES

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.

Some of the primary objectives of stabilization include:

- Improve poor subgrade conditions.
- Provide dust control.
- Provide moisture control.
- Upgrade marginal base materials.
- Improve workability.
- Improve strength and durability.

Each of these objectives is 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. For example, a contractor's sole desire may be to expedite construction by using lime on a project to dry and stabilize wet, soft clays. If this objective is satisfied, then the contractor has been successful. Any additional improvements incurred in the process, such as increased strength and durability, are certainly beneficial.

3. TYPES OF STABILIZATION ACTIVITY

There are three primary modes of stabilization associated with the chemicals considered in this manual: cementing, modifying, and waterproofing. All four principal stabilizers (portland cement, lime, lime-fly ash, and asphalt) exhibit some form of cementing action, given adequate stabilizer contents. Portland cement, lime, and lime-fly ash generally react to form cement-like materials. This reaction is referred to as a pozzolanic reaction. Once the mixture has set up, a hardened matrix of cement and soil particles results.

The cementing action experienced by asphalt is considerably different. No chemical reactions take place; the asphalt material coats the soil particles and binds them together. The result, once the asphalt cures, is a semi-hardened mixture of binder and soil particles.

Occasionally, the use of considerable amounts of stabilizer to provide cementing action is restricted due to costs. In this case, smaller quantities of stabilizer can be added to a soil in order to modify it. Although cement, lime, and asphalt can all serve in this capacity, lime is particular effective at reducing the plasticity and changing the texture of clay soils.

Asphalt stabilization provides waterproofing by coating the soil or aggregate particles with asphalt, a barrier is created which retards the absorption of moisture by the particles. Lime, cement, and lime fly-ash provide a degree of moisture resistance by reducing capillary action which reduces the amount of water that can move upward through the stabilized soil into the pavement structure.

The process by which an additive accomplishes the stabilization activity depends on the nature of the additive and how it interacts with the soil being stabilized. Soil-Stabilizer interactions are classified as follows:

- Active - Stabilizer produces a chemical reaction with the soil or aggregate (lime).
- Passive (Inert) - Stabilizer produces no chemical reaction; only physical actions are effected (asphalt).
- Intermediate - Stabilizer produces chemical reaction within itself and forms a physical bond with the soil or aggregate (cement, lime fly-ash).

4. STABILIZER SELECTION

This section presents the selection process to be followed in determining the most economical additive for a pavement stabilization project. While each additive has an ability 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 No. 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 guides have been developed to assist the engineer in the stabilization 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.[9] This process is illustrated in figure 4.

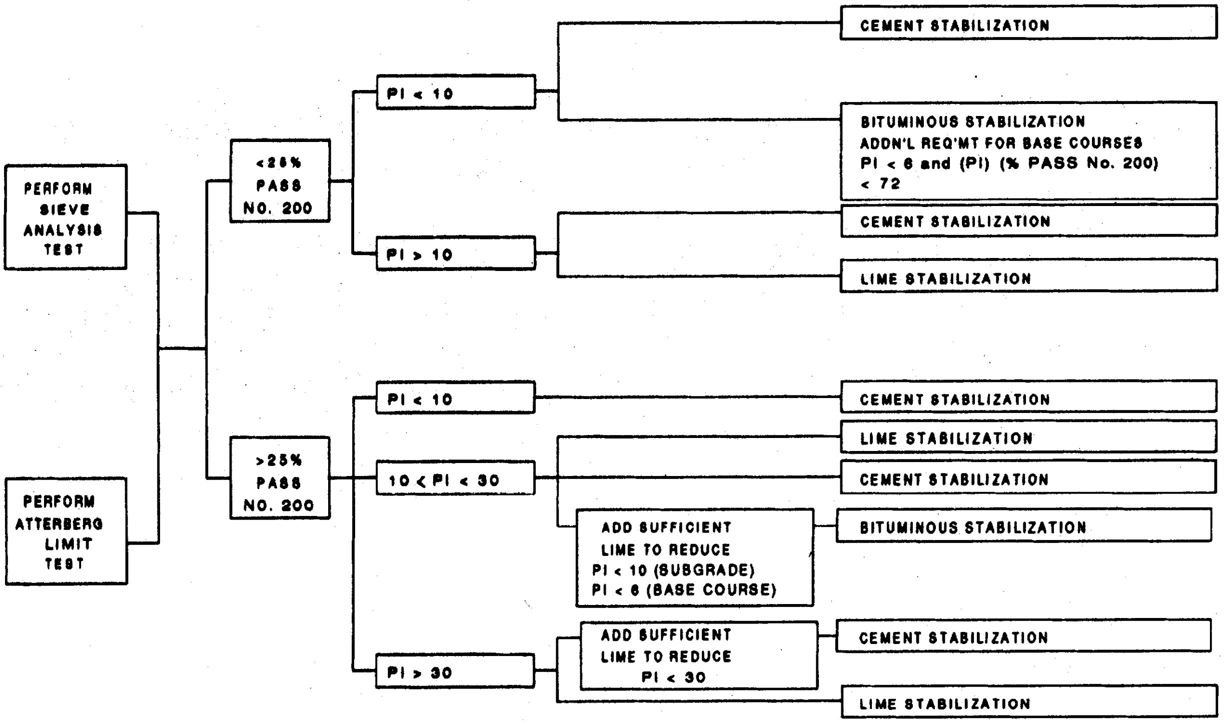

Figure 4. The Soil Stabilization Index System (SSIS) selection procedure.[9]

More detailed guides published by individual agencies such as the Air Force, illustrated in table 1, suggest stabilization methods for particular soil types based on their location in the pavement structure and the purpose or function of their use (i.e.,

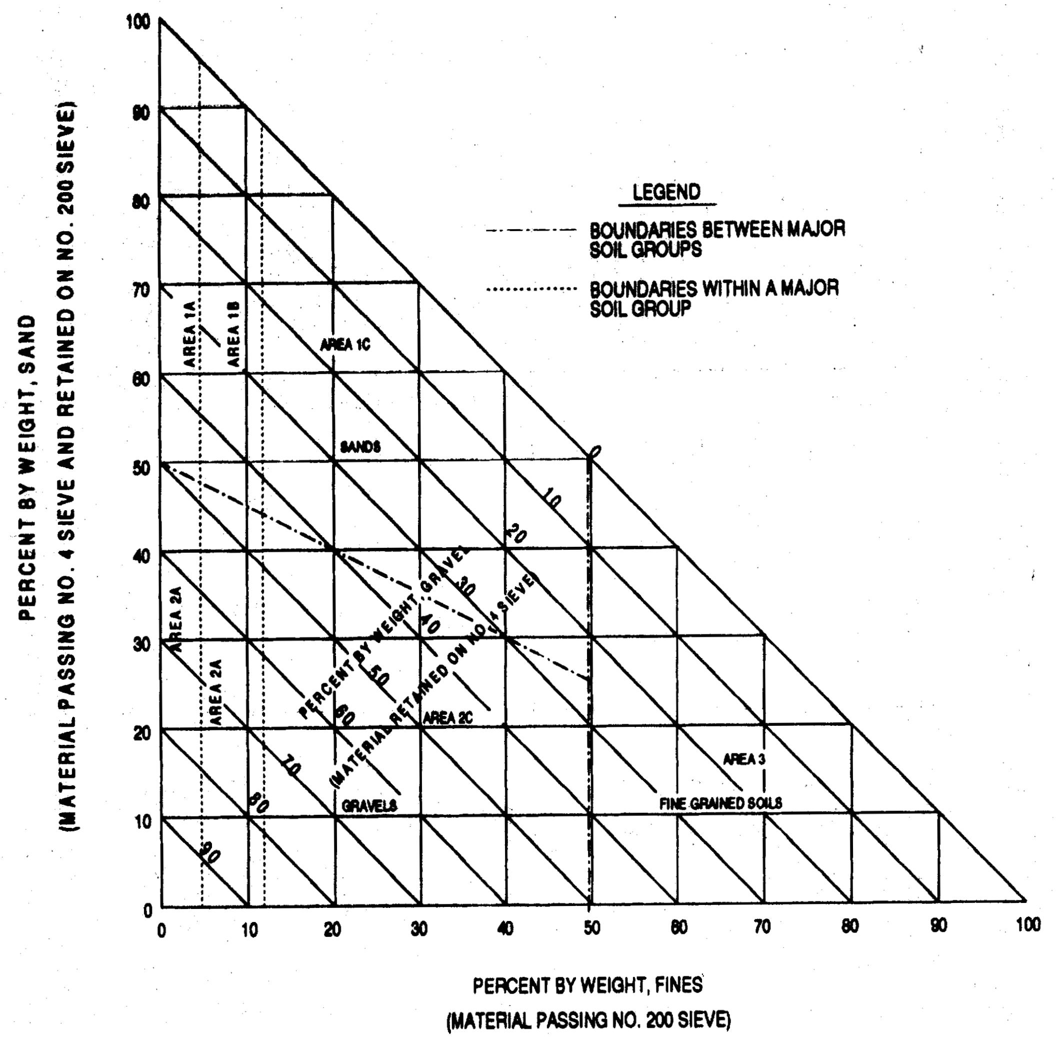

Figure 5. Gradation triangle for aid in selecting a commercial stabilizing agent.[9]

load carrying characteristics, waterproofing, etc.).[10] The gradation triangle shown in figure 5 is used by the Army. The Air Force utilizes the following soil properties to determine the proper type of stabilizer:[10]

- Percent material retained on No. 4 sieve.
- Percent material passing No. 200 sieve.
- Percent material passing No. 4 sieve and retained on No. 200 sieve.
- Atterberg Limits.
    - Liquid Limit (LL), and Plasticity Index (PI).

Table 1. Guide for selecting a stabilizing additive.(25)

Area	Soil Classification	Type of Stabilizing Additive Recommended	Restriction on LL and PI of soil	Restriction on Percent Passing No. 200 Sieve(a)	Remarks
1A	SW, SP	(1) Bituminous			
		(2) Portland Cement			
		(3) Lime-Cement-Fly Ash	PI not to exceed 25		
1B	SW-SM, SP-SM, SW-SC, SP-SC	(1) Bituminous	PI not to exceed 10		
		(2) Portland Cement	PI not to exceed 30		
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
1C	SM, SC, SM-SC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	
		(2) Portland Cement	(b)		
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
2A	GW, GP	(1) Bituminous			Well-graded material only. Material should contain at least 45% by weight of material passing No. 4 sieve.
		(2) Portland Cement			
		(3) Lime-Cement-Fly Ash	PI not to exceed 25		
2B	GW-GM, GP-GM, GW-GC, GP-GC	(1) Bituminous	PI not to exceed 10		Well-graded material only. Material should contain at least 45% by weight of material passing No. 4 sieve.
		(2) Portland Cement	PI not to exceed 30		
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
2C	GM, GC, GM-GC	(1) Bituminous	PI not to exceed 10	Not to exceed 30% by weight	Well-graded material only. Material should contain at least 45% by weight of material passing No. 4 sieve.
		(2) Portland Cement	(b)		
		(3) Lime	PI not less than 12		
		(4) Lime-Cement-Fly Ash	PI not to exceed 25		
3	CH, CL, MH, ML, OH, OL, ML-CL	(1) Portland Cement	LL less than 40 and PI less than 20		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means.
		(2) Lime	PI not less than 12		

(a) Soil classification corresponds to MIL-STD-619B. Restriction on liquid limit, plastic limit, and plasticity index is in accordance with Method 103 in MIL-STD-621A.

(b) PI < 20 + [(50 - percent passing No. 200 sieve)/4].

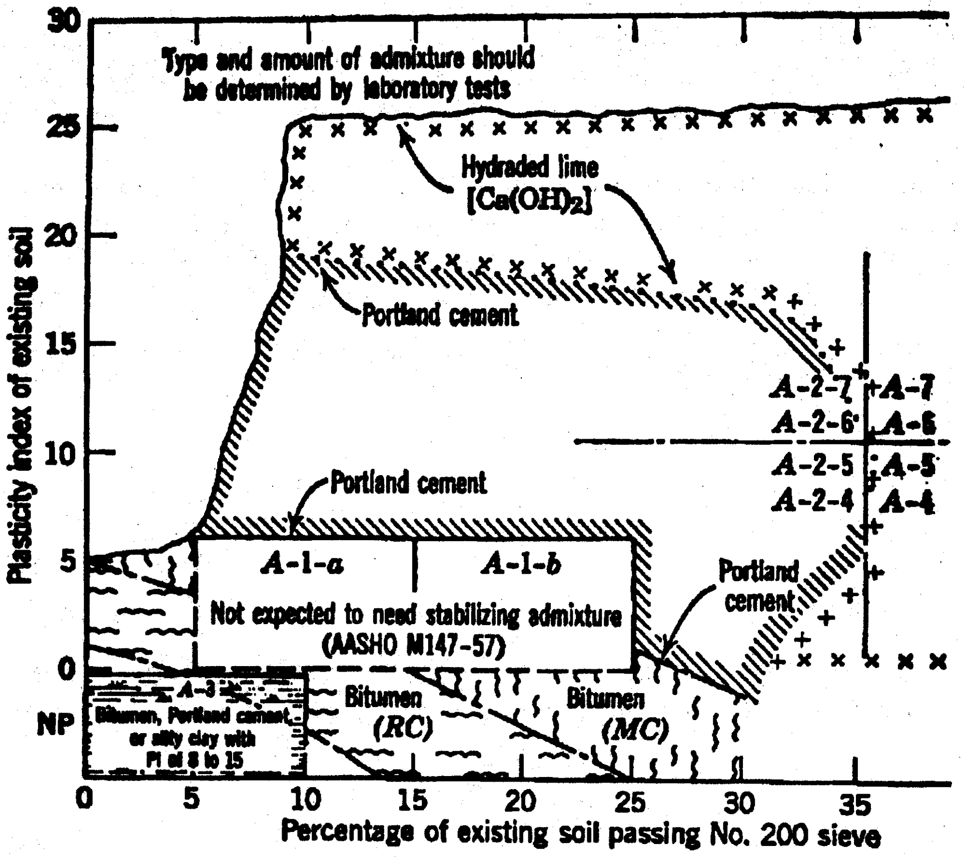

Figure 6. Suggested stabilizing admixtures suitable for use with soils.[11]

Figure 6 illustrates the stabilizer selection method presented in the early 1960's by Oglesby and Hewes.[11] This method is a modification of the original work conducted by the Division of Physical Research, Bureau of Public Roads. It utilizes the Plasticity Index and percent passing the No. 200 sieve, together with the AASHO Soil Classification System, to determine the appropriate stabilizer. While superseded by more modern systems, it indicates the important physical properties known to affect stabilization performance.

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.

Criteria for 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 and/or more than 25 percent passing the No. 200 sieve.

Experience has shown that lime will react with medium, moderately fine, and fine-grained soils to decrease plasticity, increase workability, reduce swell, and increased strength.(12) Soils classified according to the Unified System as CH, CL, MH, ML, SC, SM, GC, GM, SW-SC, SP-SC, SM-SC, GW-GC, GP-GC, or GM-GC should be considered as capable of being stabilized with lime. Soils classified by 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.(9) Experience has 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.(12,13)

Among the various stabilizers, lime is most capable of producing the largest changes in soil properties. The precise effects produced by lime when mixed with appropriate soils is discussed in chapter 4.

Criteria for 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 produce a floating aggregate matrix have given the best results. Suggested soil gradings to meet this floating aggregate matrix concept should fall within the band specified in table 2.(17) 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 suitable for cement stabilization as shown in figure 5.(9) The PI should be less than 30 for the sandy materials while the PI should be less than 20 and the liquid limit less than 40 for the 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, for soil cement, should not exceed the number indicated from the following equation:

$$PI \leq 20 + [(50 - \text{Fines Content})/4]$$

The amount of cement additive required for a particular soil depends upon whether the soil is just being modified or if full strength stabilization is desired. For example, if the intent is merely to reduce the PI of the soil, then small percentages (3 percent or less) of cement can be incorporated. Larger percentages 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 effect various cement contents will be discussed in detail in Chapter 5.

Table 2. Grading limits for cement stabilization of well-graded granular materials.[17]

Sieve Size	Percent Passing
Passing No. 4	Minimum of 55
Passing No. 10	Minimum of 35
Passing No. 10, Retained No. 200	Minimum of 25

Criteria for Asphalt Stabilization

Asphalt stabilization is a passive form of stabilization that is accomplished by using any of three asphalt products:

- Asphalt cement.
- Asphalt cutback.
- Asphalt emulsion.

Each of these materials are discussed in depth in chapter 6.

As mentioned previously, the concepts of waterproofing and cementation characterize the use of asphalt in soil and base stabilization. With thorough mixing and sufficient compaction, fine-grained soil particles can be successfully waterproofed using 2 to 3 percent asphalt binder. By doing so, the stabilized soil retains a somewhat uniform and low moisture content, thereby negating many of the adverse effects brought about by water.

If stability achieved through cementation is the main criteria, then greater percentages of asphalt binder (5 to 7 percent) can be incorporated. In this case, granular materials are coated and bound together by the asphalt. Stability is achieved by the frictional resistance of the aggregate particles and the cohesion supplied by the binder.

Asphalt stabilization works best on granular soils with low PI fines. This includes many well-graded gravels classified as GW, GM, and GC, and sands in the SW, SP, SM, and SC groupings. In these soils, the asphalt provides cohesion to the mass and waterproofs any clay constituents that may be present. All three types of asphalt can be used for stabilizing granular soils; however, restrictions on the use of asphalt cutbacks do exist in places due to environmental problems.

Depending on the plasticity characteristics and the amount of material passing the No. 200 sieve, fine-grained soils may be stabilized with asphalt. In most instances, however, the extremely large surface areas of the fine particles results in inadequate coating of the particles with asphalt. The result of this is strength loss due to water infiltration.

Some of the earliest criteria for asphalt stabilization were developed by the Highway Research Board Committee on Soil-Bituminous Roads.(18) These criteria were revised and published by Winterkorn and are illustrated in table 3.(19) The latest recommendations for gradation and plasticity characteristics given by the American Road and Transportation Builders Association (ARTBA) are provided in table 4. The Asphalt Institute (TAI) recommends that soils to be treated with asphalt possess the following properties:

- Less than 25 percent passing the No. 200 sieve.
- Sand equivalent less than 25.
- Plasticity Index less than 6.

The sand equivalent, as seen in the ARTBA and TAI recommendations, is an indication of the presence of clay-type fines. Typically, soils having a sand equivalent value greater than 35 can be successfully stabilized with asphalt. Soils with sand equivalents of 20 to 30 may be suitable for asphalt stabilization, provided that the soil particles are effectively waterproofed. Stabilization of materials such as clay-gravels with sand equivalents less than 20 is generally not successful.

Several other investigators have proposed suitable materials for asphalt stabilization. Table 5 presents the suitability of various soils having various percentages of minus No. 200 material and different liquid limit and PI ranges.(20) The requirements given in table 6 for emulsion stabilization by The Asphalt Institute's Pacific Coast Division suggest that the percent minus No. 200 should be in a range of 3 to 15, the PI should be less than 6, and the product of the PI and percent minus No. 200 should not exceed 60.(21) Table 7 illustrates guidelines set forth by Dunning and Turner for emulsion stabilization.(22)

Air Force recommendations are shown in table 8.(23) Although the manual specifically recommends gradations 6, 7, 8, and 9 for soil stabilization, it is believed that all gradations are practical, provided they are economically feasible.

Table 3. Types of soil-bitumen and characteristics of soils empirically found suitable for their manufacture.(19)

Sieve Analysis	Soil Bitumen,[a] %	Sand Bitumen, %	Waterproofed Granular Stabilization, %		
Passing:			A	B	C
1.5 in	-	-	100		
1 in	[b]	-	80 - 100	100	
0.75 in	-	-	65 - 85	80 - 100	100
No. 4	> 50	100	40 - 65	50 - 75	80 - 100
No. 10	-	-	25 - 50	40 - 60	60 - 80
No. 40	35 - 100	-	15 - 30	20 - 35	30 - 50
No. 100	-	-	10 - 20	13 - 23	20 - 35
No. 200	10 - 50	< 12; < 25[c,d]	8 - 12	10 - 16	13 - 30
Liquid Limit	< 40	-	-	-	-
Plasticity Index	< 18	-	< 10; < 15	< 10; < 15	< 10; < 15[e]
Field Moisture Equivalent	-	< 20[c]	-	-	-
Linear Shrinkage	-	< 5[c]	-	-	-

[a] Proper or general.

[b] Maximum size not larger than 1/3 of layer thickness; if compacted in several layers, not larger than thickness of one layer.

[c] Lower values for wide and higher values for narrow gradation band of sand. If more than 12 percent passes, restrictions are placed as indicated on field moisture equivalent and linear shrinkage.

[d] A certain percentage of No. 200 or filler material is indirectly required to pass supplementary stability test.

[e] Values between 10 and 15 permitted in certain cases.

Table 4. Grading and plasticity requirements for soil bitumen mixtures.(26)

Sieve Size	Amounts Finer than each Laboratory Sieve (Square Openings), percent by weight	
	Sands and/or Silty Sands	Semi-Processed, Crusher, Pit, or Bank Run
1 5 in	100	100
1 in		80 to 100
No. 4		25 to 85
No. 200	0 to 25	3 to 15
Sand Equivalent, percent or Plasticity Index	30 min 6 max	30 min 6 max

Table 5. Engineering properties of materials suitable for bituminous stabilization.(20)

Sieve Size	Sand-Bitumen	Soil-Bitumen	Sand-Gravel-Bitumen
.5 in			100
1 in	100		
0.75 in			60 - 100
No. 4	50 - 100	50 - 100	35 - 100
No. 10	40 - 100		
No. 40		35 - 100	13 - 50
No. 100			8 - 35
No. 200	5 - 12	good: 3 - 20 fair: 0 - 3 and 20 - 30 poor: > 30	
Liquid Limit		good: < 20 fair: 20 - 30 poor: 30 - 40 unusable: > 40	
Plasticity Index	< 10	good: < 5 fair: 5 - 9 poor: 9 - 15 unusable: > 12 - 15	< 10

Includes slight modifications later made by Herrin.

Table 6. Grading, plasticity, and abrasion requirements for soils suitable for emulsified asphalt-treated base course.[21]

Sieve Size	Percent Passing by Weight		
	2-in maximum	1.5-in maximum	0.75-in maximum
2.5 in	100		
2 in	90 - 100	100	
1.5 in		90 - 100	
1 in			100
0.75 in	50 - 80	50 - 80	80 - 100
No. 4	25 - 50	25 - 50	25 - 50
No. 200	3 - 15	3 - 15	3 - 15

Other Requirements

Plasticity Index	6 maximum
Resistance Value	75 minimum
Loss in LA Abrasion Machine	50 percent maximum

Product of Plasticity Index and the percent passing No. 200 sieve shall not exceed 60.

Table 7. Guidelines for emulsified asphalt stabilization.[22]

	Good	Fair	Poor
Percent Passing No. 200	3 - 20	0 - 3, 20 - 30	> 30
Sand Equivalent	>25	15 - 25	< 15
Plasticity Index	< 5	5 - 7	> 7

Table 8. Aggregate gradation specification limits for bituminous pavements.[23]

Surface Course

Sieve Designation (Square Openings)	Percentage by Weight (Passing)														
	1-1/2-in. Maximum			1-in. Maximum			3/4-in. Maximum			1/2-in. Maximum			3/8 in. Maximum		
	Gradation 1			Gradation 2			Gradation 3			Gradation 4			Gradation 5		
	a	b	c	a	b	c	a	b	c	a	b	c	a	b	c
1-1/2-in.	100	100	100	---	---	---									
1-in.	79-95	83-96	86-98	100	100	100	---	---	---						
3/4-in.	---	---	---	80-95	84-96	90-98	100	100	100	---	---	---			
1/2-in.	61-75	66-79	71-84	68-86	74-89	79-93	80-95	84-96	87-98	100	100	100	---	---	---
3/8-in.	---	---	---	---	---	---	---	---	---	79-94	81-95	86-96	100	100	100
No. 4	45-54	48-60	54-66	45-60	52-68	60-75	55-70	61-74	67-80	59-73	64-80	72-85	75-95	78-95	80-95
No. 10	31-43	37-49	43-55	32-47	39-54	47-62	40-54	46-60	54-66	43-57	50-64	57-70	56-76	60-80	62-84
No. 40	16-25	20-29	25-34	16-26	21-32	26-37	22-31	26-35	31-40	23-33	27-37	31-42	26-44	29-47	32-50
No. 80	10-17	12-19	15-22	10-18	13-21	15-24	12-20	15-23	19-26	13-20	16-23	19-26	14-28	16-30	18-32
No. 200	3-6	3.5-6.5	4-7	3-7	3.5-7.5	4-8	3-7	3.5-7.5	4-8	4-8	4-8	4-8	5-9	6-10	7-11

Binder Course

	Gradation 6			Gradation 7			Gradation 8			Gradation 9		
	a	b	c	a	b	c	a	b	c	a	b	c
1-1/2-in.	100	100	100	---	---	---						
1-in.	73-95	75-95	75-95	100	100	100	---	---	---			
3/4-in.	---	---	---	72-95	75-95	81-96	100	100	100	---	---	---
1/2-in.	55-73	59-77	62-80	61-82	65-85	69-89	70-95	74-95	77-95	100	100	100
3/8-in.	---	---	---	---	---	---	60-80	64-84	68-88	71-95	75-95	78-95
No. 4	35-51	39-55	42-58	38-54	48-66	42-60	47-60	47-65	52-70	50-71	54-75	59-80
No. 10	23-38	27-42	31-46	25-41	29-45	34-50	28-46	33-51	36-54	32-53	36-57	41-62
No. 40	11-21	13-23	15-25	12-23	14-25	17-28	14-26	16-28	18-30	16-29	18-31	21-34
No. 80	6-14	7-15	8-16	7-16	8-17	10-18	8-18	9-19	10-20	10-20	11-21	12-22
No. 200	3-7	3-7	3-7	3-7	3-7	3-7	3-7	3-7	3-7	4-9	4-9	4-9

All High-pressure Tire and Tar-rubber Surface Courses

	Gradation 10			Gradation 11		
	a	b	c	a	b	c
1-in.	100	---	---	---	---	---
3/4-in.	84-97	---	---	100	---	---
1/2-in.	74-88	---	---	82-96	---	---
3/8-in.	68-82	---	---	75-90	---	---
No. 4	54-67	---	---	60-73	---	---
No. 10	38-51	---	---	43-57	---	---
No. 20	26-39	---	---	29-43	---	---
No. 40	17-30	---	---	19-33	---	---
No. 80	9-19	---	---	10-20	---	---
No. 200	3-6	---	---	3-6	---	---

According to a University of Illinois study, materials that are suitable for asphalt treatment include:(24)

- 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.

Recent projects have used paving grade asphalt cements for foamed asphalt road mixing. Most state agencies do not specifically identify requirements for subbases and bases but rather specify requirements for surface courses and indicate that these materials are suitable for other pavement layers.(18)

Criteria for Fly-Ash Stabilization

Fly ash is normally used in stabilization operations to act as a pozzolan and/or filler. A pozzolan is siliceous and aluminous in nature, fly ash, a Pozzolan, reacts with calcium 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 the calcium in the hydrated lime or portland cement in aqueous systems.

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

Aggregates which have been successfully utilized in lime-fly ash mixtures 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. Lime-cement-fly ash stabilization is typically used on coarse-grained soils having no more than 12 percent material passing the No. 200 sieve. In addition, it is recommended that the PI of the minus No. 40 sieve fraction not exceed 25.

Criteria for the use of Combination Stabilizers

Combination stabilizers discussed in this section 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.

5. SUMMARY

The criteria presented in this chapter for selecting an additive represent a wide range of expertise. The general grouping of soils by the soil classification schemes do not truly provide a distinctive method of selecting the appropriate additive. The use of a sieve analysis and the Atterberg Limits provide for a more unique separation of soil properties, and hence, their behavior.

Once a stabilizer is selected, detailed laboratory tests should be performed to determine desirable additive quantities. These tests are outlined in chapter 3 of this volume 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 9.

Table 9. 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).

CHAPTER 3 LABORATORY TESTING PROCEDURES

1. INTRODUCTION

Laboratory testing of stabilized materials in an integral part of soil and base stabilization procedures. Larger projects cannot be economically completed without an extensive testing program to define the operational conditions of the project and smaller projects can often benefit from the added reliability provided in the testing process. Testing will first be required to show if stabilization is appropriate, and if deemed so, testing must be done to assess the economics and effectiveness associated with varying amounts of available stabilizers. Only if this data is available can knowledgeable decisions be made with regard to the type and amount of stabilizer(s) to be used on a particular project. Testing is advantageous even on the smaller projects, where the completion of several rapid tests might indicate a reliable stabilizer content.

Material test properties are often used in designing the pavement structure. Most pavement design methods, theoretical or empirical, take into consideration the strength and durability properties of the materials being used in the pavement system. Thus, improved material quality can be accounted for in the design, if required. It is often necessary to use correlations between material tests normally performed for stabilized materials and the more common tests performed on untreated materials. These conversions are readily available.

The most common use for laboratory testing is for mixture design in addition to the thickness selection process described above. Both objectives need not be accomplished to achieve a level of economic benefit from laboratory testing. The cost of this testing is generally quite small, particularly for the larger projects, in comparison with the total cost of the project.(27)

This chapter presents the procedures and applications of the more pertinent laboratory tests used today. The general classes of testing that provide data useful in selection of stabilizer type, and amount include:

- Moisture limit determinations.
- Density determinations.
- Strength tests.
- Durability tests.

Relevant ASTM or AASHTO testing methods should be followed for all procedures discussed here.(28,29,30,31,32)

2. MOISTURE LIMIT DETERMINATIONS

Moisture limit tests are used to describe the relative influence moisture has on the ability of a material to perform in a structure. These are commonly referred to as the liquid limit (LL), the plastic limit (PL), and the plasticity index (PI). These soil descriptors are highly accurate indicators of the effect a stabilizer additive produces in altering the behavior of the soil. These determinations are made on soils before and after the addition of a stabilizer to judge the improvement provided to the soil.

Improved moisture limits translate into a soil with improved workability, less plasticity problems, less volume change susceptibility, and even a limited strength increase. These tests also indicate when a particular stabilizer does not impart any significant improvement to the properties for which they are being tested. The limit determinations are normally not sufficient for designing pavement thicknesses. The relevant standards include:

- ASTM D 4318 - Liquid Limit, Plastic Limit, and Plasticity Index of Soils (formerly ASTM D 424).
- AASHTO T 89 - Determining the Liquid Limit of Soils.
- AASHTO T 90 - Determining the Plastic Limit and Plasticity Index of Soils.

3. DENSITY AND COMPACTION DETERMINATIONS

The ability to obtain density in a material is a direct indication of the load carrying capacity of the material. If the density of a material can be improved, the engineering characteristics of that soil can also be improved. The proper completion of density determinations is required for proper sample preparation to evaluate the effect of a stabilizer additive on the long term strength improvements provided to the soil. If density testing is not performed correctly, an appropriate judgement of the effectiveness of the stabilizer cannot be established.

The appropriate determination of density in the laboratory and the ability to judge compaction in the field are necessary in determining if the proper level of compaction is being achieved. The mechanics of stabilization and strength gain provided by the stabilization are all highly tied to the development of a high level of compaction in the treated soil. Regardless of the laboratory indications of adequate stabilization, inadequate field compaction will guarantee that strength and durability will not be achieved in the field. This will further guarantee a poor performance of the pavement when exposed to environmental or load stresses.

There are separate and distinct compaction and evaluation procedures for untreated soils and soils treated with different stabilizer additives. The most notable difference is for bituminous stabilized materials. The appropriate application of the relevant compaction criteria provides the following:

- Establish optimum moisture content and maximum density of the untreated soil which serve as target values for construction of stabilized soils, most importantly the optimum moisture content.
- Establish laboratory controls for stabilized samples providing for standard conditions used to judge the effectiveness of the additive on strength under similar conditions.
- Establish field controls used to verify adequacy of field construction in obtaining samples with appropriate conditions to allow stabilization to proceed as planned in the design phase.

General compaction procedures can be found in AASHTO T 99 (*Moisture-Density Relations of Soils Using a 5.5-lb Rammer and a 12-in Drop*) or AASHTO T 180 (*Moisture-Density Relations of Soils Using a 10-lb Rammer and an 18-in Drop*). There are specific recommendations for moisture density in cement treated soils in ASTM D 558 and AASHTO T 134 (*Moisture Density Relations of Soil-Cement Mixtures*).

Requirements for density in bituminous stabilized materials will normally be taken from the design procedure selected for the material being used and the procedure, Hveem or Marshall. There are special requirements for measuring the density of compacted bituminous mixtures and these should be consulted to establish appropriate measurement techniques. These requirements are ASTM D 2726 (*Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Saturated Surface Dry Specimens*) and ASTM D 1188 (*Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens*).

4. STRENGTH TESTS

There have been a number of tests proposed to characterize the strength properties of stabilized soils. Some of these tests are empirical in nature in that their usefulness lies in a correlation of the test result with field performance.(33) Other tests have been developed in recent years which characterize the response of the material to deformation or load and are applicable to the mechanistic design approach. These tests cover the range of static and dynamic loading conditions.

The contents of this section are divided into six categories of strength related testing procedures as follows:

- Compression testing.
- Tension testing.
- Stability testing.
- Elasticity testing.
- Fatigue testing.
- Bearing capacity testing.

The category used by any State agency will normally depend on the agency's experience with a material's performance and its typical failure pattern in service.

Compression Tests

Compression testing represents the class of testing that is readily available to State testing agencies and provides for a great deal of information for all stabilizer additives. The procedures discussed here include:

- Unconfined Compression.
- Triaxial - Rapid Shear.

Triaxial - Rapid Shear

The triaxial rapid shear test is used to determine the shear strength of a soil. In the test, a soil specimen is encased in a rubber membrane and subjected to an equal, all-around confining pressure (normally air). A vertical axial load is then applied at a constant deformation rate until the specimen fails. By repeating the test using various lateral pressures, a Mohr-Coulomb failure envelop can be generated from which the cohesion (c) and internal friction (ϕ) parameters of the soil can be determined. The shear strength of a soil can then be computed from the general Coulomb equation:

$$s = c + \sigma_d \cdot \tan \phi$$

where:

s = shear strength, psi
c = cohesion, psi
σ_d = applied vertical stress, psi
ϕ = internal friction angle

The test is commonly run as a closed system to reflect the drainage characteristics of the soil. It may be conducted in accordance with ASTM D 2850 (*Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression*).

Figure 7 is a schematic of the triaxial testing cell apparatus. Several types of triaxial cells and loading mechanisms exist for which testing procedures vary. Although resilient modulus is generally the strength parameter utilized in mechanistic design equations, shear strength limitations are acknowledged in mechanistic design programs. Correlations have been developed between shear strength and commonly used strength parameters, such as CBR and k-value, for use in empirical-based pavement design equations. The rapidity of the rapid shear test makes it more desirable than the time-consuming CBR and k-value tests. Stabilization typically produces a substantial increase in cohesion with only a minor increase in friction angle.

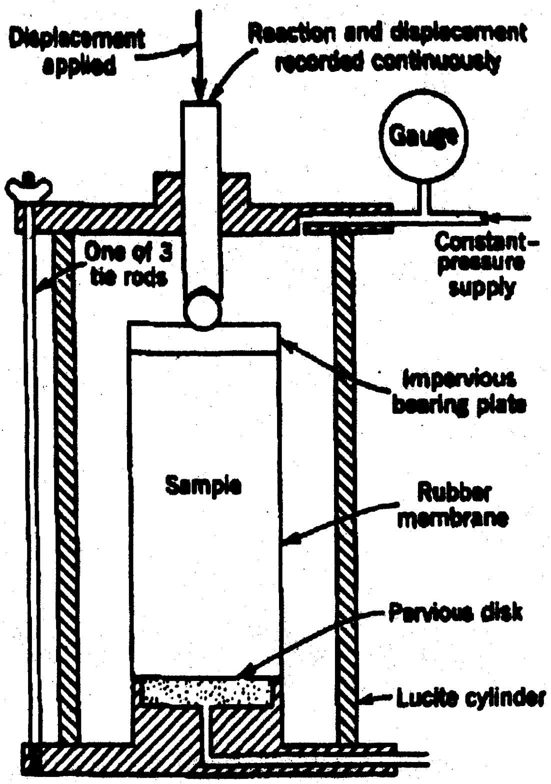

Figure 7. Schematic of triaxial cell.

Unconfined Compression

The unconfined compression test is similar to the triaxial compression test with the exception that no confining pressure is employed. Test specimens (field-extracted cores or laboratory-molded cylinders) are typically 4 in diameter and 8 in height; however, smaller dimensions are occasionally used particularly for laboratory compacted samples of fine grained cohesive soils.

The test basically consists of applying a vertical axial force at a constant rate until the specimen fails along a shear plane or by bulging. Vertical deformations are measured along with the applied load increments. The unconfined compressive strength is found by dividing the maximum applied load by the cross-sectional area of the applied load.

Unconfined compressive tests are typically performed at a specified time following sample preparation, depending on the curing time specified in the relevant specifications:

- ASTM D 1074 - Compressive Strength of Bituminous Mixtures.
- AASHTO T 167 - Compressive Strength of Bituminous Mixtures.
- ASTM D 1633 - Compressive Strength of Molded Soil-Cement Cylinders.

- ASTM D 2166 - Unconfined Compressive Strength of Cohesive Soil.
- AASHTO T 208 - Unconfined Compressive Strength of Cohesive Soil.

Stability Testing

The strengths of asphalt-treated materials are often evaluated in the context of stability. Stability is defined as the resistance of a material to displacement under applied loads. Such resistance comes primarily in the form of the frictional resistance furnished by the aggregate and the cohesion introduced by the bituminous binder.

Cohesion increases with asphalt content to the point where the aggregate particles are well covered with a film of binder. Little, if any, additional cohesion is afforded beyond that point. The cohesion provided by asphalt gives great resistance to dynamic loads but normally yields in the presence of static loads.

The most important element in the overall stability of an asphalt-aggregate mixture is the frictional resistance provided by the aggregate. Clearly, aggregate possessing rough surfaces and irregular shapes will provide greater frictional resistance, thereby yielding greater stability. The level of friction developed in a mixture is also influenced by the degree and method of compaction.

Two tests that have been developed to measure the resistance to deformation of asphalt-aggregate mixtures are the Marshall stability test and the Hveem stabilometer/cohesiometer test. Brief descriptions of these tests are provided below.

Although the concept of stability is very important, the principles behind it have not yet been implemented into theories relating to performance. Rather, an array of supplemental tests have been combined with both the Marshall and Hveem stability tests, resulting in the Marshall and Hveem methods of mix design. In these two mix design methods, specifications have been established that require minimum levels of stability based upon the observed field performance and measured stability of past mixes. Hence, the Marshall and Hveem stability tests are empirical in nature.

Many agencies have adopted one of these mix design methods and have modified the stability requirements to reflect their own experiences with observed field performance. Marshall and Hveem mixture design methods exist for all three types of asphalt-aggregate mixtures (i.e., asphalt concrete, emulsified asphalt, and cutback asphalt).(34,36) Although the procedures for preparing and testing the various asphalt-aggregate specimens differ somewhat, the concepts are essentially the same.

Hveem Stabilometer and Cohesiometer

The Hveem stabilometer and cohesiometer testing devices were developed by the California Division of Highways to measure a material's frictional resistance and cohesive strength. Test specimens (laboratory-molded or field-cored) are typically 4 in (101.6 mm) in diameter and 2.5 in (63.5 mm) thick. Figures 8 and 9 show the stabilometer and cohesiometer testing apparatuses, respectively.

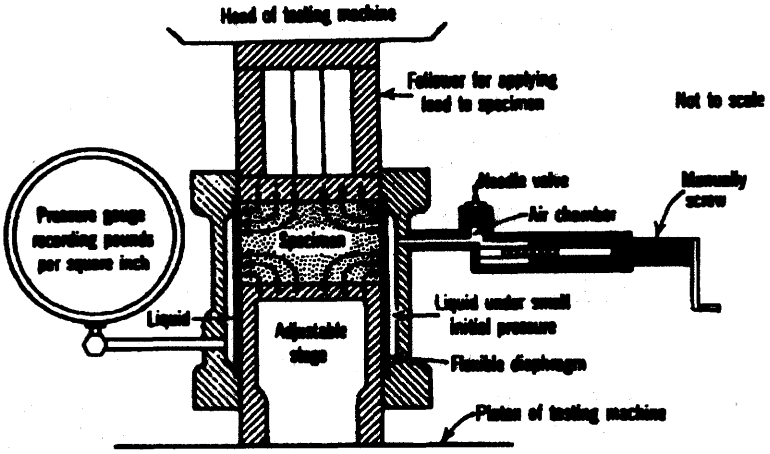

Figure 8. Schematic of the Hveem stabilometer.

In the stabilometer test, specimens are placed between the cylindrical loading heads and are subjected to a vertical load applied at a rate of 0.05 in/min. Fully-cured specimens are loaded to a vertical pressure of 400 psi. The lateral pressure developed in the fluid confining the sides of the specimen is then recorded and the stability is then determined from the following equation:

$$S=22.2/[P_hD_2/(P_h-P_y)+0.222]$$

where:

S = Hveem stability value

P_v = vertical pressure, psi

P_h = horizontal pressure at the instant P_v is recorded, psi

D_2 = number of turns of the displacement pump to change horizontal pressure from 5 to 100 psi

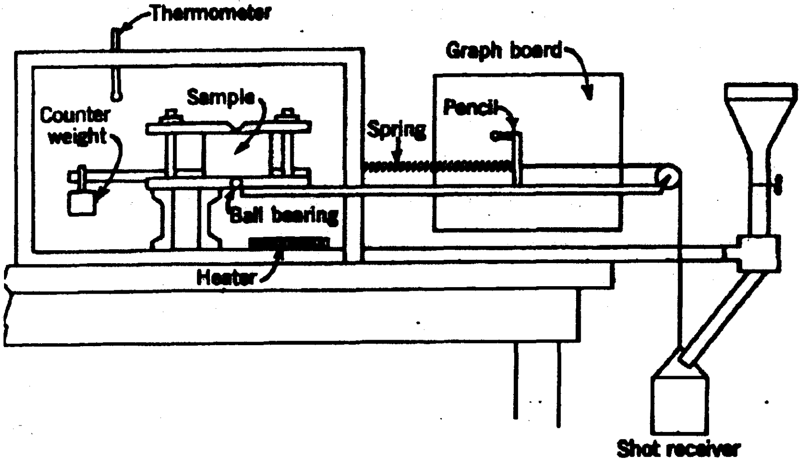

Figure 9. Schematic of the Hveem cohesiometer.

In theory, a liquid would exhibit an Hveem stability value of 0 while a perfect solid would display a value of 100. Stability values for asphalt-aggregate mixtures typically range between 30 and 40.

The Hveem stabilometer primarily measures the frictional resistance of an asphalt-aggregate mixture. The extent to which cohesion is included in the measurement of a material's Hveem stability is not known. The measurement of cohesion is achieved through the use of the Hveem cohesiometer.

Relevant test specifications include:

- AASHTO T 246 - Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus.
- ASTM D 1560 - Resistance to Deformation and Cohesion of Bituminous Mixtures by Means of Hveem Apparatus.

