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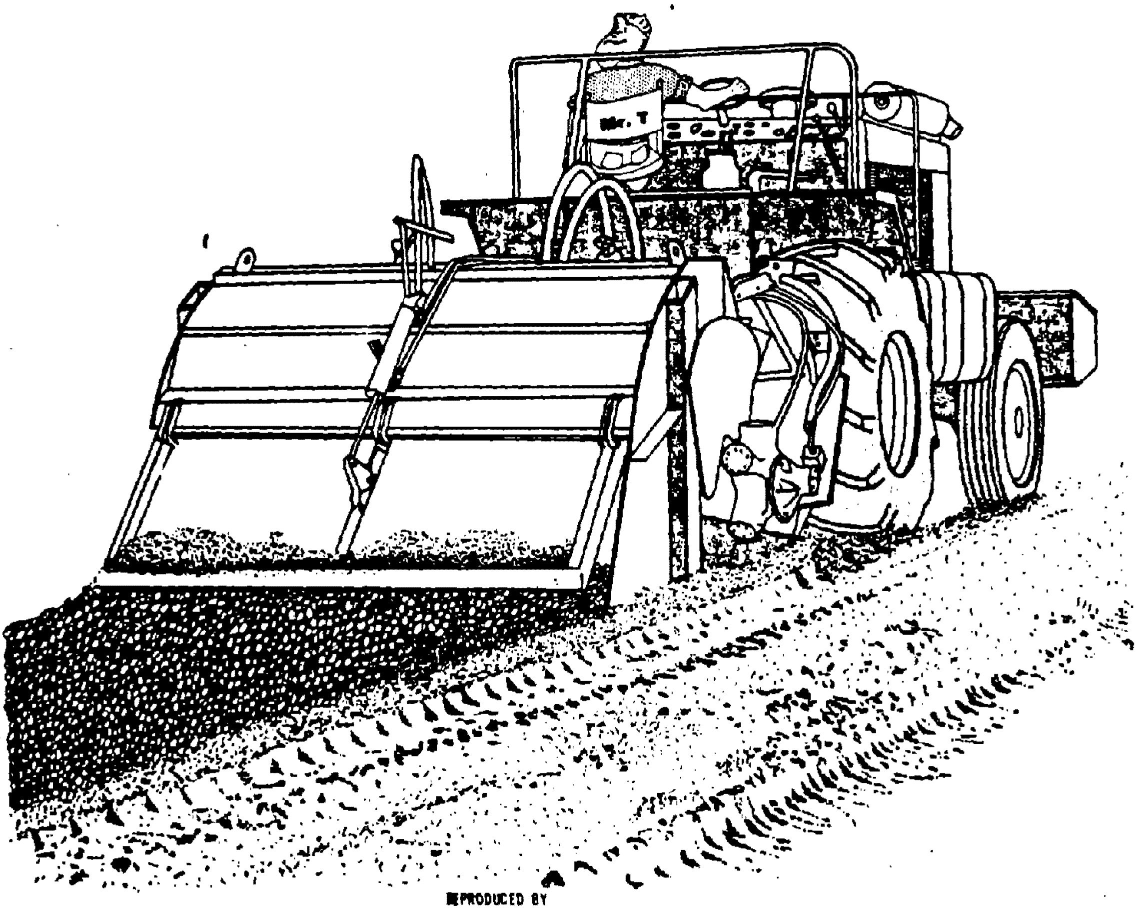
SOIL STABILIZATION IN PAVEMENT STRUCTURES
A USER'S MANUAL

VOLUME 2

MIXTURE DESIGN CONSIDERATIONS



U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
Office of Development
Implementation Division
OCTOBER 1979