Marshall

The commonly-used Marshall test measures frictional resistance and cohesion in combination. In the test, cylindrical test specimens (laboratory-molded or field-cored), 4 in (101.6 mm) in diameter and 2.5 in (63.5 mm) thick, are placed in semi-circular testing heads which resemble an open-ended adjustable collar. The device is then loaded at a rate of 2 in/min (51 mm/min), causing the specimen to deform.

Stability is measured as the maximum load applied, which occurs at the moment the specimen begins to yield. The deformation at this point is also measured and referred to as "flow." While frictional resistance comprises much of the stability measured and cohesion plays a large role in flow, neither account solely for each property.

The Marshall test, like the Hveem test, is empirical in nature. Very little theory is associated with the test as it merely gives indication of the relative stability of various asphalt-aggregate mixtures. The Marshall test, in conjunction with the bulk specific gravity test, density and voids analyses, and moisture absorption test, comprise the Marshall method of mix design. Several agencies use the Marshall method of mix design, specifying minimum values of stability and certain ranges of flow.(34)

The relevant test specifications for Marshall testing include:

- ASTM D 1559 - Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus.
- AASHTO T 245 - Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus.

Tensile Testing

Direct Tensile

Direct Tensile testing is not normally conducted on stabilized materials because of the difficulty with sample preparation. For this test, a cylindrical sample with a length to diameter ratio of 2 must be bonded to the endcaps, typically with epoxy. The end caps are separated by a constant rate, and the sample fails in tension.

Split-Tensile

The split-tensile or indirect tensile test is the most common method of determining the tensile strength of stabilized materials. The test consists of applying a compressive load at a constant rate along the diametral plane of the specimen until it splits. The applied load creates a relatively uniform tensile stress perpendicular to and along the diametral plane. The split generally occurs along this diametral plane and the tensile strength is determined from the following equation:

$$\sigma_t = (2 \cdot P)/(\pi \cdot t \cdot d)$$

where:

σ_t = tensile strength, psi
P = maximum applied load, lb
t = specimen thickness, in
d = specimen diameter, in

The split-tensile test is shown in figure 10 and has been used with all types of stabilized soils. For asphalt stabilized soils, it is most applicable to dense-graded mixtures since a greater degree of testing difficulty is experienced with open-graded mixtures. The test is occasionally used on lime, cement, and fly ash stabilized soils; however, the flexural test is more common with these types of stabilized soils.

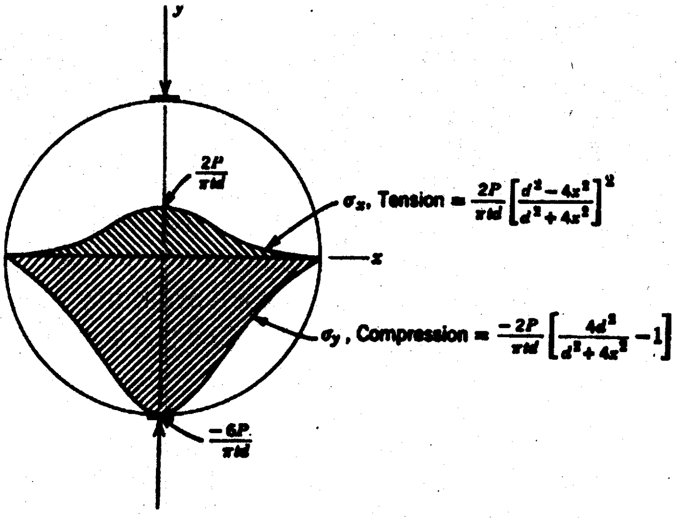

Figure 10. Indirect tensile test stress distribution from diametral loading.

A relevant procedure for conducting this test is given in ASTM C 496 (*Splitting Tensile Strength of Cylindrical Concrete Specimens*).

Flexural Strength

The flexural strength test is a common test for determining the flexural tensile strength of stabilized soils. This test can be performed using either third-point loading (ASTM C 78 - *Flexural Strength of Concrete*) or center-point loading (ASTM C 293 - *Flexural Strength of Concrete*) procedures. Figures 11 and 12 are schematics of the two loading devices which may be used.

Normally, several 6 x 6 x 20 in (152 x 152 x 508 mm) beams are prepared from a design batch mix and then tested in flexure at a specified curing period. Both methods involve placement of a test specimen in the loading apparatus and applying a load at a constant rate until the beam ruptures. The flexural strength, or modulus of rupture (R), for third-point and center-point loading methods are determined from the following equations:

$$R = (P \cdot l)/(b \cdot d^2) \quad \text{(Third-Point Loading)}$$
$$R = (3 \cdot P \cdot l)/(2 \cdot b \cdot d^2) \quad \text{(Center-Point Loading)}$$

where:

R = modulus of rupture, psi
P = maximum applied load, lb
l = beam length, in
b = beam width at point of fracture, in
d = beam depth at point of fracture, in

The preferred loading technique for flexure is third-point loading. With third-point loading, a pure moment with zero shear is created in the middle third of the beam. Failure generally occurs at the weakest point near or within this middle section. In center-point loading, substantial shear forces and unknown stress concentrations at the point of load application occur along the line of rupture. Beams are forced to break in the proximity of the load application point. This usually results in higher strengths since the weakest element is most likely displaced from the concentration of load. Less variability in flexural strength is associated with third-point loading since a greater portion of the specimen is being subjected to the stresses induced.

A version of the flexural strength test outlined in ASTM D 1635, Flexural Strength of Soil-Cement Using Simple Beam with Third-Point Loading, is specifically for soil-cement mixtures and uses 3 x 3 x 11.25 in (76 x 76 x 290 mm) beam specimens in third-point loading.

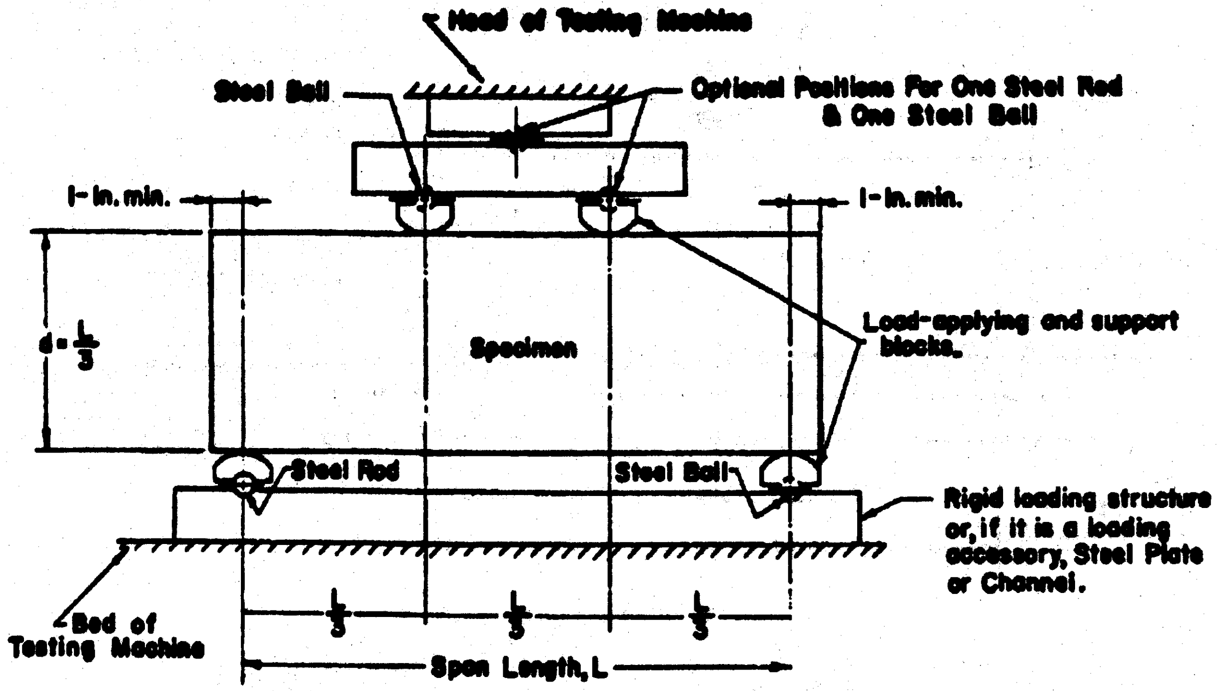

Figure 11. Third-point loading apparatus.

Usage:

Lime: Good; increase in flexural strength with extended curing.
Lime-Fly Ash: Flexural test used, but flexural strength often estimated as 0.2xUC
Cement: Routinely used for cement treated materials: f = 0.2 to 0.33 x (UC)
Lime-Fly Ash: E = 0.5 x 10^6 to 2.5 x 10^6

Repeated-Load Elasticity and Fatigue Life Testing

Repeated-load tests are useful in characterizing the elastic and fatigue properties of stabilized soils. Elasticity is typically determined by measuring a material's resiliency in a repeated load-deformation sequence. Most commonly used tests include the triaxial compression, diametral, dynamic compression, and flexural beam.

Fatigue is the phenomenon of cracking or fracture under a repeated stress having a maximum value less than the tensile strength of the material. While repeated compression tests have been used in the past for fatigue modeling, the flexural beam test is by far most appropriate for examining fatigue trends.

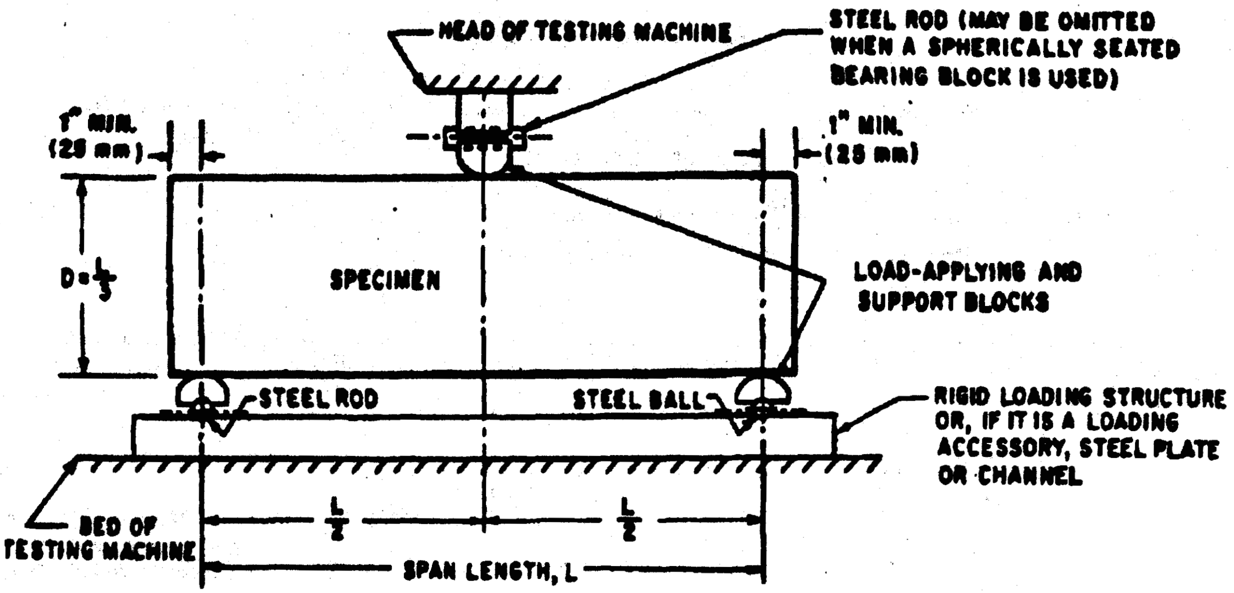

Figure 12. Center-point loading apparatus.

Triaxial Compression (Resilient Modulus)

The triaxial cell apparatus shown previously in figure 7 is used in resilient modulus testing. In this test, a rubber-encapsulated material specimen 4 in (102 mm) in diameter and 8 in (204 mm) thick is subjected to a constant confining pressure (as in the rapid shear test) and a repeated vertical deviator stress. The triaxial cell is configured in the same manner as the dynamic modulus test which will be presented here also. The resulting elastic strains are measured, from which the resilient modulus is calculated as shown below:

$$E_R = \sigma_d/\varepsilon_r$$

where:

E_R = resilient modulus, psi
σ_d = applied vertical stress, psi
ε_r = resilient axial strain, in/in

The test is conducted over a range of temperatures for asphalt mixtures and many variations in load, loading frequency, load duration, and confining pressure can be imposed. Relatively low stiffness materials, such as cement and lime modified soils, can be tested using this test method. However, medium stiffness materials, such as asphalt and lime stabilized soils, are most suitable for this type of test.

ASTM D 3496 - Preparation of Bituminous Mixture Specimens for Dynamic Modulus Testing. (Preparation Only)

AASHTO T 274 - Resilient Modulus of Subgrade Soils contains the basis, but is currently undergoing revisions and will be resubmitted for approval in the future.

Diametral (Resilient Modulus)

The resilient modulus of medium and high stiffness stabilized materials can be determined by using the diametral loading device shown in figure 13.(35) A repetitive and controlled load is applied diametrically to a specimen, which causes dynamic deformations across the horizontal diametral plane. Transducers are used to measure these deformations. Knowledge of the dynamic load and the recoverable horizontal deformation allows the resilient modulus to be calculated using the following equation:

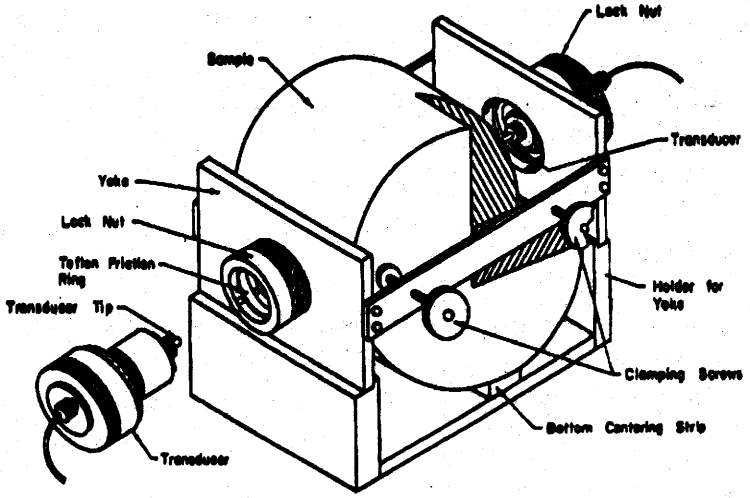

Figure 13. Diametral resilient modulus device.(35)

$$E_R = P \cdot (\nu + 0.27)/(t \cdot \delta_h)$$

where:

E_R = resilient modulus, psi
P = repeated load, lb
ν = Poisson's ratio
t = specimen thickness, in
δ_h = recoverable horizontal deformation, in

Poisson's ratio is defined as the ratio of lateral strain (ε_l) to axial strain (ε_a), caused by a load parallel to the axis in which ε_a is measured.(27) The value of Poisson's ratio is dependent upon the material and is either estimated or is determined during the test. Typical ranges for Poisson's ratio of various stabilized materials are:

• cement treated	0.10 to 0.25
• lime-fly ash treated	0.10 to 0.20
• asphalt treated	0.35 to 0.45

The test, ASTM D 4123 - Indirect Tension Test for Resilient Modulus of Bituminous Mixtures, generally covers a range of temperatures, loads, loading frequencies, and load durations. Typically, specimens 4 in (102 mm) in diameter and 2 in (51 mm) thick are tested at three different temperatures: 41, 77, and 104 °F (5, 25, 40 °C). In addition, at each temperature, one or more loading frequencies (typically 0.33, 0.5, and 1.0 Hz) are imposed.

The diametral resilient modulus test is among the more popular tests for determining elastic modulus. The test is rapid, easy to perform, and provides values of resilient moduli similar to those obtained in the flexural modulus test. It is particularly suitable for testing asphalt-aggregate mixtures because it can be used to test laboratory-compacted Marshall and Hveem specimens as well field-cored specimens.(35)

Usage:

Asphalt: Dense- and Open-Graded EAMs, Modified Hveem
LFA: Resilient Modulus; 100,000 to 250,000 for low quality, 250,000 to 500,000 for medium quality, and > 500,000 for high quality.(4)

Compressive - Dynamic Modulus

In this sinusoidal axial compression test, material specimens 4 in (102 mm) in diameter and 8 in (204 mm) thick are subjected to a repeated compressive load. The resulting recoverable axial strain may be measured by wire strain gages or LVDTs mounted on the loading ram. A device is shown in figure 14. The data is used in calculating the dynamic modulus as follows:

$$|E^*| = \sigma_o/\varepsilon_o$$

where:

$|E^*|$ = resilient modulus, psi
σ_o = applied axial stress, psi
ε_o = recoverable axial strain, in/in

As with the resilient modulus tests, the dynamic modulus test is conducted at multiple temperatures (41, 77, and 104 °F [5, 25, and 40 °C]) and loading frequencies (1, 4, and 16 Hz). It should be noted that, besides the testing device, the significantly higher loading frequencies used in this test distinguish it from the resilient modulus tests discussed above. In general, the dynamic modulus of a material specimen is

somewhat greater than the resilient modulus of that same specimen. The relevant test procedure for this test are:

ASTM D 3496 - Preparation of Bituminous Mixture Specimens for Dynamic Modulus Testing.
ASTM D 3497 - Dynamic Modulus of Asphalt Mixtures.

Usage:

Cement: Lean concrete - 5 to 6×10^6 psi, course-grained treated materials - 2 to 3×10^6 psi, fine-grained treated materials - 0.5 to 1.5×10^6 psi.

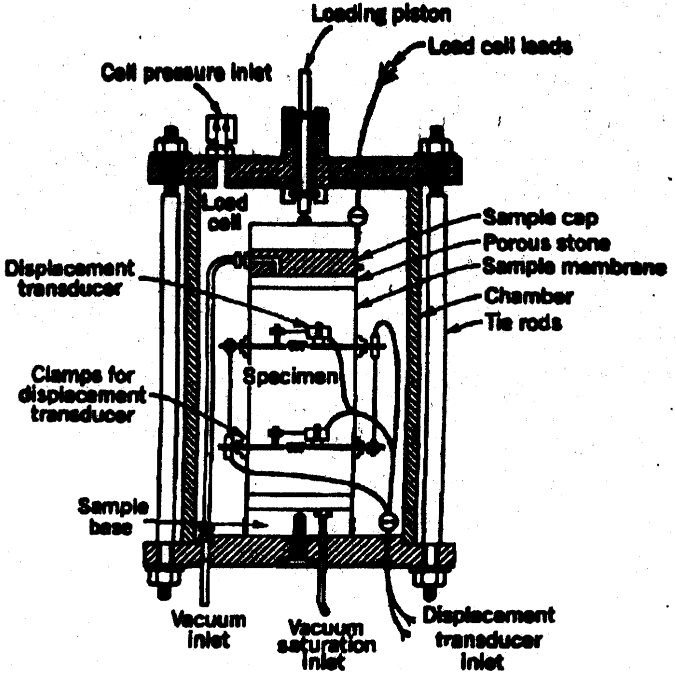

Figure 14. Dynamic compression device.

Flexural Beam (Resilient Modulus & Fatigue)

Typically, a 3.5 x 3.5 x 15 in (89 x 89 x 381 mm) beam specimen is prepared and placed in either a center-point or third-point repetitive loading. As mentioned earlier, the third-point loading is preferred. Repeated flexural loads of haversine wave form are applied to the beam with a 0.1 second duration and 0.4 second rest period. An approximate load of 10 percent of that used to deflect the specimen is applied to force the beam back to its original position. Because progressive damage to the beam is incurred, its stiffness decreases with load applications, and flexural stiffness is based upon initial loading conditions and is computed from the following equation:

$$E_s = [Pa(3L^2 - 4a^2)/(48\ I\ \delta]$$

where:

E_s = flexural stiffness, psi
P = dynamic load applied to deflect beam, lb
a = $0.5 \cdot (L - 4)$, in
L = reaction span length, in
I = specimen moment of inertia about centerline, in^4
δ = dynamic beam deflection at center point, in

The beam sample is formed using ASTM D 3202, Preparation of Bituminous Mixture Beam Specimens by Means of California Kneading Compactor. Figure 15 is a schematic of the repeated third-point flexural device. The dynamic deflection of the beam at the center, as in most deflection testing, is measured by a linear differential transformer (LVDT). The test is normally performed at 70 °F (21.1 °C) using stress levels ranging from 30 to 300 psi.

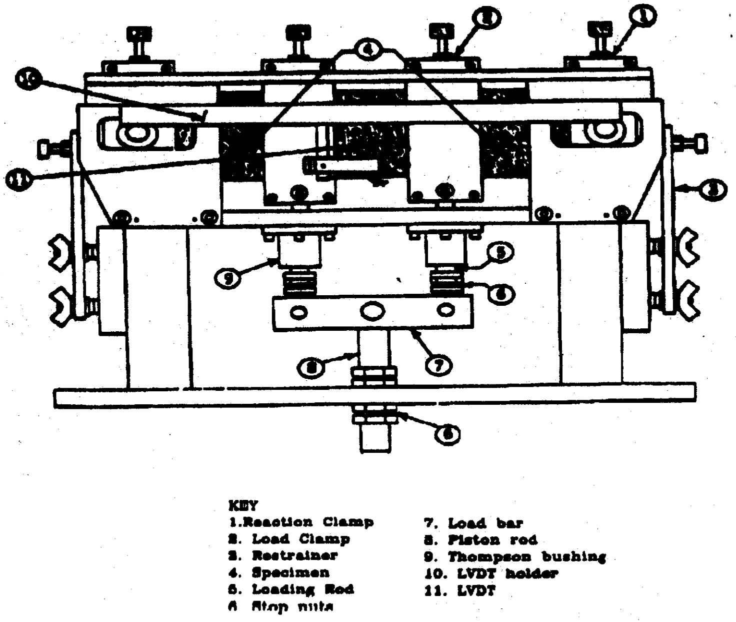

Figure 15. Schematic of repeated flexure apparatus.

Bearing Tests

California Bearing Ratio (CBR)

The California Bearing Ratio test is a load-deformation test that provides relative strength values useful in determining pavement layer thicknesses for some of the empirical design procedures. Originating with the California Department of Transportation, the test has enabled engineers to develop empirically-based equations where the CBR indicates the quality of a material in relation of that of an excellent base course (i.e., crushed stone).

The test which is performed in accordance with ASTM D 3668, Bearing Ratio of Laboratory Compacted Soil-Lime Mixtures, is essentially consists of driving a piston (3 in^2 in end area) at a uniform rate [0.05 in/min (1.3 mm/min)] into a compacted specimen (prepared in accordance with AASHTO T-99) previously soaked in water for 96 hours. The specimen is enclosed in a steel mold and surcharged at the top with a series of metal rings. Load-deformation data are gathered as the specimen is penetrated and the CBR is usually determined by utilizing the loads associated with 0.1 in (2.54 mm) or 0.2 in (5.08 mm) penetration.

The CBR is computed as follows:

$$CBR = (P_s/P_{cs}) \cdot 100$$

where:

CBR = California bearing ratio, %
P_s = load carried by sample specimen at penetration of X in.
P_{cs} = load carried by crushed stone specimen at penetration of X in.

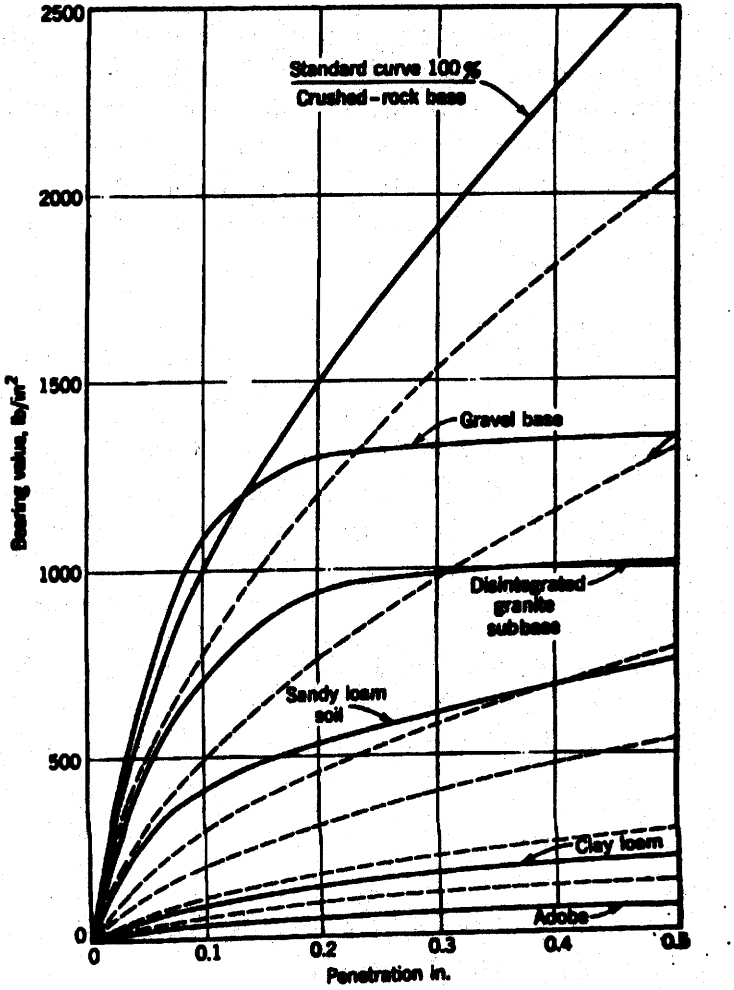

Figure 16. CBR load deformation curves for typical soils.

Figure 16 shows CBR load deformation curves for a variety of soils. As can be seen, the load-deformation curve for crushed stone is used as the standard by which other materials are compared. It is also apparent that curve profiles vary with soil type. This is important because different highway agencies use penetration levels that are unique to their situations. Thus, it is not possible to directly compare the CBR designs of agencies with differing standards.

The CBR test is often criticized because it does not simulate the shearing forces that develop in the underlying structure. It is maintained that the mold provides excessive confinement, allowing certain soils to better resist the penetration of the piston. The CBR test is also limited in applicability. Only untreated soils or treated soils that have not obtained considerable strength are suitable for CBR analysis. There are no practical applications for CBR values greater than 100, as exhibited by high strength cemented materials.

Usages:

Lime: Not appropriate, CBR > 100 typically
Cement: Not appropriate for coarse-grained treated soils; Applicable for fine-grained treated soils up to CBR = 100.

5. DURABILITY TESTS

The durability of cement, and occasionally lime and lime-fly ash, are frequently assessed by ASTM D 559-82 and AASHTO T 135, Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures, and ASTM D 560 and AASHTO T 136, Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures. These tests were designed to determine whether the soil-cement would stay hard or whether expansion and contraction on alternate freezing-thawing and moisture changes would cause the soil-cement to soften.(27)

Weight Loss

Typically, specimens 4 in (102 mm) in diameter and 4.5 in (114.3 mm) thick are compacted in a mold at optimum moisture content. Specimen weights are obtained after extrusion from the mold and the specimens are then cured 7 days in a moist room. Following the curing period, the specimens are subjected to a series of 12 wet-dry or freeze-thaw cycles. The wet-dry cycle consists of 5 hours of water immersion and 42 hours in a 160 °F (71 °C) oven. The freeze-thaw cycle consists of 24 hours in a -10 °F (-23 °C) freezer cabinet and 23 hours in a chamber with a temperature of 70 °F (21 °C) and a relative humidity of 100 percent. In both tests, specimens are lightly brushed with a wire brush at the end of each cycle. Final specimen weights are obtained and the percent soil-cement loss is determined by the following equation:

$$\textbf{Percent Loss} = (A/B) \cdot 100$$

where:

A = original specimen weight minus final specimen weight.
B = original specimen weight.

Experience has shown that the freeze-thaw test is generally the critical test except for mixtures containing relatively large amounts of silt and clay.(27)

Residual Strength

In recent years, it has become desirable to assess the effects of freeze-thaw on lime, fly ash, and cement treated materials in terms of residual strength. While the compressive strength is the residual strength parameter of choice today, it is believed by some that the residual tensile strength and for resilient modulus should be the criteria for assessing freeze-thaw damage. Regardless of which strength parameter is used, the testing procedure adheres to ASTM D 560 for the most part, with strength testing being conducted at the end of the 7-day curing period and at the end of the freeze-thaw cycling. ASTM C 593, "Strength Loss upon Vacuum Saturation" is a vacuum saturation procedure that evaluates the freeze-thaw durability in the laboratory.

Stripping

A problem with asphalt stabilized mixtures is the development of stripping, a separation of the asphalt from the aggregate in the presence of water. Most mix design procedures for these liquid asphalt/aggregate combinations include a means of establishing the effect of moisture on strength, and these procedures should be utilized.(35,36) If these procedures are not available, ASTM D 1664, Coating and Stripping of Bitumen-Aggregate Mixtures, may be used, or ASTM D 4867, Effect of Moisture on Asphalt Concrete Paving Mixtures may be used.

6. SUMMARY

Laboratory testing of stabilized materials must be done when stabilization is being used to provide an improved material with properties that will be used in the pavement design process. Material variability and its effect on the strength properties is such that performance improvements cannot be assumed, and the amount of the stabilizer required for specific performance improvements must be established and verified.

Each stabilizer additive has its own mix design procedure and specific set of tests which determine the individual properties each different material and stabilizer combination can achieve. The tests establish durability limits, strength levels, and can indicate when problems would be encountered in the field. This provides the engineer with more information to ensure the final product. The testing procedures should be adhered to provide the certainty required for in the pavement design.

CHAPTER 4 LIME STABILIZATION

1. INTRODUCTION

Lime is among the oldest soil stabilizing agents known. The early Romans used lime in the construction of the Appian Way and many other roads. Similar applications have been recorded in the ancient civilizations of Greece, India, and China.

Since World War II, the use of lime as a stabilizer has increased substantially. In the U.S. alone, it has been incorporated into construction projects in every state and has an annual average usage of about 100 million yd^2 of 6-in equivalent thickness. Lime stabilization has been used in a broad spectrum of projects, including freeways and highways, county and municipal roads, military and commercial airfield pavements, earth dams, drainage canals, and public and private parking areas.

2. TYPES OF LIME

The term lime refers specifically to calcium oxide (quicklime) and calcium hydroxide (hydrated lime), both burned forms of limestone (calcium carbonate). Quicklime, available in granular or pulverized form, is often treated with water to produce hydrated lime in coarse or finely ground form. Lime for stabilization should not be confused with inert carbonates such as limestone, agstone, or aglime which are used in agricultural applications.

Several types of lime are commercially available. Calcitic quicklime (CaO) and dolomitic quicklime ($CaO + MgO$) are produced by calcining (burning) calcitic and dolomitic limestone, respectively. By the controlled addition of water to quicklime, three types of hydrated lime can be produced: high-calcium [$Ca(OH)_2$], monohydrated dolomitic [$Ca(OH)_2 + MgO$], and dihydrated dolomitic [$Ca(OH)_2 + Mg(OH)_2$. Typical properties of commercial varieties of quicklime and hydrated lime are summarized in table 10.

Several forms of lime, including products with varying degrees of purity, have been successfully utilized for many years as soil stabilizing agents. The most commonly used products are commercial hydrated high-calcium lime, monohydrated dolomitic lime, calcitic quicklime, and dolomitic quicklime. A steady rise in the use of quicklime has been experienced in the U.S. in the last 20 years. Estimated at 10 percent in 1979, quicklime now accounts for approximately 35 percent of all lime used in stabilization.(37)

Table 10. Properties of commercial limes.(38)

A. Quicklime

Constituent	High Calcium Range, %*		Dolomitic Range, %*	
CaO	92.25 - 98.00		55.50 - 57.50	
MgO	0.30 - 2.50		37.60 - 40.80	
SiO_2	0.20 - 1.50		0.10 - 1.50	
Fe_2O_3	0.10 - 0.40		0.05 - 0.40	
Al_2O_3	0.10 - 0.50		0.05 - 0.50	
H_2O	0.10 - 0.90		0.10 - 0.90	
CO_2	0.40 - 1.50		0.40 - 1.50	
Specific Gravity	3.2 - 3.4		3.2 - 3.4	
Specific Heat at 100 °F (38 °C)	Btu/lb 0.19	(J/kg) (442)	Btu/lb 0.21	(J/kg) (488)
Bulk Density, pebble lime	pcf 55 - 60	(kg/m^3) (880 - 960)	pcf 55 - 60	(kg/m^3) (880 - 960)

B. Hydrates

	High Calcium		Monohydrated Dolomitic		Dihydrated Dolomitic	
Principal constituent	$Ca(OH)_2$		$Ca(OH)_2 + MgO$		$Ca(OH)_2 + Mg(OH)_2$	
Specific gravity	2.3 - 2.4		2.7 - 2.9		2.4 - 2.6	
Specific heat at 100 °F (38 °C)	Btu/lb 0.29	(J/kg) (674)	Btu/lb 0.29	(J/kg) (674)	Btu/lb 0.29	(J/kg) (674)
Bulk density	pcf 25 - 35	(kg/m^3) (400 - 560)	pcf 25 - 35	(kg/m^3) (400 - 560)	pcf 30 - 40	(kg/m^3) (480 - 640)

* Percentage by weight.

By-product lime, available from various manufacturing processes, also provides a source of lime that may be suitable for use in stabilization. Common types of by-product lime are:

- Lime kiln dust — collected from the draft of the calcining process in lime production operations.
- By-product (carbide) lime — resulting from production of acetylene gas.

Fresh by-product lime can be just as effective in stabilization as hydrated lime and quicklime, at a cost saving. However, by-product limes can vary in quality to the extent that consistent results are difficult to obtain. For instance, commercial hydrates usually are more finely divided and have higher specific surfaces than carbide (by-product) limes. These irregularities are generally attributed to the operation of the kiln, the inherent variability of the coal used, and lagoon storage methods.

Another by-product lime is polyhydrate lime. This type, developed in Chicago, is produced by hydrating a mixture of lime kiln dust and normal quicklime. The quicklime is essentially used to upgrade the lime kiln dust, initiating the hydration reaction. Although the by-product hydrate is not chemically equivalent to normal commercial hydrated lime, it has been successfully used in soil stabilization.(39) From a cost standpoint, polyhydrate lime is considerably more expensive than lime kiln dust, yet is cheaper than commercial hydrated limes and quicklimes.

In the past, there has been concern as to whether calcitic lime [$Ca(OH)_2$] or monohydrated dolomitic lime [$Ca(OH)_2 + MgO$] is the more effective lime stabilizer. Studies by Thompson and the Portland Cement Association have shown that high calcium limes are generally more effective for modifying soil plasticity.(40,41) Dolomitic limes produced higher cured strength in Thompson's study, but the PCA investigation indicated that, "most soils do not respond preferentially to dolomitic monohydrate or hydrated calcitic lime stabilizations for strength improvement."(41) Therefore, both types are, in general, satisfactory for use in soil stabilization.

Most types of lime [exclusive of dihydrated dolomitic, $Ca(OH)_2 + Mg(OH)_2$] are appropriate if a quality soil-lime mixture meeting strength, durability, and economic criteria can be obtained. Laboratory testing may be used to indicate the effectiveness of any of the lime types. Properties of the soil being stabilized may have a much greater influence on the soil-lime reaction than lime type or source.

In most instances, considerations of local availability and cost are more significant than lime type in selecting a lime source. Figure 1 in Chapter I indicates the location of commercial lime plants in the U.S. It is apparent that significant hauling distances may be involved if lime stabilization is to be used in certain areas.

Lime specifications have been prepared by many groups and agencies. Chemical and physical properties (primarily particle size) are normally the major factors considered in a lime specification.

AASHTO M216 is an example of a specification for the use of lime for soil stabilization. ASTM C 977-89 is a specification for quicklime and hydrated lime for soil stabilization that is being used by many agencies today.

Appropriate quality control testing should be conducted during the course of a project to ensure the quality and uniformity of the lime being incorporated into the construction. Producer certification of the lime is used in some cases in lieu of "on the job" lime testing.

3. SOIL-LIME REACTIONS

Lime is used extensively to modify the engineering properties of fine-grained soils and the fine-grained fractions of more granular soils. It is most effective in stabilizing plastic clays capable of holding large amounts of water. The particles of such clays have highly negatively-charged surfaces that attract free cations (positive-charged ions) and water dipoles. As a result, a highly diffused water layer shown in figure 14 forms around the clay particles, thereby separating the particles and causing the clay to become weak and unstable. The extent to which this occurs depends on the amount of water present and the morphology and mineralogy of the clay.(37)

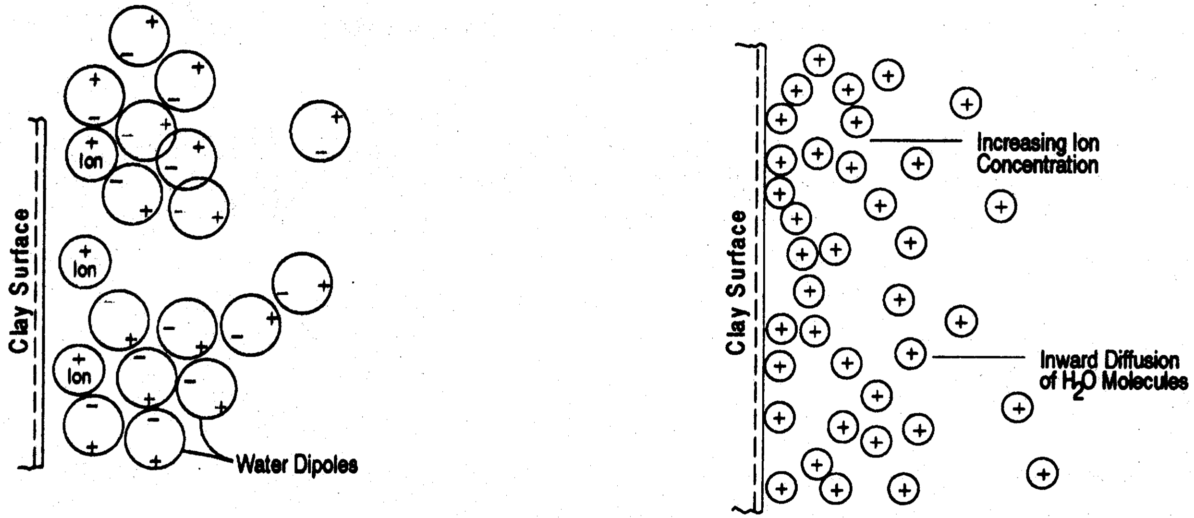

Figure 17. Formation of a diffused water layer around clay particle.(37)

The addition of lime to a fine-grained soil in the presence of water initiates several reactions. The primary reactions, cation exchange and flocculation-agglomeration, take place rapidly and produce immediate improvements in soil plasticity, workability, uncured strength, and load-deformation properties.

Depending on the characteristics of the soil being stabilized, a soil-lime pozzolanic reaction may also occur, resulting in the formation of various cementing agents which further increase mixture strength and durability. Pozzolanic reactions are time and temperature dependent. Therefore, given appropriate temperatures, strength development is gradual but continuous for long periods of time. Temperatures less than 55 to 60 °F (12.8 to 15.6 °C) retard the reaction while higher temperatures accelerate the reaction.(42)

A fourth reaction which may occur in the lime is carbonation. This chemical reaction, in which lime reacts with atmospheric carbon dioxide to form a relatively insoluble carbonate, is detrimental to the stabilization process. It can be avoided by properly expedited and sequenced construction procedures which avoid prolonged exposure to the air and/or rainfall.

Cation Exchange and Flocculation-Agglomeration

Practically all fine-grained soils display cation exchange and flocculation-agglomeration reactions when treated with lime in the presence of water. The reactions occur quite rapidly when soil and lime are intimately mixed.

Assuming equal concentrations, the general order of replaceability of the common cations is given by the Lyotropic series, $Na^+ < K^+ < Ca^{++} < Mg^{++}$.(43) In general, higher valence cations replace those of lower valence, and larger cations replace smaller cations of the same valence. The addition of lime to a soil in a sufficient quantity supplies an excess of Ca^{++} which replaces the weaker metallic cations from the exchange complex of the soil. This exchange of cations causes a reduction in the size of the diffused water layer, thereby allowing clay particles to approach each other more closely, or flocculate. In some cases, however, the exchange complex is practically Ca^{++} saturated before the addition of lime. As a result, cation exchange is minimized or does not occur.

Flocculation and agglomeration produce an apparent change in texture, with the clay particles "clumping" together into larger-sized "aggregates". According to Herzog and Mitchell, the flocculation and agglomeration are caused by the increased electrolyte content of the pore water and as a result of ion exchange by the clay to the calcium form.(44) Diamond and Kinter suggested that the rapid formation of calcium-aluminate-hydrate cementing materials is significant in the development of flocculation-agglomeration tendencies in soil-lime mixtures.(45)

The net result of cation exchange and flocculation-agglomeration of particles is as follows:(37)

- Substantial reduction and stabilization of the adsorbed water layer.

- Increased internal friction among the agglomerates and greater aggregate shear strength.
- Much greater workability due to the textural change from a plastic clay to a friable, sand-like material.

Soil-Lime Pozzolanic Reaction

The reactions between lime, water, soil silica, and alumina to form various cementing-type materials are referred to as soil-lime pozzolanic reactions. The cementing products are primarily calcium-silicate-hydrates and calcium-aluminate-hydrates, the same hydrates formed during the hydration of portland cement. Although a wide variety of hydrate forms can be obtained, the basic pozzolanic reaction is illustrated in the following equations:

$$Ca(OH)_2 \rightarrow Ca^{++} + 2(OH)^-$$
$$Ca^{++} + OH^- + SiO_2 \text{ (soluble clay silica)} \rightarrow \text{calcium-silicate-hydrate (CSH)}$$
$$Ca^{++} + OH^- + Al_2O_3 \text{ (soluble clay alumina)} \rightarrow \text{calcium-aluminate-hydrate (CAH)}$$

Possible sources of silica and alumina in typical fine-grained soils include clay minerals, quartz, feldspars, micas, and other similar silicate or alumino-silicate minerals, either crystalline or amorphous in nature. The clay minerals and amorphous materials are the only important sources in most soils.

When a significant quantity of lime is added to a soil, the pH of the soil-lime mixture is elevated to approximately 12.4, the pH of saturated lime water. This is a substantial pH increase for natural soils. The solubilities of silica and alumina are greatly increased at these elevated pH levels.(45) Thus, as long as enough residual calcium from the lime remains in the system and the pH remains high enough to maintain solubility, the pozzolanic reaction will continue.(37)

The extent to which the soil-lime pozzolanic reaction proceeds is influenced primarily by natural soil properties. With some soils, the pozzolanic reaction is inhibited, and cementing agents are not extensively formed. Thompson has termed "reactive" those soils that react with lime to produce substantial strength increase.(46) A strength increase of greater than 50 psi (345 kPa^2) after a 28-day curing period at 73 °F is considered reactive. Soils displaying less than a 50 psi (345 kPa^2) strength increase are deemed non-reactive. In such cases, extensive pozzolanic strength development will not be achieved, regardless of lime type, lime percentage, or curing conditions of time and temperature.

Several soil properties and characteristics influence the lime-reactivity (i.e., the ability of the soil to react with lime to produce cementitious materials) of a soil. These include:

- Soil pH.
- Organic carbon content.
- Natural drainage.
- Excessive quantities of exchangeable sodium.
- Clay mineralogy.
- Degree of weathering.
- Presence of carbonates.
- Extractable iron.
- Silica-sesquioxide ratio.
- Silica-alumina ratio.

Detailed summaries concerning the effects of soil properties on lime reactivity are contained in references 46, 47, and 48.

Carbonation

Lime carbonation is an undesirable reaction which may also occur in soil-lime mixtures. In this reaction, lime reacts with carbon dioxide to form calcium carbonate, as shown below.

$$CaO + CO_2 \rightarrow CaCO_3$$

Prudent construction practices are necessary to minimize lime carbonation. Actions to be avoided are long exposure of the lime to air prior to mixing with the soil, and long, intensive mixing and processing times. It is recommended that prior to mellowing, the mixture be compacted using a pneumatic roller. Design considerations for location of subdrains should be carefully considered, as with other cementitious materials, migration of the stabilizer into the drain system can develop carbonation which may clog drainage systems and compound soil moisture problems.

4. SOILS SUITABLE FOR LIME STABILIZATION

Since the beneficial effects of lime stabilization are the result of various reactions between the fines portion of the soil and lime, fine-grained soils, such as clay and silty-clay, respond most favorably. A minimum clay content of approximately 10 percent and a plasticity index greater than 10 are desirable, although benefits have been noted for lower PI silty soils containing less clay.

For low PI sands and non-plastic soils, a pozzolan additive is needed to produce the necessary lime-silica reaction. Fly ash, volcanic ash, and expanded shale fines are examples of pozzolans that have been successfully incorporated. Suitable stabilization of organic soils may be difficult with normal lime contents.

5. TYPICAL PROPERTIES OF LIME STABILIZED SOILS

In general, all fine-grained soils exhibit decreased plasticity, improved workability, and reduced volume change characteristics when mixed with lime; however, not all soils exhibit improved strength, stress-strain, and fatigue characteristics. It should be emphasized that the properties of soil-lime mixtures are dependent on many variables, the most important of which are soil type, lime type, lime percentage, compacted density, and curing conditions (time, temperature, and moisture). The properties of a lime-treated soil are, therefore, not "static values" but will vary in response to changes in the variables listed above.

The effects of lime treatment on pertinent soil properties can be classified as immediate and long-term. Immediate effects are achieved without curing and are of interest primarily during the construction stage. They are attributed to cation exchange and flocculation-agglomeration. Long-term effects take place during and after curing and are important from a strength and durability standpoint. While these effects are generated to an extent by cation exchange and flocculation-agglomeration, they are primarily a result of pozzolanic strength gain. Consideration for the properties affected by lime treatment are provided herein.

Uncured Mixtures

Plasticity and Workability

The addition of lime to a soil decreases the liquid limit and increases the plastic limit, resulting in a substantial reduction of the PI. In some cases, the soil may become nonplastic. Generally, high initial PI and clay content soils require greater quantities of lime to achieve the nonplastic condition, if it can be achieved at all.

The first increments of lime are usually most effective in reducing the plasticity of the soil. The silty and friable texture of the treated soil causes a marked increase in workability, expediting subsequent manipulation and placement of the treated soil. Figure 18 illustrates the manner in which lime influences plasticity.

Moisture-Density Relations

For a given compactive effort, soil-lime mixtures have a lower maximum dry density and a higher optimum moisture content than the untreated soil. This phenomenon normally persists with the incorporation of additional lime. Maximum dry density reductions of 3 to 5 lb/ft^3 (48.1 to 80.1 kg/m^3) and optimum water content increases of 2 to 4 percent are common. Further reductions in maximum dry density and increases in optimum moisture content can be expected if the mixture is allowed to cure so that substantial cementing occurs.

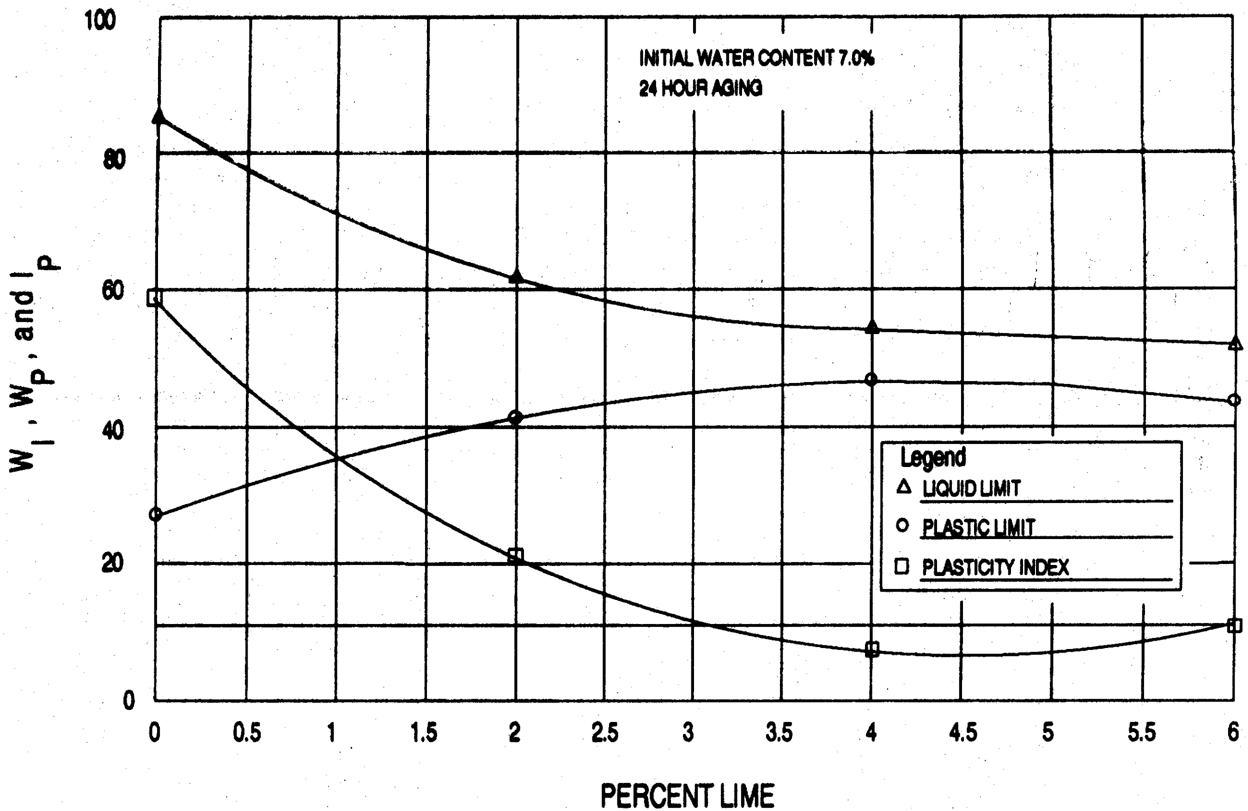

Figure 18. Effects of lime on liquid limit, plastic limit, and plasticity index for clay soil.(49)

Since moisture-density relationships are constantly changing, it is important that the appropriate moisture-density curve is utilized for field control purposes. If curing has occurred, it may be impossible to achieve density; however, it is important to realize that it is not necessary to achieve that density because the reduction is not due to poor compaction but to the fact that the material is different.(50) Figure 19 illustrates the effects of compaction effort, lime content, and aging on the dry density of a clay soil.

Swell Potential

Swelling potential and swelling pressures are reduced markedly by lime treatment. These reduced swelling characteristics are primarily attributed to the decreased water affinity of the calcium-saturated clay and the formation of a cementitious matrix which can resist volumetric expansion. CBR swell values of lime-treated soils vary, but it is not uncommon to decrease swell to less than 0.1 percent.(51) Mitchell and Raad, in considering additive treatments for swell control, concluded that lime continues to be the most effective additive for stabilization of expansive soils.(52)

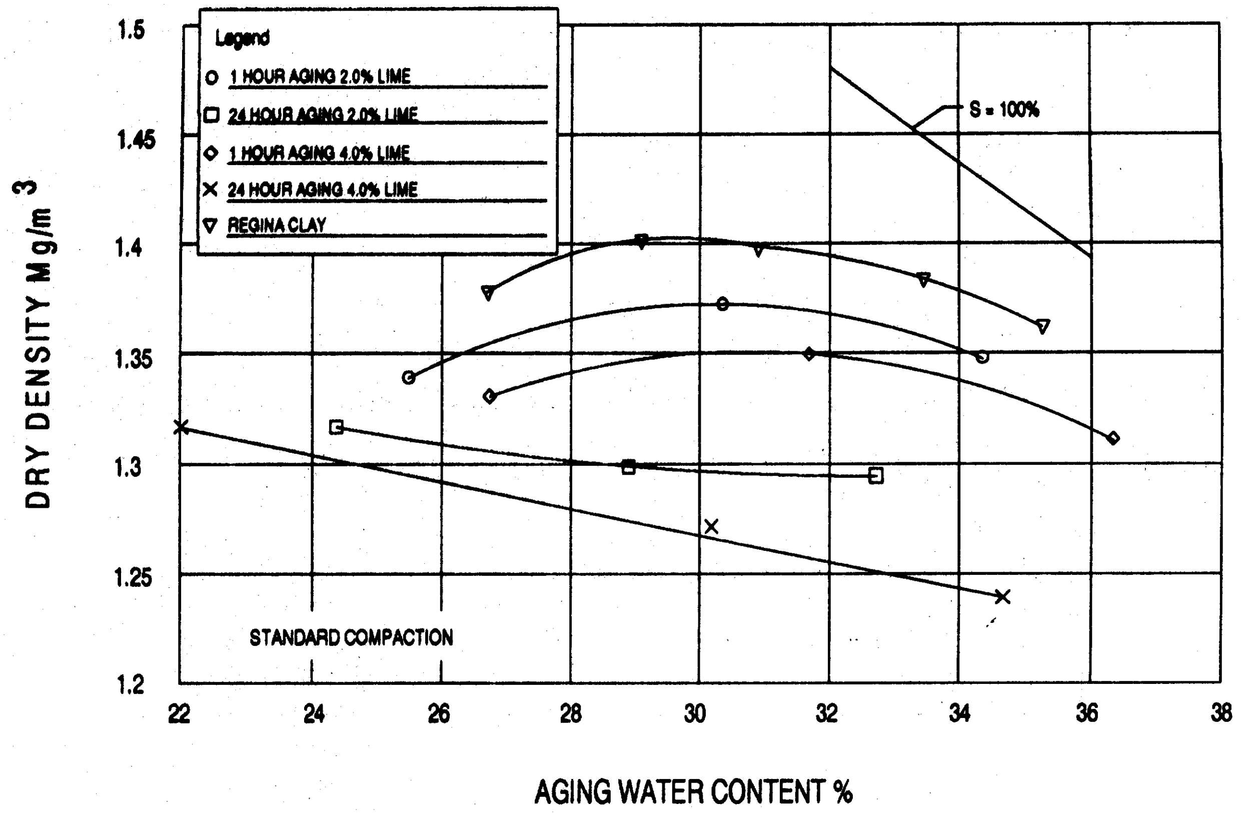

Figure 19. Effects of compaction effort, lime content, and aging on dry density of clay soil.[49]

Strength and Deformation Properties

Immediate increases in the strength and deformation properties of soils are apparent with the incorporation of lime. These improvements are a result of the textural changes produced by cation exchange and flocculation-agglomeration. They can be characterized in terms of shear strength, CBR, cone index, static-compression modulus of elasticity, and resilient modulus. Figure 20 illustrates the increase in CBR of a low plasticity clay treated with varying amounts of lime. The immediate effects of lime treatment on the resilient behavior of fine-grained soils can be seen in figure 21. Chapter 3 should be consulted for laboratory test procedures to evaluate the effect of the additive.

It is apparent that the immediate strengthening effects of lime treatment are substantial. These immediate effects often prove beneficial as they help to provide a stable working platform for pavement construction operations. As curing progresses and the soil-lime pozzolanic reaction proceeds, the soil-lime mixture will develop much higher levels of strength and stiffness characteristics.

Cured Mixtures

Strength/Deformation Properties

Unconfined Compression

The unconfined compression test is a simple and effective means of evaluating the strength properties of treated soils. Although the strength increase of a lime-treated soil depends considerably on the type and percentage of lime used and the curing period, the primary factor influencing lime-soil reactions is the soil type. If a soil is nonreactive, substantial strength cannot be developed.

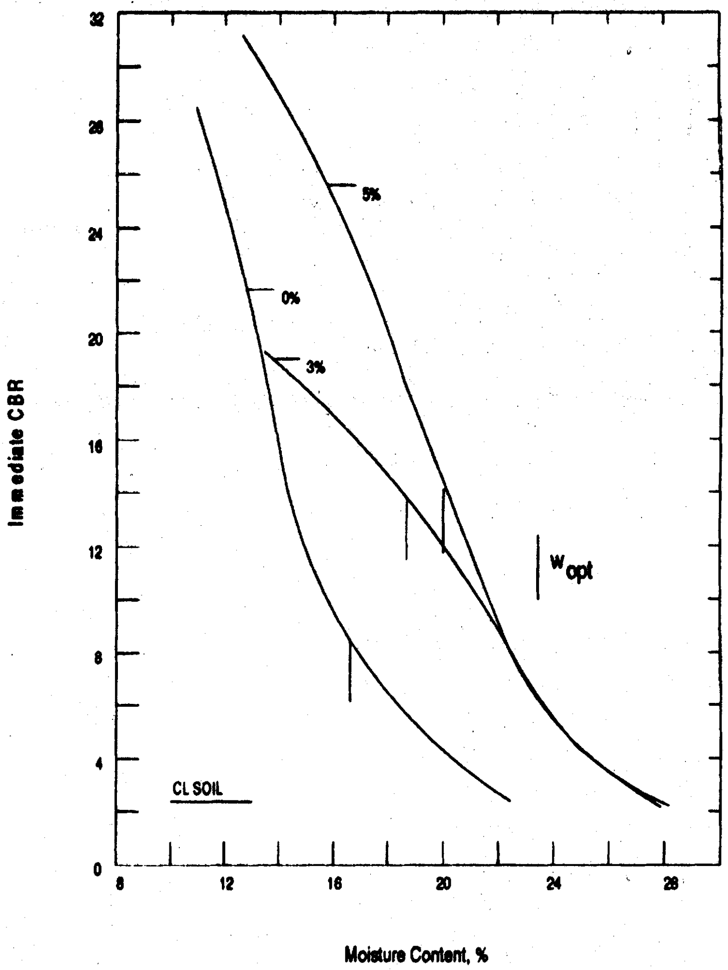

Figure 20. CBR-moisture content relations for natural and lime-treated (3%,5%) CL soil (AASHTO T-99 compaction).(53)

Soil-lime mixture strength increases for 39 Illinois soils cured 28 days at 73 °F (23 °C) varied considerably.(54) Nonreactive soils showed little or no compressive strength gain while some soils displayed strength increases in excess of 265 psi (1724 kPa). A majority of the soils experienced strength increases greater than 100 psi (690 kPa) and extended curing of the same mixtures (56 days at 73 °F [23 °C]) produced strength increases for soil-lime combinations that exceeded 625 psi (4310 kPa).

Extensive California test data indicate a wide range of strength increases for 5 percent lime treatment and 6-month curing (ambient lab temperature, sealed curing to preserve moisture content).(55) The maximum strength increase achieved was 770 psi (5300 kPa), and 30 of the 41 soils evaluated developed compressive strength increases in excess of 100 psi (690 kPa). Field data indicate that with some soil-lime mixtures, strength continues to increase for up to ten years or more.

The difference between the compressive strength of the natural and lime-treated soils is an indication of the degree to which the soil-lime-pozzolan reaction has proceeded.(56) A substantial strength increase indicates that the soil is reactive with lime and can be stabilized to produce a quality paving material.

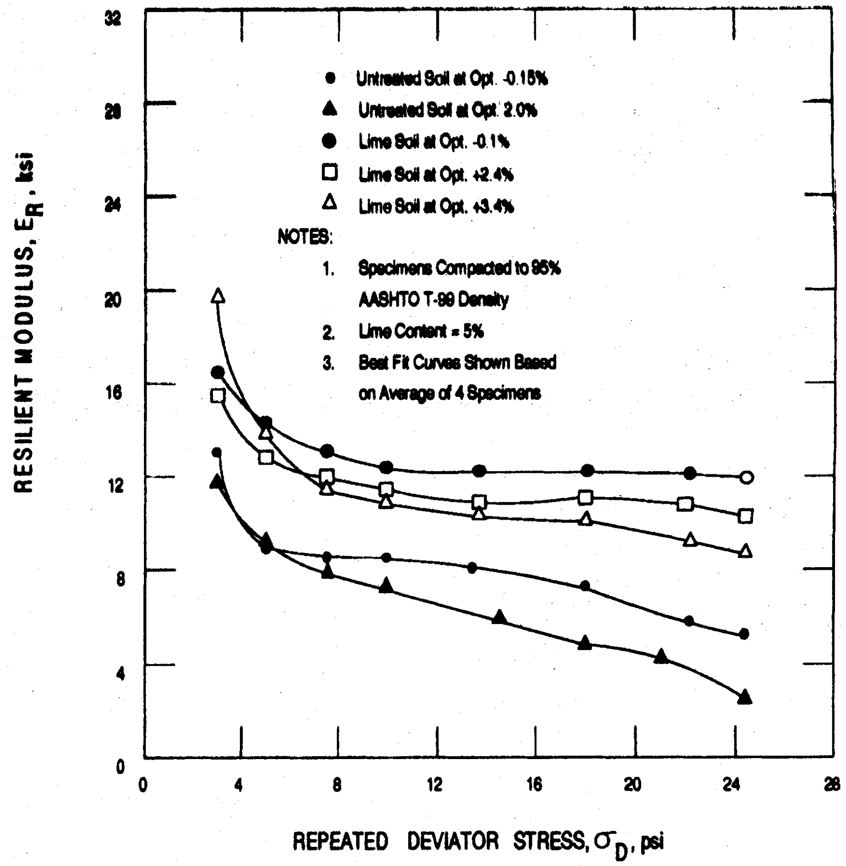

Figure 21. Effect of lime treatment and variable compaction moisture on resilient response of Flanagan B soil.(89)

Shear Strength

The major effect of lime on the shear strength of a reactive fine-grained soil, as determined by triaxial testing, is to produce a substantial increase in cohesion with some minor increase in friction angle (ϕ). This increase in cohesion largely reflects the amount of cementing which occurs in lime-reactive soils. In addition, the increase is of greatest significance at the low confining pressures normally considered to exist in a flexible pavement structure.

For typical lime-reactive Illinois soils, the friction angle for cured soil-lime mixtures ranged from 25° to 35°.(57) The cohesion of the mixtures was substantially increased compared to the natural soils, and cohesion continued to increase with increased compressive strength. A linear regression equation was developed from these observations:

$$C = 9.3 + 0.292 \cdot q_u$$

where:

C = cohesion, psi
q_u = unconfined compressive strength, psi

It is apparent that large shear strengths can easily be developed in cured soil-lime mixtures. It has been demonstrated that if high-quality mixtures are used in typical flexible pavement structures, the strengths would be adequate to prevent shear failure.(57) Shear-type failures generally have not been observed and reported for field service conditions.

Tensile Strength

Tensile strength properties of soil-lime mixtures are of concern in pavement design because of the slab action that is afforded by a material possessing substantial tensile strength. Although both the split-tensile test and the flexural test are used to evaluate the tensile strengths of soil-lime mixtures, the flexural test is most commonly used.

As with unconfined compressive strength, split-tensile strength depends upon the soil-lime mixture and curing conditions. The similarity between the two parameters has been documented in at least two studies. Thompson estimated a ratio of 0.13 between split-tensile strength and unconfined compressive strength.(58) Tulloch et. al. found the ratio to be considerably lower, as illustrated below.(59)

$$\sigma_t = 6.89 + 0.506 \cdot q_u$$

where:

σ_t = split tensile strength, psi
q_u = unconfined compression strength, psi

Typical flexural strengths and corresponding split-tensile strengths of soil-lime mixtures subjected to various curing conditions are given in table 11.(51) As can be seen, for a particular mixture, the ratio of flexural strength to split-tensile strength decreases as strength increases. This ratio is different for each soil-lime mixture.

A realistic estimate of 25 percent for flexural strength can be achieved with the assumption that the ratio of flexural strength to split-tensile strength is 2.0 and, that the ratio of split-tensile strength to unconfined compressive strength, as given by Thompson, is 0.13.

California Bearing Ratio (CBR)

CBR testing procedures have been used extensively to evaluate the strength of lime stabilized soils. Many agencies have arbitrarily adopted this technique due to their familiarity with the test. In reality, however, the CBR is inappropriate for characterizing the strength of cured soil-lime mixtures.

Thompson conducted extensive CBR tests on various reactive and nonreactive Illinois soils.(51) CBR tests were run on three types of specimens: untreated soil, lime-treated soils cured for 48 hours at 120 °F (48.9 °C), and uncured lime-treated soils placed in a 96-hour soaking cycle immediately after compaction. Test results are provided in table 12.

Table 11. Tensile strength properties of soil-lime mixtures.(59)

Soil	Percentage of Lime	Curing Time,[a] hr	Flexural Strength σ_f, psi	Indirect Tensile Strength σ_t, psi	σ_f/σ_t
Bryce B	5	24	92	42	2.2
		48	105	53	2.0
		96	122	88	1.4
Champaign County Till[b]	3	48	69	-	-
		96	93	-	-
Fayette C	5	24	66	46	1.4
		96	166	126	1.3
Illinoian till, Sangamon County	3	24	86	35	2.5
		48	164	92	1.8
		96	202	106	1.9
Sable B[b]	3	48	63	-	-
		96	77	-	-
Wisconsin loam till	3	24	83	35	2.4
		48	140	63	2.2
		96	157	78	2.0

Note: 1 psi = 6.9 kPa; t,°F = (t,°C/0.55) + 32. [a] At 120 °F. [b] Test not conducted for indirect tensile strength σ_t.

Table 12. CBR values for selected soils and soil-lime mixtures.(51)

Soil	Natural Soil		Percentage of Lime	Soil-Lime Mixtures			
				No Curing[a]		48-hr Curing at 120 °F	
	CBR, %	Swell, %		CBR, %	Swell, %	CBR, %	Swell, %
Reactive Soils							
Accretion Gley 2	2.6	2.1	5	15.1	0.1	351.0	0.0
Accretion Gley 3	3.1	1.4	5	88.1	0.0	370.0	0.1
Bryce B	1.4	5.6	3	20.3	0.2	197.0	0.0
Champaign Co. till	6.8	0.2	3	10.4	0.5	85.0	0.1
Cisne B	2.1	0.1	5	14.5	0.1	150.0	0.1
Cowden B	7.2	1.4	3	-	-	98.5	0.0
Cowden B	4.0	2.9	5	13.9	0.1	116.0	0.1
Cowden C	4.5	0.8	3	27.4	0.0	243.0	0.0
Darwin B	1.1	8.8	5	7.7	1.9	13.6	0.1
East St. Louis clay	1.3	7.4	5	5.6	2.0	17.3	0.1
Fayette C	1.3	0.0	5	32.4	0.0	295.0	0.1
Illinoian B	1.5	1.8	3	29.0	0.0	274.0	0.0
Illinoian till	11.8	0.3	3	24.2	0.1	193.0	0.0
Illinoian till	5.9	0.3	3	18.0	0.9	213.0	0.1
Sable B	1.8	4.2	3	15.9	0.2	127.0	0.0
Nonreactive Soils							
Fayette B	4.3	1.1	3	10.5	0.0	39.0	0.0
Miami B	2.9	0.8	3	12.7	0.0	14.5	0.0
Tama B	2.6	2.0	3	4.5	0.2	9.9	0.1

[a] Specimens were placed in 96-hr soak immediately after compaction.

Strength improvements are apparent with both uncured and cured soil-lime specimens. The improvements in engineering properties of the uncured mixtures are primarily a result of cation exchange and flocculation-agglomeration. While strength gains are moderate, it is evident that benefits can be realized without prolonged curing.

The tremendous increase in CBR values for cured specimens reflects the extensive development of pozzolanic cementing agents. For those mixtures that display CBR values of 100 or more, it is quite apparent that CBR test results have little practical significance. Thus, the CBR value is only useful as a strength indicator when extensive pozzolanic cementing action has <u>not</u> developed, either due to a lack of curing time or nonreactivity of the treated soil.

Fatigue Strength

Flexural fatigue strength is related to the number of loads that can be carried at a given stress level, and it is an important consideration in the evaluation of soil-lime mixtures. Swanson and Thompson illustrated how, for typical highway pavement loading conditions, the flexural strength rather than the shear strength is generally the limiting factor in the application of soil-lime mixtures in base and subbase courses.(60)

Figure 22 shows flexural fatigue response curves for various Illinois soils. The response curves of cured soil-lime mixtures are analogous to those curves normally obtained for materials having similar cementitious products, such as lime-fly ash-aggregate mixtures and portland cement concrete. The allowable stress in the pavement layer under loading to produce a fatigue life of 5 million stress repetitions of the lime-soil mixtures varied from 41 to 66 percent of the ultimate flexural strength, with an average of 54 percent.

Soil-lime mixtures continue to gain strength with time, and the ultimate strength of the mixture is a function of curing period and temperature. The magnitudes of the flexural stress repetitions applied to the mixture, however, are relatively constant throughout its design life. Therefore, as the ultimate strength of the material increases due to curing, the stress level, as a percent of ultimate strength, will decrease and the fatigue life of the mixture will increase.

<u>Deformation Properties</u>

Stress-strain properties are essential for properly analyzing the behavioral characteristics of a pavement structure containing a soil-lime mixture structural layer. The marked effect of lime on the compressive stress-strain properties of fine-grained soils is shown in figure 23.(61) The failure stress is increased, and the ultimate strain is decreased for soil-lime mixtures relative to the natural soil.

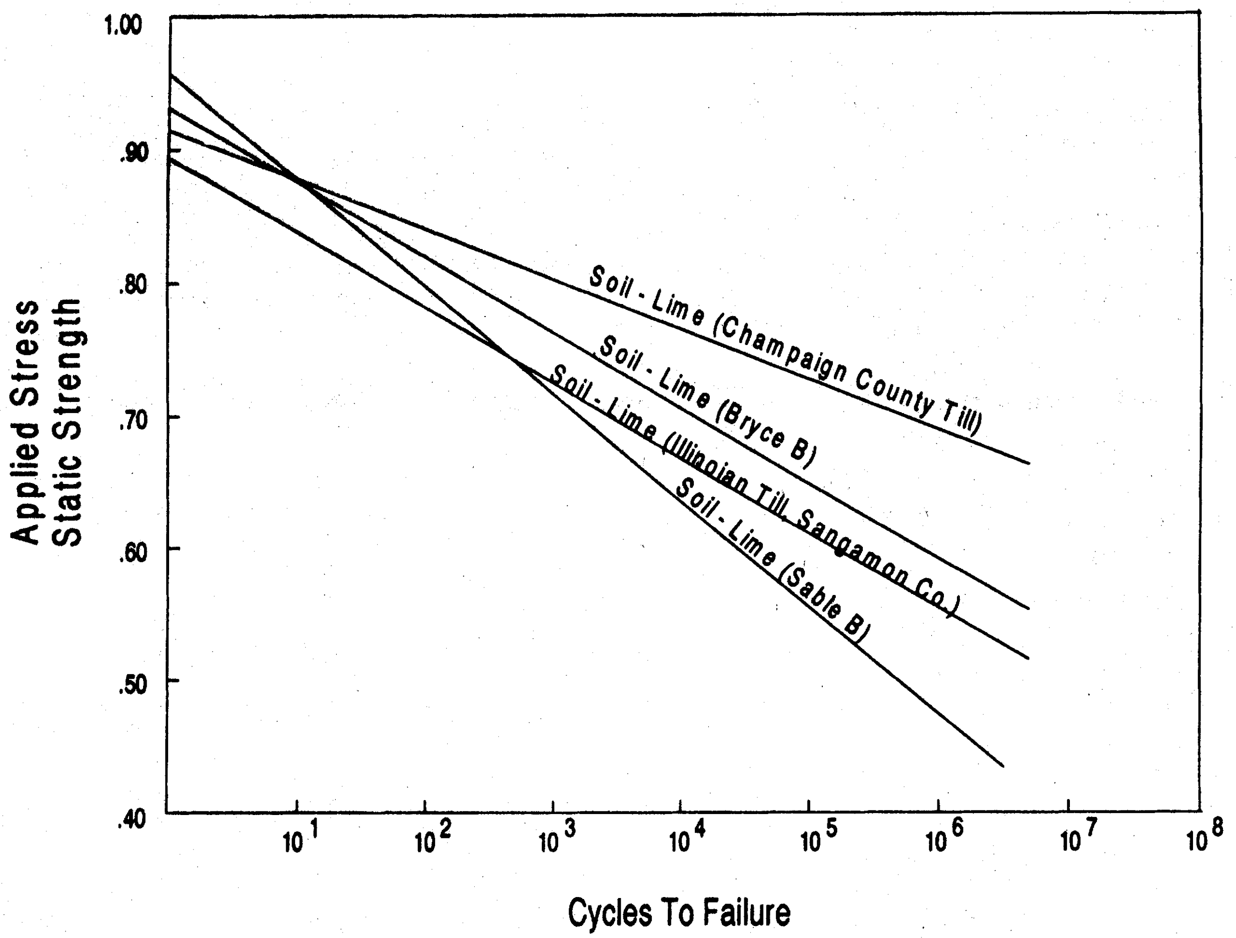

Figure 22. Flexural fatigue response curves.(60)

An extensive study involving the triaxial compression testing of various lime-stabilized Illinois soils provided two significant findings.(57) First, it was found that lime-soil mixtures are strain sensitive, having ultimate strains of approximately 1 percent at maximum compressive stress. Secondly, it was found that the compressive modulus of elasticity at a confining pressure of 15 psi (103.4 kPa) can be estimated from the unconfined compressive strength of the soil-lime mixture according to the following equation:

$$E = 9.98 + 0.124 \cdot q_u$$

where:

E = compressive modulus of elasticity, ksi

q_u = unconfined compressive strength, psi

Repeated or dynamic compressive loading data for soil-lime mixtures are being developed with the resilient modulus procedures now being implemented, but not a great deal of information is currently available. Suddath and Thompson evaluated the resilient moduli for a 4 percent lime-treated Goose Lake Clay mixture, a highly lime reactive soil, following curing periods of 2, 7, 14, and 28 days.[61] The cured static compressive strengths ranged from about 125 to 250 psi (862 to 1724 kPa). However, the resilient moduli for repeated compressive stresses equal to approximately 50 percent of the mixture's compressive strength varied only from approximately 70,000 to 125,000 psi (483,000 to 862,000 kPa). In contrast, static modulus of deformation data for the same mixtures were substantially lower and varied from approximately 16,000 to 45,000 psi (110,000 to 310,000 kPa). The actual amount of strength increase provided will depend entirely on the reactive nautre of the soil.

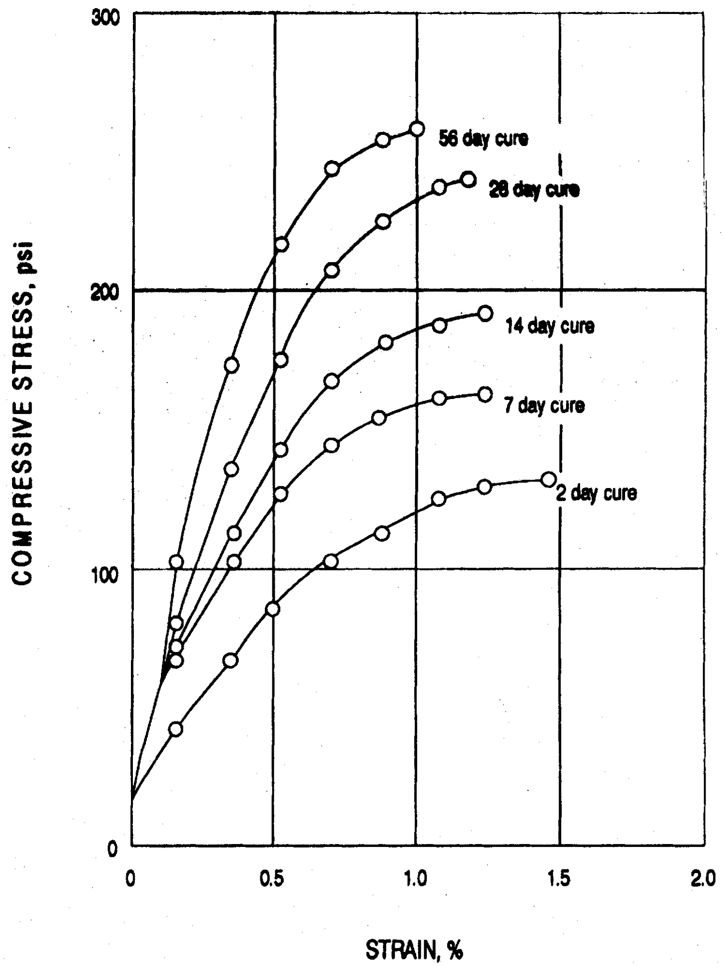

Figure 23. Compressive stress-strain relations for cured soil-lime mixtures (goose lake clay + 4% lime).[61]

Flexural Moduli

For soil-lime pavement layers possessing high shear strength, the flexural stresses in the mixture may be the controlling design factor. In view of this fact, flexural moduli of elasticity have been evaluated for typical cured soil-lime mixtures.[51] Various fine-grained soils stabilized with lime were made into 2 x 2 x 9 in (5.08 x 5.08 x 22.86 mm) beams and cured for 2 and 4 days at 120 °F (48.9 °C). With the use of strain gauges, the beams were then tested under third-point loading conditions. The flexural modulus of elasticity was calculated from the moment-curvature relationships for the beams. Figure 24 shows the relationship generated between the flexural strength and the flexural modulus of elasticity. Obviously, as the flexural strength increases with curing time, so does the modulus of elasticity.

Poisson's Ratio

Only limited data are available for Poisson's ratio of lime-soil mixtures. Reported values at stress levels below 25 percent of ultimate compressive strength ranged from 0.08 to 0.12, with an average of 0.11.(51) At stress levels between 50 and 75 percent of ultimate compressive strength, the ratio ranged from 0.27 to 0.37, with an average of 0.31. A value of 0.15 to 0.20 is quite reasonable for analysis purposes. Figure 25 illustrates the influence of stress level on Poisson's ratio for soil-lime mixtures.

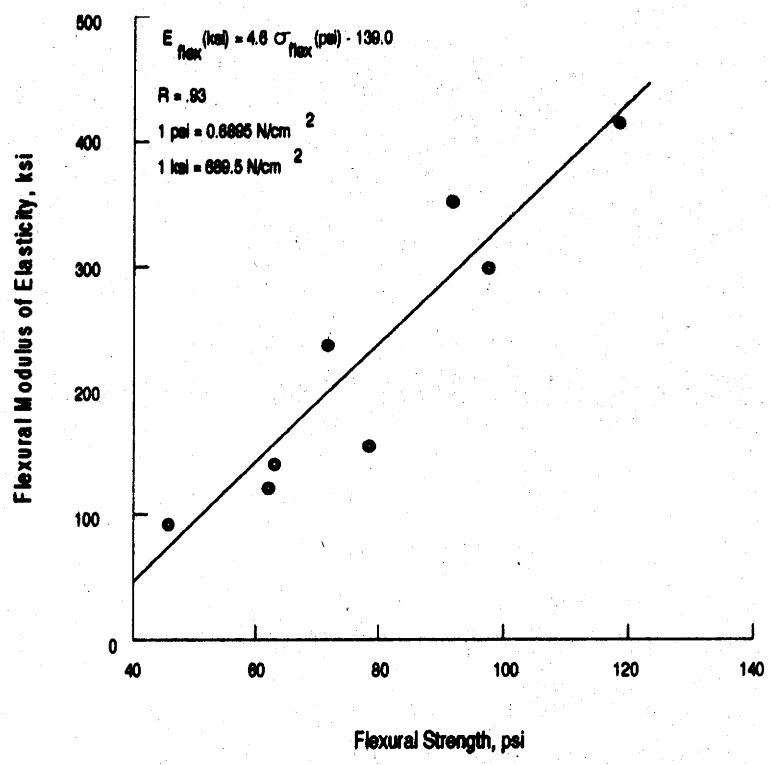

Figure 24. Relationship between flexural strength and flexural modulus for soil-lime mixtures.(51)

Shrinkage

Shrinkage associated with the loss of moisture from the stabilized soil is of importance relative to the problem of "shrinkage cracking" of the materials and reflective cracking through overlying paving layers. A study conducted by Dempsey and Thompson on four Illinois soils indicated that lime treatment decreased shrinkage potential.(61) Field moisture content data for lime-treated soils suggest that the moisture content changes in the stabilized material are not large and the in-situ water content stabilizes at approximately optimum.

Theoretical calculations based on laboratory shrinkage data as well as field service data from many areas indicate that for typical field service conditions shrinkage of cured soil-lime mixture will not be extensive. A comprehensive California study of the field performance of lime-treated roadways showed that only 20 percent of the pavements with soil-lime bases displayed evidence of shrinkage cracking, ultimately reflecting through the surface course.(62)

Durability

Durability characteristics are an important consideration in the evaluation of a paving material. This is particularly true with low-volume roads where the effects of environment (temperature and moisture) may be more pronounced due to the reduced thickness of base and subbase layers and the use of either thin surface courses or surface treatments.

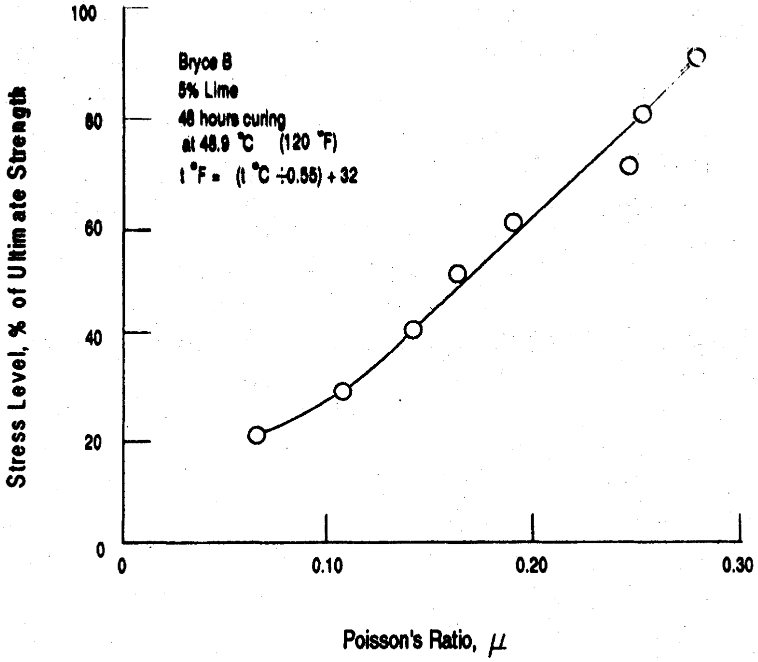

Figure 25. Influence of stress level on Poisson's ratio.(51)

Prolonged exposure to water produces only slight detrimental effects. An Illinois study found that the ratio of soaked to unsoaked compressive strength of soil-lime mixtures is quite high, at approximately 0.7 to 0.85.(63) The soaked soil-lime specimens in the study seldom achieved 100 percent saturation and, in most cases, the degree of saturation was in the range of 90 to 95 percent. Similar response to soaking has been noted in extensive studies conducted by the Road Research Laboratory, United Kingdom.(64)

Pavement systems may experience two general types of freeze-thaw action. Cyclic freeze-thaw takes place in the material when freezing occurs as the advancing frost line moves through the layer and then thawing subsequently occurs. Heaving conditions develop when a quasi-equilibrium frost line condition is established in the stabilized material layer. The static frost line situation provides favorable conditions for moisture migration and subsequent ice lens formation and heaving, if the material is frost susceptible.

Depending on the nature of the prevailing climate in an area, either cyclic freeze-thaw or heaving action or both may occur. Extensive field pavement temperature data or heat flow model studies are required to accurately characterize the field temperature environment. Thompson and Dempsey demonstrated that the most expeditious way of characterizing the field temperature environment is through the use of a theoretical heat flow model.(65) Their data indicated that for Illinois the dominate frost action form was cyclic freeze-thaw and not heaving.

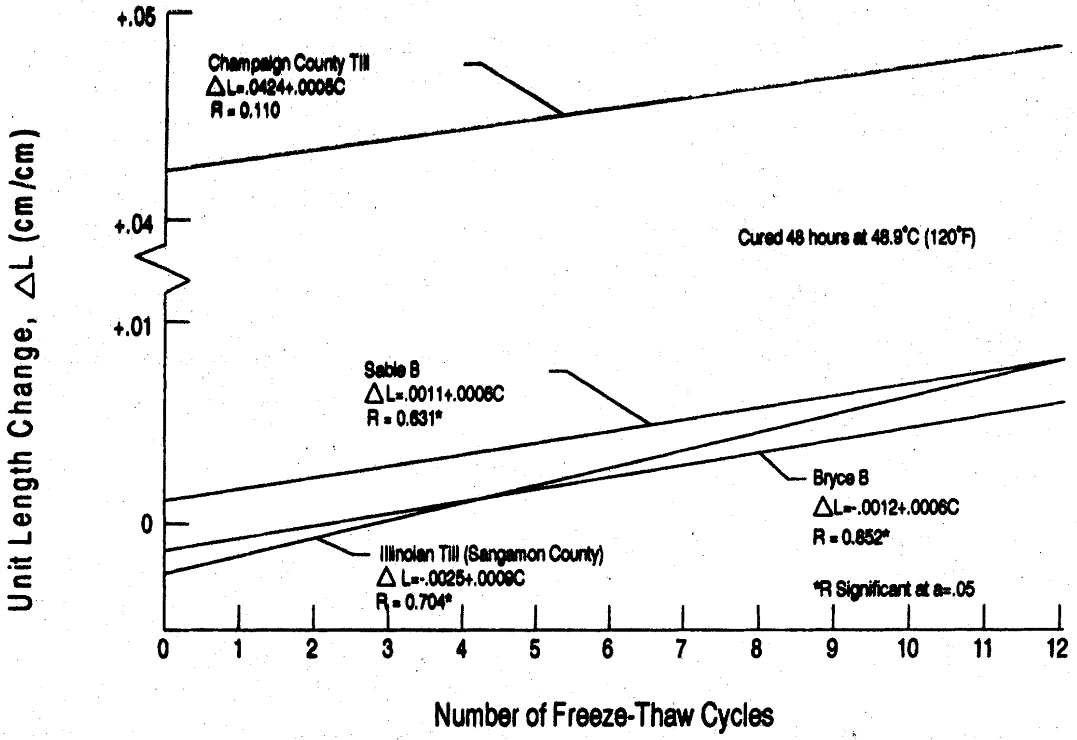

Figure 26. Influence of freeze-thaw cycles on unit length change (48-hour curing).(66)

Cyclic Freeze-Thaw

In zones where freezing temperatures occur, freeze-thaw damage may result. This damage is generally characterized by volume increase and strength reduction, as shown in figures 26 and 27.(66)

The initial unconfined compressive strength (0 freeze-thaw cycles) is a good indicator of freeze-thaw resistance. Dempsey and Thompson observed average strength loss rates of 9.4 psi/cycle for 48-hour cured specimens and 18.5 psi/cycle for 96-hour cured specimens.(66) While cyclic freeze-thaw reduces the strength of highly-cured soil-lime mixtures at a greater rate, it is much more desirable to have obtained a high strength, stabilized mixture prior to the first freeze-thaw cycle.