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ERRATA

Soil Stabilization in Pavement Structures
A User's Manual

Volume 2

page 11, line 19:	"leterature" should read literature
page 15, line 6:	"United" should read Unified
page 29, line 17:	"Illinios" should read Illinois
page 29, line 23:	"exclusively" should read exclusive
page 29, line 24:	"$Mb(OH)_2$" should read $Mg(OH)_2$
page 30, line 11:	"Termperatures" should read Temperatures
page 30, line 16:	"inimately" should intimately
page 32, line 13:	"less lay" should less clay
page 32, line 14:	"stabiliation" should read stabilization
page 32, line 26:	"approximately" should read appropriately
page 33, line 2:	"expediates" should read expedites
page 37, line 3:	"(32)" should read (52)
page 37, line 40:	"66 percen tof" should read 66 percent of
page 38, lines 25, 27:	"psl" should read psi
page 41, line 7:	"dominate" should read dominant
page 41, line 35:	"(65, 65)" should read (65)
page 43, line 1:	"3010" should read 3-10
page 44, line 34:	"that in any..." should read <u>than</u> in any
page 44, line 40:	"invironmental" should read environmental
page 45, line 11:	"Minimun" should read Minimum
page 45, line 3 in Table 13:	"kPa" should read psi
page 53, line 13:	"infludence" should read influence
page 68, line 11:	"build" should read built
page 69, line 8:	should read ... Chemical <u>and</u> physical property...
page 69, line 37:	"necleic" should read nucleic
page 71, line 34:	should read $(UC)_d = (UC)_{do} + K \log (d/do)$
page 71, line 40:	should read, and 1.4 <u>times</u> the 7-day strength
page 72, 73, 76, 78, Figures 24, 25, 26 and 30:	The unit conversions should read 1 psi = 6.89×10^3 Pa = 6.89×10^{-3} MN/m^2
page 73, line 3:	"strengh" should read strength
page 74, line 8:	"uncer" should read under
page 76, Figure 29:	The unit conversion should read 1 psi = 6.89×10^3Pa = 6.89×10^{-6}GN/m^2
page 77, line 1:	"R_C can be extressed by:" should read can be expressed by:
page 77, line 18:	$m = 0.04(10)^{-.186C}$
page 78, line 9:	"percen" should read percent
page 80, line 11:	"consequent" should read subsequent
page 82, Table 20:	unit conversion at bottom of table should read 1 psi = 6.89×10^3 Pa = 6.89×10^{-3} MN/m^2
page 87, line 21:	"setting time of" should read setting time
page 88, line 19(text)	"CMS-sh" should read CMS-2h
page 140, line 19:	"(3.81/minute)." should read (3.81 mm/minute).

(See back for Vol. 1)

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A

SOIL STABILIZATION IN PAVEMENT STRUCTURES

A USER'S MANUAL

VOLUME 2

MIXTURE DESIGN CONSIDERATIONS

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Contract No. DOT-FH-11-9406

prepared for the

FEDERAL HIGHWAY ADMINISTRATION
DEPARTMENT OF TRANSPORTATION
WASHINGTON, D. C. 20590

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October 1979

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ACKNOWLEDGMENTS

The authors wish to acknowledge the cooperation provided by the following groups who supplied information and/or photographs used in the manual.

National Lime Association
National Ash Association
Portland Cement Association
The Asphalt Institute
Chevron U.S.A., Inc.
American Association of State
  Highways and Transportation
  Officials

The authors further wish to acknowledge the assistance provided by: Dr. Dallas Little for developing design examples and Prof. Bob Gallaway for technical editing and suggestions. Sharon Tighe provided editorial assistance on the original single volume. Barbara Williams and Yunja Yu typed the final manuscript and Kay Moore provided the drafting. All their assistance is appreciated.

FOREWORD

The primary purpose of this manual is to provide background information for those engineers responsible for utilizing soil stabilization as an integral part of a pavement structure. Information is included which will allow the pavement design engineer to determine the thickness of stabilized layer(s) for a pavement in a specific climate and subjected to definable highway traffic. The construction engineer will find information on quality control, specifications and construction sequences. The materials engineer has been provided with information that will allow the determination of the type and amount of stabilizers that are suitable for a particular soil.

The manual has not been written to endorse one type of a chemical stabilizer over another. Nor is it intended to provide the specific features of one manufacturer's products. Rather, it explains the general characteristics of chemical soil stabilization and offers a method for evaluating the benefits of chemical stabilization versus the conventional mechanical stabilization operations.

A thorough study of the manual should enable the engineer to recommend where, when and how soil stabilization should be used. It may also act as an aid in solving problems that may arise on soil stabilization projects.

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I. INTRODUCTION

PURPOSE AND SCOPE

Purpose

This two volume user's manual was developed to provide guidance for pavement design, construction and materials engineers responsible for soil stabilization operations associated with transportation systems. Volume I of the manual (Pavement design and Construction Considerations) has been prepared for pavement design and construction engineers. Volume I describes a method for selection of the type of stabilizers as well as pavement thickness design methods and construction information. Quality control, guide specifications, cost and energy considerations are contained in the Appendices. Volume II of the manual (Mixture Design Considerations) follows and has been prepared for materials engineers. This volume describes methods for selection of the type and amount of stabilizers. Methods of estimating stabilizer contents are presented as well as detailed test methods, mixture design criteria and typical mixture criteria.

The manual is directed to the engineer who is reasonably familiar with pavement technology, but who has limited experience with stabilized soil construction. Current technology of soil stabilization is presented in a complete but concise format such that the engineer can grasp the key elements and apply the information to his own needs. Suggested additional references are provided so that the reader may follow up on details of interest that are beyond the scope of this manual.