It has been shown that some soil-lime mixtures display autogenous healing properties.(67) If the stabilized soil has the ability to regain strength or "heal" with time, the distress produced during winter freeze-thaw cycles will not be cumulative,

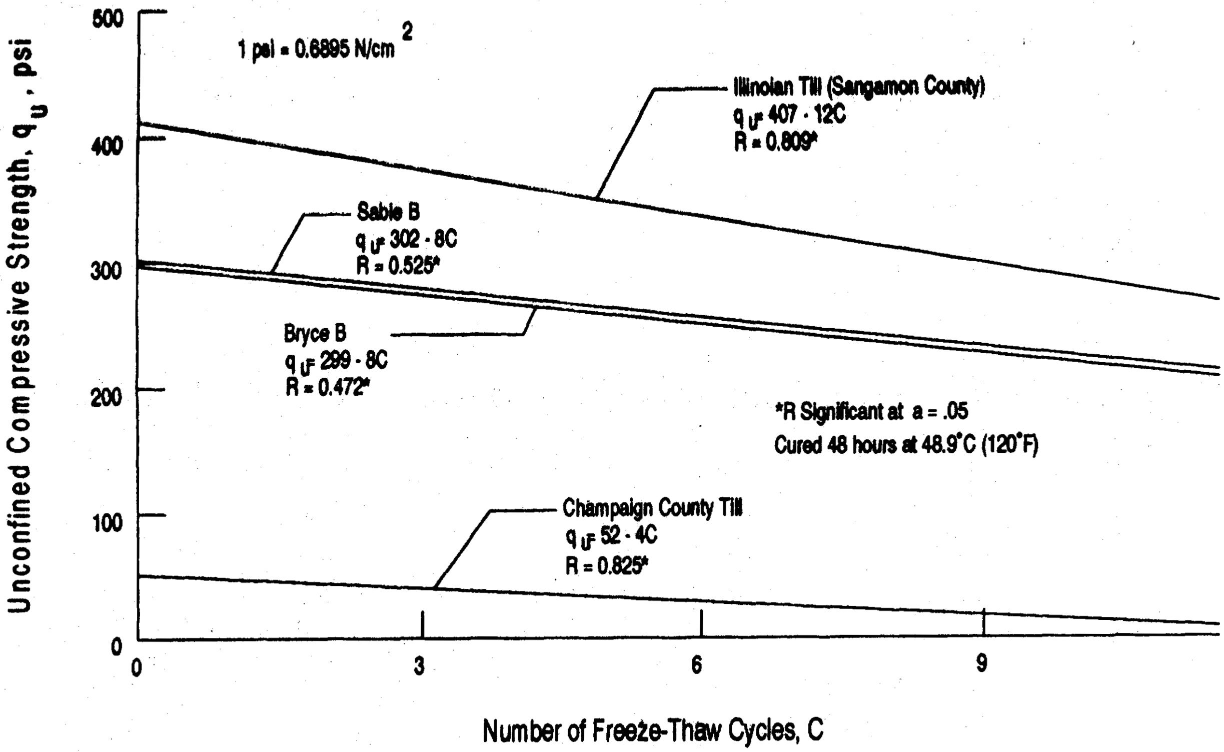

Figure 27. Influence of freeze-thaw cycles on unconfined compressive strength (48-hour curing).(66)

since autogenous healing during favorable curing conditions would serve to restore the stability of the material. This phenomenon has been investigated and confirmed by McDonald.(68)

Frost Heaving Action

Little consideration has been given in the past to the heaving resistance of soil-lime mixtures. The general philosophy regarding heaving potential of cemented systems has been based on the assumption that if a certain critical strength level is achieved, the tensile strength of the stabilized materials is sufficient to withstand the heaving pressures generated, thereby limiting the heave potential to tolerable levels.

British experience with the Road Research Laboratory Heave Test, has indicated that cement stabilized materials with a minimum cured compressive strength of 400 psi (2758 kPa) do not heave excessively and are "non-frost-susceptible".(66,69) Many agencies require 7-day cured compressive strengths of 250 to 400 psi (1723 to 2758 kPa) for lime-treated materials, which would supposedly result in "durable" materials with good resistance to heaving.

Sustained freezing of a soil-lime mixture does not cause a strength decrease.(70) It should be noted, though, that the test conditions provided for the whole specimen to be frozen, thus limiting the access of moisture since the frost line completely penetrated the specimen.

In an effort to develop preliminary data regarding the heaving resistance of soil-lime mixtures, several typical Illinois soils were considered in a pilot laboratory testing program.(71) The data indicated that the high strength materials show little or limited heave while the lower strength mixtures may heave excessively. Based on the limited data, a minimum cured strength requirement of approximately 200 psi (1379 kPa) would restrict the potential heave to less than about 2 percent. It should be noted that the British criterion for non-frost-susceptible materials is a heave of less than about 8 percent.(72)

The strength required to prevent excessive heaving of cured soil-lime mixtures, approximately 200 psi (1379 kPa), compares favorably with the strength required to restrict the heave of cement stabilized materials. British Road Research Laboratory studies conclusively demonstrated that if sufficient strength is developed in the cement-stabilized material, the material will not heave excessively (< 8%).(72)

6. SELECTION OF LIME CONTENT

In establishing an appropriate lime content for construction, the primary variable that can be altered is lime percentage, since the inherent properties and characteristics of the soil are fixed. Due to the many varied applications of lime treatment, several mixture design methods have been developed. The majority of these methods follow the general principle that the mixture provide satisfactory performance given a particular position in the pavement structure. It is apparent that a wide range of soil-lime mixtures can be successfully used to accomplish differing lime treatment objectives. Normally, design lime contents are based on an analysis of the effect of various lime percentages on selected engineering properties of the soil-lime mixture. For structural layer applications, cured strength is the most appropriate property to consider.

Mixture design criteria are needed to establish the quantity of lime required to produce an acceptable quality mixture. For some soils and stabilization applications, acceptable soil-lime mixtures may not be produced regardless of the lime percentage used.

Approximate Quantities

Most fine-grained soils can be effectively stabilized (relative to a predetermined stabilization objective) with 3 percent lime (dry weight of soil basis).

Under normal field construction conditions, 2 to 3 percent lime is the minimum quantity that can be effectively distributed and mixed with a fine-grained soil.

A quick test for determining lime content has been developed by Eades and Grim and is suggested for use when detailed testing cannot be performed.(73) In this procedure, the pH of soil-lime mixtures containing various lime percentages are measured with a pH meter. The idea is to add sufficient lime to ensure a pH of 12.4 for sustaining the strength-producing, soil-lime pozzolanic reaction. The technique does not evaluate mixture quality in terms of strength and no indications are given as to the lime-reactivity of a soil.

Mixture Design Methods and Criteria

Mixture design criteria are needed to evaluate the adequacy of a given soil-lime mixture. Criteria will vary depending on the stabilization objectives and anticipated field service conditions (i.e., environmental factors, wheel loading considerations, design life, etc.). It is therefore apparent that mixture design criteria may range over a broad scale and should be based on a careful consideration of the specific conditions associated with the stabilization project.

Mixture design criteria can be broken down into two categories according to stabilization objectives. The first category relates to situations where the major objective is soil modification (e.g., PI reduction, improved workability, reduced swell potential, and immediate strength increase). These improvements are essentially brought about by cation exchange and flocculation-agglomeration.

The second set of criteria centers on strength improvement generated by the pozzolanic reaction between the soil and lime. Soil-lime mixtures are often evaluated on their ability to perform well as base or subbase courses; thus, strength and durability are key parameters.

Most current design criteria specify a minimum cured compressive strength and the design lime content is the percentage that produces maximum strength for given curing conditions. Because stress and durability exposure differ for various depths in the pavement structure, strength requirements for base materials are generally higher than for subbase materials. Table 13 lists typical strength requirements specified by a few highway agencies.

A summary of various soil-lime mixture design procedures is provided in Chapter 3. While these procedures do not require a durability test, soil-lime durability should not be overlooked. As discussed previously, durability can be addressed by subjecting soil-lime specimens to moisture and freeze-thaw actions and then conducting laboratory strength tests to determine strength losses.

Table 13. Typical specimen curing and strength requirements.

Agency	Specimen Curing	Minimum Strength Requirements, psi Base Course	 Subbase
California DOT	Mixture is "loose cured" 24 hour prior to compaction.		
Illinois DOT	48 hour @ 120 °F (48.9 °C)	150	100
Louisiana DOT	7 day moist room, 8 hour air drying @ 140 °F (60 °C), 8 hour cooling, and 10 day capillary soaking at a confining pressure of 1 psi (AASHTO T 212)	100	50
Texas State Dept. of Highways and Public Transportation	AASHTO T 220	100	50
Virginia Dept. of Highways and Transportation	72 hour @ 120 °F (48.9 °C)	Not Used	150

Mixture design criteria can be validated only on the basis of actual field performance. Numerous studies (references 62, 68, and 74 through 86) have been conducted which serve as examples of validation activities in widely separated geographic areas.

Mixture design criteria developed for use with a particular mixture design procedure and geographic location must not be applied indiscriminately to other areas. Careful consideration should be given to all aspects of a stabilization problem before adopting any criteria. As a starting point, it is suggested that the test methods and criteria presented in Chapter 3 be utilized.

7. SUMMARY

The fundamental principles of soil-lime reactions, namely cation exchange, flocculation/agglomeration, pozzolanic reaction, and carbonation have been presented to illustrate the nature of the lime reaction with the soil. These reactions are important to the understanding of the function of lime stabilization. Although soil-lime reactions are complex, a basic understanding of them coupled with field experience, should provide adequate technology for successful lime treatment of a large number of soils under a wide variety of conditions. More extensive and detailed background information on these reactions can be found in work by Diamond and Kinter and by Stocker.(87,88)

The principal soils which can be stabilized and improved with lime include the fine grained soils with appreciable clay content to render them lime-reactive as determined in the laboratory. Caution must be observed to ensure that organics are not present which can interfere with the lime reactions.

Since the strength properties, to be used in the pavement design procedure, of a reactive soil-lime mixture change with curing due to the development of additional cementing products, it may not be justified to conduct elaborate tests to precisely evaluate mixture properties that will soon change during field curing. It may be more desirable to use simple tests, such as the unconfined compressive strength or split-tensile strength tests, to evaluate the quality of the mixtures. Other pertinent properties can be estimated utilizing previously developed correlations.

Durable soil-lime mixtures can be obtained when reactive soils are stabilized with quality lime. Although some strength reduction and volume change may occur due to cyclic freeze-thaw during the initial winter following construction, the residual strength of the stabilized materials should be adequate to meet field service requirements, if proper curing and construction are performed. Durability considerations must be taken into account in establishing the mix composition and selecting engineering properties for use in pavement design.

CHAPTER 5 CEMENT STABILIZATION

1. INTRODUCTION

In 1915, a street in Sarasota, Florida, was constructed using a mixture of shells, sand, and portland cement mixed with a plow and compacted. Since then, cement treatment has become one of the most widely used forms of soil stabilization for highways. A 1.5- mi (2.4 km) section of soil-cement base constructed near Johnsonville, South Carolina, is considered the first engineered soil-cement road and it remains in service today.

Large usage of soil-cement for airfield construction during World War II was followed in the 1950's by extensive highway construction, with an annual usage of soil-cement of 50 million yd^2 (41.8 million m^2). Roads were built to high standards, and central plant mixing was introduced. Cement-treated bases have been used in many miles of the Interstate highway system, as well as for streets, low volume roads, and parking areas. Most recently, cement treatment has been used for the recycling and reconstruction of old pavements and for pavements to support heavy industrial vehicles.

Many in-service, cement stabilized soil pavements are 6 in (152 mm) thick. This thickness is generally adequate for secondary roads and residential streets. Soil-cement thicknesses of 7 to 8 in (178 to 203 mm) are widely used for primary roads and high traffic volume secondary roads. Although soil-cement pavements with thicknesses of 9 in (229 mm) or greater have not been common in the past, considerably greater thicknesses are now being used for pavements to support heavy industrial vehicles.

2. TYPES OF CEMENT-AND-SOIL MIXTURES

Cement stabilized soil is soil to which cement and water have been added to improve its natural qualities and make it more stable.(90) Although there are several types of cement stabilized soils, there are primarily two categories associated with highway construction:

- Soil-cement.
- Cement-modified soil.

Soil-cement is a precise mixture of pulverized soil, portland cement, and water which, upon hydration, becomes a hard, durable paving material. It contains sufficient cement (usually greater than 3 percent by weight of soil) to pass standard durability tests and enough moisture for maximum compaction, resulting in a significant strength increase. Soil-cement is occasionally referred to as cement-treated base or cement stabilized aggregate.

Cement-modified soil is an unhardened or semi-hardened mixture of soil and water to which relatively small quantities of cement have been added to improve the chemical and physical properties of the soil and to produce a better paving material. In particular, the addition of cement decreases the liquid limit and increases the plastic limit of the soil such that the plasticity index (PI), volume-change capacity, and water-holding capacity are substantially reduced. Furthermore, bearing values and shear strengths may be somewhat increased, providing a more stable soil.

Cement-modified soil contains less cement than is required to produce soil-cement. As a result, the mixture undergoes limited or no hardening. Typically, cement-modified soil is used as an improved subgrade material while soil-cement is used as a pavement base or subbase material. Although soil-cement is the primary focus here, cement-modified soils will be discussed briefly toward the end of this chapter.

3. TYPES OF PORTLAND CEMENT

Portland cement is an energy-rich anhydrous tricalcium silicate (C_3S)* with excess lime. Approximately 50 percent (by weight) of the cement consists of C_3S while another 25 percent consists of dicalcium silicate (C_2S)*. The remaining 25 percent is primarily composed of calcium aluminates (C_3A and C_4AF)* and calcium sulfate dihydrate (CSH_2)*.

Unhydrated cements contain a range of particle sizes, with an average particle diameter of the order of 10 m (10×10^{-6}m). Although the surface area of portland cement powder is only about 0.3 m^2/gm, the cement gel after hydration has a surface area of about 300 m^2/gm. This large surface area is responsible for the cementing action of cement pastes by adhesion forces to adjacent surfaces. Cement stabilization resembles lime stabilization in many ways, except that with cement, pozzolanic material is present in the cement initially and need not be derived from the soil itself.

Several different cement types have been used successfully for cement stabilization of soils. Normal portland cement (Type I) and air-entraining cement (Type IA) were used extensively in the past and gave about the same results. At the present time, Type II cement has largely replaced the Type I cements, as greater sulfate resistance is obtained for nearly the same cost and rate of strength gain, which may not be important in stabilization. This greater sulfate resistance is achieved by limiting the tricalcium aluminate (C_3A)* content to 8 percent.

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* $C = CaO$, $S = SiO_2$, $H = H_2O$, $A = Al_2O_3$, $F = Fe_2O_3$

In some soils, higher strength can be obtained by using high early strength (Type III) cement. Type III cement contains finer particles and a greater C_3S content which makes for faster hydration and a more rapid strength development. Over the first 24 hours of curing, Type III cement attains nearly twice as much strength as Type I cement.

The presence of alkalis (Na_2O and K_2O) may, from time to time, cause problems with certain aggregates. In such cases where an alkali-aggregate reaction occurs, maximum contents of Na_2O and K_2O may be specified to limit the reactivity. Chemical and physical property specifications for portland cement can be found in ASTM C150. This is not an important consideration in stabilization.

4. SOIL-CEMENT REACTIONS

Cement acts as both a cementing agent and modifier.(90) Hydration of the calcium silicates produces cementitious paste in the form of calcium silicate hydrate (CSH) and free lime in the form of calcium hydroxide (CH).

Calcium silicate hydrate, termed tobermorite, is the predominant cementing compound in hydrated portland cement. In primarily coarse-grained soils, the cement paste bonds soil particles together by surface adhesion forces between the cement gel and particle surfaces.

In fine-grained soils, the clay phase may also contribute to the stabilization through solution in the high pH environment and reaction with the free lime from the cement to form additional calcium silicate hydrate. It is believed that this reaction contributes to the strength of silty clay material constituents, thus accounting for reductions in plasticity and expansion properties. In this manner, the cement acts as a modifier.

The crystalline structure formed by the set cement is mainly extraneous to the soil particles. This structure can be disrupted by subsequent swelling of soil particles or particle groups if an insufficient cement content is used. Disruption of the cement structure can also be caused by certain salt solutions (e.g., sulfates), although some of these salts if present initially may have a beneficial effect.

5. SOILS SUITABLE FOR CEMENT STABILIZATION

A wide range of soil types may be stabilized using portland cement. Yet, cement treatment of granular soils, such as sand and gravel, is generally more effective and economical due to ease in pulverization and mixing and the smaller quantities of required cement. Fine-grained soils of low to medium plasticity can be stabilized as well, but not as effectively as the coarse-grained soils. If the PI exceeds about 30 percent, cement becomes difficult to mix with the soil. Generally, in this case, lime is added first to reduce the PI and improve workability prior to the addition of cement.

A soil may be acid, neutral, or alkaline and still respond well to cement treatment. Although certain types of organic matter, such as undecomposed vegetation, may not influence stabilization adversely, organic compounds, such as nucleic acid and dextrose, act as hydration retarders and reduce strength. When such organics are present they absorb calcium ions from the hydrating cement, resulting in a reduction of pH and precipitation of an alumina-silica gel, and inhibiting the normal hardening process. If the pH of a 10:1 mixture (by weight) of soil and cement 15 minutes after mixing is at least 12.1, it is probable that organics, if present, will not interfere with normal hardening.(91)

Although sulfate attack is known to have an adverse effect on the quality of hardened portland cement concrete, less is known about the sulfate resistance of cement stabilized soils. From two studies(92,93), it is known that the resistance to sulfate attack differs for cement-treated, coarse-grained and fine-grained soils and is a function of sulfate concentrations.

Sulfate-clay reactions can cause deterioration of fine-grained soil-cement.(93) On the other hand, granular soil-cements do not appear susceptible to sulfate attack. In some cases the presence of small amounts of sulfate in the soil at the time of mixing may be beneficial. The use of sulfate-resistant cements may not improve the resistance of clay-bearing soils, but may be effective in granular soil-cements exposed to adjacent soils and/or groundwater containing high sulfate concentrations.

Accordingly, the sulfate content of a soil should be considered in the selection of cement as a stabilizer. Until more definitive criteria are available, the use of cement for fine-grained soils containing more than about 1 percent sulfate should be avoided.

Potable water is normally used for cement stabilization, although sea water has been found to give good results in several cases.

6. TYPICAL PROPERTIES OF SOIL-CEMENT MIXTURES

Soil-cements are easily divided into two groups: granular and fine-grained. Granular soil-cements are made using the coarser-grained cohesionless soil types (i.e., AASHTO A-1, A-2, and A-3 soils, and Unified Soil Classification System (G-) and (S-) soils). Fine-grained soil-cements are made using cohesive soils (i.e., AASHTO A-4, A-5, A-6, and A-7 soils, and Unified Soil Classification System (C-) and (M-) soils).

The properties of soil-cement mixtures are strongly dependent on density, water content, and confining pressure. The development of generalized property relationships is further complicated by the fact that cement content, curing time and conditions, and the deleterious effects of past loadings and weathering are also important. Thus, measurement of properties under one set of conditions may yield data of limited value for other conditions. On the other hand, an easily measured property that can be used to indicate other properties needed for design can be useful. Such a property is the unconfined compressive strength.

In general, for a given cement content, the higher the density, the higher the strength of soil and cement mixtures. Both water content at compaction and compaction method may be important in cohesive soil and cement mixtures.(94)

Compaction Characteristics

The addition of cement to a soil generally causes some change in both the optimum water content and maximum dry density for a given compactive effort. The direction of this change is not usually predictable. The flocculating action of the cement tends to give an increase in optimum water content and a decrease in maximum density; whereas the high specific gravity of the unhydrated cement (3.1) relative to the soil tends to produce a higher density. The gradation of the unhydrated portland cement relative to that of the soil can be important, as it influences the packing of soil particles.

A delay between mixing and compaction leads to a decrease in both density and strength for a fixed compactive effort.(95) If, however, the compactive effort is increased so that the original density is obtained, and provided no significant amount of cement hydration occurs during the delay period, then no strength loss is observed.

Strength

The strengths of soil and cement mixtures may range from less than 10 psi to more than 2000 psi (from 1 to 15,000 kPa), depending on such factors as type of loading, cement content, and curing conditions. In general, the highest strengths are associated with mixtures prepared from cohesionless soils, and the less plastic the soil, the smaller the deformation required to cause failure.

Compressive Strength

The unconfined compressive strength is probably the most widely used measure of the effectiveness of cement treatment. It may be as low as 200 psi (1,400 kPa) for fine-grained soil cements (cement requirement as low as 3 percent). Figure 28 shows that a linear relationship can be used to approximate the relationship

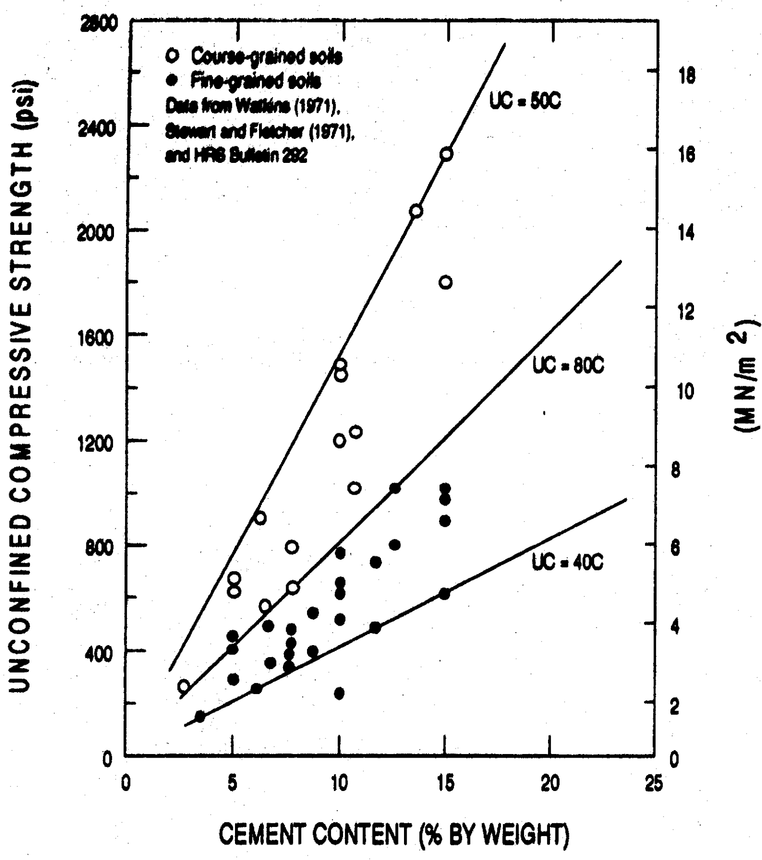


Figure 28. Relation between cement content and unconfined compressive strength for soil and cement mixtures. (Equations give strength in psi)

between compressive strength and cement content, for cement contents up to 15 percent and a curing period of 28 days.

The increase of unconfined compressive strength with curing time for several soil and cement mixtures is shown in figure 29. The relationship between strength and curing time for a given soil and cement mixture can be given by:

$$UC_d = UC_{d_o} + K \cdot \log (d/d_o)$$

where:

UC_d = unconfined compressive strength, psi, at an age of d days

UC_{d_o} = unconfined compressive strength, psi at an age of d_o days

K = 70 °C for granular soils and 10 °C for fine-grained soils

C = cement content in percent by weight

The 28-day strength was found to be 1.7 times the 7-day strength by Dunlap et. al.,(91) and 1.4 times the 7-day strength is suggested by Williams.(96) A value of 1.5 times the 7-day strength would seem a reasonable value for estimating purposes.

Tensile Strength

Flexural beam tests, direct tension tests, and the split tension (Brazilian) test have all been used to evaluate the tensile strength. The results of several studies have indicated that the flexural strength is about 20 to 33 percent of the unconfined compressive strength. Data for some soils are shown in figure 30. In low strength mixtures, the flexural strength is a greater proportion of the compressive strength (up to 33 percent) than in high-strength mixtures (down to less than 20 percent). A good approximation for the flexural strength is:

$$f = 0.51 \cdot (UC)^{0.88}$$

where:
f = flexural strength, psi
UC = unconfined compressive strength, psi

Values of tensile strength deduced from the results of flexure, direct tension, and split tension tests may differ due to the effects of stress concentrations and differences between moduli in tension and compression. Raad et al.(97) have shown that the split tensile test yields values that do not deviate by more than 13 percent from the actual tensile strength. Because of the simplicity of the split tension test, it is suitable for use in practice. In this test, a vertical compressive load is applied to a cylinder of the cemented soil which lies with its longitudinal axis parallel to the surface of the testing machine platens. Failure develops by splitting along the vertical axial plane. The tensile strength is given by:

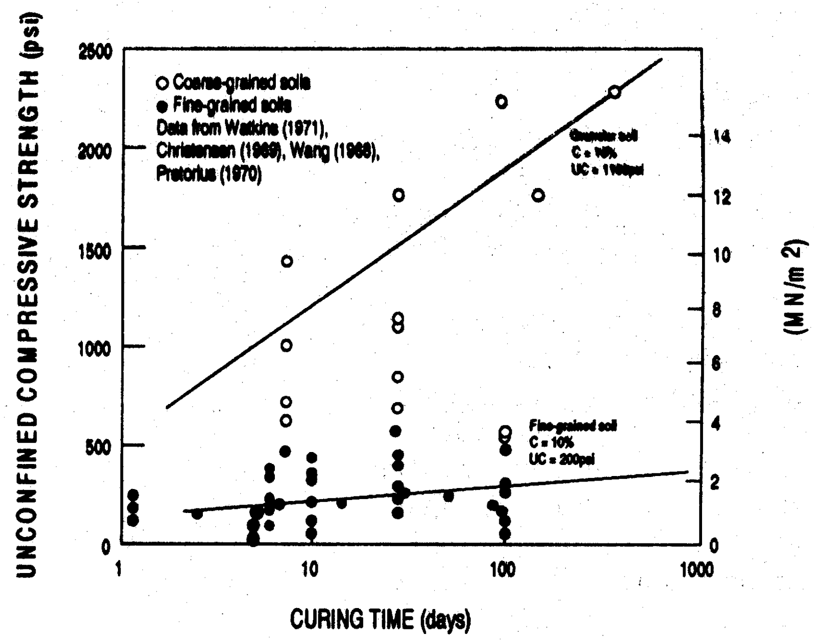


Figure 29. The effect of curing time on the unconfined compressive strength of some soil cement mixtures.

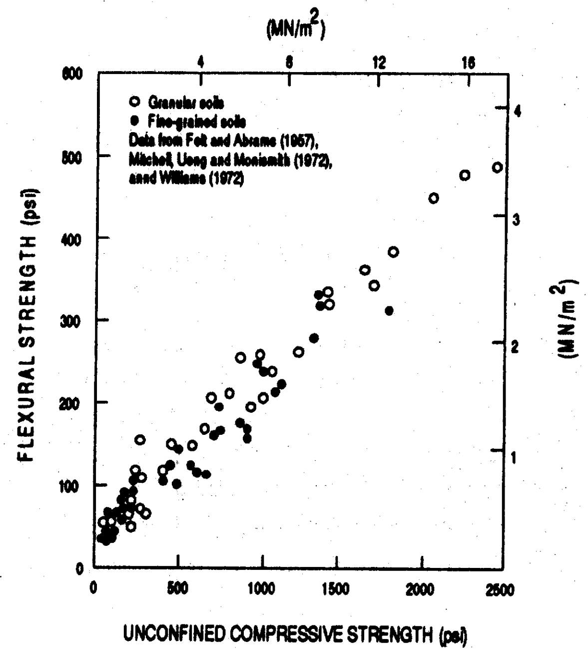


Figure 30. The relation between unconfined compressive strength and flexural strength of soil and cement mixtures.

$$f_t = 2 \cdot P / \pi \cdot D \cdot L$$

where:

f_t = tensile strength, psi
P = load at failure, lb
D = cylinder diameter, in
L = cylinder length, in

Griffith crack theory has been found useful for characterizing the strength of cement-treated soils under various combinations of major (σ_1) and minor (σ_3) principal stresses.(98,99,100) Normalized strength data (failure stresses divided by the unconfined compressive strength) for several soils are summarized in figure 31.

With this figure and a knowledge of the unconfined compressive strength, principal stress combinations causing failure can be estimated directly.

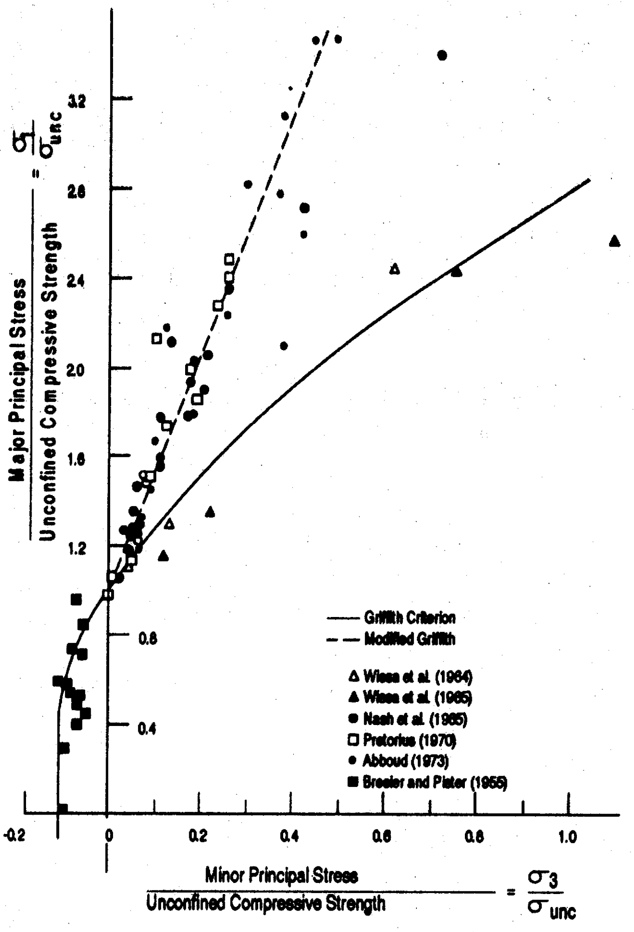


Figure 31. Failure envelope for cement-treated soils.(102)

California Bearing Ratio (CBR)

The relationship between unconfined compressive strength and CBR for some granular and fine-grained soil and cement mixtures is shown in figure 32. The difference between the relationships for fine-grained and granular-treated soils probably results from the uncertainty associated with the application of the CBR test to coarse-grained soils. The meaning of CBR values greater than 100 percent in relation to pavement design and performance is not clear. Accordingly, the high values of CBR in figure 32 can be interpreted as a strength index only.

Deformation Characteristics and Moduli

In general, the stress-deformation behavior of cement stabilized soils is non-linear and stress dependent. However, for many soils and treatment levels, and within limited loading ranges, the material may be assumed to be linearly elastic under repeated loadings.

Deformation moduli may range from about 10,000 psi (70,000 kPa) to several million psi (several GN/m^2), depending on soil type, treatment level, curing time, water content, and test conditions. Cement-treated fine-grained soils have modulus values near the lower end of the range, whereas granular soil-cements exhibit the higher values.

Different relationships between modulus and strength apply to different soil types, as may be seen from the data plotted in figure 33. The values shown were determined from the small strains developed in longitudinal vibration tests, and so the moduli are some 10 to 15 percent higher than would be obtained by static loading tests.(96)

Figure 34 illustrates the general form of stress-strain curves for cement-treated soils in static compression. The shape of such curves can be represented using the hyperbolic relationships proposed by Duncan and Chang.(101)

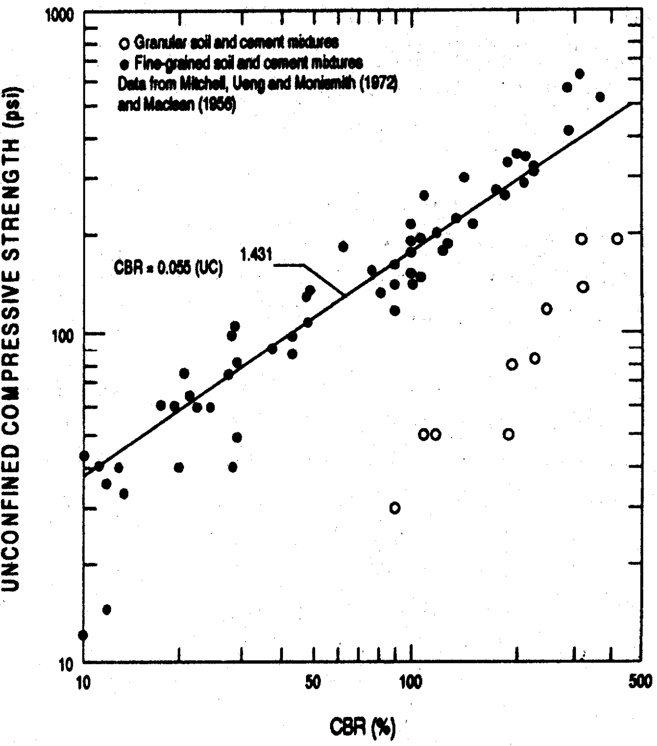


Figure 32. The relation between CBR and the unconfined compressive strength of soil and cement mixtures.

Although the modulus under repeated loading conditions depends on soil type, cement content, compaction and curing conditions, and test type, the unconfined compressive strength, which depends on the same variables, is a useful correlating parameter. Beyond some number of load repetitions, in the range of a few hundred to 10,000, the resilient modulus in compression, M_{rc}, can be expressed by:

$$M_{rc} = K_c \cdot (\sigma_1 - \sigma_3)^{-k_1} \cdot (\sigma_3)^{k_2} \cdot (UC)^n$$

where:

M_{rc} = resilient modulus, psi
UC = unconfined compressive strength, psi

$(\sigma_1 - \sigma_3)$ = deviator stress, psi
σ_3 = confining pressure, psi
K_c = material constant
k_1 = 0.2 to 0.6
k_2 = 0.25 to 0.7
n = $1.0 + 0.18 \cdot C$
C = cement content in percent by weight

Determination of k_1, k_2, and K_c requires separate measurements of M_{rc} under at least two values of σ_3 and two values of $(\sigma_1 - \sigma_3)$, and a regression analysis.

If it is assumed that confining pressure has no effect on resilient modulus in flexure, then, from the results of beam tests:

$$M_{rf} = K_f \cdot (10)^{m \cdot UC}$$

where:

M_{rf} = resilient modulus in flexure, psi
K_f = material constant, from regression of data
UC = unconfined compressive strength, psi
m = $0.04 \cdot (10)^{-0.186 \cdot C}$
C = cement content in percent by weight

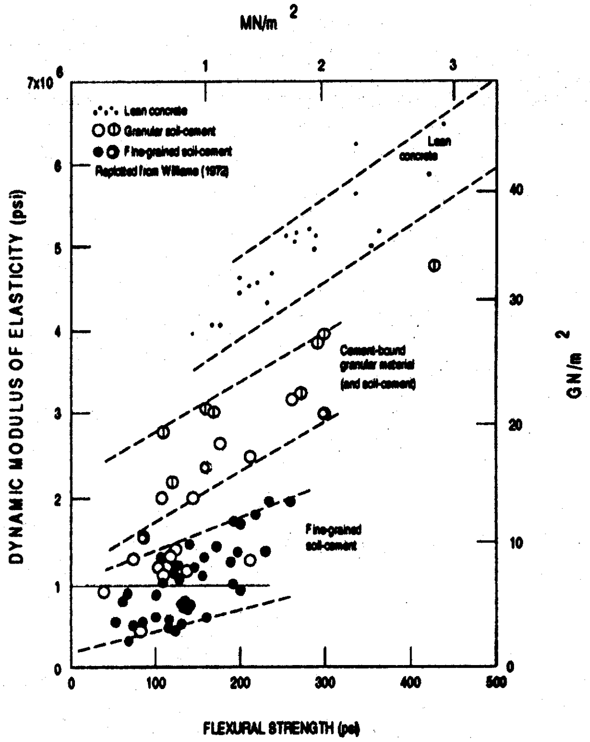


Figure 33. Relationship between flexural strength and dynamic modulus of elasticity for different cement treated materials.

Poisson's Ratio

At working stress levels for pavement bases and treated subgrades, Poisson's ratio is in the range of 0.1 to 0.2 for treated granular soils. Treated fine-grained soils exhibit somewhat higher values, with a typical range of 0.15 to 0.35.

Fatigue Behavior

Cement-treated soils are susceptible to fatigue failure after repeated applications of stresses. Fatigue in flexure is of greatest interest because of its relevance to pavement cracking. Some general observations concerning the fatigue behavior of cement-treated soils are:

- Fatigue life is shorter under repeated direct tensile stresses than in compression.
- Flexural fatigue is greatly reduced for repeated stress levels less than 50 percent of the flexural strength.
- The flexural fatigue of soil-cement can be related to radius of curvature according to:(103,104)

$$R_c/R = a \cdot N^{-b}$$

where:

R_c = critical radius of curvature (i.e., the radius of curvature causing failure under static loading), in
R = radius of curvature leading to failure under N load applications, in
a = $(h^{1.5})/(2.1 \cdot h - 1)$
h = slab thickness, in
b = 0.025 for granular soil-cements and 0.050 for fine-grained soil-cements
N = number of load applications

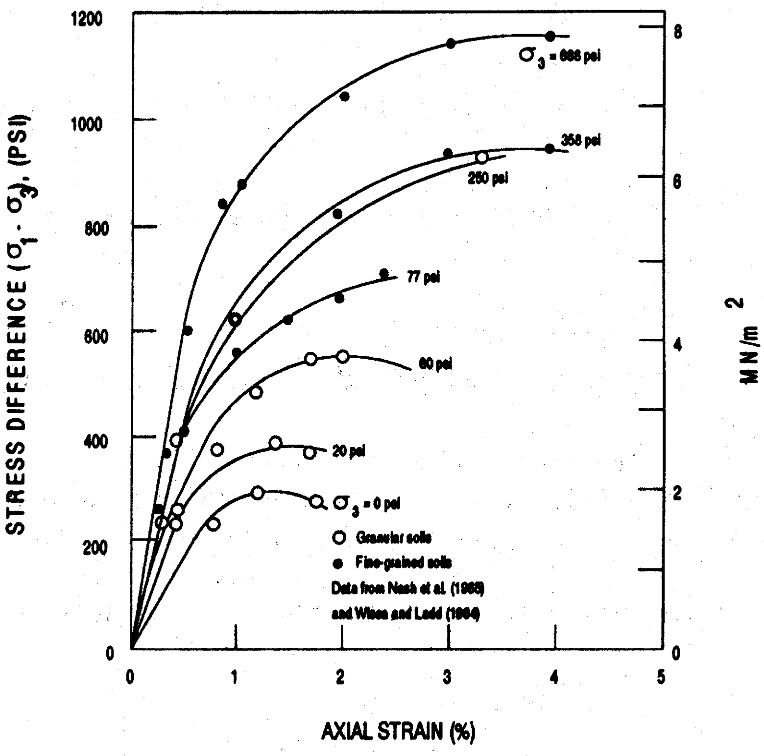


Figure 34. Typical stress-strain behavior for soil and cement mixtures.

- A generalized relationship that permits analysis of fatigue under repeated changes in both the major (σ_1) and minor (σ_3) principal stresses has been verified by Raad et. al.(102) A stress factor, F, is defined as:

$$F = [(\sigma_1 - \sigma_3)^2]/[8\cdot(\sigma_1 + \sigma_3)] \qquad \text{for } \sigma_1 + 3\cdot\sigma_3 > 0$$

$$F = -\sigma_3 \qquad \text{for } \sigma_1 + 3\cdot\sigma_3 < 0$$

where compression stress is positive.

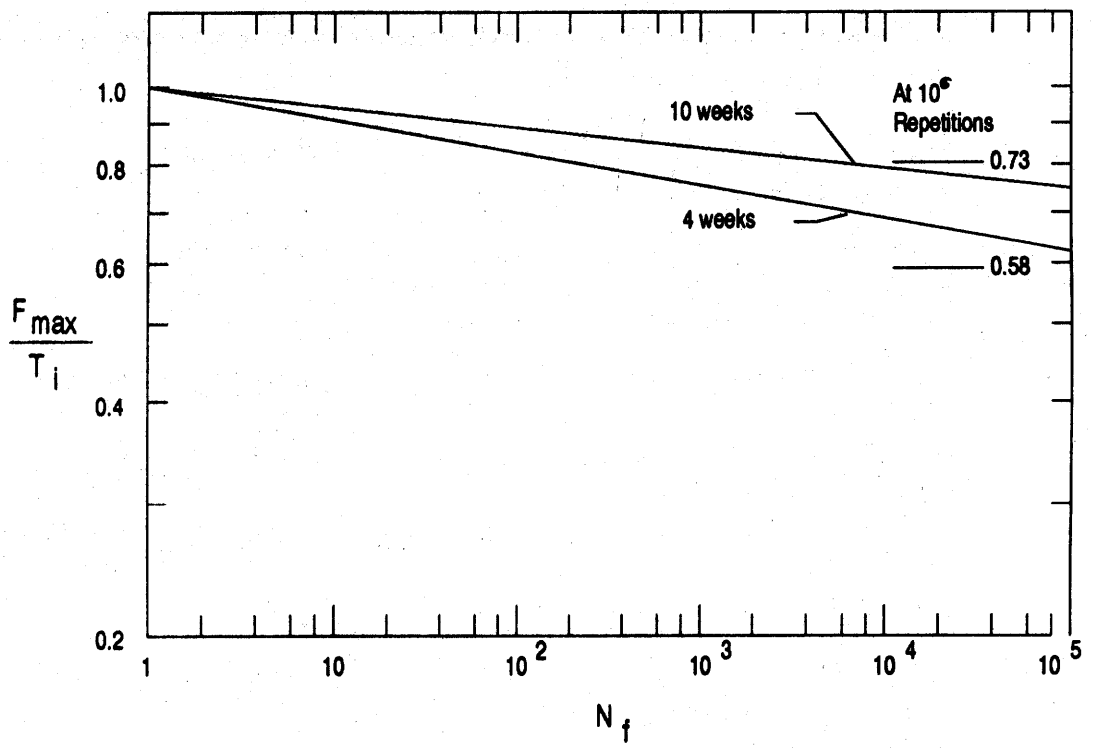


Figure 35. Suggested fatigue failure criteria for cement-treated soils.(102)

Repeated tensile stresses cause a progressive decrease in tensile strength from its initial value T_i. When the strength drops to F, cracking failure is initiated. A relationship between F_{max}/T_i and the number of stress repetitions of N_f to cause failure that fits available fatigue data well is shown in figure 35. The two curves shown pertain to different times after treatment.

Shrinkage

Cement-treated soils exhibit shrinkage on curing and drying in an amount that depends on cement content, soil type, water content, degree of compaction and curing conditions. Some amount of shrinkage cracking should be considered inevitable in soil-cement pavement slabs.

Field observations indicate these cracks can be appreciable at spacings of 10 to 20 ft (3 to 6 m). The smaller crack spacings are usually associated with the higher clay content soils. Because of the likelihood of shrinkage cracks in soil-cement road bases, it is important to consider edge loading conditions in thickness design and to provide surface sealing so that water is prevented from entering the subgrade and leading to loss of support.