The information in this manual was obtained from an extensive review of published literature from the United States and abroad. In addition, the authors' familiarization and experience, coupled with correspondence and project site visits, have provided a background from which to extract pertinent information of direct use to the design and construction engineer. In short, it is presented as a state-of-the-practice of soil stabilization for pavements.

Every attempt has been made to present information that is technically correct and that can be applied with reasonable confidence. The methodology is provided with regard to acceptable testing methods that are generally known in the field and these are suitably referenced. Where newer or less known techniques or other information may not be readily available, it is provided in the manual. 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 as to designs and application of the technology contained therein.

Scope

This volume provides the engineer with information to perform the following:

1. Select the type or types of stabilizers suitable for a specific soil (Chapter II)
2. Determine lime contents for a particular soils based on results from laboratory tests, typical property values and an understanding of physical-chemical reactions (Chapter III)
3. Determine lime-fly ash contents for particular soils based on results from laboratory tests, typical property values and an understanding of physical-chemical reactions (Chapter IV)
4. Determine portland cement contents for particular soils based on results from laboratory tests, typical property values and an understanding of physical-chemical reactions (Chapter V)
5. Determine asphalt contents for particular soils based on results from laboratory tests, typical property values and an understanding of stabilization mechanisms (Chapter VI)
6. Determine combination stabilizer contents for particular soils based on results from laboratory tests, typical property values and an understanding of physical-chemical reactions (Chapter VII)

Stabilization as used in this manual refers to the use of the following chemicals for base and subbase stabilization:

1. lime
2. Lime-fly ash
3. Portland cement
4. Asphalt
5. Combinations of the above

The use of other chemical stabilizers is not discussed. Mechanical stabilization is not considered in detail; however, certain comparisons can be made between mechanical and chemical stabilization by use of this manual.

BACKGROUND

The concept of soil improvement or modification through stabilization with additives has been around for several thousand years. At least as early as 5000 years ago, soil was stabilized with lime or pozzolans. Although this process of improving the engineering properties of soils has been practiced for centuries, soil stabilization did not gain significant acceptance for road and airfield construction in the U.S. until after World War II. Today, stabilization with lime, lime-fly ash, portland cement, and bituminous materials is popular in only some areas of the country.

One of the major concerns in recent years has been shortage of conventional 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 also experienced shortages. The reasons include lack of the raw materials, environmental and zoning regulations which prohibit mining and production of aggregates, and land use patterns which make aggregate deposits inaccessible. These factors, and others, combine to produce an escalation of aggregate costs, with a resultant increase in highway construction and maintenance costs. Consequently, there is a great need to find more economical replacements for conventional aggegates. A natural result is that attention be focused on substitute materials such as stabilized soils and marginal aggregates that can be upgraded through stabilization.

Another area of concern has been the energy crisis brought on by the temporary shortage of petroleum experienced in the early and late 1970's. It is rapidly becoming a practice to consider the energy demands of a project as well as cost. In terms of highway construction materials, the trend will be toward the use of materials which require less energy input in their production, handling, and placement. A recent study (2) revealed that the energy requirements for producing the materials for various asphalts, crushed stone, and portland cement concrete pavements ranged from 30 to 96 percent of the total energy required for production, handling, and placing of various pavements. Since relatively small quantities of binders such as lime, cement, fly ash, and asphalt can be used to improve pavement layers using stabilization technology, total energy demands may be reduced as well as costs.

Utilization of the major stabilizers considered in this manual will depend on several factors (discussed in Chapter II) including the availability of the materials. 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 1974. (3) Fly-ash, as produced from burning of coal, is not produced in quantities shown in Figure 2, but the actual location is not noted. (3). With more recent changes to coal as a fuel, these figures may change rapidly over the next few years. For example, Figure 3 shows 1977 figures for coal/ash tonnages by region in the U.S. (4). Portland cement plant sites are shown in Figure 4. (5). Asphalt production by location is somewhat more difficult to pin down, 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.

In summary, existing literature suggests that soil stabilization is a desired design alternative. Specifically, soil stabilization may provide the following engineering advantages as compared to conventional unstabilized construction materials:

1. Function as a working platform (construction expediency)
2. Reduce dusting
3. Water-proof the soil
4. Upgrade marginal materials
5. Improve strength
6. Improve durability
7. Control volume change of soils
8. Improve workability of soil
9. Drying of wet soils
10. Reduce pavement thickness requirements
11. Conserve aggregate materials
12. Reduce cost
13. Conserve energy
14. Provide a temporary or permanent wearing surface.