Summary

The numerical values indicated in this section are typical for normal conditions. Final design values in any case should be based, whenever possible, on carefully conducted tests in which the anticipated field conditions are simulated as closely as possible.

6. SELECTION OF CEMENT CONTENT

Approximate Quantities

Table 14 lists the usual cement requirements for soil-cement for various soil types classified according to the AASHTO and Unified systems. An approximate cement content may be selected from this table. It should be remembered that the cement content ranges indicated are for soil-cement, a hardened material that will pass rather severe durability tests. For many applications (e.g., treated subgrades, subbases, low volume roads, etc.), satisfactory modification may be achieved using lower cement contents (see section 7).

Detailed Testing

For major projects, and when soil-cement meeting specified durability conditions is required, a more detailed testing program is needed. The flow diagram in figure 36 may be used as a basis for determination of the cement content. The pH determination is used to establish whether sufficient deleterious organic matter is present to inhibit cement hydration. The sulfate determination will establish the possibility of adverse sulfate reactions. Reference 91 contains specific details of the test procedures.

Table 14 indicates the usual cement contents for moisture-density, strength, wet-dry, and freeze-thaw durability testing. Detailed test procedures are given in the Portland Cement Association Soil-Cement Laboratory Handbook(106) and by the following ASTM Test Standards, which are approved also as American National

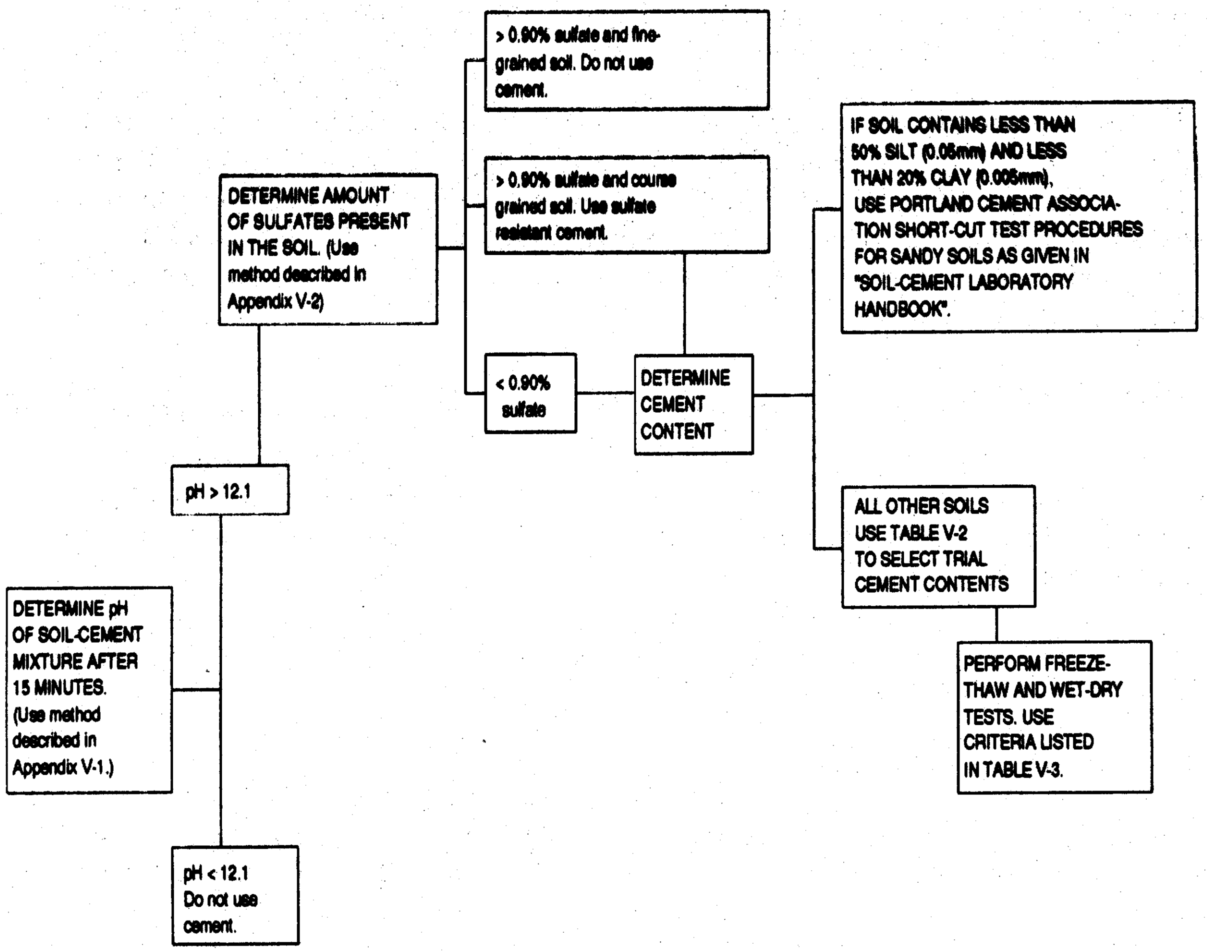


Figure 36. Subsystem for nonexpedient base course stabilization with cement.(91)

Standards by the American National Standards Institute:

> ASTM D 558-82 (Reapproved 1990): Test for Moisture-Density Relations of Soil-Cement Mixtures.
>
> ASTM D 559-89: Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures.
>
> ASTM D 560-89: Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures.
>
> ASTM D 1632-87: Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory.

ASTM D 1633-84: Test for Compression Strength of Molded Soil-Cement Cylinders.

ASTM D 2901-82 (Reapproved 1986): Test for Cement Content of Freshly Mixed Soil-Cement.

Criteria for satisfactory performance of soil-cement in the durability tests are listed in table 15. Cement contents sufficient to prevent weight losses greater than the values indicated after 12 cycles of wetting-drying-brushing or freezing-thawing-brushing are adequate to produce durable soil-cement.

Soil-cement mixes designed in this way can generally be expected to perform satisfactorily as roadway base courses. An exception to this is the case of cement-treated uniform sands. Recent experience shows that with low-cost, low-volume roads, excessive shrinkage cracks develop if the full cement requirement is used. An unsightly pavement develops as a result, and slippage of thin (1 to 1.5 in [25 to 40 mm]) asphaltic concrete surfacings may occur. Although some shrinkage cracking is inevitable, as noted earlier, it can be minimized in uniform sands if the cement and water contents are held to a minimum while still obtaining a desired compressive strength, usually about 300 psi (2,000 kPa).

Table 14. Cement requirements for various soils.(105)

AASHTO Soil Classification	Unified Soil Classification*	Usual range in cement requirement		Estimated cement content and that used in moisture-density test	Cement contents for wet-dry and freeze-thaw tests
		Percent by volume	Percent by weight	Percent by weight	Percent by weight
A-1-a	GW, GP, GM, SW, SP, SM	5 - 7	3 - 5	5	3 - 5 - 7
A-1-b	GM, GP, SM, SP	7 - 9	5 - 8	6	4 - 6 - 8
A-2	GM, GC, SM, SC	7 - 10	5 - 9	7	5 - 7 - 9
A-3	SP	8 - 12	7 - 11	9	7 - 9 - 11
A-4	CL, ML	8 - 12	7 - 12	10	8 - 10 - 12
A-5	ML, MH, CH	8 - 12	8 - 13	10	8 - 10 - 12
A-6	CL, CH	10 - 14	9 - 15	12	10 - 12 - 14
A-7	OH, MH, CH	10 - 14	10 - 16	13	11 - 13 - 15

* Based on correlation presented by Air Force. Amounts should be increased when stabilizing an A horizon soil.

Table 15. Criteria for soil-cement as indicated by wet-dry and freeze-thaw durability tests.

AASHTO Soil Group	Unified Soil Group	Maximum Allowable Weight Loss, percent
A-1-a	GW, GP, GM, SW, SP, SM	14
A-1-b	GM, GP, SM, SP	14
A-2	GM, GC, SM, SC	14
A-3	SP	14*
A-4	CL, ML	10
A-5	ML, MH, CH	10
A-6	CL, CH	7
A-7	OH, MH, CH	7

* The maximum allowable weight loss for A-2-6 and A-2-7 soils is 10 percent.

Additional Criteria

1. Maximum volume changes during durability test should be less than 2 percent of the initial volume.
2. Maximum water content during the test should be less than the quantity required to saturate the sample at the time of molding.
3. Compressive strength should increase with age of specimen.

Criteria other than the durability tests for mix design are used by some agencies. Among the tests used are unconfined compression, triaxial compression, and flexural beam tests. The Portland Cement Association has short-cut test procedures that can be used for determining the cement content required to make soil-cement using sandy soils.

7. CEMENT-MODIFIED SOILS

Cement-modified soils can be classified into two groups according to predominant grain size:

- Cement-modified granular soils (soils containing 35 percent or less of silt and clay)

- Cement-modified silt and clay soils (soils containing more than 35 percent of silt and clay)

Granular soils, not acceptable for use as pavement base materials because of slightly excessive plasticity index or poor gradation, can be modified with portland cement to reduce or eliminate plasticity and to increase bearing values to an acceptable level. This type of treatment, which is done extensively, provides a more stable base for bituminous wearing surfaces. Figure 37 illustrates the reduction in PI produced by the addition of cement to a substandard granular base material. It also shows the permanency of the PI reduction as measured over a 10-year period.

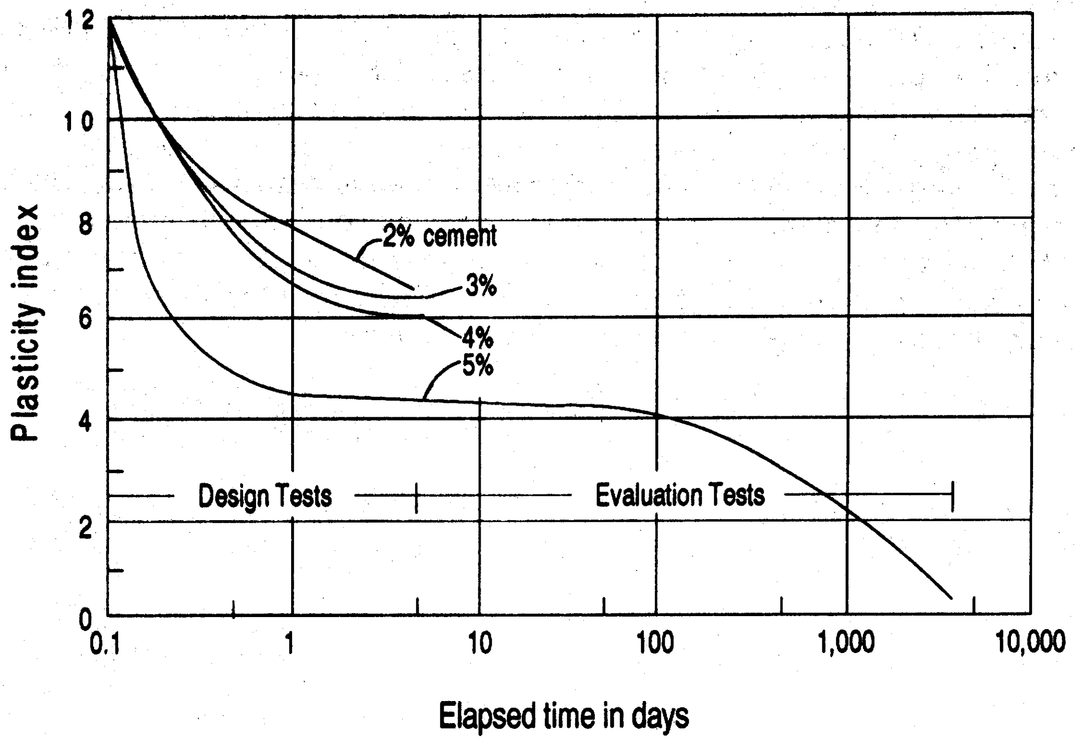


Figure 37. Plasticity index versus cement content.[107]

Fine-grained subgrades and highway fills are often treated with cement to increase bearing strength and reduce volume changes and plasticity. The effect of cement on swell for the same soil is shown in figure 38. The effect of cement in reducing plasticity of an A-7-6 clay soil is shown in figure 39.

The altered soil properties provide greater subgrade support and a stable working platform on which to compact the base course. Cement-modified subgrades also provide an effective solution to the problem of fatigue failures caused by high deflections of the pavement.[105]

Cement-modified silt and clay soils are not recommended for pavement bases. The small quantities of cement do not improve silt and clay soils sufficiently to justify their use as base materials.

8. SUMMARY

Cement stabilization is most effective for granular materials, or fine grained soils with little plasticity. Gradations with fines which produce a floating matrix for the aggregate particles are most amenable to this form of stabilization. The amount of cement produces either a soil-cement mixture, or a cement-modified soil. The soil cement will have the highest strength increase, while the cement-modified soil is done principally to provide a material with improved construction properties. Cement stabilization produces the highest strength increase of all additives, but care must be taken when producing high strength stabilized bases as shrinkage cracking can occur, requiring increased maintenance expenditures.

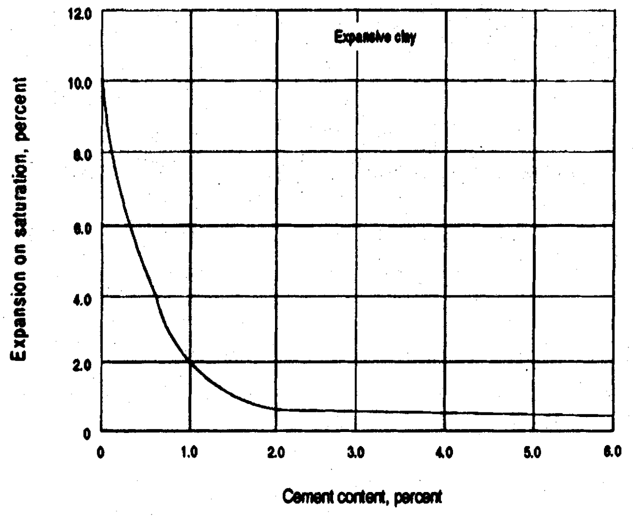


Figure 38. Expansion versus cement content for an expansive clay.(107)

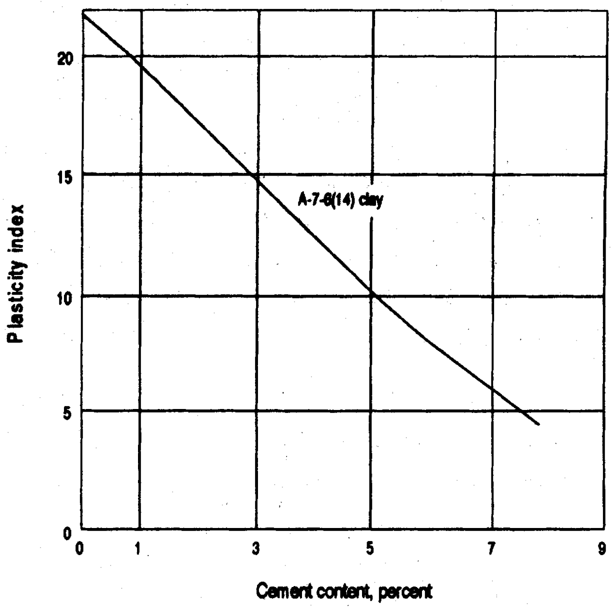


Figure 39. Plasticity Index versus cement content for a clay.(107)

CHAPTER 6 ASPHALT STABILIZATION

1. INTRODUCTION

Through the ages, asphalt has played a significant role in the construction and maintenance of societies' infrastructures. As early as 3800 B.C., asphalt was being used as mortar for building stones and paving blocks. The first use of asphalt for streets in the United States was in 1870, when an asphalt pavement was laid in Newark, New Jersey.

While asphalt is primarily used in pavement surfacing, surface treatment, patching, and sealing, it is also occasionally used in soil stabilization. Asphalt was first used as a stabilizer in the U.S. in 1930. At that time, the States of Florida, Oklahoma, and South Carolina all constructed road bases using asphalt stabilized soils or aggregates. Since then, thousands of miles of pavement have been built utilizing asphalt treated materials.

While many of the early asphalt stabilization projects were done in-place, a large percentage of the stabilization projects in the last 30 to 40 years have used central plant mixed material. In fact, a survey of state practices published by the Highway Research Board in 1970 showed that at least 85 percent of the asphalt stabilization projects in the 20 years prior to the survey utilized hot or cold central plant mixing operations as shown in figure 40.(108)

Despite the existence of more common stabilizers, such as lime and cement, asphalt possesses properties and characteristics that make its use quite desirable in certain situations. In general, it is used as a stabilizer for the following reasons:

- Waterproofing fine-grained subgrade soils.
- Construction expediency.
- Upgrading of marginal materials.
- Reduction of pavement layer thickness, thereby reducing costs and conserving materials and energy.
- Provide temporary and permanent wearing surfaces.
- Reduce dusting.

2. TYPES OF ASPHALT

Virtually all asphalts used in the United States are products of the distillation of crude petroleum.(109) A variety of asphalt types and grades are available, of which asphalt cement is the basic component. Asphalt cement is the hard, high-molecular weight fraction of the crude oil, which at ambient temperatures takes the form of a semi-solid. Because it is virtually impossible to apply asphalt cement in this state, it is made liquid in one of three ways:

- Heat.
- Addition of a solvent or diluent.
- Emulsification with water.

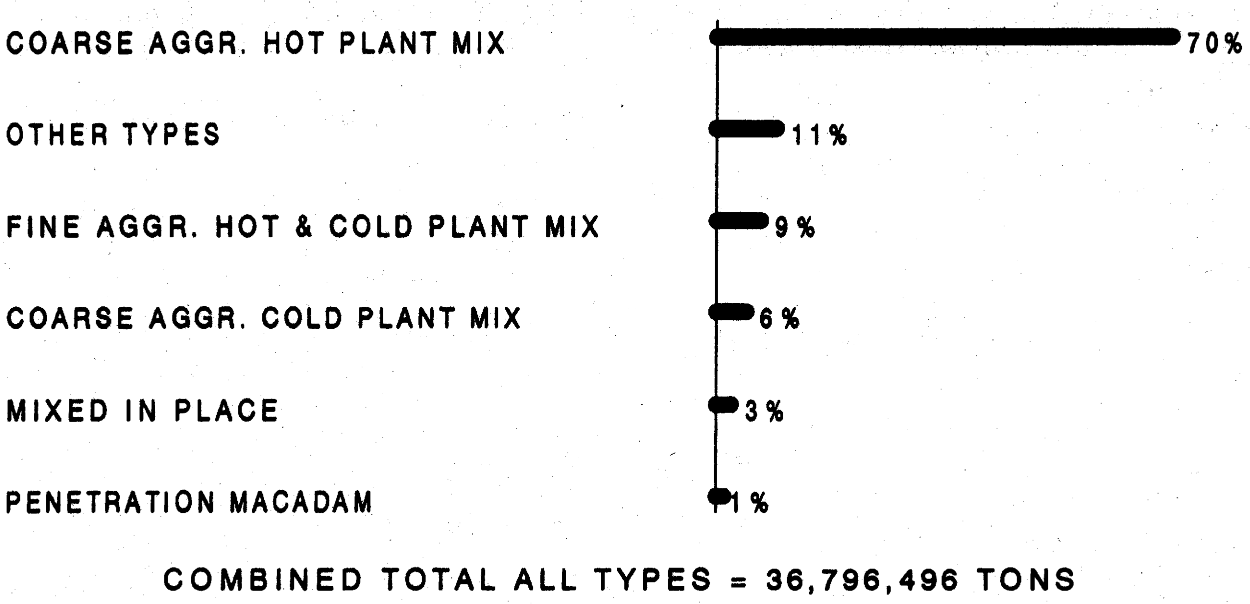


Figure 40. Bituminous bound base courses - practice in United States, all States reporting (Alaska only State not using this type construction).

Asphalt Cements

Asphalt cements are graded on the basis of consistency at a standard temperature. Three primary specifications are utilized to grade asphalts on this basis:

1. Penetration at 77 °F (25 °C) of original asphalt.
2. Viscosity at 140 °F (60 °C) of original asphalt.
3. Viscosity at 140 °F (60 °C) of laboratory-aged asphalt.

Standard asphalt cement specifications have been developed by ASTM and AASHTO and are shown in table 16. Typical penetration grades are 40-50, 60-70, 85-100, 120-150, and 200-300. Typical viscosity grades are AC-5, AC-10, AC-20, and AC-40.

Because of their semi-solid consistency at ambient temperatures, asphalt cements must be heated to obtain a mixing and spraying viscosity. Asphalt cements are normally used in central plants with heated aggregates; however, soft asphalt cements have been mixed in-place and some hard asphalts have been used in in-place foaming operations. The strengthening of mixtures utilizing asphalt cements occurs as the heat required for mixing, laydown, and compaction dissipates and the asphalt cement stiffens.

Table 16. Asphalt specifications.

Material		Specification	
		AASHTO	ASTM
Asphalt Cement	Penetration basis	M 20	D 946
	Viscosity basis	M 226	D 3381
Cutback	Rapid curing	M 81	D 2028
	Medium curing	M 82	D 2027
	Slow curing	M 141	D 2026
Emulsion	Anionic	M 140	D 977
	Cationic	M 208	D 2397

Cutback Asphalts

Cutbacks are combinations of asphalt cement and a petroleum diluent blended to provide viscosities suitable for mixing and spraying at relatively low temperatures. Cutbacks are graded based upon curing time and consistency. Curing time is varied by the solvent used in cutting back the asphalt cement, while the viscosity (consistency) is controlled by the amount of solvent. Rapid-cure cutbacks (RC) use a naphtha or gasoline type solvent, medium-cure cutbacks (MC) use kerosene-type solvents, and slow-cure cutbacks (SC) use low volatility oils or are made during the refining process.

Grade designations for viscosity graded RC, MC, and SC materials are typically as shown below:

1. RC-70, RC-250, RC-800, RC-3000.
2. MC-30, MC-70, MC-250, MC-800, MC-3000.
3. SC-70, SC-250, SC-800, SC-3000.

The lower limit of the viscosity range for the grade of cutback is given in the material designation. The upper viscosity limit is twice that of the lower limit. For example, an RC-70 is a rapid curing cutback with a viscosity between 70 and 140 centistokes at 140 °F (60 °C).

While cutbacks are occasionally mixed in a central plant, most are used for in-place operations. Regardless of which operation is utilized, it is desirable but not always possible to heat cutbacks to aid distribution and mixing. Curing is usually necessary after mixing and prior to compaction to allow the solvent to evaporate, thereby producing a stiffer mix. Final strength is achieved when the solvent has evaporated, leaving the original asphalt cement.

Emulsified Asphalts

Emulsified asphalts are mixtures of asphalt cement, water, and an emulsifying agent. The emulsifying agent coats the minute droplets of asphalt cement, keeping them suspended in the water. The result is a liquid asphalt which can be readily applied to soils.

Asphalt emulsions are divided into three categories, based upon the charge imparted to the asphalt particles by the emulsifying agent used:

1. Anionic (negatively charged).
2. Cationic (positively charged).
3. Nonionic (uncharged).

Today, only anionic and cationic emulsions are commercially available. Of these two, cationic emulsions work successfully with a wider range of materials.

Emulsions are further classified on the basis of how quickly the asphalt will break, or set (i.e., revert to asphalt cement).(109) The classifications for anionic emulsions are rapid-setting (RS), medium-setting (MS), and slow-setting (SS) with the corresponding classifications for cationic emulsions CRS, CMS, and CSS, respectively. The setting characteristics of an asphalt emulsion are determined primarily by the type and amount of emulsifying agent present.

The characteristics of the asphalt cement used to manufacture an emulsion and the viscosity of that emulsion are utilized in defining the grade. For example, a major difference between the CRS-1 and CRS-2 (cationic rapid-setting emulsions) is the viscosity of the emulsion. CRS-2 is more viscous than CRS-1. Additionally, the major difference between CMS-2 and CMS-2h (cationic medium-setting emulsions) is the penetration of the base asphalt cement. The "h" in CMS-2h indicates that a harder asphalt cement was used in manufacturing the emulsion.

Additional grades of emulsions include high float medium-set (HFMS) emulsions and quick-setting (QS) emulsions. HFMS emulsions are frequently used in stabilizing granular soils as they provide a thicker coating of the aggregate particles, which reduces the stripping action of water. Quick-setting and rapid-setting emulsions, on the other hand, are rarely used in stabilization. Their fast setting times generally do not allow for sufficient mixing.

Some medium-setting emulsions may contain small amounts of petroleum solvents (up to 12 percent) to aid mixing and provide stockpiling capability to mixtures made with the emulsion. The strength gain in an emulsion stabilized soil develops first from the setting of the emulsion, and then from the evaporation of the water, leaving the original asphalt cement.

A review of the above descriptions of asphalt products indicates that a large number of asphalts are available for soil stabilization purposes. ASTM specifies 49 different asphalts. Selection of the type of asphalt for a given stabilization use is discussed later. In general, asphalt cements are used in hot central plant operations, while medium and slow curing cutbacks and medium and slow setting emulsions can be used for in-place stabilization operations.

3. MECHANISMS OF ASPHALT STABILIZATION

The mechanisms involved in the stabilization of soils and aggregates with asphalt differ greatly from those involved in cement and lime stabilization. The basic mechanism involved in asphalt stabilization of fine-grained soils is waterproofing, which improves strength and durability. Soil particles or soil agglomerates are coated with asphalt, resulting in a membrane that prevents or impedes the penetration of water which, under normal conditions, would result in a decrease of shear strength, compressive strength, tensile strength, flexural strength, and elastic modulus. From a durability standpoint, asphalt-coated soil particles are much more resistant to the detrimental effects of water, such as volume change due to alternating wet-dry and/or freeze-thaw cycles.

In non-cohesive materials, such as sands and gravel, crushed gravel, and crushed stone, two basic mechanisms are active: waterproofing and adhesion. The asphalt coating on the cohesionless materials provides a membrane which prevents or hinders the penetration of water and thereby reduces the tendency of the material to lose strength and other desirable properties in the presence of water.

In the second mechanism, adhesion, the asphalt acts as a binder, holding the aggregate particles together. This cementing effect increases shear strength by increasing cohesion while the effect of the asphalt on the angle of internal friction is minimal. Other property improvements resulting from the asphalt cement include an increase in tensile strength, compressive strength, flexural strength, and elastic modulus.

In addition to the benefits cited above for asphalt stabilization, the stabilized layer may prevent surface water from penetrating into the subgrade, preventing strength loss in the subgrade materials. In surface course applications, the asphalt binder has the capability of eliminating or reducing the occurrence of raveling, washboarding, loss of fines, etc., under traffic.

4. SOILS SUITABLE FOR ASPHALT STABILIZATION

Fine-Grained Soils

Fine-grained soils may be stabilized with asphalt, depending upon the plasticity characteristics of the soil and the amount of material passing the No. 200 sieve. Due to the extremely high surface area of the finer soil particles, a large percentage of asphalt would be required to coat all of the soil surfaces. Since this is virtually impossible, agglomerations of particles are coated with economical percentages of asphalt. The gradation of fine-grained soils suitable for asphalt stabilization is shown in table 17 as soil-bitumen. As noted in this table, the amount of material passing the No. 200 sieve should be less than 25 percent. In addition, the PI should be less than 10 to ensure that adequate mixing is possible. If proper mixing is not obtained, the plastic fines may swell upon contact with water, resulting in a substantial loss of strength.(110)

Coarse-Grained Soils

Cohesionless soils (PI less than six) suitable for asphalt stabilization are shown in table 17 under sand-bitumen and sand-gravel-bitumen. Cohesionless soils identified as suitable for hot mix asphalt concrete by AASHTO, ASTM, and states, counties, and cities are, in general, acceptable. Asphalt-stabilized materials made with well- or dense-graded aggregates have higher strength than the more one-sized sand-asphalt mixtures.

5. TYPICAL PROPERTIES OF ASPHALT-STABILIZED SOILS

In order to provide a bituminous mixture to satisfy the needs of a particular engineering application, the following mixture properties should be defined:

- Stability.
- Durability.
- Fatigue behavior.
- Tensile behavior.
- Stiffness.
- Flexibility.
- Workability.

Table 17. Engineering properties of materials suitable for bituminous stabilization.(110)

Percent Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel Bitumen
1-1/2 in			100
1 in	100		
3/4 in			60 - 100
No. 4	50 - 100	50 - 100	35 - 100
No. 10	40 - 100		
No. 40		35 - 100	13 - 50
No. 100			8 - 35
No. 200	5 - 12	Good: 3 - 20 Fair: 0 - 3 and 20 - 30 Poor: > 30	
Liquid Limit		Good: < 20 Fair: 20 - 30 Poor: 30 - 40 Unusable: > 40	
Plasticity Index	10	Good: < 5 Fair: 5 - 9 Poor: 9 - 15 Unusable: > 12 - 15	10

Includes slight modification made by Herrin.
1 in = 25.4 mm.

Few tests have been developed to indicate the workability and flexibility of bituminous stabilized materials. Gradation limits and compaction tests have been used to control workability in the uncompacted mixes while elongation and certain tensile tests on the compacted mixes have been used to measure flexibility.

Stability, stiffness, tensile properties, fatigue behavior, and durability of asphalt mixtures have been defined by a number of investigators, and typical properties are available. However, prior to a delineation of these properties, it must be realized that unlike most other stabilized materials, these properties are highly dependent upon the temperatures at which the test is conducted and the rate of loading or rate of elongation utilized by the test method. Other important variables which control asphalt-stabilized mixture properties include:

- Type of asphalt.
- Type and gradation of the aggregate.
- Density of the compacted mixture.
- Curing and/or aging conditions.

Currently, several methods are being used by highway agencies in the design of bituminous stabilized soils and aggregates. Specifications and criteria are almost exclusively based on stability, durability, and gradation requirements. Some agencies do not have durability requirements and thus stability becomes the only laboratory test parameter utilized for mixture design.

Strength

The most widely accepted strength tests today are the Hveem and Marshall stability tests; several states use unconfined compression testing. Other tests used for strength determinations include the Hubbard-Field, the triaxial compression, repeated load triaxial, and various penetration type tests, including the California Bearing Ratio, the Iowa Bearing Value, and the Florida Bearing Value. Some of these tests are discussed in volume II, chapter 3, while others are outdated and are therefore not discussed.

Typical criteria and hence typical values for Hveem, Marshall, and unconfined compressive strength are shown in table 18. Methods of sample preparation, test temperatures, and curing conditions prior to testing vary widely from state to state. Attempts have been made to establish one standard method but success has been limited.(111) Most of the criteria presently utilized were originally developed for surface courses and adapted to base course design and may or may not be suitable for emulsion mixes.

Durability

Durability tests which have been utilized for control of bituminous stabilized mixtures include the California Moisture Vapor Susceptibility Test, the Immersion Compression Test, the Swell Test, and Vacuum Saturation Tests. These water-susceptibility tests are usually performed on Hveem or Marshall stability samples or unconfined compression test samples, and acceptance criteria are based on a percent of retained strength (70 percent) or a minimum stability after soaking.

Freeze-thaw and wet-dry durability-type tests for bituminous stabilized mixtures are nearly non-existent. The water saturation test coupled with freezing and thawing developed by Lottman(112) and the Root-Tunnicliff modification have been developed for asphalt concrete mixtures, but may have some limited applicability to stabilized materials.

Table 18. Design methods and criteria for asphalt stabilized base courses.(108)

A. Hveem Method

State	Stability, lbs	Percent Air Voids	Percent Voids Filled With Asphalt	Cohesiometer
California	35 min	4 - 6	80 - 85	300 min
Colorado	30 - 45	3 - 5	75	
Hawaii	35 min	5 - 10		
Nevada	30 - 37 min	3 - 5		
Oklahoma	35 min	8 max		
Oregon	30 min	10 max		150 min
Texas	30 min			
Washington	20 min			50 min

B. Marshall Method

State	Stability, lbs	Flow, 0.01 in	Percent Air Voids	Percent Voids Filled With Asphalt
District of Columbia	750 min	8 - 16	3 - 8	65 - 75
Georgia	1800 min	8 - 16	3 - 6	65 - 75
Kansas	800 - 3000	5 - 15	1 - 5	70 - 85
Kentucky	1100 - 1500	12 - 15	4 - 6	
Mississippi	1600	16 max	5 - 7	50 - 70
New Jersey	1100 - 1500	6 - 18	3 - 7	
North Carolina	800	7 - 14	3 - 8	
North Dakota	400 min	8 - 18	3 - 5	
Pennsylvania	700 min	6 - 16		60 - 85
Rhode Island	750 min		3 - 8	
South Carolina	1200 - 3000	6 - 12		
South Dakota		8 - 18	3 - 5	
Wyoming	1000 min			

C. Unconfined Compressive Strength

State	Load, psi	Percent Air Voids	Percent Voids Filled With Asphalt
Colorado	200 - 400	3 - 5	80 - 85
Oregon	150 min		

1 in = 25.4 mm; 1 lb_f = 4.448 N; 1 psi = 6.89 x 10^3 Pa

Fatigue Behavior

The fatigue behavior of bituminous stabilized materials has been reviewed by Witczak, Epps and Monismith, Pell, and Santucci, and Kallas.[113,114,115,116,117] All of these reviews indicate the relative importance of asphalt type, aggregate gradation, aggregate type, air void content, and other mixture variables. Santucci has offered some typical fatigue curves for asphalt concrete and emulsion mixtures shown in figure 41, and for cement treated emulsion mixtures in figure 42. Kallas investigated the effect of different emulsion types, moisture contents and curing times. Resulting fatigue curves are shown in figures 40 (a&b). These emphasize the effect that can be produced in these performance tests when different curing conditions develop.

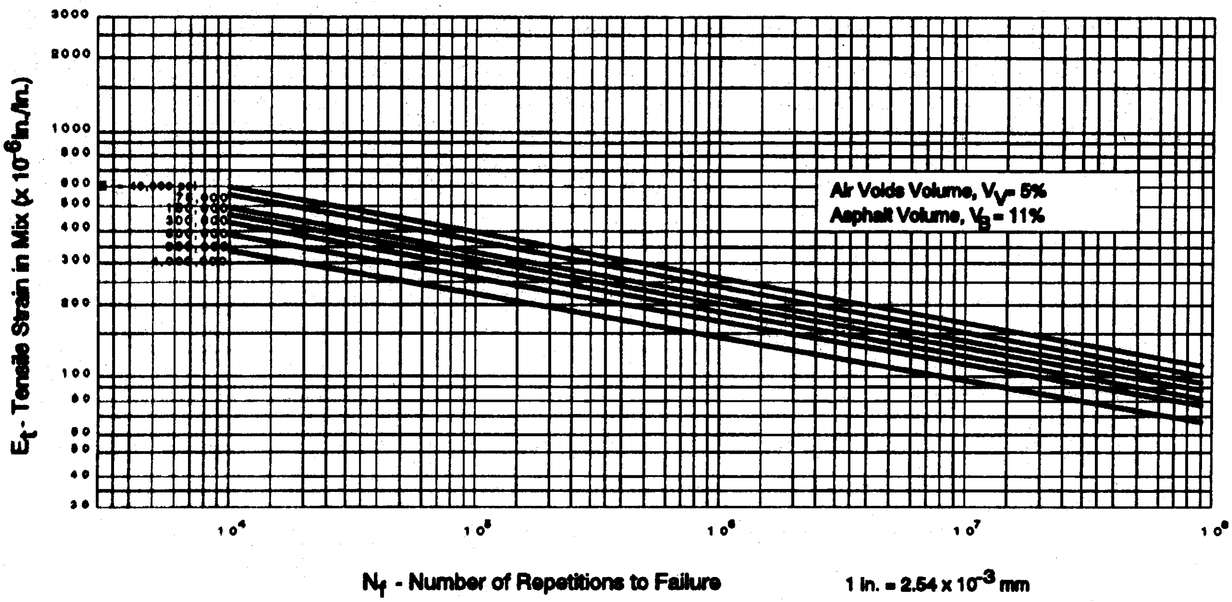


Figure 41. Fatigue criteria for asphalt and emulsion mixes.[6]

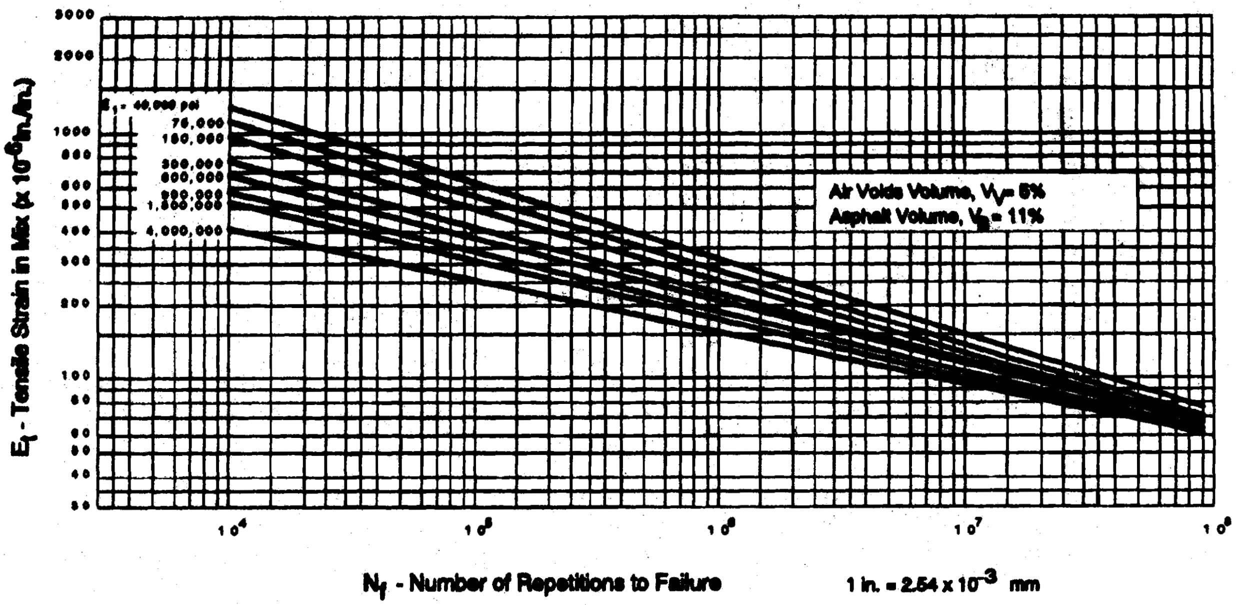


Figure 42. Fatigue criteria for cement-modified asphalt emulsion mixes.[(6)]

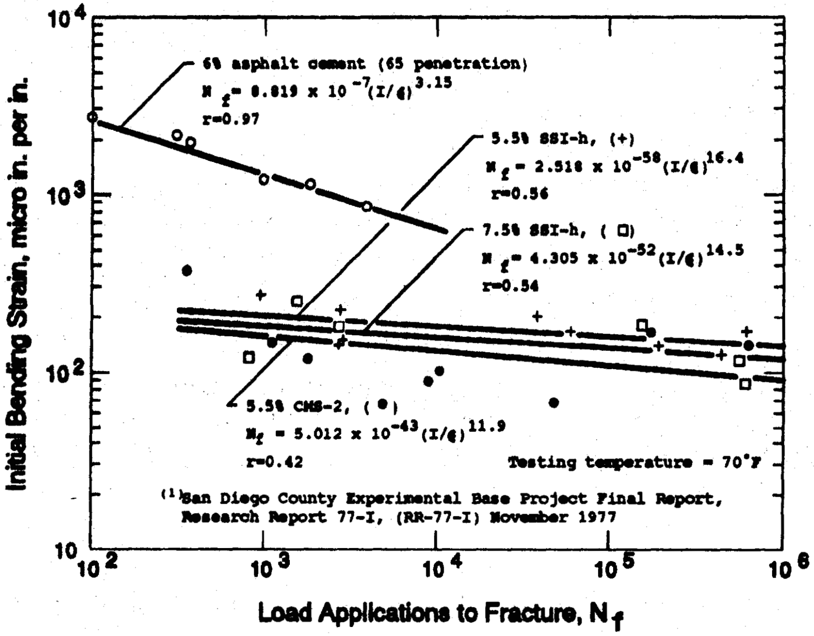


Figure 43a. Strain-fracture life relationships for crushed gravel emulsified asphalt mixes containing less than 1% moisture and for crushed gravel asphalt cement mix.

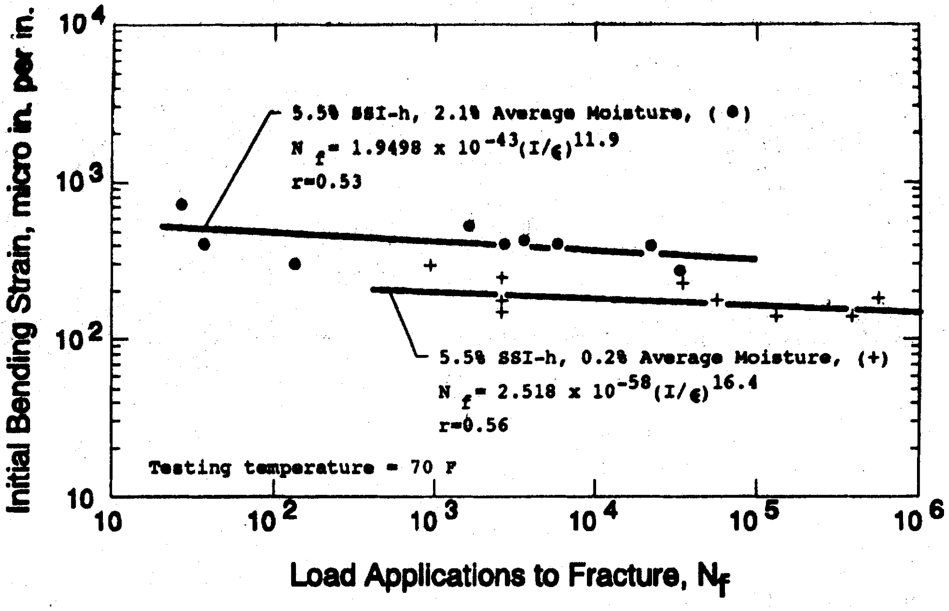


Figure 43b. Strain-fracture life relationships for crushed gravel emulsified asphalt mixes with moisture contents of 0.2 and 2.1 percent.

Tensile Properties

A wide variety of tensile tests, including direct tension, indirect tension, dumbbell, and "dornprobe", have been performed on bituminous stabilized mixtures. The most popular test at present appears to be the indirect tension or split-tensile test. This test has been utilized by many universities, research institutes, and others to define tensile properties both prior to and after water susceptibility tests. Tensile strength is largely dependent upon voids, curing, rate of loading, temperature, and age. Typical values obtained under conditions simulating highway loadings are on the order of 70 to 800 psi (480 to 5,500 kPa).

Stiffness

Stiffness of an asphalt stabilized mixture is generally defined as the ratio of the applied stress to the observed strain for a test performed at a particular temperature and rate of loading. It is basically an "elastic" modulus at rapid rates of loading. The resilient modulus (M_R), defined as the ratio of the applied repeated stress to the recoverable strain, closely approximates stiffness.

Values of M_R can be obtained from tests such as the diametral, flexural beam, direct tension, and triaxial compression. A comparison of modulus values of asphalt concrete specimens measured by all four tests was conducted by Schmidt.(118) Results showed agreement to within about 25 percent.

The most popular test for measuring resilient modulus is the diametral. It can be used to test laboratory compacted Marshall or Hveem specimens and extracted pavement cores. In addition, the test is rapid and convenient and simulates field stress states well.