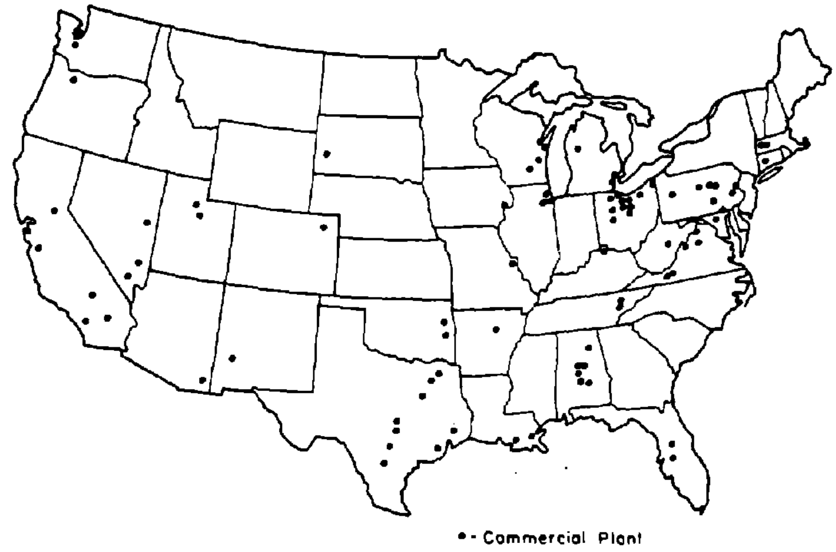


Figure 1. Commercial lime plants in the United States, 1974 (National Lime Association) (3).

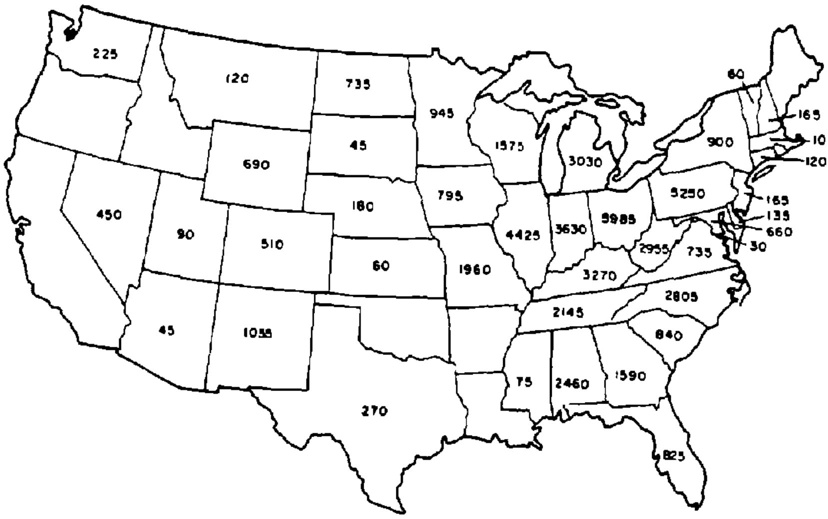


Figure 2. Approximate ash production (in 1,000s of tons) by major electric utilities, 1973. (3)

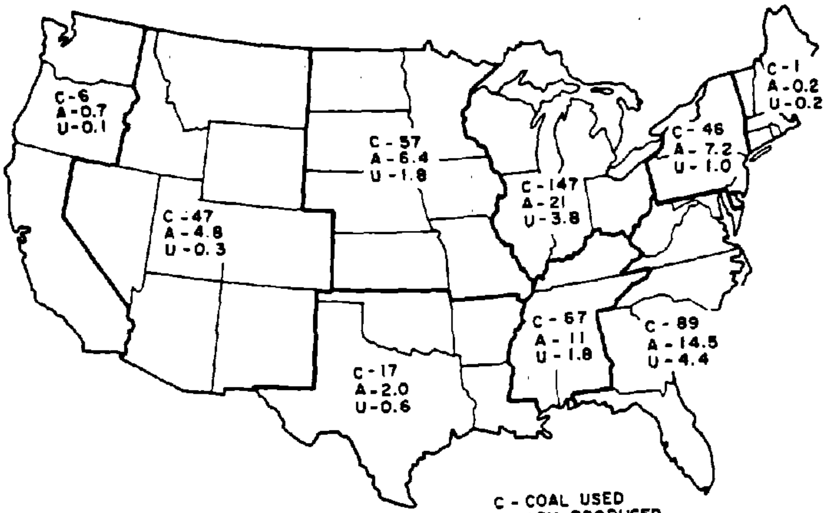


Figure 3. Fly ash availability (4).

Figure 4. Portland cement plant sites (5).

DEFINITIONS

Discussion of soil and aggregate stabilization requires use of terminology that may not be familiar, or at least needs to be defined for clarification. The following terms are presented with brief definitions as used in the manual.

General Definitions

Soil (Earth) - 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) (3).

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 by changing the gradation of the soil by the addition or removal of particles or by densifying or compacting the soil.

Aggregate - A granular material of mineral composition such as sand, gravel, shell, slag, or crushed stone, used with a cementing medium to form mortars or cement, or alone as in base courses, railroad ballast, etc.

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 abbrebiation AASHTO is also used.

ASTM - An abbreviation used to designate the American Society for Testing and Materials.

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 abbrebiation used to designate a mixture of lime-fly ash-aggregates.

LFCA - An abbreviation used to designate a mixture of lime- and cement-fly ash aggregates.

LFS - An abbrebiation used to designate a mixture of lime-fly ash and soil.

Definitions Associated with Portland 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). Portland cement will be referred to as cement in this manual.

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 initmate 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 inimate mixture of pulverized soil, portland cement, and water. Significantly smaller cement contents are used in cement-modified soil than in soil-cement.

Plastic Soil-Cement - A hardened material formed by curing an intimate mixture of pulverized soil, portland cement, and enough water to produce a mortarlike consistency at the time of mixing and placing. Plastic soil-cement can be used for highway ditch linings.

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, of which asphalts, tars, pitches, and asphaltites are typical.

Asphalt - A dark brown to balck cemetitious material in which the predominating constituents are bitumens which occur in nature or are obtained in petroleum processing.

Asphalt cement - A fluxed or unfluxed asphalt specially prepared as to quality and consistency for direct use in the manufacture of bituminous pavements, and having a penetration at 77 F (25 C) of between 5 and 300, under a load of 100 g applied for 5 s.

Cut-back asphalt - Petroleum residiums which have been blended with distillates.

Anionic emulsion - A type of emulsion such that a particular emulsifying agent establishes a predominance of negative charges on the discontinuous phase.

Cationic emulsion - A type of emulsion such that a particular emulsifying agent establishes a predominance of positive charges on the discountinuous phase.

Liquid bituminous materials - Those having a penetration at 77 F (25 C), under a load of 50 g applied 1 s, of more than 350.

II. SELECTION OF STABILIZER TYPE

INTRODUCTION

Stabilization of subgrade soils and aggregates by mechanical or chemical means is very common. The decision as to the proper form of stabilization (mechanical or chemical) as well as the selection of the proper chemical stabilizer is often made without the benefit of extensive field and laboratory testing. Ideally, field tests should be performed to determine the type or types and characteristics of the subgrade soils as well as to define the types and properties of borrow materials available. Laboratory tests should be performed to determine the engineering properties of mechanically stabilized and chemically stabilized soils and borrow materials. Cost and energy associated with providing the pavement structures designed with these materials should be calculated, based on engineering economic principles including first costs and maintenance and rehabilitation costs over a 20 to 30 year period. (Appendices C and D). Except for large projects, this desired engineering approach is rarely undertaken. Therefore, simplified guidelines need to be established to direct the engineer to those stabilization techniques which appear most suitable for the particular situation.

This chapter will present criteria which can be used as a guide to select the proper type of chemical stabilizer. The decision of using mechanical or chemical stabilization is one of economy and an appreciation of the engineering advantages and disadvantages of the different types of stabilization.

REVIEW OF EXISTING GUIDES

Several general guides have been published which assist the engineer in the proper selection of a stabilizer for a particular soil (7, 8, 9, 10, 11). For example, Air Force Manual AFM 88-51 (Table 1) contains information which suggests that lime is a more appropriate stabilizer for highly plastic clay soils while asphalt should be used only for the coarse and fine granular soils. More detailed guides such as those published by the Air Force (Table 2) and by Johnson (8), suggest stabilization methods for particular soil types based on both their location in the pavement structure and the purpose or function of their use (load carrying characteristics, waterproofing, etc.). Although these guides do not quantitatively indicate soil types for particular stabilizers, they do indicate the importance of recognizing the purpose of the use of the stabilizer in a particular location within the pavement structure.

More explicit selective information is being utilized by the Army and the Air Force (9). The gradation triangle shown on Figure 5 makes use of the following soil index properties to determine the proper type of stabilizer:

1. Percent material retained on No. 4 sieve,
2. Percent material passing No. 200 sieve,
3. Percent material passing No. 4 sieve and retained on No. 200 sieve and
4. Atterberg Limits.

As noted, the gradation triangle allows soils to be separated into selected areas (1A, 2A, 2B, 2C, 3). The Unified Soil Classification System is then used to further subdivide the type of soils and a stabilizer type is recommended. Restrictions for particular stabilizers within these soil groups are stated in terms of Atterberg Limits and percent material passing the No. 200 sieve.