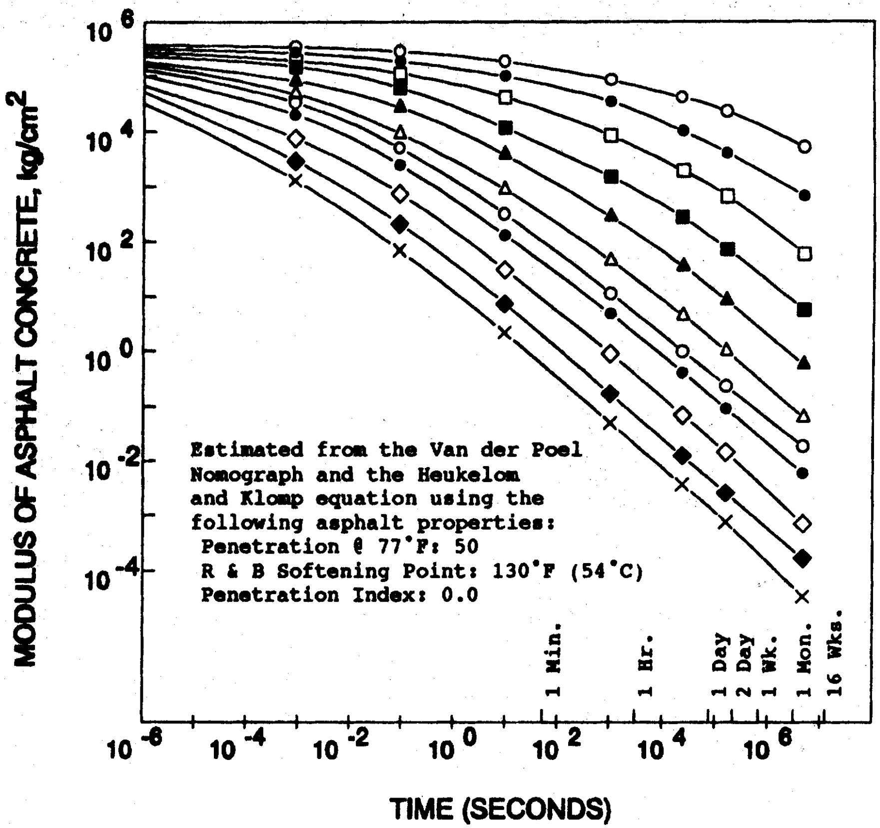


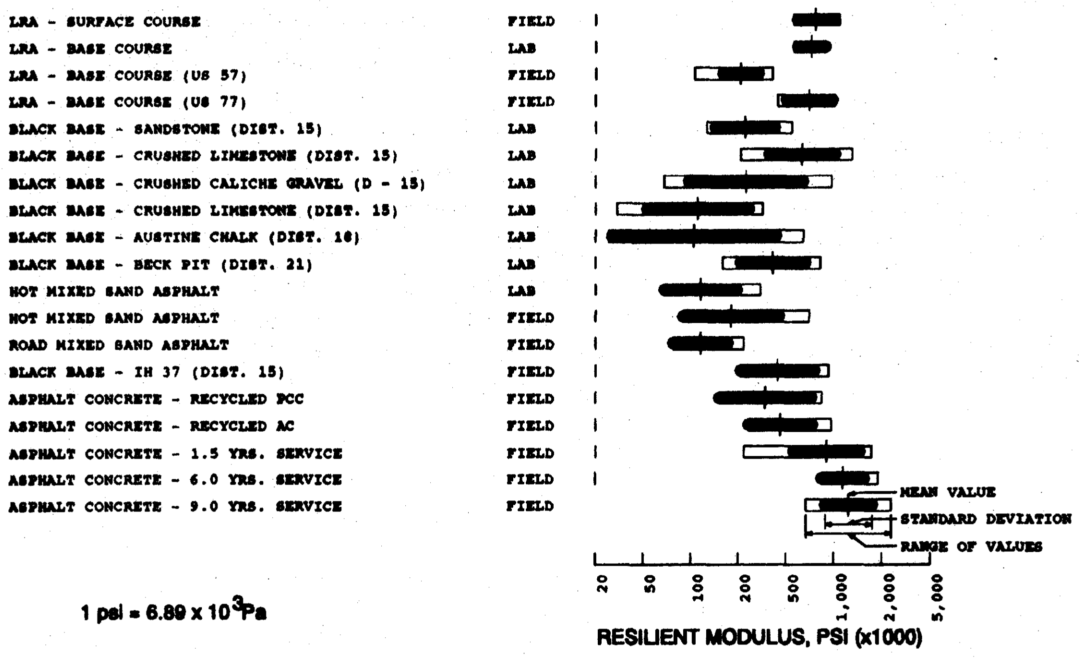
Figure 44. Variation of asphalt concrete modulus.

Figure 44 indicates the wide range of stiffness as a function of temperature and rate of loading for an asphalt cement treated material. Figure 45 illustrates ranges in resilient modulus for a wide range of asphalt cement treated materials at 73°F (23°C with a 0.1-sec load rate. Curing and moisture condition are not a factor in asphalt cement treated mixtures, but for cutbacks and emulsions these conditions must be carefully controlled, as they can alter the stiffness significantly. The study by Kallas clearly indicated the impact of different emulsion types and curing conditions.(117) Figure 46 illustrates the general effect of inadequate curing not allowing the water to evaporate, producing a lower stiffness and more sensitivity.

Summary

Current mixture design procedures are based primarily on stability and durability tests. While stability is frequently considered an indicator of material quality, durability cannot be overlooked. High stability is often obtained at the expense of lowered durability. Thus, a favorable balance between stability, durability, and other properties (density, voids, and moisture absorption) must be determined, given the conditions in which the mixture will be used.

Fatigue behavior, tensile properties, and stiffness parameters are being utilized more often for pavement structural design purposes and mix designs are being developed which provide design values. In a study conducted by Darter and Devos, procedures were developed to correlate the resilient modulus and the AASHTO structural layer coefficient of asphalt emulsion cold mixes used in base course construction.(119) Development of this procedure, accomplished with finite element techniques and the AASHTO performance equations produced the relation between marshall stability [72°F (22 °C)] and layer coefficient, a_2, as shown in figure 47.



1 psi = 6.89 x 10^3Pa

Figure 45. Typical modulus values from field and lab measurements.

6. ASPHALT SELECTION

Selection of Asphalt Type and Grade

There are four considerations which influence selection of the type and grade of asphalt for a particular use:

- Method of construction.
- Available equipment.
- Soil characteristics.
- Loading and climatic conditions.

Method of construction and available equipment will primarily determine the type of asphalt (asphalt cement, cutback, or emulsion) to be used. The corresponding grade, including viscosity and setting characteristics, will be influenced by the gradation and the amount of fine particles in the aggregate soil characteristics, the environmental conditions during and after construction, the type of mixing equipment, and the magnitude of loads expected on the pavement.

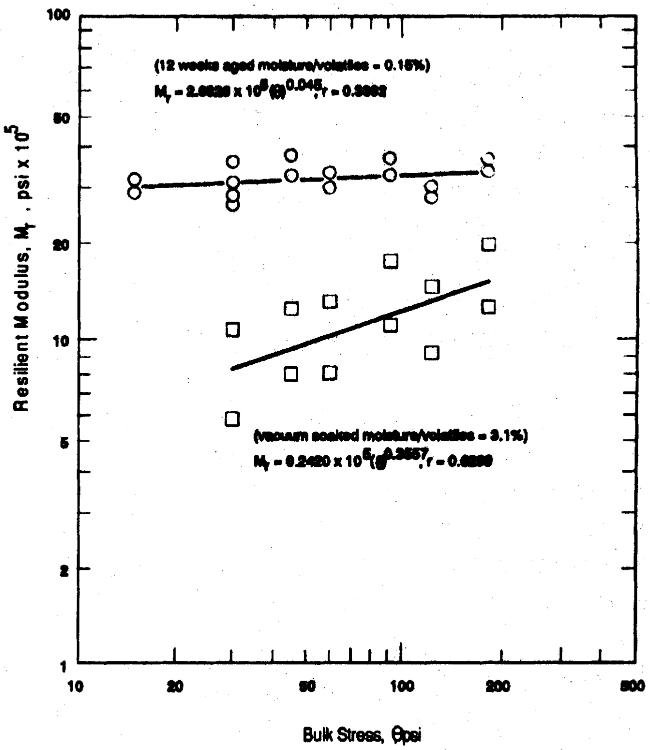


Figure 46. Effect of curing on modulus of an emulsion mixture.

Method of Construction

The basic types of construction include central plant and mixed-in-place or on-grade construction. Asphalt cements are in general limited to hot central plant mixing operations; however, soft asphalt cements have been used for mixed-in-place operations. Central plants are occasionally used for mixing liquid asphalts (cutbacks and emulsions) with aggregates. In these cases, the liquid asphalt may be mixed hot or cold with unheated or heated aggregate, recognizing the potential danger of mixing cutbacks with heated aggregates.

While some emulsions and cutbacks have been employed in warm central plant operations, the majority of these asphalts are mixed in-place. This construction method is both economical and effective. High production rates are achievable without the large investment in equipment (i.e., central plant mixer). And, although higher quality mixes are produced using central plants, successful stabilization using mixed-in-place procedures can be accomplished with proper attention.

Construction Equipment

Central plants are typically batch or continuous in operation. The drum-mixer continuous plants are the most popular plants presently used. The continuous plants utilizing pugmills for mixing are often used for cold mixing operations.

In-place equipment can be selected to provide a variety of mixing capability. The desired setting characteristic of the emulsion to be selected may often be controlled by the type of equipment selected for the job. Travel plants, rotary mixers, and motor graders are typical in-place mixing equipment. Further details on these pieces of equipment can be found in volume 1, chapter 4.

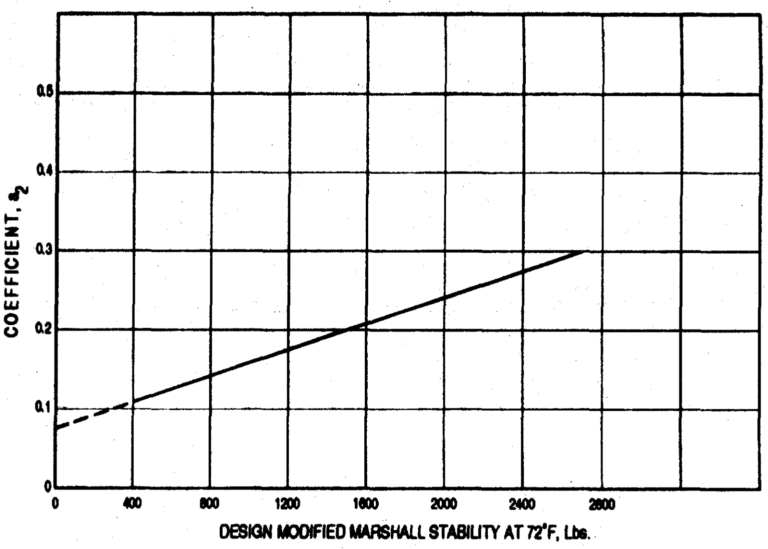


Figure 47. Structural layer coefficient for emulsion aggregate mixtures.(119)

Soil Characteristics

The selection of asphalt grade is based largely on the ability of the asphalt to adequately coat the job aggregate. Thus, aggregate characteristics such as type, gradation, fines, absorption capacity, and moisture content must be thoroughly examined. For instance, the presence of a considerable amount of fines in a soil (characteristic of fine-grained soils and dense-graded aggregates) usually necessitates the use of emulsions or cutbacks having lower viscosities and longer setting times. The long workability times associated with these asphalt types help to ensure sufficient mixing with the fine-grained particles.

Loading and Climatic Conditions

The type of loading (static or dynamic), magnitude of loading (including gross loads and wheel loads), and climatic conditions (including temperature and moisture both before and after construction) affect the selection of the type and grade of asphalt. In the case of heavy static loading, the use of a stiffer, higher quality mix is generally warranted. Such a mix can be produced at a central plant using a higher viscosity grade of asphalt cement.

Climatic conditions (i.e., moisture and temperature) can influence both the type and grade of asphalt to be used. Although moisture-laden soils can be stabilized with cutbacks or dried for use with asphalt cement, they are most suitable for stabilization with emulsions. The presence of moisture affects the coatability, workability, and compaction of mixes, and thus is a factor in selecting the grade and the amount of emulsion. Ideally, there should be enough moisture in the soil to assist in coating and workability; however, the moisture content of the finished mixture should be reduced sufficiently to permit the emulsion to break prior to compaction.

Temperature also influences the grade of asphalt to be used. Low viscosity asphalts are suitable for use in cold climates, while high viscosity asphalts are conducive for use in warm climates. As a general rule, the most satisfactory results are obtained when the most viscous liquid asphalt that can be readily mixed into the soil is used.(20)

Asphalt Selection Process

Asphalt-stabilized materials used as surface courses, base courses, or subbases generally require different types of asphalts. Asphalt cements are normally used with hot central plant operations and are popular binders for surface and base courses. Their use is generally warranted on projects demanding quality over construction expediency, particularly high volume roads or roads which are in severe climates.

Emulsions and cutbacks, on the other hand, are typically used with mixed-in-place operations and are utilized extensively for subbase and base course construction. While most emulsion applications have been on low- to medium-volume roads, they have recently gained acceptance for use as subbase and base course construction in high traffic volume roads.

The use of cutbacks has declined in the last few years as a result of problems with air quality, safety, and the alternate use of cutter stocks for more important purposes. Concurrently, the use of emulsions has increased due to the concern for energy and the environment. Unlike the cutbacks, emulsions do not require as large an amount of petroleum solvent and, unlike the asphalt cements, they can be used without additional heat. Both of these factors contribute to energy savings. Furthermore, there are little or no hydrocarbon emissions from asphalt emulsions, limiting pollution of the environment.

Once a particular type of asphalt has been selected, consideration must be made for an appropriate asphalt grade. Tables 19 (asphalt cement), 20 (asphalt cutback), and 21 (asphalt emulsion) are excellent guides for selecting the grade.

Figures 45 and 46 provide guidance for selecting the type of emulsion (i.e., anionic or cationic) to be used.

Selection of Asphalt Content

Approximate Quantities

An in-depth laboratory testing program is mandatory for determining the exact asphalt content to be incorporated into a particular mix. However, quick estimates of the amount of asphalt to be used in the field or as a starting point in comprehensive laboratory testing can be obtained by using established tables and charts.

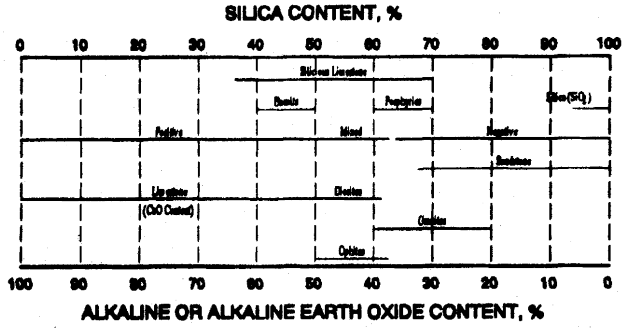


Figure 48. Aggregate composition.

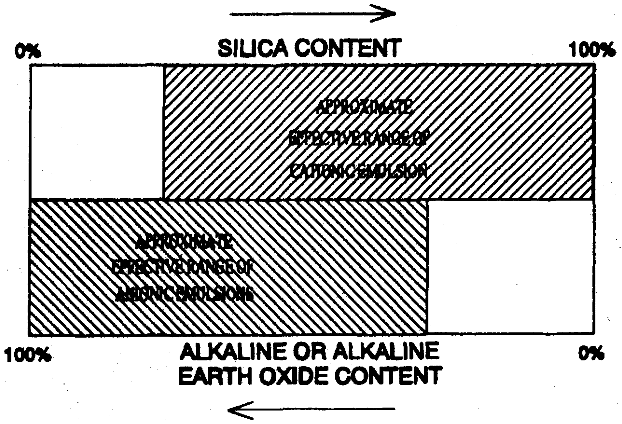


Figure 49. Applicability of emulsions with aggregate composition.

Table 19. Recommendations for selection of paving asphalt.(121)

Thickness of Asphalt Concrete, in[5]	Climate	AASHTO M 20	AASHTO M 226	West Coast[4]
≤ 3	Cold[1]	200 - 300	AC-5	AR-1000
	Moderate[2]	85 - 100	AC-10	AR-4000
	Hot[3]	85 - 100	AC-10	AR-4000
4 - 6	Cold	120 - 150	AC-5	AR-2000
	Moderate	85 - 100	AC-10	AR-4000
	Hot	60 - 70	AC-20	AR-8000
> 7	Cold	120 - 150	AC-15	AR-2000
	Moderate	60 - 70	AC-10	AR-8000
	Hot	40 - 50	AC-20	AR-16,000

[1] Normal minimum daily temperature* of 10 °F (-12 °C) or less; for extremely low temperatures, special studies are recommended.

[2] Normal maximum daily temperature* of 90 °F (32 °C) or less.

[3] Normal maximum daily temperature* greater than 90 °F (32 °C).

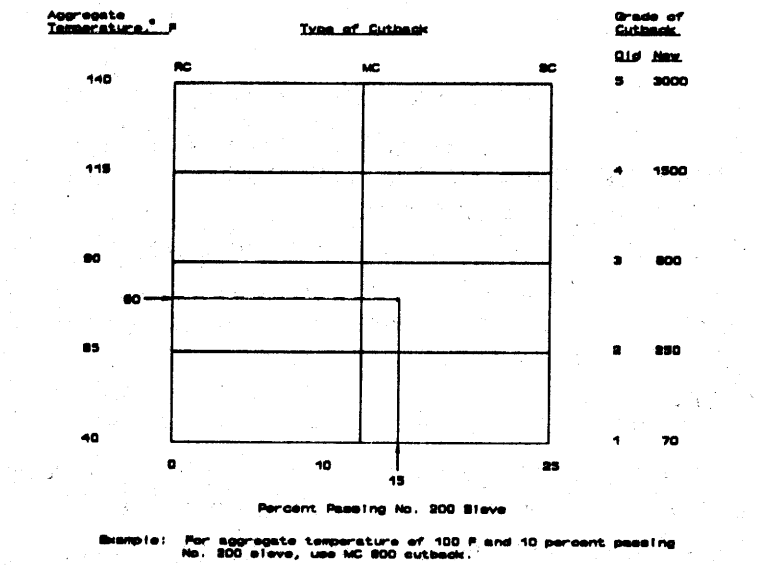
[4] Uniform Pacific Coast Specifications for AR-graded Paving Asphalts.

[5] Total thickness of asphalt concrete; surface plus base.

After NCHRP Report 9-4, "Minimizing Premature Cracking in Asphalt Concrete Pavements."

*As per U.S. Weather Bureau climatological reports.

Table 20. Selection of type of cutback for stabilization.(122)



Example: For aggregate temperature of 100 F and 10 percent passing No. 200 sieve, use MC 800 cutback.

Table 21. Selection of emulsified asphalt type.(123)

Grade Designation	Preferred Usage		
ASTM	Aggregate	Rain Resistance	Construction Method
SS-1 SS-1h	Damp to wet dense-graded aggregates, high sand content gravels, poorly or well-graded sands.	Dependent on dehydration and absorption.	Central Mix or Travel Plant
CSS-1 CSS-1h			
CMS-2 (CMS-2h)	Dry or damp low sand content gravels, well-graded or silty sands.	Resistant to early rainfall.	Travel Plant or In-Place Mixing
MS-1 MS-2 MS-2h	Dry or damp processed open-graded aggregates.	Resistant to early rainfall.	Central Mix or Travel Plant
CMS-2 CMS-2h			

Note: Figures 46 and 47 can be used as a basis for selecting anionic or cationic emulsions. The geologic type of aggregate is located on figure 46 and the approximate silica or alkaline earth oxide content determined. These contents are utilized to enter figure 47 to select the type of emulsion.

Table 22 provides means of approximating the asphalt cement content for hot, central plant mixes. As can be seen, aggregate shape and texture are the sole factors in this table for determining asphalt content. The equation in table 23, developed by the Asphalt Institute, can be used to determine the amount of cutback to be used with a granular soil, given the aggregate characteristics listed. The U.S. Navy assembled a chart for determining the emulsion requirements, given certain percentages of fine- and coarse-grained materials, displayed in table 24.

Detailed Testing

A detailed laboratory testing program is essential to the successful design of bituminous stabilized mixes. The most commonly used techniques to determine asphalt content are the Hveem and Marshall mix design methods. ASTM, AASHTO, the Asphalt Institute, Chevron Asphalt, and the U.S. Air Force have standardized these procedures and established mixture-design acceptance criteria for them.(34,123,124)

Hveem and Marshall mixture-design criteria exist for asphalt cement, cutback, and emulsion stabilized mixtures. The principal features of the Hveem method of mix design are a centrifuge kerosene equivalent (C.K.E.) test on aggregate (for estimation of asphalt requirement) stabilometer, and cohesiometer, swell, and moisture vapor susceptibility tests conducted on compacted mix specimens. The Marshall method features a detailed density-voids analysis and a stability-flow test conducted on compacted mix specimens with a moisture susceptibility requirement.

Asphalt Cement

Material stabilized with asphalt cement may employ the standard ASTM and AASHTO criteria for the Hveem or Marshall test methods commonly used for asphalt concrete surface courses. The acceptance criteria will vary, depending on the agency. Common acceptance criteria for a number of states were shown in table 18. It should be noted that some agencies suggest that a lower stability can be specified for base or subbase materials if the test is performed at the standard 140 °F (60 °C) temperature or a similar stability required for the base or subbase provided the test is conducted at a lower temperature, say 100 °F or 77 °F (38 °C or 25 °C). Suggested criteria are shown in tables 25 and 26 for Hveem and Marshall test procedures, respectively. A standard curing procedure is not required as part of this design method.

Table 22. Selection of asphalt cement content.

Aggregate Shape and Surface Texture	Percent Asphalt by Weight of Dry Aggregate*
Rounded and Smooth	4
Angular and Rough	6
Intermediate	5

*Approximate quantities which may be adjusted in field based on observation of mix and engineering judgement.

Table 23. Determination of quantity of cutback asphalt.[125]

p = 0.02(a) + 0.07(b) + 0.15(c) + 0.20(d)	
Symbol	Definition
p	Percent of residual asphalt by weight of dry aggregate*.
a	Percent of mineral aggregate retained on No. 50 sieve.
b	Percent of mineral aggregate passing No. 50 and retained on No. 100 sieve.
c	Percent of mineral aggregate passing No. 100 and retained on No. 200 sieve.
d	Percent of mineral aggregate passing No. 200 sieve.

*Percent cutback can be obtained by referring to table 23 and utilizing the following equation:
percent cutback = [percent residual asphalt (p)/(100 - percent solvent)] x 100

Table 24. Emulsified asphalt requirements.[122]

Percent Passing No. 200 Sieve	Pounds of Emulsified Asphalt per 100 pounds of Dry Soil when Percent Passing No. 10 Sieve is –					
	< 50	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

Table 25. Hveem design criteria.[34]

Test Category	**Heavy**		**Medium**		**Light**	
Test Property	**min.**	**max.**	**min.**	**max.**	**min**	**max.**
Stabilometer Value	37	-	35	-	30	-
Swell	less than 0.30					

The following notes must be made relative to the Hveem design procedure:

- Although not a routine part of this design method, an effort is made to provide a minimum percent air voids of approximately 4 percent.
- All criteria, and not stability value alone, must be considered in designing an asphalt paving mix.
- Hot-mix asphalt bases which do not meet the above criteria when tested at 140 °F (60 °C) should be satisfactory if they meet the criteria when tested at 100 °F (38 °C) and are placed 4 inches (102 mm) or more below the surface. This recommendation applies only to regions having climatic conditions similar to those prevailing throughout most of the United States. Guidelines for applying the lower test temperature in regions having more extreme climatic conditions are being studied.

Table 26. Marshall design criteria.[34]

Traffic Category	Heavy		Medium		Light	
No. of Compaction Blows Each End of Specimen	75		50		35	
Test Property	min.	max.	min.	max.	min.	max.
Stability, all mixtures, lb N	 750 3,336	-	 500 2,224	-	 500 2,224	-
Flow, all mixtures, 0.01 in (0.25 mm)	8	16	8	18	8	20
Percent Air Voids, Surfacing or Leveling Base	 3 3	 5 8	 3 3	 5 8	 3 3	 5 8
Percent Voids in Mineral Aggregate	See table below					

The following notes must be made relative to the Marshall design procedure:

- Laboratory compactive efforts should closely approach the maximum density obtained in the pavement under traffic.
- The flow value refers to the point where the load begins to decrease.
- The portion of the asphalt cement lost by absorption into the aggregate particles must be allowed for when calculating percent air voids.
- Percent voids in the mineral aggregate is not to be calculated on the basis of the ASTM bulk specific gravity for the aggregate.
- All criteria, and not stability value alone, must be considered in designing an asphalt paving mix.
- Hot-mix asphalt bases which do not meet the above criteria when tested at 140 °F (60 °C) should be satisfactory if they meet the criteria when tested at 100 °F (38 °C) and are placed 4 inches (102 mm) or more below the surface. This recommendation applies only to regions having climatic conditions similar to those prevailing throughout most of the United States. Guidelines for applying the lower test temperature in regions having more extreme climatic conditions are being studied.

Cutback Asphalt

Design procedures for mixtures stabilized with cutbacks have been standardized by the Asphalt Institute.(125) A summary outline of the design methods for both Hveem and Marshall test procedures are shown in figure 50. Details of the mixture fabrication, curing, and testing procedures can be found in reference 13. Design criteria are shown in tables 27, 28, and 29. The critical elements of the above-suggested procedure are control of the mixing temperature, volatile content at compaction, method of curing prior to testing, test temperature, and water susceptibility test. It should be noted that the test temperature is 77 °F (25 °C) and not the 140 °F (60 °C) normally associated with Hveem and Marshall testing of asphalt cement-stabilized materials.

Emulsified Asphalt

Design procedures for mixtures stabilized with emulsions have also been prepared for Hveem and Marshall testing procedures, but are not standardized at present. A summary outline of suggested design methods for both the Hveem and Marshall test procedures is shown in figure 51. The suggested procedures are based on procedures outlined by the Asphalt Institute.(126) Criteria for the Hveem and Marshall procedures are shown in tables 30 and 31, respectively. Details of the mixture fabrication, curing, and testing techniques can be found in reference 23.

The critical elements of both procedures are the moisture content during mixing and compaction, method of curing prior to testing, test temperature, and water susceptibility test. As with cutback mixtures, testing for emulsified mixes is conducted at 77 °F (25 °C) in both methods. Furthermore, in the Hveem method, the Resistance R-value and Cohesiometer C-value are obtained rather than the Stabilometer S-value.

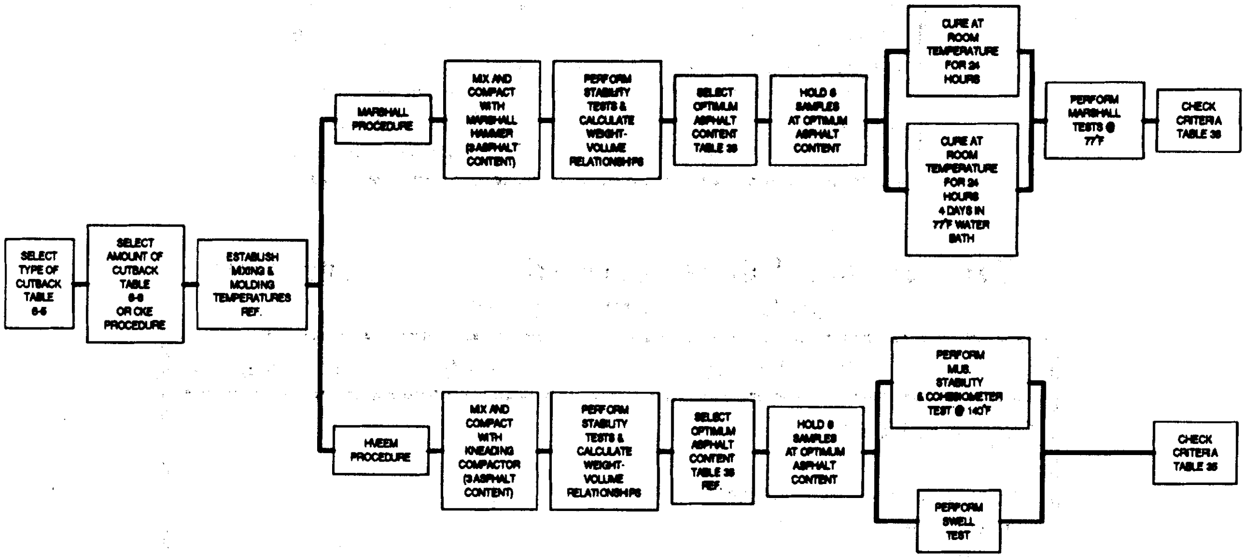


Figure 50. Selection procedure for asphalt cements.

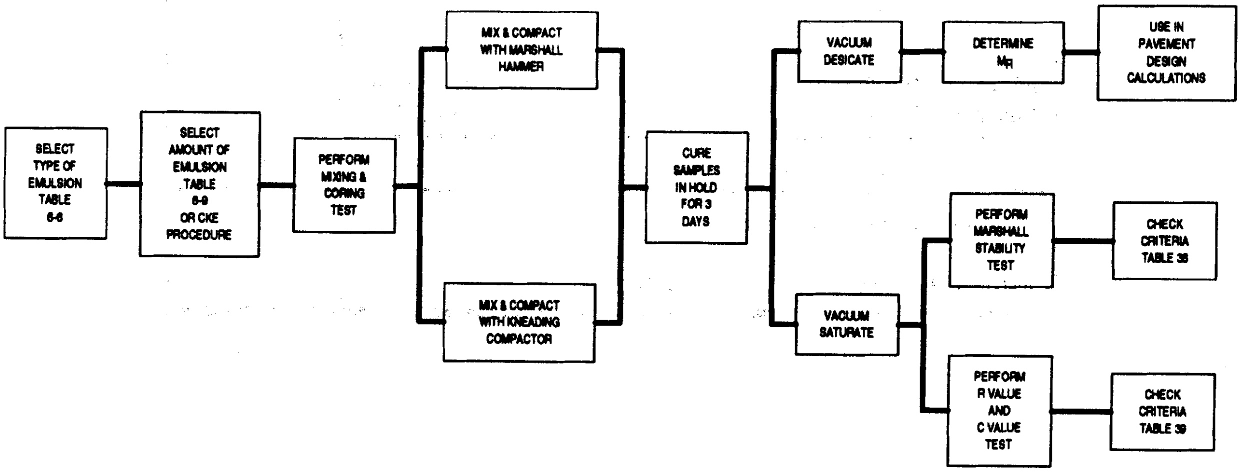


Figure 51. Selection procedure for cutbacks.

Table 27. Suggested criteria for cutback asphalt mixes.[125]

Test	Requirement
Stabilometer value	30 min.
Moisture Vapor Susceptibility (Stabilometer value)	20 min.
Swell	0.30 in max. (0.76 mm max.)

Table 28. Marshall design criteria for paving mixtures containing cutback asphalt.[125]

Test Property	Minimum	Maximum
Degree of Curing		
Percent solvent evaporated		
Maintenance Mixtures	25	
Paving Mixtures	50	
Number of Hammer Blows		
Hand Compactor	75	
Percent Air Voids in Compacted Mix	3	5
Percent Voids in Mineral Aggregate (VMA)		
Stability, lb (N) at 77 °F (25 °C)		
Maintenance Mixtures	500 (2224)	
Paving Mixtures	750 (3336)	
Flow, units of 0.01 in (0.25 mm)	8	16
Percent Stability Retention		
After 4 days in water at 77 °F (25 °C)	75	

Table 29. Minimum percent voids in mineral aggregate.[125]

USA Standard Sieve No.	Nominal Maximum Particle Size, in	Minimum % VMA
No. 16	0.0469	23.5
No. 8	0.093	21
No. 4	0.187	18
3/8 in	0.375	16
1/2 in	0.500	15
3/4 in	0.750	14
1 in	1.0	13
1-1/2 in	1.5	12
2 in	2.0	11.5
2-1/2 in	2.5	11

7 SUMMARY

While stabilization has been conducted using all types of asphalt products, the most common forms are the emulsion and the cutback materials. Because asphalt stabilization is accomplished primarily through the coating and waterproofing action of the asphalt films surrounding the soil or aggregate particles, the construction requirements play a significant role in determining the suitability of this form of stabilization. Both fine and coarse grained materials can be stabilized. Because of difficulty in obtaining complete coating, high plasticity materials are difficult to construct. Soils with high amounts of fines require significantly more asphalt for complete coating, and stabilization may be more economical with another stabilizer. Generally the PI should be less that 10, and the fines less than 25 percent to assure proper construction can be achieved.

Example Problem

The use of tables 17 through 31 and figures 41 through 49 can best be explained by the use of an example problem. A secondary road is to be constructed in a rural area of Southwest Texas. The surface soils in this area are primarily silica sands. Samples of the soil have been obtained, from which the following was found:

Percent passing 3/8-in sieve = 100
Percent passing No. 4 sieve = 90
Percent passing No. 10 sieve = 82

Percent passing No. 40 sieve = 45
Percent passing No. 80 sieve = 30
Percent passing No. 200 sieve = 15
Material smaller than 0.05 mm (silt clay combined) = 12
Material smaller than 0.002 mm (clay) = 9
Liquid Limit = 20
Plastic Index = 9

Table 17 indicates that from a gradation and plasticity standpoint the soil will make a fair soil-asphalt subbase. Because of its fine-grained nature, the soil is most suitable for stabilization with either a cutback or emulsion. A hot mix operation is discouraged for a soil of this nature.

Cutback Stabilization

Table 20 can be used to select the type of cutback and table 23 can be used to determine the approximate quantity of cutback required. The anticipated aggregate temperature at the time of construction is expected to be 80 °F (27 °C). Table 20 indicates that this soil can best be stabilized with a MC-800. The equation contained in table 23 will allow for the engineer to estimate the quantity of cutback. Plotting the results of the soil gradation on aggregate grading charts indicate the following:

$$a = 50,\ b = 25,\ c = 10 \text{ and } d = 15$$

Thus the percent residual asphalt by weight of dry aggregate is 7.25 (from table 23). The amount of MC-800 will be 7.25/0.80 or 8.5 percent, as the MC-800 to be used has 15 percent solvent.

Marshall or Hveem stability tests should be performed as shown in figure 48, and at asphalt contents of 7.5, 8.5, and 9.5. Results of these tests should be compared with appropriate criteria presented in tables 27, 28, and 29.

Emulsion Stabilization

Figures 46 and 47 and table 21 can be used to select the type and grade of emulsion. Figures 46 and 47 suggest that a cationic emulsion will be best suited for this siliceous soil. Table 21 indicates that a CSS-1 or a CSS-1h emulsion are preferred. A CSS-1h will be selected due to the summer pavement temperatures.

Table 24 estimates that 7.9 percent emulsion will be required for this soil. Marshall or Hveem stability tests should be performed as shown in figure 49 at emulsion contents of 7, 8, and 9 percent. Results of these tests should be compared with appropriate criteria presented in tables 30 and 31.

Table 30. Design criteria for emulsified asphalt-aggregate mixtures.

Test Property		Base Mixtures	Surface Mixtures
REISTANCE R_t Value at 73 ± 5 °F	Early Cure (23 hrs)	70 min.	N. A.
	Fully cured and water immersed (72 hrs)	78 min.	N. A.
STABILOMETER S Value at 140 ± 5 $_o$F		N. A.	30 min.
COHESIOMETER C Value at 73 ± 5 °F	Early Cure (23 hrs)	50 min.	N. A.
	Fully cured and water immersed (72 hrs)	100 min.	N. A.
COHESIOMETER C Value at 140 ± 5		N. A.	100 min.
Agregate Coating (%)		50 min.	75 min.

Table 31. Emulsified asphalt-aggregate mixture design criteria based on Marshall procedures.(109)

Test Property	Minimum	Maximum
Stability, lb (N) at 72 °F (22.2 °C)		
Paving Mixtures	500 (2224)	-
Percent Total Voids		
Compacted Mix (granular mixes, no requirement for sand)	2	8
Percent Stability Loss		
After 4 days soak at 72 °F (22.2 °C)	-	50
Percent Absorbed Moisture		
After 4 days soak at 72 °F (22.2 °C)	-	4
Aggregate Coating, percent	50	-

CHAPTER 7 LIME-FLY ASH STABILIZATION

1. INTRODUCTION

The use of lime-fly ash-aggregate (LFA) mixtures in road construction has steadily increased since this mixture was introduced in the United States in the early 1950's. From its initial application in private and low volume roads, the material has gained acceptance to the point where it is now in the specifications as an accepted material for bases and subbases in a number of states and many countries around the world. The material has also been approved for use by both the Federal Highway Administration (FHWA) and the Federal Aviation Administration (FAA). It is currently used in the construction of the full range of pavements from low volume roads to the heavy duty pavements normally used for airport pavements and port facilities. Several million tons of lime-fly ash-aggregate mixtures are used on an annual basis in pavement construction.

In-service pavements built with LFA mixtures, or lime-cement-fly ash-aggregate (LCFA) mixtures, have base thicknesses which range from 4 to 6 in (100 to 150 mm) for light duty parking facilities, to 6 to 10 in (150 to 250 mm) for medium and heavy duty pavements, and up to 30-in (750 mm) for the very heavy duty pavements. A wearing surface is always used with these materials. The wearing surface can be either a bituminous surface treatment for light duty pavements, or hot mix asphalt concrete ranging from 2 to 6 in (50 to 150 mm) in thickness for medium and heavy duty pavements.

2. TYPES OF FLY ASH

Fly ash is the particulate matter in the stack gas that results from the burning of coal, lignite, or like materials. Fly ash is collected from flue gasses of smoke stacks by mechanical means such as cyclonic or bag house collectors or with electrostatic precipitators. The collected fly ash is a very finely divided, powdery substance, composed primarily of silica and alumina, but also containing oxides of iron, calcium, magnesium and sulfur as secondary ingredients.(127)

The characteristics of the fly ash are largely determined by the type of coal burned, the type of combustion equipment utilized, type of air quality control equipment, and the method of handling the fly ash. Estimates of fly ash characteristics can be made prior to plant "start-up"; however it is important that the physical-chemical properties of the fly ash be determined after the plant has reached a relatively steady state operation.

There are two major types of fly ash that are currently used in construction today, class "F" and class "C". Class F is obtained from burning Bituminous coal and generally contains less than or equal to 10 percent Calcium Oxide (CaO). Class C is obtained from burning subbituminous or lignitic coal, and generally contains greater than or equal to 10 percent CaO.

Collected fly ash can be stored either in a dry state or conditioned state. The dry state generally requires storage in silos or other protected bins. The conditioned state refers to fly ash which is dampened while it is stored to reduce dust problems. Fly ash can also be sluiced into storage ponds. In the dry state, fly ash is chemically and physically stable and will not change with time. In the conditioned state, fly ash which contains significant quantities of CaO may take on a "set" and will require further processing. These fly ashes must be crushed to a reasonably fine state before they can be used in mixes (see the discussion of fly ash fineness below). Conditioned fly ashes which do not take on a set are chemically and physically stable and can be stored indefinitely in this state and be used in mixes without any further processing. Fly ashes which are stored in sluice ponds will usually segregate by particle size and may react chemically. These factors can result in fly ash which will not produce a product as uniform or as desirable as fly ash which is stored in the dry or conditioned state. As a rule, pond ashes are not suitable for use in lime-fly ash mixes except as mineral fillers.

Specifications for dry fly ash for use in LFA mixtures are given in ASTM C 593 "Fly Ash and Other Pozzolans for Use with Lime". Conditioned fly ash which takes on an initial set must be crushed before use, but is usually not crushed to the same degree of fineness as dry fly ash. While it has been shown that the amount of minus No. 200 material in the fly ash is a most critical factor in the reactivity of fly ash, most specifications for crushed fly ash specify a fineness in the following range for production control:

Sieve Size	Percent Passing
1/2 in (12.2 mm)	100
3/8 in or #4	80 - 90
#10	65 - 75

As a practical matter, however, these fly ashes should be checked periodically to ensure that they contain a desired percentage of fines (material passing the No. 200 sieve).

3. SOIL, LIME-FLY ASH REACTIONS

Reactions which occur in lime-fly ash-water systems are quite complex. Several studies have provided basic information pertaining to these reactions. Based on his own laboratory investigations as well as a review of other studies documented in the literature, Minnick concludes that the major cementing compounds formed in lime-fly ash mixtures are probably calcium silicate hydrates, with other less common compounds and minerals such as ettringite.(128) Low-sulfate sulfoaluminates may also be formed under favorable conditions.

The amorphous glassy component in the fly ash provides the constituent elements which form the complex silicate and aluminate compounds. The strength and durability of the lime-fly ash mixtures are directly related to the quantity of cementitious compounds formed by the reaction of the lime ($Ca(OH)_2$ and MgO) and the fly ash constituents. Since these reactions are also affected by time and temperature, the curing time and curing conditions have a significant effect on the properties of the mixes.

The reactivity of fly ash from various sources is quite variable. For this reason, fly ash from each source should be carefully tested for reactivity before approved for use in LFA and LCFA mixes. Monitoring programs should be set up to establish the uniformity of the fly ash from each source. Several studies have established some basic properties of fly ash which are at least indicative of the fly ash reactivity.(129,130,131,132) These include:

- Increased percentage of fly ash passing the No. 325 (45 m) sieve, which increases the pozzolanic activities of fly ash(129)
- Increased surface area(130,131)
- Increased SiO_2 content(133,134)
- Increased $Si_2 + R_2O_3$ content ($R = Ca^{++}$ or Mg^{++})(131)
- Increased $SiO_2 + Al_2O_3$ content(130)
- Low carbon content(129)
- Low loss on ignition(131)
- Increased alkaline content(131)

Minnick et. al. emphasized that "no single test on fly ash will predict the performance of that material in compositions in which it is used", but that "it is far more preferable to combine factors or develop multiple factors in making performance predictions."(132)

In addition to the primary reaction between lime and the fly ash, the lime may also react with the fines in the material being stabilized. Soil-lime reactions that may occur are cation exchange, flocculation-agglomeration, and a soil-lime pozzolanic reaction as discussed in Chapter 4, Lime Stabilization.

Cation exchange and flocculation-agglomeration reactions take place quite rapidly and cause decreased plasticity of the fines and some immediate strengthening. The plasticity reduction improves workability and allows easier mixing with materials that contain substantial quantities of plastic fines.

Reaction products from the soil-lime pozzolanic reaction contribute to the development of the cementitious matrix in the stabilized mixture. Similar secondary soil-lime reactions have been noted for soil-cement mixtures containing lime-reactive fines.

4. SOILS SUITABLE FOR LIME-FLY ASH STABILIZATION

Lime and fly ash treatment can significantly improve the properties of fine grained soils and soil aggregate mixtures. The mechanism by which this improvement is achieved and the degree of improvement is greatly affected by the mineralogy and the fineness of the soil. While there have been a number of successful applications in which natural fine grained soils have been treated with lime and fly ash, the technology for use of this method of treatment is not sufficiently developed to permit a generalized statement of the effect of the treatment on the soils. Some general considerations for selection of soil type for treatment are given below.