Oglesby and Hewes (10) have presented a method of determining stabilizer types which was modified after the original work of the Division of Physical Research, Bureau of Public Roads (Figure 6). This method utilizes the Plasticity Index and percent passing the No. 200 sieve together with the American Association of State Highway Officials (AASHO) Soil Classification System for the purpose of stabilizer selection.

Additional criteria for selecting the type of chemical stabilizer are available in leterature pertaining to particular types of stabilizers. For example, lime stabilization literature contains information which indicates the types of soils which can be most readily stabilized with lime. Available criteria based on available literature are presented for lime, lime-fly ash, portland cement, and asphalt stabilization.

Table 1. Most Effective Stabilization Methods for Use with Different Soil Types (7)

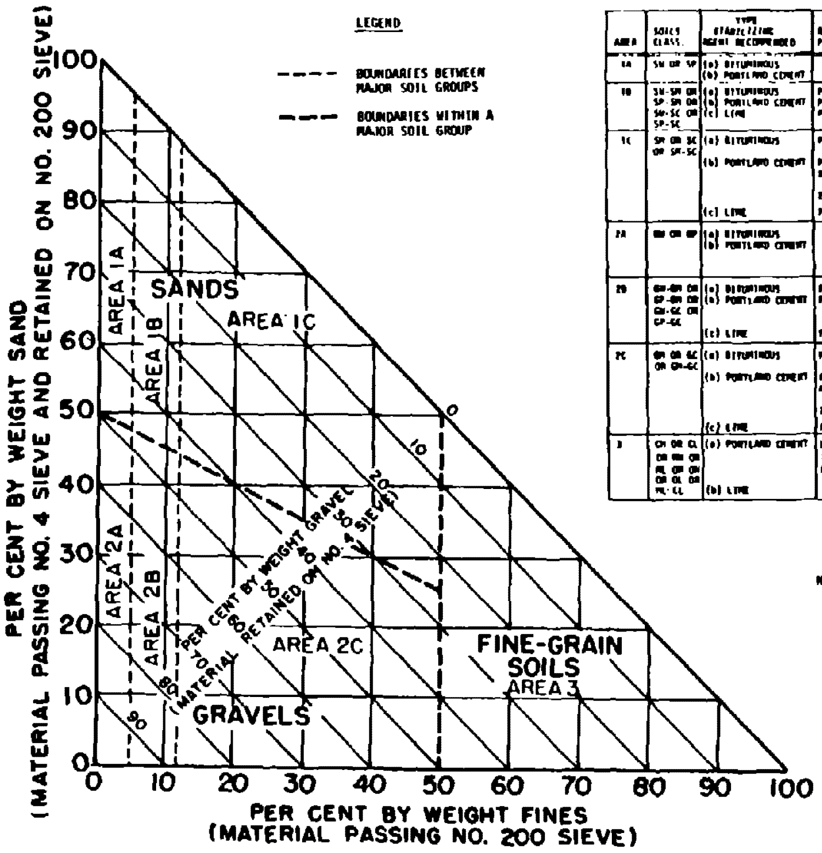
Soil Types	Most Effective Stabilization Methods
1. Coarse granular soils	Mechanical blending, soil-asphalt, soil-cement, lime-fly ash
2. Fine granular soils	Mechanical blending, portland cement stabilization, lime-fly ash, soil-asphalt, chlorides
3. Clays of low plasticity	Compaction, portland cement stabilization, chemical waterproofers, lime modification
4. Clays of high plasticity	Lime stabilization

Table 2. Soil Types and Stabilization Methods which Appear Best Suited for Specific Applications (7).

Purpose	Soil Type	Recommended Stabilization Methods
1. Subgrade Stabilization		
A. Improved load carrying and stress distributing characteristics	Coarse granular	SA, SC, MB, C
	Fine granular	SA, SC, MB, C
	Clays of low PI	C, SC, CMS, LMS, SL
	Clays of high PI	SL, LMS
B. Reduce Frost Susceptibility	Fine granular	CMS, SA, SC, LF
	Clays of low PI	CMS, SC, SL, CW, LMS
C. Waterproofing and improved runoff	Clays of low PI	CMS, SA, CW, LMS, SL
D. Control of shrinkage and swell	Clays of low PI	CMS, SC, CW, C, LMS, SL
	Clays of high PI	SL
E. Reduce resiliency	Clays of high PI	SL, LMS
	Elastic silts and clays	SC, CMS
2. Base Course Stabilization		
A. Improvement of substandard materials	Fine granular	SC, SA, LF, MB
	Clays of low PI	SC, SL
B. Improved load carrying and stress distributing characteristics	Coarse granular	SA, SC, MB, LF
	Fine granular	SC, SA, LF, MB
C. Reduction of pumping	Fine granular	SC, SA, LF, MB, membranes
3. Shoulders (unsurfaced)		
A. Improved load carrying ability	All soils	See Section 1A above, Also MB
B. Improved durability	All soils	See section 1A above
C. Waterproofing and Improved runoff	Plastic soils	CMS, SL, CW, LMS
D. Control of shrinkage and swell	Plastic soils	See section 1E above
4. Dust palliative	Fine granular	CMS, CL, SA, oil or bituminous surface spray
	Plastic soils	CL, CMS, SL, LMS
5. Ditch Lining	Fine granular	PSC, CS, SA
	Plastic soils	PSC, CS
6. Patching and Reconstruction	Granular soils	SC, SA, LF, MB

KEY:

C	Compaction	LMS	Lime Modified Soil
CMS	Cement Modified Soil	MB	Mechanical Blending
CL	Chlorides	PSC	Plastic Soil Cement
CS	Chemical Solidifiers	SA	Soil Asphalt
CW	Chemical Waterproofers	SC	Soil Cement
LF	Lime Fly Ash	SL	Soil Lime



AREA	SOILS CLASS.	TYPE STABILIZING AGENT RECOMMENDED	RESTRICTION ON LL, PL & PI OF SOIL	RESTRICTION ON (-200 MATL)	REMARKS
1A	SW OR SP	(a) BITUMINOUS (b) PORTLAND CEMENT			
1B	SW-SM OR SP-SM OR SW-SC OR SP-SC	(a) BITUMINOUS (b) PORTLAND CEMENT (c) LIME	PI NOT TO EXCEED 10 PI NOT TO EXCEED 30 PI NOT LESS THAN 12		
1C	SM OR SC OR SM-SC	(a) BITUMINOUS (b) PORTLAND CEMENT (c) LIME	PI NOT TO EXCEED 10 PI NOT TO EXCEED 20 AND BY EQ $20 + \frac{50 - \text{FINES CONT.}}{4}$ PI NOT LESS THAN 12	NOT TO EXCEED 30% BY WT	
2A	GW OR GP	(a) BITUMINOUS (b) PORTLAND CEMENT			WELL-GRADED MATL ONLY MATL SHOULD CONTAIN AT LEAST 45% BY WT OF MATL PASSING NO. 4 SIEVE
2B	GW-GM OR GP-GM OR GW-GC OR GP-GC	(a) BITUMINOUS (b) PORTLAND CEMENT (c) LIME	PI NOT TO EXCEED 10 PI NOT TO EXCEED 30 PI NOT LESS THAN 12		WELL-GRADED MATL ONLY MATL SHOULD CONTAIN AT LEAST 45% BY WT OF MATL PASSING NO. 4 SIEVE
2C	GM OR GC OR GM-GC	(a) BITUMINOUS (b) PORTLAND CEMENT (c) LIME	PI NOT TO EXCEED 10 PI NOT TO EXCEED AS INDICATED BY EQ. $20 + \frac{50 - \text{FINES CONT.}}{4}$ PI NOT LESS THAN 12	NOT TO EXCEED 30% BY WT	WELL-GRADED MATL ONLY MATL SHOULD CONTAIN AT LEAST 45% BY WT OF MATL OF MATL PASSING NO. 4 SIEVE
3	CH OR CL OR MH OR ML OR OH OR OL OR ML-CL	(a) PORTLAND CEMENT (b) LIME	LIQUID LIMIT LESS THAN 40 AND PI LESS THAN 20		ORGANIC AND STRONGLY ACID SOILS FALLING WITHIN THIS AREA ARE NOT SUSCEPTIBLE OF STABILIZATION BY ORDINARY MEANS

NOTE: (A) Soil classification corresponds to MIL-STD-619B. Restriction on Liquid Limit, Plastic Limit and Plasticity Index is in accordance with Method 102 in MIL-STD-621A.

Table 5. Gradation triangle for aid in selecting a commercial stabilizing agent.