The methods used for placing the LFA and LCFA mixtures vary and can have some effect on performance of the material. LFA or LCFA mixtures which incorporate fine- or coarse-grained aggregates can be mixed in-place, but are generally plant-mixed and shipped to the site for better uniformity.(135) LFA and LCFA mixtures with natural soils are generally mixed in-place using dry lime and dry fly ash, though some research has investigated the use of lime-fly ash slurry pressure injection (LFASPI) to stabilize active clay soils with some success.(136)

Fly ashes are normally used in lime-fly ash mixes as a pozzolan and as a filler for the voids. Since the particle size of fly ash is normally larger than the voids in the fine grained soils, it is not appropriate to use fly ash as a filter in fine grained soils. Thus the only role for the fly ash in stabilization of fine grained soils is that of a pozzolan. It is noted, however, that clays are often pozzolanic in nature (Chapter 4). Consequently, silts are generally considered the most suitable soil type for treatment with lime and fly ash. Research has been done on the use of lime-fly ash-soil mixtures incorporating fine grained soils, such as silts and clays, which occur naturally at the site. (See references 137 through 144). The resulting mixtures, if designed to be economically competitive with other methods of construction, are usually not of as high quality as the LFA mixtures, in part because of the initial lack of mechanical stability in the unstabilized soils and the greater tendency towards frost-susceptibility in the fine grained soils. Nevertheless, the lime-fly ash-soil mixtures have been found to be highly serviceable and economical in the following three areas of roadway construction:

- Base course for secondary roads, parking lots, and so on, where heavy traffic loads are not anticipated.
- Subbase beneath conventional pavements.
- Subgrade improvements to provide additional support for the pavement and/or remedy undesirable subgrade conditions to expedite construction.

A more detailed discussion on the stabilization of fine grained soils with lime and fly ash can be found in reference 145.

Because of the number of variables involved, and the state of technology, each lime-fly ash-soil mixture must be carefully evaluated for properties and characteristics.

Aggregates

Aggregates which have been successfully used in LFA mixtures cover a wide range of types and gradations, including sands, gravels, crushed stones, and several types of slag. (See references 129,130,131,146 through 151). Aggregates should be of a gradation such that, when mixed with lime, fly ash and water, the resulting mixture is mechanically stable under compaction equipment and capable of being compacted in the field to high density. Further, the aggregate should be free from deleterious organic or chemical substances which may interfere with the desired chemical reaction between the lime, fly ash, and water, and should consist of hard, durable particles which are free from soft or disintegrated pieces.

Fine grained aggregate mixtures have generally produced materials of greater durability than coarse grained mixtures. However, mixtures with coarser aggregate gradations are generally more mechanically stable and may possess higher strength at an early age. With time, however, mixtures with fine-grained aggregates may ultimately develop strengths which equal or exceed those obtained with coarse grained aggregates. The key to the ultimate strength development lies in the lime-fly ash matrix rather than the aggregate. Newly released guide specifications and ASTM C593 do not have a gradation requirement, but rely solely on the actual cured strength.(152) Typical aggregate gradations which have been specified for use in LFA mixtures are shown in table 32. Some optimum gradation ranges for well-graded aggregates are given in table 33. Other typical requirements for aggregates appear in table 34.

Table 32. Typical aggregate specifications for LFA mixtures.

Sieve Size	Percent Passing (by weight)			
	Illinois	Pennsylvania		Ohio
2 in	---	100	100	100
1.5 in	100	---	---	---
1 in	90-100	---	---	---
3/4 in	---	52-100	70-100	---
1/2 in	60-100	---	---	50-85
3/8 in	---	36-70	58-100	---
No. 4	40-70	24-50	45-80	35-60
No. 8	---	---	---	15-45
No. 16	---	10-30	25-50	10-35
No. 40	0-25	---	---	---
No. 50	---	---	---	3-18
No. 100	---	---	6-20	---
No. 200	0-10[1]	0-10	---	1-7
	0-15[2]			

[1] Gravel, [2] Crushed Stone and Slag,

Table 33. General requirements for gradation of aggregate for the plant-mix base course.

Sieve Designation (square openings)	Percent passing (by weight)		
	A	B	C
2 in	100	---	---
1.5 in	---	100	---
1 in	55-85	70-95	100
3/4 in	50-80	55-85	70-100
No. 4	40-60	40-60	40-65
No. 40	10-30	10-30	15-30
No. 200	5-15	5-15	5-15

Table 34. Other typical requirements for aggregates.

Property	Illinois	Pennsylvania	Ohio	FAA
Sodium Sulfate Soundness (AASHTO-T104)	<25%	<20%	<15%	<12%
Los Angeles Abrasion (AASHTO-T96)	<45%	<55%	---	---
Plasticity Index	<9	<6	---	<6
Liquid Limit	---	<25	---	<25

5. TYPICAL PROPERTIES OF LIME-FLY ASH STABILIZED SOILS

Pozzolanic reactions which give LFA mixtures their long-term strengths are influenced by many factors, including ingredient materials, proportions, processing, moisture content, field density, and curing conditions. The pozzolanic nature of fly ash and its reaction with lime is discussed in Chapter 4, with details on how characteristics of the fly ash itself affect the reaction.

Thorough mixing is essential for an LFA mixture to develop its maximum possible strength. The time required to achieve a uniformly blended product depends upon the type and efficiency of the available mixing equipment, mixture proportions, and to some extent, on the ingredients themselves.

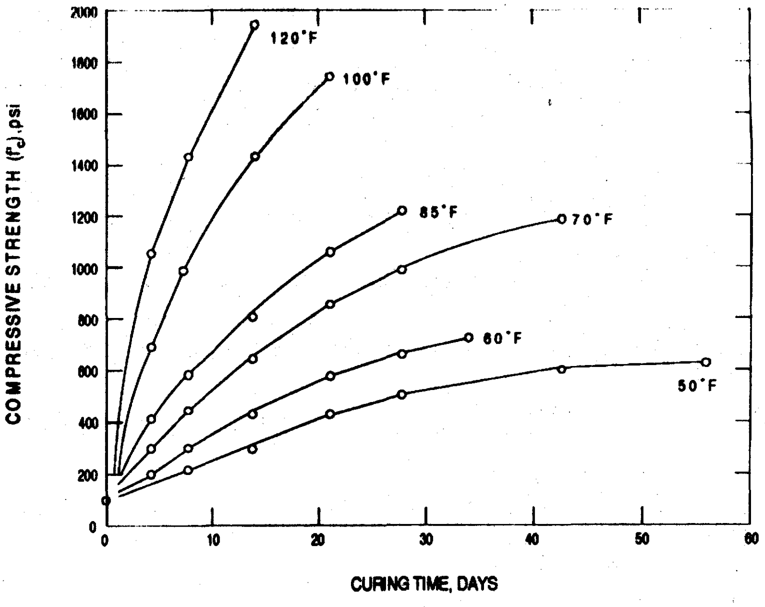
Because of the combined effects of time and temperature on the strength development of LFA mixtures shown in figure 52, it is difficult to specify combinations of curing conditions which simulate field conditions. One method of taking into account the combined effects of temperature and time is to combine the two variables into a single variable called a degree-day.(153) Procedures for establishing reasonable cut-off dates for construction based on the degree-day concept are contained in NCHRP Syntheses 37.(134)

While curing at low temperatures retards the reaction process of LFA mixtures, and almost entirely stops the reaction below 40 °F (4 °C), reduced temperatures or even freezing of the mixtures have no apparent permanent detrimental effects on the chemical properties of the constituents.(154,155) Although these materials may be subjected to a significant number of freeze-thaw cycles in the field during winter months, increases in strength are again developed with rising temperatures during spring and summer, as is illustrated in figure 53.

Under acceptable curing conditions, pozzolanic reactions in LFA mixtures continue as long as sufficient lime and fly ash are available to continue the reaction.

Cores taken from pavements over a 10-year period indicate a continuing development in the strength of the mixture with time, as is shown in figure 50.

This continuing pozzolanic process can manifest itself in a phenomenon called autogenous healing, which is one of the properties of LFA mixtures and is discussed later in this section.(156,157,158) There are a number of recorded cases where distressed areas caused by improper loading of LFA pavements during the early cure stages have healed with time. This can only occur however, if there are sufficient quantities of unreacted lime and fly ash available to provide the necessary reaction components.



1 psi = 6.89 x 10^3 Pa

°F = 5/9 (°C-32)

Figure 52. Curing time effect on strength of LCFA mixtures at various temperatures.

Admixtures

In an effort to accelerate development of early strength and improve the short-term durability characteristics of LFA mixtures, thereby permitting extension of the construction period later into the fall, admixtures have been added to accelerate or complement the lime-fly ash reactions. Most of the work in this area has been with chemicals in liquid suspension or in powdered form.

Portland cement is an effective admixture for use in LFA mixtures. The early strength development associated with the hydration of portland cement complements the slower strength development associated with some lime-fly ash reactions (see references 151,159,160,161).

Certain other admixtures (e.g., water-reducing agents) may also give beneficial results. However, the use of many admixtures may be impractical due to handling problems and prohibitive costs.

Compressive Strength

Properly designed LFA mixtures compacted to a high relative density and properly cured may ultimately develop compressive strengths well in excess of 3,000 psi (20,680 kPa). Materials cured for seven days at 100 °F (38 °C) normally develop compressive strengths in the range of 500 to 1,000 psi (3,450 to 6,890 kPa). These same materials are likely to develop compressive strengths in excess of 1,500 psi (10,340 kPa) after one or two years in service, as is shown in figure 53.

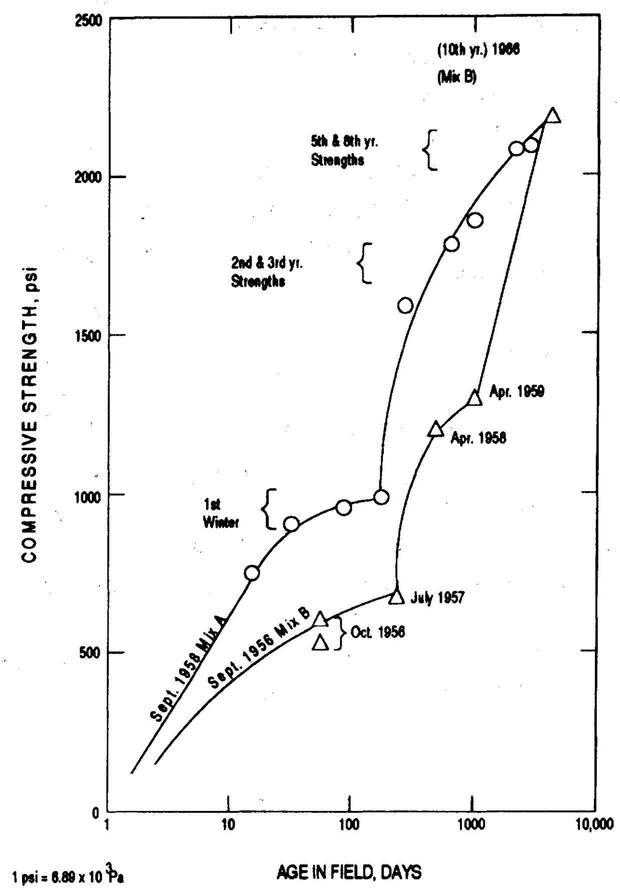


Figure 53. Effect of age on compressive strength.

Flexural Strength

LFA mixtures, like all granular and/or cementitious materials, are significantly stronger in compression than in tension. Thus, the tensile strength is a critical indicator of quality. Because pure tensile strength is difficult to measure in these types of mixtures, an effective alternate method of evaluating the tensile capacity is through a determination of the flexural strength or modulus of rupture. Although flexural strength can be determined directly from tests, most agencies estimate the flexural strength by taking a ratio of the material's flexural strength to compressive strength. The ratio of flexural to compressive strength for most LFA mixtures is between 0.18 and 0.25. An average value of 0.20 for the ratio is a good, conservative, engineering estimate of the flexural strength, as is illustrated in figure 54.(162)

Durability

Durability is a measure of a material's ability to perform in an unfavorable environment and is the most important property in the performance of these mixtures in areas subject to freeze-thaw cycles and the use of deicing salts. Properly designed LFA mixtures meet durability criteria for high quality base materials. Several methods for evaluating the durability of LFA mixtures have been developed (see references 129,162 through 167). These include a strength loss upon vacuum saturation, detailed in ASTM C 593, a minimum strength criterion based on a residual strength following freeze-thaw cycling, and a weight loss on freeze-thaw.(128,130,162,165,168)

Stiffness

The stiffness of LFA mixtures is usually expressed in terms of their modulus of elasticity (E) in bending. Typical E values for LFA mixtures range from 0.5 to 2.5 x 10^6 psi (3.4 to 17.2 x 10^6 kPa). Specific values depend on whether a tangent modulus or secant modulus is used, as the relationship between stress and stain is non-linear. Figure 55 illustrates a typical stress-strain relationship using moment as a stress indicator and curvature as a strain indicator. The expected range of E values for a specific LFA mixture is a function of several factors, in particular the aggregate characteristics such as particle hardness and gradation, the degree of compaction and the extent and type of curing of the mixture (see references 147, 162,169,170,171).

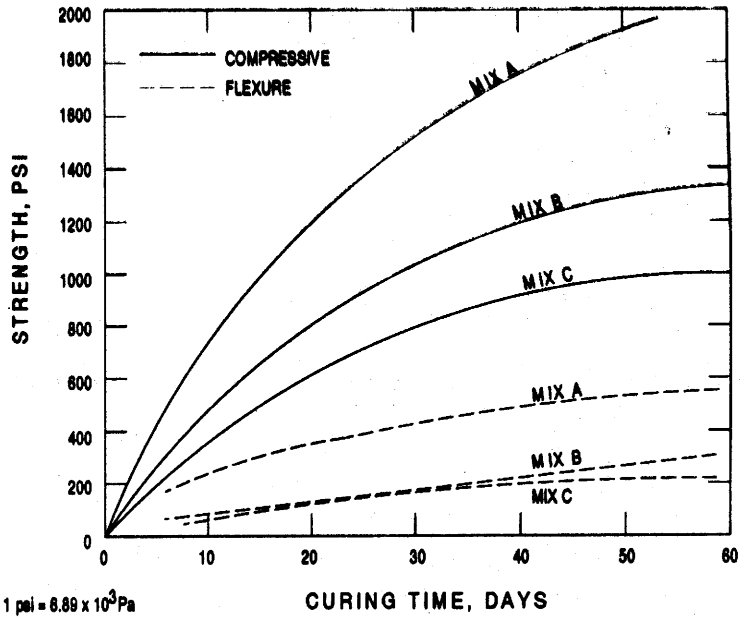


Figure 54. Flexural andcompressive strengths ofLFA mixtures cured at ambient temperature.

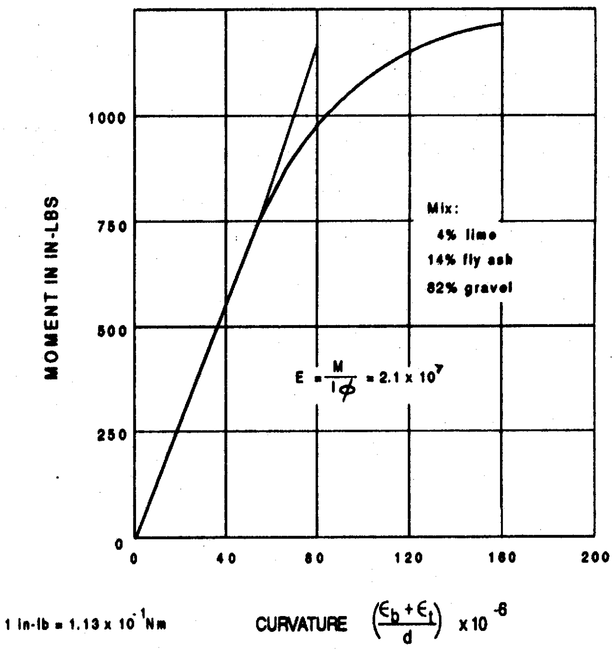


Figure 55. Moment-curvature relationship for LFA mixtures.

Autogenous Healing

Autogenous healing is the ability of LFA mixtures have to heal or re-cement across cracks by a self-generating mechanism, as illustrated in figure 55. The degree to which autogenous healing occurs is dependent upon many factors, including:

- The age at which the mixture cracks.
- The degree of contact of the fractured surfaces.
- The curing conditions.
- The availability of reaction products (lime and fly ash).
- The moisture conditions.

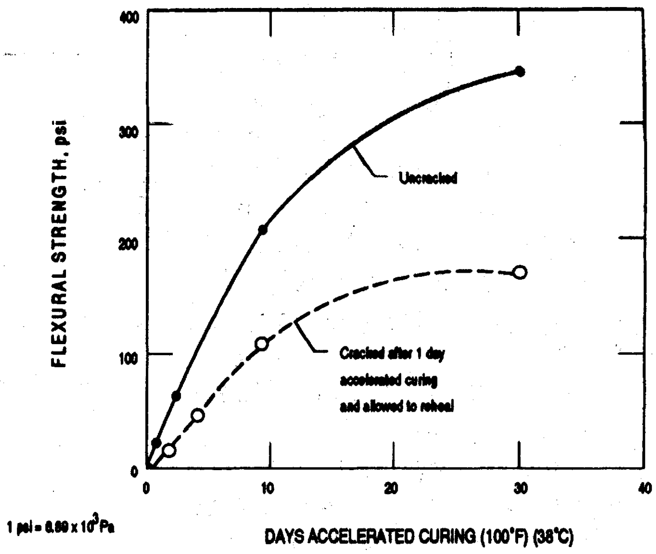


Figure 56. Effects of fracture on strength of LFA mixtures.

Autogenous healing in LFA mixtures produces a material that is less susceptible to deterioration under repeated loading and is more resistant to attacks by the elements than other materials which develop this characteristic to a lesser extent (see references 129,158,162,163,164,168,174).

Fatigue

Like all paving materials, LFA mixtures can fail under repeated loading at stress levels considerably less than the ultimate stress required to cause failure in a single load application. This is illustrated in figure 57. However, because of autogenous healing characteristics, LFA mixtures are less susceptible to failure by fatigue than most other paving materials. This is due to the healing process, which can provide a greater curing effect than the damage being caused by the repeated loads.(162,170) Unless fatigue failure occurs during the first few days of loading, it is not normally a factor in the performance of these pavements, when properly designed.

Poisson's Ratio

The Poisson's ratio of LFA material usually varies somewhat with the intensity of the applied stress. However, for most mixtures, this ratio usually remains relatively constant at a value of about 0.08 at stress levels below approximately 60 percent of ultimate, and then increases at an increasing rate with the stress level to a value of about 0.3 at failure, as illustrated in figure 58.(156,162) For most calculations, Poisson's ratio for LFA mixtures can be taken as between 0.10 and 0.15 without appreciable error.

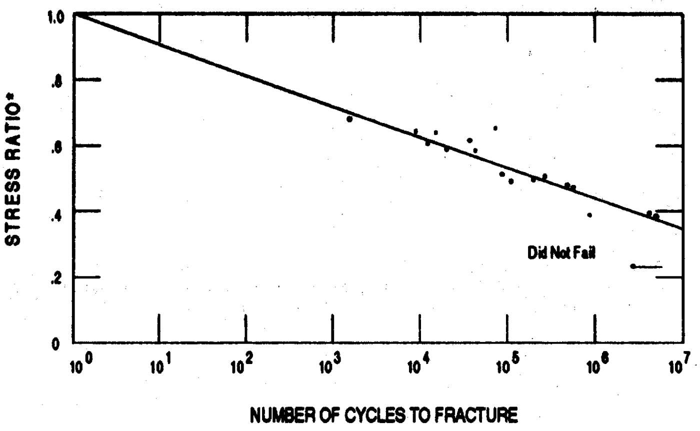


Figure 57. Fatigue relationship for LFA mixture.

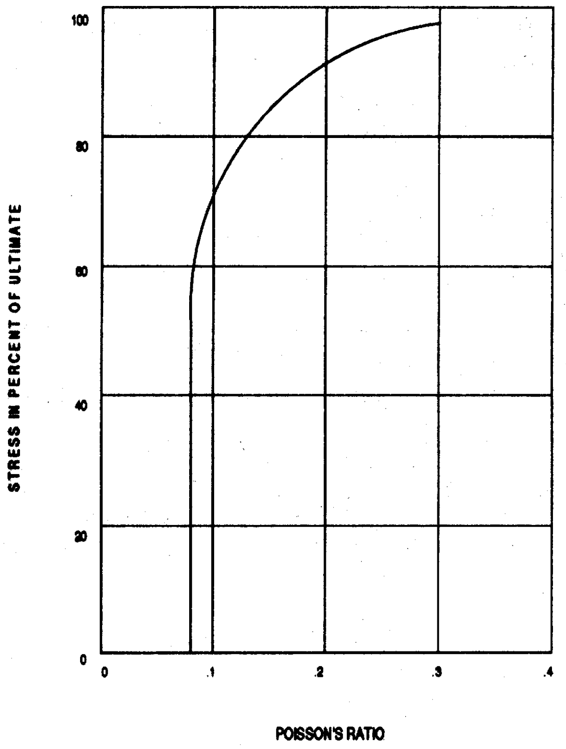


Figure 58. Poisson's ratio for various stress levels.

Coefficient of Thermal Expansion

Hardened LFA materials, like all stabilized paving materials, are subject to dimensional changes with changing temperature. The coefficient of thermal expansion of LFA mixtures is influenced primarily by the aggregates and the moisture content of the materials, as is illustrated in figure 59. Typical values for the coefficient are about the same as for concrete at the same moisture content, or approximately 6 to 10 x 10^{-6} inches per inch per degree Fahrenheit (see references 149, 156, 157, 162). This similarity has a serious implication for the construction of pavement bases using these mixtures, with newer guide specifications requiring sawing for crack control, and joint sealing in the asphalt surface.(152)

Leaching Potential of Fly Ash

The leaching potential for fly ash used in soil stabilization should be determined because of the possibility that heavy metals from the ash could migrate to groundwater systems.(172) Research efforts have investigated the leachability of the two predominant types of fly ash; subbituminous fly ash and lignite fly ash.(172,173) In two studies conducted by the Louisiana Department of Transportation and Development, fly ash-soil mixtures were leached following the E.P.A. multiple extraction procedure and analyzed by Inductively Coupled Argon Plasma Spectrometry (ICAP). Table 35 lists the elements of both the lignite and the subbituminous fly ashes, and the quantities of each element found in each ash type. Also presented in table 35 are the percentages of the elements which were leached from soil-fly ash mixtures which were prepared using a fly ash-soil ratio of 30:70.

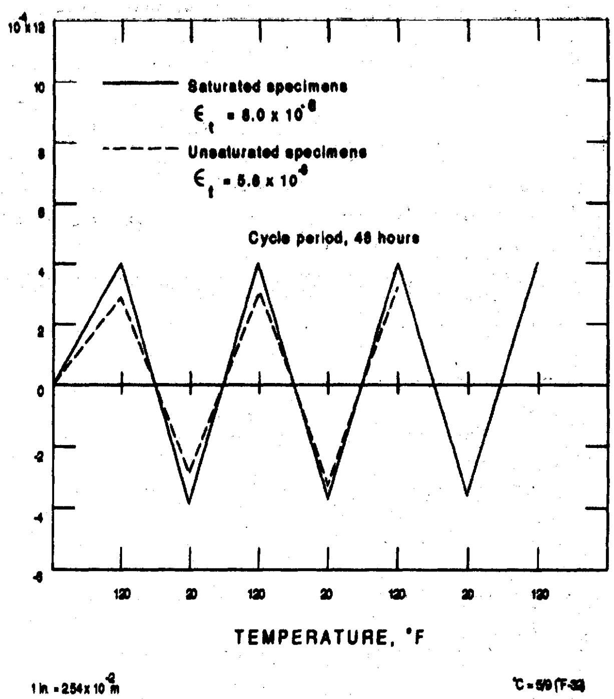


Figure 59. Change in length of cured LFA mixtures.

Neither of the two fly ashes evaluated produced levels of heavy metals in excess of the Resource Conservation and Recovery Act (RCRA) Standards, though both produced heavy metal concentrations exceeding the National Interim Primary Drinking Water (PDW) Standards with some elements. The elements which exceeded PDW standards for each fly ash type are:

<u>Lignite</u>	<u>Subbituminous</u>
Arsenic	Arsenic
Cadmium	Cadmium
Manganese	Iron
	Lead
	Manganese

Table 35. Bulk Analysis and Leachability Potential of Lignite and Subbituminous Fly Ashes.

Element	Lignite Fly Ash		Subbituminous Fly Ash	
	Bulk Analysis (milligram per kilogram)	Percent Leachate from 30:70 Ash:Soil Material (%)	Bulk Analysis (milligram per kilogram)	Percent Leachate from 30:70 Ash:Soil Material (%)
Aluminum	13,100	0.15	13,660.00	0.70
Arsenic(1)	150	4.30	150.00	3.20
Cadmium(1)	3	15.00	1.60	56.30
Calcium	7,100	100.00	31,720.00	> 100.00
Copper(1)	140	0.00	220.00	0.00
Iron	61,300	< 0.01	28,240.00	0.04
Lead(1)	100	0.10	118.00	2.40
Magnesium	630	100.00	181.00	> 100.00
Manganese(1)	900	5.30	150.00	9.10
Molybdenum	20	7.50	25.00	7.20
Nickel	80	0.80	65.00	5.60
Phosphorous	350	1.70	5,750.00	0.55
Potassium	9,300	0.20	7,290.00	2.50
Silicon	68,400	2.70	225,410.00	0.80
Zinc(1)	60	0.00	153.00	0.00

[1] Heavy metals

Though no detrimental effects are expected as a result of fly ash stabilization, further research should be done in this area. The leachability of heavy metal elements from LFA mixtures should also be evaluated.

6. SELECTION OF LIME-FLY ASH CONTENTS

Approximate Quantities

The relative proportions of each constituent used in specific LFA mixtures vary over a range. Effective mixtures have been prepared with lime contents as low as 2 percent and as high as 8 percent, while fly ash contents varied from a low of 8 percent to a high of 36 percent (see references 131,146,156,174). Typical proportions are 2.5 to 4 percent lime and 10 to 15 percent fly ash. In some instances, small quantities (0.5 to 1.5 percent) of Type I portland cement have been used to accelerate the initial rate of strength gain in LFA mixes. Mix design procedures which have been developed are discussed below.

Detailed Testing

The acceptability of LFA and LCFA mixtures is determined by applying selected design criteria. Most mixture design procedures include both strength and durability criteria.

Minimum cured compressive strength and maximum weight loss criteria are specified by the Illinois Department of Transportation and the FAA as shown in table 36. The Pennsylvania Department of Transportation has a durability requirement, but not a strength requirement. ASTM C 593 specifies a minimum cured vacuum saturation strength of 400 psi (2,760 kPa).

Table 36. Specified Design Criteria for LFA and LCFA Mixtures

Agency	Minimum Compressive Strength, psi	Maximum Weight Loss[1], percent
ASTM C 593	400	—
Illinois Department of Transportation	400	10
Ohio Department of Transportation	400	10
Pennsylvania Department of Transportation	not specified	14
Federal Aviation Administration	400	14

[1] After 12 cycles of freeze thaw - ASTM D560, 1 psi = 6.89×10^3 Pa

The residual strength approach emphasizes that a sliding scale of quality should be specified, depending on the field service conditions anticipated for the mixture.(175) For example, little freeze-thaw action occurs in an LFA base course in the south-Central United States, but many freeze-thaw cycles occur in a base course constructed in the Northern States. In fact, Illinois is divided into three separate zones for the purpose of establishing stabilized durability criteria.

The objective of the mixture design procedures is to develop the proper proportions of lime (cement), fly ash, and aggregate. A flow diagram for the mix design procedure for LFA and LCFA mixes is shown in figure 60. The design mixture must satisfy three criteria:

- Possess adequate strength and durability for its designated use,
- Be easily placed and compacted, and
- Be economical.

For a given set of component materials (lime, cement, fly ash, and aggregates), the factors that can be varied are the lime to fly ash ratio and the ratio of lime plus fly ash to the aggregate fraction. If cement is used with lime, the ratio of lime to cement is also variable. It is often more economical to blend aggregates from several sources to achieve a blend which gives superior performance than to use just one aggregate source and vary the binder components.(151,164)

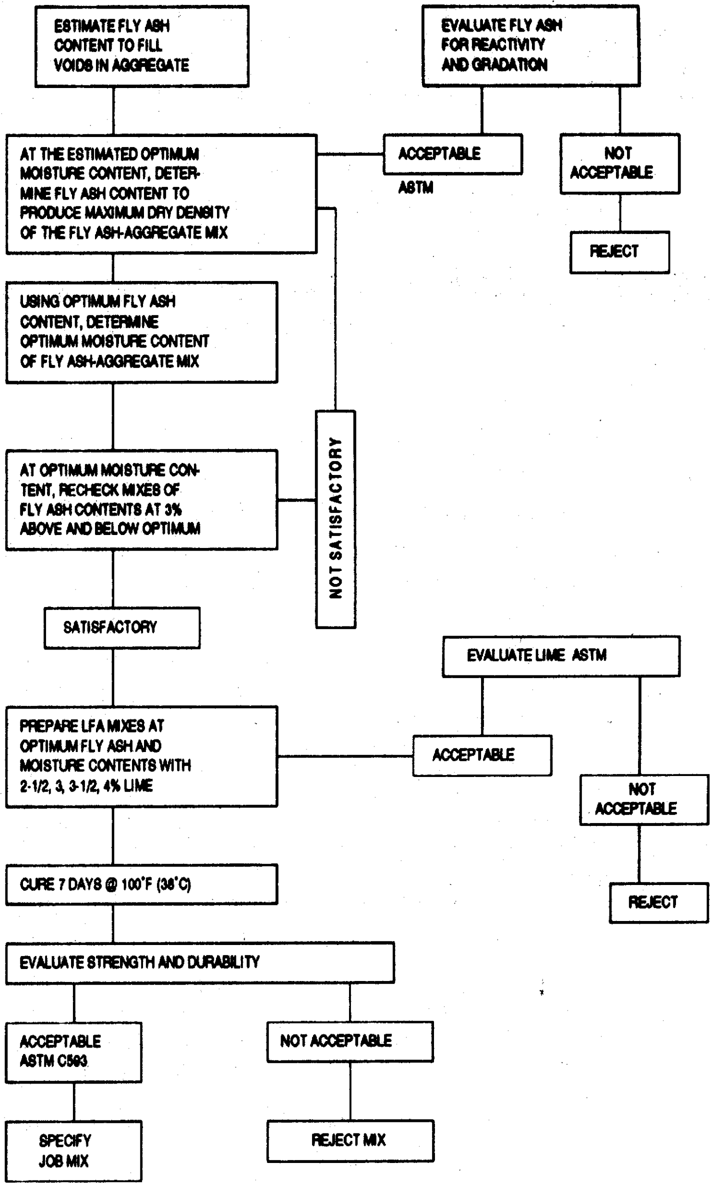


Figure 60. Mixture flow diagram.

The quality of LFA and LCFA mixtures, as measured by their strength and durability, is closely related to the quality of the cementitious matrix in the mixture. This matrix can be defined as the lime plus the fly ash and that portion of the aggregate finer than the No. 4 sieve. Only if there is sufficient matrix material to float the coarser aggregate fraction is it possible to achieve the high compacted density which is essential for good strength and durability in the mixture.(164) In general, the more uniform the particle-size distribution of the aggregate (dense graded), the lower the quantity of lime plus fly ash needed to achieve a highly compacted density in the matrix. Care must be taken, however, that the proportion of lime and fly ash in the matrix is sufficient to provide a good chemical reaction.(164) Also, sand aggregates with single-sized particles and sands devoid of minus No. 200-sized particles may require high fly ash content to serve as a filler or void reducer as well as a pozzolan in the mixture (see references 151, 160, 164, 170).

Figures 61 and 62 illustrate the variation of density and compressive strength with lime plus fly ash contents for both coarse- and fine-grained aggregates. To achieve a quality mixture, it is necessary that the amount of lime plus fly ash be slightly in excess (2 to 3 percentage points) of that required for maximum density. As indicated earlier, poorly graded materials, such as the Plainfield sand in figure 62, require a higher lime plus fly ash content because of the volume of voids to be filled.

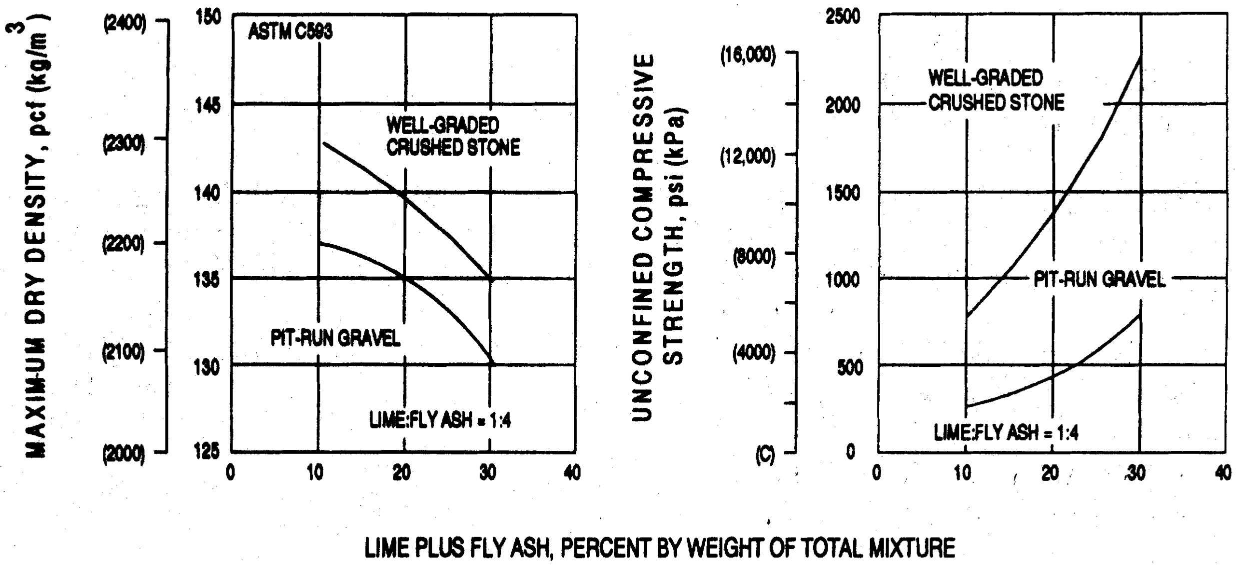


Figure 61. Variation of maximum density and compressive strength for LFA mixture.

The proper proportions of lime to fly ash, or lime to cement to fly ash, must be based on laboratory mix design data. These ratios do not remain constant, but are a function of the aggregate and fly ash properties and the rate of strength development in the mixture. Lime to fly ash ratios of 1:2 to 1:7 have been evaluated and found acceptable, but most mixtures have a ratio of about 1:3 or 1:4 for reasons of economy and quality.(170)

After the lime plus fly ash to aggregate ratio has been determined, the mixture should be evaluated and adjusted for quality by changing the lime to fly ash or lime to cement to fly ash ratios. This is done by preparing trial mixes, curing them for prescribed periods of time at a prescribed temperature [ASTM C 593 specifies 7 days

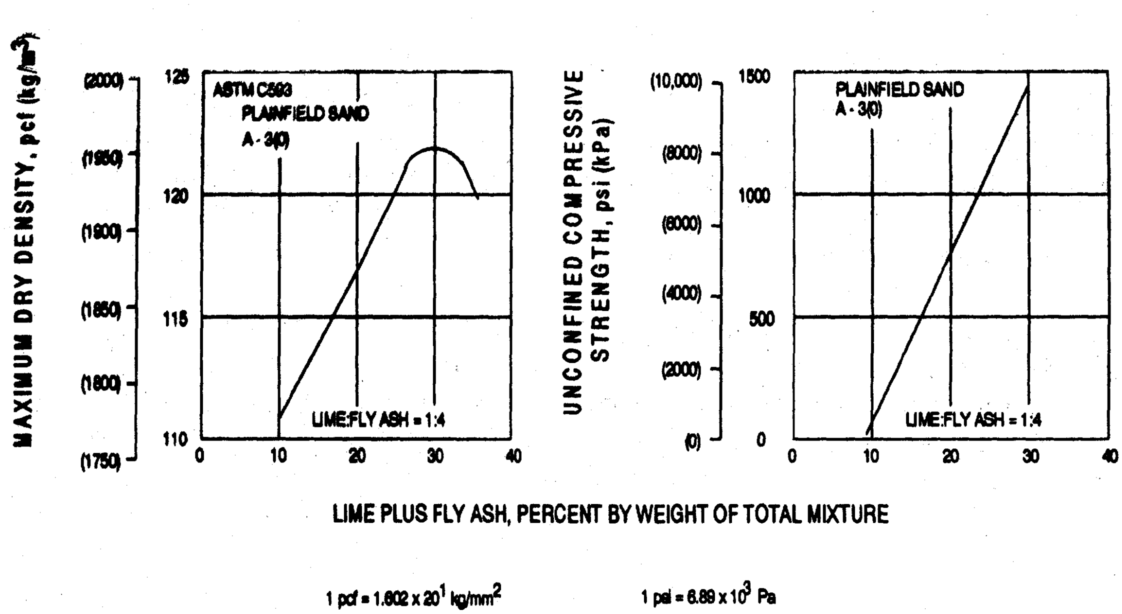


Figure 62. Variation in maximum density and compressive strength for LFA mixtures.

at 100 °F (38 °C)], and testing for strength, durability, and the rate of strength development. This last test requires curing at various temperatures for varying time periods. Durability requirements for these materials are given in ASTM C 593.

When lime, cement, and fly ash requirements have been established, the designated mix must be adjusted to compensate for construction variability. The amount of adjustment needed is related to the level of quality control provided by the producer. For typical operations, the lime plus fly ash content should be increased by about 2 percentage points, and the lime content by about 0.5 percent.(171)

In some instances, a less structured approach to mix design is used, and typical mixture proportions are evaluated for adequacy and quality. As a guide to selecting appropriate component ratios, the four mixtures shown in table 37 have provided highly serviceable mixtures for normal construction operations.

Laboratory Testing Program

Different laboratory tests are described below for the characterization of LFA and LCFA mixtures. A more complete description of the testing procedures are given in Chapter 3.

Moisture-Density Relationship. Moisture-density tests are conducted in the usual manner as described in AASHTO T180-74 with the exception of the compactive effort used. In table 38, several of the different compactive efforts in common use are summarized. In each case, 4-in (102-mm) diameter by 4.6-in (117-mm) high, 1/30-cf (9.4×10^{-4}-m^3) molds are used.

It is important to note that compacted density has a very significant effect on the cured strength of LFA and LCFA mixtures. Strength or durability criteria based on one compactive effort cannot be applied to mixtures prepared in accordance with procedures using other compactive efforts for design.

Compressive Strength Tests. Standard Proctor-sized specimens (4-in [117-mm] high) are most commonly used to evaluate the compressive strength of cured LFA or LCFA mixtures. Aggregate particles larger than 3/4 in (19 mm) are normally scalped from the aggregates and discarded. For fine-grained aggregate mixtures, such as those containing fine sand, 2-in (51-mm) diameter by 4-in (102-mm) high specimens have also been used, but there is difficulty in correlating the results from the two sizes of specimens.

Table 37. Typical LFA mixtures.

	Mix Aggregate			
	Crushed Stone	Gravel	Sand	Slag
Mix Component	(a) Mix Proportions - Percent by Weight[1]			
Aggregate	82 - 87.5	77 - 87.5	65 - 82	60 - 82
Fly Ash	10 - 14	10 - 18	15 - 30	15 - 35
Lime[2]	2.5 - 4	2.5 - 5	3 - 5	3 - 5
Sieve Size	(b) Aggregate Gradation - Percent Passing			
1 in	100	100	100	100
3/4 in	90 - 100	90 - 100	100	100
1/2 in	60 - 85	60 - 85	100	100
No. 4	50 - 75	50 - 75	90 - 100	90 - 100
No. 40	10 - 20	7 - 15	20 - 40	10 - 40
No. 100	2 - 5	3 - 6	0 - 3	0 - 2

[1] Based on total mix dry weight, [2] Lime or lime plus cement at a 3:1 ratio,

Table 38. Specified compactive efforts for LFA and LCFA mixtures

Agency	Procedure Designation	Compactive Effort[1]
Illinois Department of Transportation	---	10/18/3/25
Ohio Department of Transportation	ASTM C593	10/18/3/25
Pennsylvania Department of Transportation	PTM 106	5.5/12/3/25
Federal Aviation Administration	FAA T611	10/18/5/25

[1] Hammer weight (pounds)/height of drop (inches)/number of layers/blows per layer

It is essential to maintain a closely controlled environment during the curing of LFA and LCFA mixtures because both time and temperature have a profound effect on the strength and durability of these mixtures. Curing conditions (time in days and curing temperature) should always be specified along with the strength data. The standard curing conditions for these materials are 7 days and 100 °F (38 °C). For evaluation of the rate-of-strength development, other times and temperatures are specified, such as 28 days at 70 °F (21 °C), 7 days at 50 °F (10 °C), 14 days at 72 °F (22 °C) and 2 days at 130 °F (54 °C). The method for converting various times and temperatures to equivalent degree-days is explained in reference 134.

Durability Tests. Three procedures have been extensively utilized for evaluating the freeze-thaw durability of LFA and LCFA mixtures. The freeze-thaw brushing procedure included in ASTM D560 is basically modeled after the soil-cement procedure (AASHTO T136-70). Application to practical situations is problematic because the temperature conditions utilized in the ASTM D560 procedure do not simulate field conditions.(146) The "weight loss" factor determined in the AASHTO procedure has no physical significance in terms of basic engineering properties (strength, stiffness, etc.)

Automatic freeze-thaw testing equipment which accurately simulates field conditions has been developed which more closely approximates the expected field conditions.(129) Compressive strength after freeze-thaw cycling (5 or 10 cycles) is used to characterize LFA and LCFA mixture durability. Details of the test procedure are presented in reference 131.

The vacuum saturation test procedure now in ASTM C593 is a rapid technique (approximately one hour). The justification for using the vacuum saturation is the excellent correlation between the compressive strengths of vacuum saturation specimens and freeze-thaw specimens.(130,175) The procedure has been incorporated into a recommended practice for base stabilization prepared th the American Coal Ash Association (ACAA).

7. SUMMARY

Lime-fly ash stabilization is useful in non-plastic soils where the lime will react with the fly ash to provide the strength increase for stabilization. This combination can be used with sands, silts, and aggregates, and provides a means of gradation improvement by filling the voids in the compacted material. Suitable handling and testing must be conducted with the flyash due to its inherent variability and consistency. The handling and curing of this mixture closely follows those for lime or cement stabilization. Adequate compaction is necessary, and maintenance of adequate water content for curing are critical to ensure complete development of strengths as determined in the laboratory, and low temperatures should be avoided. The mix design procedures with lime-fly ash is more involved because the proportion of lime to fly ash must be determined, and then the percentage of the lime-fly ash combination in the aggregate must be determined.

CHAPTER 8 COMBINATION AND OTHER STABILIZERS

1. INTRODUCTION

The use of combination stabilizers has not yet received widespread application in the United States because most agencies prefer to utilize one stabilizer and avoid the handling and construction requirements of a multi-component stabilization system. However, the advantage in utilizing combination stabilizers is that one of the stabilizers in the combination compensates for the lack of effectiveness of the other in treating a particular aspect or characteristic of a given soil. For instance, in clay areas that are devoid of base material, lime has been used jointly with other stabilizers, notably portland cement or asphalt, to provide base courses for secondary roads and residential streets. Since portland cement cannot be easily mixed with plastic clays, the lime is first incorporated into the soil to make it friable, thereby permitting the cement or asphalt to be adequately mixed. While such stabilization practice might be more costly than the conventional single stabilizer methods, it may still prove to be economical in areas where base aggregate costs are high.

While there are many types of combination stabilizers, there are four combination stabilizers which are predominant and thus are given the most consideration. These are:

- Lime-cement
- Lime-asphalt
- Lime-emulsified asphalt
- Cement-emulsified asphalt.

Some of the remaining combinations which are used throughout the world include:

- Rice husk ash (RHA)-lime-cement(176)
- RHA-lime(177)
- Cinder ash-lime(177)
- Gypsum-granulated blast furnace slag-cement-lime(178),
- LD converter slag-lime(179)

These last five combinations are currently being used outside of the United States in places such as Nigeria, India, and Japan where different materials are available for use in stabilization projects.

Other forms of stabilization using proprietary chemicals have been performed on a local basis, with the most common stabilizer being salt, principally calcium chloride.(186,187) This chemical has been used extensively on unsurfaced gravel roads for dust control, and to maintain cohesion through water control.

2. COMBINATION STABILIZER REACTIONS

Lime-Cement Combinations

Combinations of lime and cement are often acceptable as a construction expedient. Lime added to the soil increases the workability and mixing characteristics of the soil as well as reducing its plasticity. Cement can then be mixed with the lime-modified soil to provide rapid strength gain.

Details of lime and its reactions are covered in Chapter 4. In general, lime reacts readily with most plastic soils containing clay, either the fine-grained clays or clay-gravel types. Such soils range in Plasticity Index (PI) from 10 to over 50 percent. Lime may react with some silts but should not normally be expected to react, and lime will not react with sandy soils.

Details of cement and cement reactions are discussed in Chapter 5. While cement is difficult to use alone for heavy clays or highly plastic soils, lime can be used to initiate cation exchange and flocculation-agglomeration reactions which produce immediate changes by reducing the plasticity and improving the workability of these soils. Addition of cement ensures rapid strength development in the mixture. This combination is especially advantageous when rapid strength gain is required under cooler weather conditions.

Lime-Asphalt Combinations

All asphalt products that are currently being produced may be mixed with a variety of sand, soil, or aggregate and soil mixture. The more viscous asphalt materials may require mixing in a plant, while more fluid materials may be mixed in-place with soil-aggregate materials. Although stabilization has been quite effective with many soils, a major problem is a decreased resistance to moisture with some mixtures and this influence is more marked with increased temperature.(180)

Combinations of lime and asphalt have been effective in relieving moisture problems. The addition of lime helps prevent stripping at the asphalt-aggregate interface and increases the stability of the mixture. This reduction of stripping potential maintains the structural value of the mixture, which maintains the integrity of the pavement structure.

Lime slurry pretreatment of the soil or aggregate at one percent or more by dry weight of lime has been effective in raising the modulus value in some cases, and in imparting almost complete water resistance. The mechanism has lime in itself acting as a binder by separately forming a crystalline structure of lime-mortar, cementing the aggregate particles together. In combination with asphalt, the lime action appears to be synergistic with the binding action of the asphalt. The gains in strength and water resistance of the lime-asphalt stabilized material can be far greater than simply the sum of the two binding actions of lime and asphalt taken separately.(180) A further observation is that lime improves the workability of some soil-aggregate materials (and hence the other properties noted above) through the cation exchanges discussed in Chapter 4.

Lime- or Cement-Emulsified Asphalt Combinations

Research in emulsified asphalt mixtures has indicated that curing is the key factor to ensure adequate performance of emulsion aggregate mixtures. The curing of the emulsion-treated material requires loss of water from the mixture. When an emulsion treated aggregate is placed, initial strength is gained when the emulsion breaks. Curing is the continued process of removing the emulsion water through evaporation. Thus, curing can only continue until such time as a surface is placed. If curing is not allowed to continue to completion, final strength may never be achieved. Elevated mixture temperatures and introduction of various additives into the emulsion represent the results of research efforts to increase rapid strength gains. Rapid early strength gain lessens the negative impact of premature sealing of the mixture.

In recent investigations, hydrated lime or portland cement has been used to promote early strength gain of the emulsified asphalt-treated materials. Terrel and Wang have shown that the rate of development of strength in emulsified asphalt mixtures is greatly accelerated by the addition of cement.(181) Figure 63, from the Terrel and Wang study, shows that when an emulsified asphalt mixture is uncured, it behaves essentially like an untreated granular material (i.e., M_R is stress dependent). After varying amounts of curing, the material becomes less stress dependent and more like asphalt concrete. Figure 64 illustrates how small amounts of portland cement can enhance the early modulus gain for emulsified asphalt mixtures. Emulsion mixtures that might not cure to usable strength in a reasonable length of time (say, because of cool, damp weather) can be improved through the use of cement or lime. Schmidt and Graf also demonstrated high moisture resistance of emulsified asphalt mixtures pretreated with lime or cement slurries.(180) Addition of lime or cement to emulsified asphalt mixtures would probably result in higher modulus values and provide better resistance to water at all stages of curing.

3. SELECTION OF STABILIZER CONTENT

The selection of the proper combination stabilizer to be used will depend on the soil type and on various tests to identify the materials. The procedure is much the same as for other stabilizers and reference should be made to earlier chapters for details.

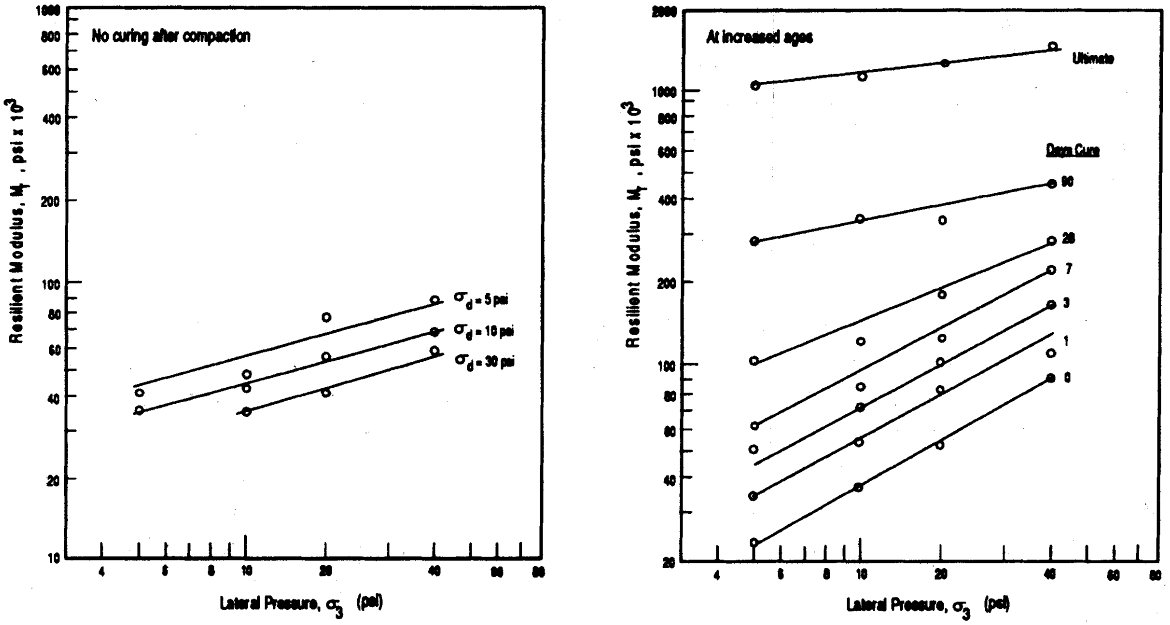


Figure 63. Development of resilient modulus for SM-K emulsion mixture, 68 °F.(182)

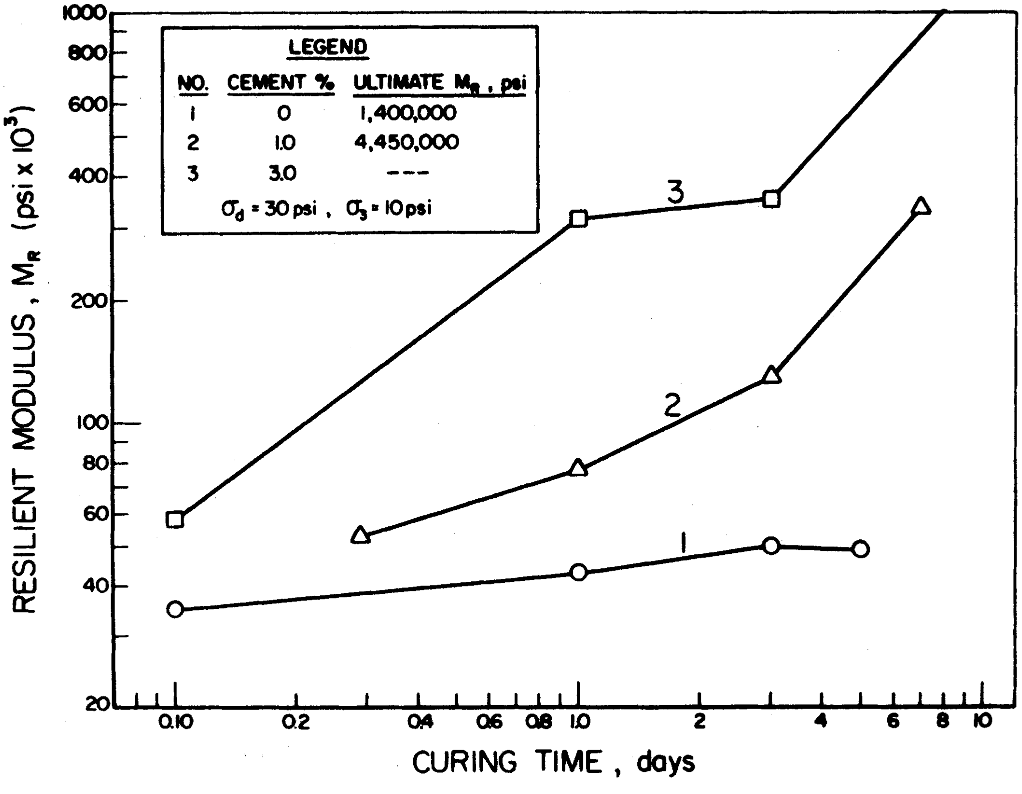


Figure 64. Resilient modulus v. curing time.(182)

Stabilizer selection procedures can then be based on the percent passing the No. 200 sieve and the PI. With the results of the tests, a combination stabilizer can be selected through the process suggested in figure 65. In general, combination stabilizers are best utilized for soils that have more than 25 percent passing the No. 200 sieve and a PI greater than 10.

The various amounts of each individual stabilizer can then be determined by the methods outlined in previous chapters of this manual. The general purpose of combination stabilizers is to first pretreat the soil to alter its properties prior to applying the dominant stabilizer. Normally the quantity of the first stabilizer applied will be less than the second. Approximate quantities of combinations are discussed below.

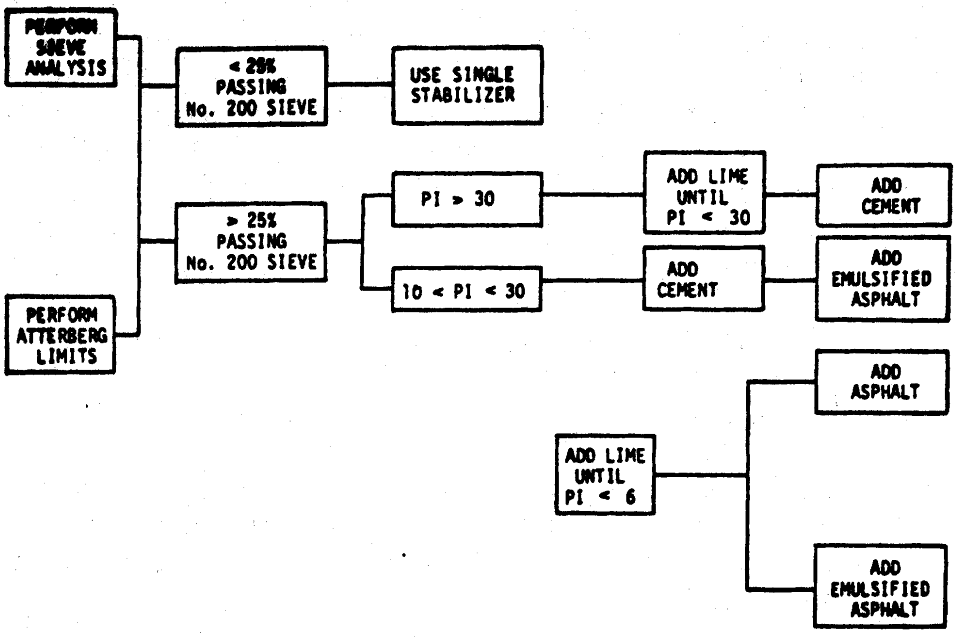


Figure 65. Selection of combination stabilizers.

Approximate Quantities

Lime-Cement. Since cement cannot be easily mixed with plastic clays, 1 to 3 percent of lime (by weight) can be first incorporated into the soil before about 3 to 10 percent cement is added. The amount of lime and cement added depends on the type of soil. Because of the different reactions, more hydrated lime is required than quicklime in the lime-cement mixture to achieve the same end result.

Lime-Asphalt. Pretreatment of aggregates with at least 1 percent of lime in a slurry form for emulsified asphalt, or pulverized form with cutback or asphalt cement, can minimize the strength loss from water in asphalt-treated mixtures. In general, 1 to 3 percent of lime (by weight) can be used with 4 to 7 percent asphalt (by weight of asphalt) in the mix for soil stabilization purposes.

Lime-Emulsified Asphalt. The addition of a small amount of lime to emulsified asphalt mixes at the time the asphalt emulsion is added to the aggregate has a profound effect on the rate of strength development as well as the ultimate strength level attained. About 1 to 3 percent lime (by weight) can be combined with 4 to 8 percent emulsified asphalt (by weight of residual asphalt) in the mix.

Cement-Emulsified Asphalt. The addition of small amounts of cement on the order of approximately 1.5 percent (by weight) to emulsion-treated mixes assists in the development of early stiffness as compared to the same mix without cement. Care must be taken not to incorporate too much cement; a ratio of cement emulsion on the order of 1 to 5 (based on residual asphalt) appears appropriate to ensure adequate early stiffness without excessive embrittlement, or early break in the emulsion.

Detailed Testing

The quantity of stabilizer to be utilized should be determined by means of suitable laboratory tests. These attempt to simulate field conditions of weathering and other durability processes and include a strength determination.

Cyclic freeze-thaw or wet-dry actions are the major durability factors that must be considered for some combination stabilizer mixtures. The extent of freeze-thaw and wet-dry action is dependent on the location of material in the pavement structure, geographical location, and climatic variability.

The laboratory tests necessary for determining strength and/or durability for the combination stabilizers are listed below. Detailed information on the laboratory test procedures can be found in Chapter 3.

Lime-Cement. The strength and the durability of these mixtures when they are subjected to cyclic environmental conditions are the characteristics most commonly tested and evaluated. Sample preparation is most commonly done using a standard AASHTO compaction cylinder with varying moisture contents and admixture levels. The curing of the samples is generally achieved using guidelines found in ASTM C 593. The tests which are most commonly performed are listed below:

- Unconfined compression test (ASTM C 39)
- Flexural tensile test (ASTM C 293 or C 78)
- Split tensile test (ASTM C 496)
- Vacuum saturation test(184) (ASTM C 593)
- Iowa freeze-thaw test(182)
- Automatic freeze-thaw test(183)

Further details on these and other tests discussed in this section can be found in Chapter 3.

Lime-Asphalt. The techniques for evaluating strength characteristics of lime-asphalt stabilized materials depend upon the soil type. For fine-grained materials these would be absorption tests and Hubbard-Field tests. For coarse-grained (gravel, gravel-sand) materials, tests would include the California Bearing Ratio tests and triaxial tests. Durability tests for lime-asphalt stabilized materials are similar to those listed for lime-cement stabilized materials. A list of tests is given below (further details on these tests can be found in Chapter 3:

- Absorption test (ASTM C 128) (Fine-grained)
- Hubbard-Field test (Fine-grained)
- California Bearing Ratio test (ASTM D 1883) (Coarse-grained)
- Triaxial test (ASTM D 3397) (Coarse-grained)
- Vacuum saturation test(184) (ASTM C 593)
- Iowa freeze-thaw test(182)
- Automatic freeze-thaw test(183)

Lime- or Cement-Emulsified Asphalt. The strength tests for the lime-emulsified asphalt and cement-modified asphalt can be determined either by the Hveem or Marshall test, or resilient modulus tests. Details of these tests can be found in Chapter 3. The durability tests listed for lime-cement and lime-asphalt can also be used for lime- or cement-emulsified asphalt stabilized materials. A list of applicable tests which can be run is given below:

- Hveem stabilometer test (ASTM D 1560)
- Resilient modulus test (ASTM D 4123)
- Vacuum saturation test(184) (ASTM C 593)

- Iowa freeze-thaw test(7)
- Automatic freeze-thaw test(8)

4. LIMITATIONS AND PRECAUTIONS

Climatic and/or Construction Limitations

Lime-stabilized soils are relatively slow curing and require some warm weather to harden properly. Cement hydration also ceases when temperatures are near or below 40 °F.(183) Lime-cement stabilization therefore should not be carried out in cold weather. As a general rule, lime-cement stabilization should not be attempted when the soil temperature is below 40 °F (5 °C) and there is little prospect of the weather improving in the next day or two. During cold weather conditions, lime-cement stabilized soils should be protected by a suitable covering of hay, straw or other protective material to prevent freezing for a period of 7 days after placement and until they have hardened.

If heavy vehicles are allowed on the lime-cement stabilized soil prior to a 7 to 10-day curing period, damage to the structural layer may occur. However, light vehicles may be allowed on the stabilized material with the expectation that any damage that is incurred will be subjected to autogenous healing as the curing process continues(185). This autogenous healing is dependent on four factors:

- Initial curing period before inducement of flaws
- Length of curing period after inducement of flaws
- Curing conditions
- Levels of stabilizing agents

The amount of autogenous healing seems to be most dependent on the second factor, the length of the curing period after inducement of the flaws. This is also related to the initial curing period in that the greater the amount of curing which takes place before the flaws, the less likely it is that there will be sufficient curing left to heal the flaws.

All lime-cement stabilized bases require a wearing surface of at least a bituminous seal coat. An unprotected lime-cement stabilized base has poor resistance to the abrasive action of continued traffic.

Hot, dry weather is preferred for all types of lime-asphalt stabilization. If thin lifts of lime-asphalt stabilized material are being placed, the air temperature should be 40 °F (5 °C) and rising, and the compaction equipment should be used immediately after laydown operations. Adequate compaction can be obtained at freezing temperatures if thick lifts are used with hot-mixed, hot-laid asphalt stabilization operations.

From a strength standpoint, too much lime or cement in a stabilized mixture is not a problem. However, excessive asphalt in the mix will cause reduction in soil strength. Excess asphalt will be evident on the top, sides, and bottom of the compacted laboratory samples.

The use of lime-emulsified asphalt or cement-emulsified asphalt should not be attempted during periods of rain or if the probability of rain exists. Unbroken emulsions subjected to rain can be further diluted and completely lost by runoff. Also, a longer breaking and curing time should be anticipated during periods of high humidity. Temperatures preferably should be above 60 °F (16 °C). During hot, dry weather conditions, it is advantageous to moisten the soil prior to the application of emulsion.

Heavy vehicles should not be allowed on lime-emulsified asphalt or cement-emulsified asphalt stabilized soils prior to a 7 to 10 day curing period in order to avoid damage to the structural layer.

Safety Precautions

In using lime, contact with quicklime or prolonged contact of hydrated lime with moist skin can cause burns or skin irritation. Adequate protective clothing for workers is necessary for lime-emulsified and cement-emulsified asphalt stabilization operations. Care must also be taken when heated asphalt cement is used as a stabilizing agent.

5. OTHER COMBINATIONS

In addition to the predominant combination stabilizers discussed in the preceding pages, several alternative combinations have been identified as effective stabilization materials around the world. A brief discussion is given here on some of these combinations.

Rice Husk Ash (RHA)-Lime-Cement

Stabilization, produced from the addition of rice husk ash (RHA), has been conducted in many parts of the world, most notably India and Nigeria. Research carried out in Nigeria reported that RHA can be added to lateritic soils (described as "ferruginous, vesicular, unstratified and porous material with yellow ochres caused by high iron content) with good success.(176) RHA seems to cause flocculation of the soil particles in much the same way that lime does, which subsequently reduces the

PI of fine-grained cohesive soils. In individual tests with lime and RHA, it was determined that a lime content of approximately 5 percent produced a maximum CBR, whereas it took approximately 18 percent RHA to produce the maximum CBR results. In areas where RHA and lateritic soils are abundant (tropical and subtropical countries), the use of RHA to reduce the quantities of lime and cement necessary for stabilization could have a significant impact on making the stabilization process more efficient and more cost effective.

Rice Husk Ash-Cinder Ash-Lime

Both RHA and cinder ash have been added to lime-stabilized soils in an effort to improve the strength characteristics of the soils and to reduce the cost of the stabilization process. Research conducted in India reported that addition of 8 percent RHA or cinder ash to a 5 percent lime-soil material increased the CBR values 45 to 50 percent over the addition of just 5 percent lime.(177) Addition of these "waste" materials (RHA and cinder ash) is once again recommended as a means of improving the strength of the stabilized soils and making the stabilization process more cost effective.

Gypsum-Granulated Blast furnace Slag-Cement-Lime

Research in Japan was carried out using two types of soil (cohesive and sandy) and various combinations of cement, hydrated lime, gypsum and granulated blast furnace slag as stabilizers.(178) Using different combinations and different amounts of each stabilizing agent, Hasaba et al were able to identify the formation of ettringite, a cement, in the reaction products as one of the contributing factors in high compressive strengths. More ettringite was produced when stabilizing the cohesive soil than the sandy soil, thus resulting in higher strengths in the stabilized cohesive soil. The stabilized sandy soil also experienced significant expansion which contributed to lower compressive strengths. As with the research conducted in Nigeria and India, this research effort is exploring possible methods for disposing of industrial waste products (desulfurization by-product gypsum and granulated blast furnace slag) and improving the cost efficiency of the soil stabilization process.

LD Converter Slag-Lime

LD converter slag is another "waste" product generated in the production of steel. Researchers in Japan feel that there is a great deal of potential in the use of this slag in stabilizing both sandy and cohesive soils.(179) There was some difference in the degree that the slag improved the strength characteristics, namely that the sandy soils showed higher strengths with a lime/slag ratio of 1 whereas the cohesive soils had maximum strengths at a lime slag ratio of 3. This is believed to be a result of more active reactions between clays and lime in the cohesive soils than in the sandy soil.

6. SALTS (CALCIUM CHLORIDE)

Salt stabilization has focused primarily on the use of Calcium Chloride as an additive to improve the performance of granular materials. The stabilizing action of this additive is in its ability to attract and hold moisture, and to provide a reduction in void space in the compacted material.

In solid form it is deliquescent and absorbs moisture directly from the air. When it is in liquid state, it continues to absorb water from its surroundings, resisting evaporation. This continued attraction for water provides the principal benefit used in stabilizing unsurfaced aggregate roads. A major problem in these roads is the maintenance of integrity from dusting. As the compaction moisture evaporates, the soil binder dries out and is removed. Careful gradation control is needed to maintain a natural moisture content in the aggregate. The addition of the calcium chloride to the aggregate provides a material that maintains a moisture level in the aggregate by attracting the moisture from the atmosphere and surrounding soil.

This ability to hold moisture can improve the workability and maintainability of the aggregate surface by providing a more stable moisture condition during maintenance operations. Increased strengths may also be realized by the action of the salt/water combination in the pores of the aggregate.

Application Rates

Quantities may vary slightly with application purpose and gradation, but typical application rates for the flaked calcium chloride are from 0.4 to 0.5 pounds per square yard per inch of material for full stabilization effects. For dust control, the amount is typically reduced, in the range of 0.25 pounds per square yard per inch of material.(188)

Suitable materials

The aggregate gradations suitable for salt stabilization are those typically recommended for aggregates to be used as an unsealed surface. These gradations typically have an increased amount of fines, the amount passing the #200 sieve. A minimum value here would be 5 percent, with typical values in the range of 10 percent for a base course, and 20 percent for a wearing surface.

7. SUMMARY

Combination stabilizers are not as common as individual additives and they require more engineering and testing to ensure quality and development of the required engineering properties. Lime and\or cement in conjunction with asphalt or emulsion has been most widely used to improve initial strengths in the emulsion and as an anti-stripping additive in the asphalt cement. the more exotic combinations are used primarily overseas where these byproducts are readily available.

Salt stabilization is an important stabilizer that has widespread use as a dust palliative, and a moisture control additive in unsurfaced granular roads.

CHAPTER 9 COST DATA AND ECONOMIC ANALYSIS

1. INTRODUCTION

Decreasing highway revenues and increasing construction costs have caused many highway agencies to place greater emphasis on improved pavement management in the past few years. Highway administrators are attempting to get the best pavement performance for the highway dollars using thepavement management approach. With this emphasis, the the selection of specific rehabilitation alternatives or material preparation processes such as soil stabilization, must be evaluated in the context of the cost effectiveness of one alternative or process over another. Such a cost effectiveness study should consider the life cycle costs of the pavement.

This Chapter contains information describing costs associated with soil stabilization. To assist the engineer in performing economic analyses, a simplified economic analysis method is described which illustrates the steps an engineer should follow to perform a suitable economic analysis of roadway network to allocate his monetary resources in an optimum manner.

2. COST DATA

The cost information presented in table 39 was obtained from one midwestern state and is intended only to provide an indication of different stabilization costs. If costs for these activities are available from local agencies' historical records, they should be substituted appropriately.

Table 39. Costs Associated with Soil Stabilization

STABILIZATION ACTIVITY COSTS	$ per SY - IN
Lime stabilization subgrade	0.25
Cement stabilized subgrade	0.60
Cement stabilization base	1.60
Asphalt stabilization base	1.60
Lime-fly ash stabilization	1.40
Cold in-place recycling	0.45

3. ECONOMIC ANALYSIS

A life-cycle cost analysis should be the dominant factor in selecting different pavements designs, rehabilitation strategies, or material processes. Recent publications on life-cycle cost analysis have added greatly to the available information on this important topic.(189,190,191,192) The NCHRP synthesis on "Life-Cycle Analysis of Pavement" by D. E. Peterson is the most comprehensive document available and is recommended for future information.(193)

The following cost elements must be considered for both new construction and rehabilitation:

- Initial construction (of new construction or rehabilitation).
- Future maintenance and rehabilitation.
- Future salvage value.

Life-cycle costs can be expressed in terms of their "present worth (PW)" or their "equivalent uniform annual cost (EUAC)".[194,195] The present worth method converts all future costs to their equivalent present costs using a selected discount rate. The converted future costs can be combined with the initial construction cost to give a total present worth cost over the analysis period. The equivalent uniform annual cost method converts this present worth to an equivalent annual cost over the analysis period.

Analysis Period

The analysis period refers to the time over which the economic analysis is to be conducted. The analysis period for new pavement design is typically twenty to forty years, as specified in the 1986 AASHTO Guide. For rehabilitation work, the analysis period will usually be shorter, such as ten to twenty or more years. An analysis period of at least ten years is recommended for rehabilitation so that future costs are reasonably considered.

Performance and Design Period

The performance period is the time between the beginning of the life of an alternative and the time when major rehabilitation will be required. The performance period may or may not equal the design period. The design period is how long the pavement is supposed to last. For example, a pavement may be designed for 20 years, but due to factors not adequately considered in the design such as unexpected high rate of increase in truck traffic, it may actually last only 14 years. The design period is used in the AASHTO design procedure, but the performance period should be used in the economic comparisons of different alternatives. It is essential that the engineer review all information available on how the various pavement design or rehabilitation alternatives being considered have performed under similar conditions of climate and traffic so that realistic performance periods are developed for each alternative for use in the cost analysis.

Discount Rate

The discount rate (interest rate) or (rate of return) is used to reduce future expected costs for projects to present-day terms for economic comparison purposes. The value selected for discount rate deserves careful attention by the engineer. The rate selected normally varies between 0 to 10 percent while the actual value selected should be based upon consideration of the following:

- Interest rate currently charged to borrow capital,
- Rate of return expected of private investments,
- Rate of return expected on public works investments, and
- Risks and uncertainties associated with investments.

It should be noted that construction and rehabilitation alternatives with large initial costs and low maintenance or user costs are favored by low interest rates. Conversely, high interest rates favor strategies that combine low initial costs with high maintenance and user costs. No general recommendations can be given for this value at present.

A discount rate of 8 percent has been used in this discussion together with a 20-year analysis period for the example presented below, and does not constitute any recommendation. Present worth factors and capital recovery factors for discount rates of 6, 7, and 8 percent are shown in table 40. Values for other discount rates can be found in textbooks on engineering economy. Both present worth and the uniform annual cost methods are illustrated below. Costs are estimated in terms of dollars per square yard; however, costs in terms of dollars per lane-mile are also a convenient unit.

Life-cycle costs procedures may not be precise since reliable data for maintenance, subsequent stages of construction, salvage value, and pavement life are not always available and it is usually necessary to apply engineering judgement to make reasonable estimates. Despite these difficulties, life-cycle cost analysis is believed to provide the best potential to obtain the greatest service from a pavement construction or rehabilitation project at the lowest possible costs.

Example Problem[1]

A nine-mile pavement section is to be constructed in central Texas. Two pavement sections have been suggested for use on this roadway. Plan 1 consists of construction of a pavement containing 6 inches of lime stabilized subgrade, 8 inches of crushed stone base, and 2 inches of asphalt concrete surfacing. Overlays are scheduled on a 7-year cycle (table 41). Plan 2 consists of constructing a pavement containing 6 inches of lime stabilized subgrade, 8 inches of asphalt treated base, and

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[1] Only English units will be used for the sake of clarity.

2 inches of asphalt concrete. Overlays will not be required during the 20-year life cycle (table 42).

The following cost estimates were utilized for the initial construction:

- Lime stabilization - \$0.25 per yd^2-in.
- Asphalt stabilization - \$0.80 per yd^2-in.
- Crushed stone base - \$0.50 per yd^2-in.
- Asphalt concrete - \$1.00 per yd^2-in.

Initial construction costs are \$7.50 per yd^2 for Plan 1, and \$9.90 per yd^2 for Plan 2. Routine maintenance costs were estimated in order to complete the example.

From both a present worth and uniform annual cost basis with an 8 percent rate of return, Plan 1 is favored (\$9.72 versus \$10.16 and \$.99 versus \$1.04). It should be realized that several assumptions including pavement life, maintenance costs, rehabilitation costs, and discount rates were made. For example, if a lower discount rate were utilized, Plan 2 would be favored over Plan 1. Present worth for 0 percent discount rates are shown on tables 41 and 42. Plan 1 is \$12.33 and Plan 2 is \$10.67 for 0 percent discount rates. Selection of Plan 1 over Plan 2 should be made based on more detailed estimates of life and cost figures. The sensitivity of the analyses to these estimates should be investigated.

Table 43 is a sample calculation sheet for a rehabilitation alternative. The present worth and capital recovery factor values can be substituted as required.

Table 40. Present Worth and Capital Recovery Factors

	PRESENT WORTH FACTOR			CAPITAL RECOVERY FACTOR		
	INTEREST RATE(%)			INTEREST RATE(%)		
YEAR	6	7	8	6	7	8
1	0.9434	0.9346	0.9259	1.06000	1.07000	1.08000
2	0.8900	0.8734	0.8573	0.54544	0.55309	0.56077
3	0.8396	0.8163	0.7938	0.37411	0.38105	0.38803
4	0.7921	0.7629	0.7350	0.28859	0.29523	0.30192
5	0.7473	0.7130	0.6806	0.23740	0.24389	0.25046
6	0.7050	0.6663	0.6302	0.20336	0.20980	0.21632
7	0.6651	0.6227	0.5835	0.17914	0.18555	0.19207
8	0.6274	0.5820	0.5403	0.16104	0.16747	0.17401
9	0.5919	0.5439	0.5002	0.14702	0.15349	0.16008
10	0.5584	0.5083	0.4637	0.13587	0.14238	0.14903
11	0.5268	0.4751	0.4289	0.12679	0.13336	0.14008
12	0.4970	0.4440	0.3971	0.11928	0.12590	0.13270
13	0.4688	0.4150	0.3677	0.11296	0.11965	0.12652
14	0.4423	0.3878	0.3405	0.10758	0.11434	0.12130
15	0.4173	0.3624	0.3152	0.10296	0.10979	0.11683
16	0.3936	0.3387	0.2919	0.09895	0.10586	0.11298
17	0.3714	0.3166	0.2703	0.09544	0.10243	0.10963
18	0.3505	0.2959	0.2502	0.09236	0.09941	0.10670
19	0.3305	0.2765	0.2317	0.08962	0.09675	0.10413
20	0.3118	0.2584	0.2145	0.08718	0.09439	0.10185

Table 41. Economic Analysis of Plan 1

YEAR	COST,DOLLARS PER SQUARE YARD	PRESENT WORTH FACTOR, 8%	PRESENT WORTH DOLLARS
INITIAL COST	**7.50 initial construction**	**1.0000**	**7.50**
1		0.9259	
2		0.8573	
3	0.07 routine maintenance	0.7938	0.056
4	0.10 routine maintenance	0.7350	0.074
5	0.12 routine maintenance	0.6806	0.082
6	0.12 routine maintenance	0.6302	0.076
7	1.8 2-inch overlay	0.5835	1.050
8		0.5403	
9		0.5002	
10	0.07 routine maintenance	0.4632	0.032
11	0.10 routine maintenance	0.4289	0.043
12	0.12 routine maintenance	0.3971	0.048
13	0.12 routine maintenance	0.3677	0.044
14	1.8 2-inch overlay	0.3405	0.613
15		0.3152	
16		0.2919	
17	0.07 routine maintenance	0.2703	0.019
18	0.10 routine maintenance	0.2502	0.025
19	0.12 routine maintenance	0.2317	0.028
20	0.12 routine maintenance	0.2145	0.026

TOTAL = 12.33 TOTAL = 9.716

UNIFORM ANNUAL COST = PRESENT WORTH x CAPITAL RECOVERY FACTOR

= 9.716 x 0.10185

= 0.990

Table 42. Economic Analysis of Plan 2

YEAR	COST, DOLLARS PER SQUARE YARD	PRESENT WORTH FACTOR, 8%	PRESENT WORTH DOLLARS
INITIAL COST	9.90 initial construction	1.0000	9.90
1		0.9259	
2		0.8573	
3		0.7938	
4		0.7350	
5		0.6806	
6		0.6302	
7		0.5835	
8	0.07 routine maintenance	0.5403	0.038
9		0.5002	
10	0.10 routine maintenance	0.4632	0.046
11		0.4289	
12	0.12 routine maintenance	0.3971	0.048
13		0.3677	
14	0.12 routine maintenance	0.3405	0.041
15		0.3152	
16	0.12 routine maintenance	0.2919	0.035
17		0.2703	
18	0.12 routine maintenance	0.2502	0.030
19		0.2317	
20	0.12 routine maintenance	0.2145	0.026

TOTAL = 10.67 TOTAL = 10.164

UNIFORM ANNUAL COST = PRESENT WORTH x CAPITAL RECOVERY FACTOR

= 10.160 x 0.10185

= 1.035

Table 43. Calculation Form for Economic Analysis

YEAR	COST, DOLLARS PER SQUARE YARD	PRESENT WORTH FACTOR, _%	PRESENT WORTH DOLLARS
INITIAL COST		1.0000	
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			
11			
12			
13			
14			
15			
16			
17			
18			
19			
20			

TOTAL = _____ TOTAL = _____

UNIFORM ANNUAL COST = PRESENT WORTH x CAPITAL RECOVERY FACTOR

= _____ x _____

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APPENDIX A

QUALITY CONTROL

1. INTRODUCTION

The development of statistical based specifications has increased the importance of quality control and quality assurance in the construction of pavements, and stabilization projects. Quality assurance is the testing process whereby the purchasing agency such as the State Highway Agency (SHA) makes certain that the material it is purchasing for use in its pavements meets the specifications established for that material. Quality control, often termed process control, testing is the sequence of testing performed by the field engineer, or even the contractor himself to indicate that the process is under control and producing the expected materials that will meet the established specifications for that material. Quality control testing indicates that work has been performed in accordance with the plans and specifications, and may or may not be used as the basis for payment for the material, depending on the contract document.

Stabilization procedures have certain requirements that are not normally encountered during the more common types of construction conducted during the construction of a pavement project. Lime, cement, asphalt, and lime-fly ash are discussed here, being the main stabilization options. The discussion is general, and highlights the concerns of each stabilization procedure, and is not intended to be a thorough treatment of the subject for field personnel.

2. SCOPE

Chapter 3 of Volume II presented detailed laboratory procedures to be used to determine optimum stabilizer contents to ensure adequate strength or durability in the finished product. Additionally, individual chapters provide detailed considerations to be observed during the construction of each stabilizer type. Quality control in the field cannot be achieved if the field personnel do not understand the engineering function of each stabilizer type, the impact of construction and materials on the effectiveness of each stabilizer type, and the sensitivity of the final strength or durability of the stabilizes material to variation in material properties. The relevant portions of Volume I and II should be thoroughly read by all personnel involved with the quality control of a stabilization project.

The testing program used in the laboratory establishes the required quantities and construction procedures necessary in the field. In the field many similar tests are performed to ensure that the quality established in the laboratory is being achieved. The field tests normally include:

- Determination of In-place density
- Determination of stabilizer content
- Gradation
- Plasticity
- Moisture content

These specific quantities should be determined from the finished material, but the equipment and processes being used to prepare the material must be continually inspected to ensure that they are performing correctly and are indeed capable of producing the high quality material required.

3. CONSTRUCTION SEQUENCE

Quality control testing is performed at various stages in the construction sequence. This sequence, as detailed elsewhere is very similar for all stabilization procedures, mainly:

- Pulverization and/or scarification of the candidate material
- Application of the correct stabilizer amount
- Uniform mixing of the stabilizer
- Time sequencing of the operation
- Compaction
- Curing

The different stabilizer additives used may require slightly different procedures to test for the adequacy of each step.

Pulverization and/or Scarification

This step is normally of concern with stabilization which is done on fine-grained cohesive materials which may have a high water content. The other forms of stabilization are most commonly performed on aggregates or low plasticity material which can be evaluated with gradation testing for compliance. Pulverization for mixing is normally controlled by the percent passing the #4 sieve, which can be easily evaluated in the field as required.

Stabilizer Amount

Lime, cement,and lime-fly ash contents can be easily determined with a canvas cloth of a known area and weight. Place the cloth on the grade ahead of the additive distributor and weigh the cloth after the additive has been place on the cloth. This amount of additive per square hard can be calculated and compared to the specification amount. Asphalt amount can be established by extraction after the process is completed, but during construction, the meters on the distributer must be monitored to determine the gallons per square yard of coverage.

Mixing Uniformity

The evaluation of mixing uniformity is primarily a visual examination of the mixed material. A uniform color should result throughout the depth of mixing, and across the entire roadway. Lime mixing can be examined using a ph indicator (Phenolphthalein) sprayed on the soil-lime mixture. It will turn a reddish-pink color indicating the presence of free lime, and a pH of 12.5. The uniformity of asphalt mixing is determined by the water content in the material. Often, the mixing water content required to get uniform mixing is higher than the water content required for compaction. These should be carefully monitored.

Time Sequence of Operation

The time for completion of the various steps is important for the different additives. Lime, cement, and lime-fly ash all require the timing not be so lengthy that the water content is evaporated before the mixture is compacted. The water content must remain at optimum at the time of compaction. Asphalt stabilization with cutbacks or emulsions requires a different timing sequence. The mixing must continue until the cutbacks begin to evaporate, or the emulsion begins to break as indicated by the change in color from brown to black. this mixing period is termed aeration, and is essential to the production of a high quality bituminous stabilized material. Aeration should continue until the mixture becomes tacky.

Compaction

Density should be carefully controlled by any of the accepted procedures such as sand cone, rubber balloon, and nuclear gauges. Careful attention should be given to the uniformity of compaction across the entire width of the pavement. Compaction of lime, cement, and lime-fly ash should begin immediately after the mixing. Delays of more than several days will damage the materials and require their removal. Asphalt stabilization cannot be delayed and must begin immediately after the aeration phase. Rolling of the asphalt emulsion material should begin when the emulsion begins to indicate that it is breaking and turning from brown to black in color.

Curing

Curing operations must be conducted to seal in the moisture in lime, cement, and lime-fly ash stabilized materials. Moist curing involves the application of light amounts of water to the surface of the compacted material to prevent evaporation of the water in the mixture. Care must be taken with lime stabilization not to allow the mixture to go from wet to dry during these waterings as this will promote carbonation. Membrane curing, which is more commonly done, involves the application of a membrane such as a bituminous spray application, or a curing compound which prevents the evaporation of the mixture water.

Asphalt stabilization is very dependent on the curing process, and the process is opposite from the other forms of stabilization. The asphalt stabilized mixture must be left open to allow the solvents from the cutback or the water from the emulsion to evaporate. If the material is sealed immediately after compaction, the solvent or water will be trapped in the material and result in a very low strength which will lead to rapid premature failure. The compaction process is important to the curing, as compaction in itself serves to seal the material and reduce the evaporation. Normally a minimum of 2 - 5 days of open surface are recommended. When the layer is a structural layer, and/or the environment is not conducive, the surface may have to be left open for 10 - 14 days. The addition of cement to the emulsion is designed to provide higher early strengths to overcome the problem of early sealing.

4. STATISTICAL QUALITY CONTROL

The tests and procedures discussed in Chapter 3 and here represent the tests necessary to evaluate the composition and quality of the materials and the stabilization process. Given the speed of current construction procedures, the specifications should very clearly indicate what tests are going to be taken, how often they will be taken, and how they will be used to establish quality. It is not sufficient to take tests on a haphazard manner with inconsistent sampling schedules. The use of statistics to evaluate the test results and relate them to the entire production is mandatory to keep testing in line with construction and provide accurate determinations of quality of production.

Many of the tests described in Chapter 3, and here, can be used in a statistical sampling program to evaluate the quality of a mixture. The specific tests and their frequency must be established to maintain the low-cost nature of soil stabilization, but provide the required level of assurance that the desired product is being produced. The SHA must accept the fact that some bad material will be produced and used in their pavement. Likewise, the contractor must accept the fact that some good material may be rejected for use in the pavement. The amount of either should not be excessive or neither party will be happy with the product.

There are a number of statistical based quality control plans currently in use. Many sates are using such plans for their bituminous production, and while stabilization is not as rigorous as asphalt concrete production, the essential elements of the process are the same. An adequate statistical quality control procedure should recognize the following:

- A specific number of tests must be specified.
- The procedure for taking and reporting the test results must be stated.
- The quantity of material being produced from which the tests will be taken must be specified.
- The position of each test must be randomly selected within the quantity of material being evaluated.
- The acceptability of the tests must be clearly spelled out, giving the criteria used to judge acceptability of the results.

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