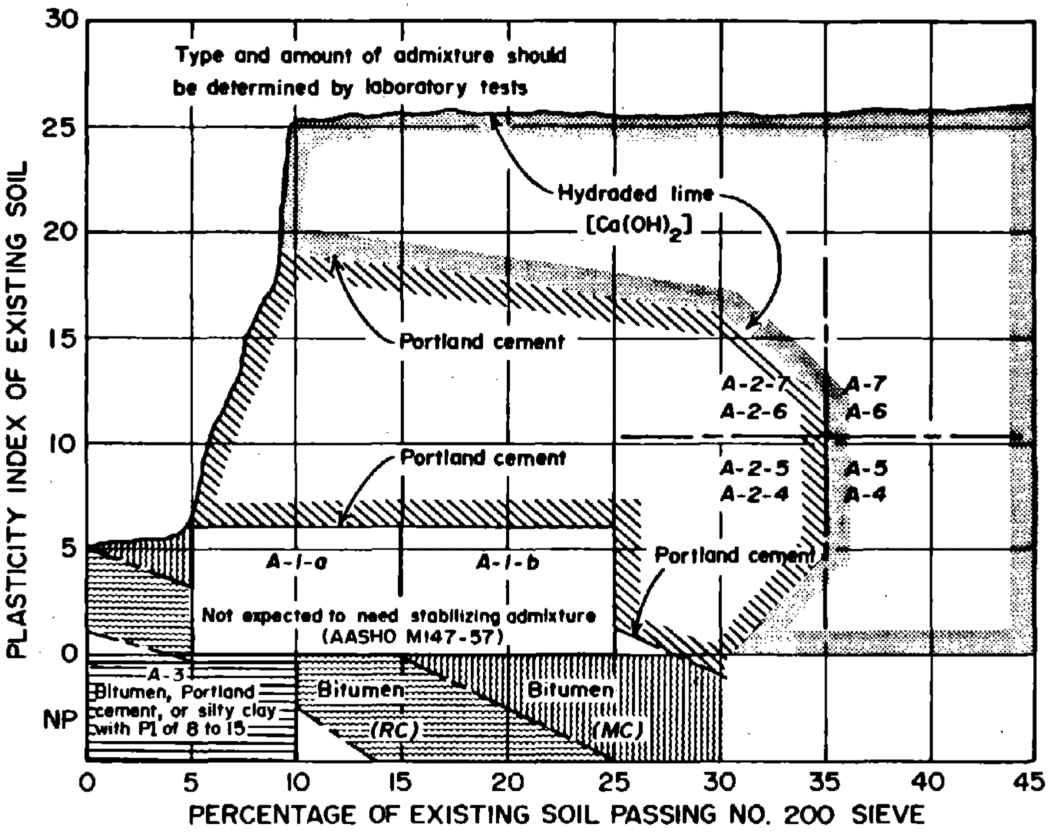


Figure 6. Suggested stabilizing admixtures suitable for use with soils.

Criteria for Lime Stabilization

Experience has shown that lime will react with medium, moderately fine, and fine grained soils to produce decreased plasticity, increased workability, reduced swell, and increased strength (12). Generally speaking, those soils classified by AASHO as A-4, A-5, A-6, A-7, and some of the A-2-7 and A-2-6 soils are most readily susceptible to stabilization with lime. Soils classified according to the United System as CH, CL, MH, ML, SC, SM, GC, GM, SW-SC, SP-SC, SM-SC, GW-GC, CP-GC, or GM-GC should be considered as potentially capable of being stabilized with lime.

Robnett and Thompson (12), based on experience gained with Illinois soils, have indicated that lime may be an effective stabilizer with clay contents ($<2\mu$) as low as 7 percent and, furthermore, soils with a P.I. as low as 8 can be satisfactorily stabilized with lime (13). Air Force (9) criteria presented in Figure 5 indicate that the P.I. should be greater than 12, while representatives of the National Lime Association (14) indicate that a P.I. greater than 10 would be a reasonable criteria to utilize.

Criteria for Cement Stabilization

The Portland Cement Association (15, 16) indicates that all types of soils can be stabilized with cement. However, well-graded granular materials that posses sufficient fines to produce a floating aggregate matrix have given the best results. Suggested gradings to meet this floating aggregate matrix concept should fall within the band specified in Table 3 (17).

Limits on Plasticity Index have been established by the Air Force (9) as shown in Figure 5 for different types of soils. As noted, the P.I. should be less than 30 for the sandy materials while the P.I. 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 gravel type materials a minimum of 45 percent by weight passing the No. 4 sieve is desirable. In addition, the P.I. of the soil should not exceed the number indicated from the following equation

$$20 + \frac{50 - \text{Fines Content}}{4}$$

Information developed by the Bureau of Public Roads (Figure 6) (10) indicates that cement should be used as a stabilizer for materials with less than 35 percent passing the No. 200 sieve and with a P.I. less than 20. Thus, this system implies that AASHO classified A-2 and A-3 soils can be best stabilized by cement while A-4, A-5, A-6, and A-7 soils can be best stabilized by lime.

Table 3. Grading Limits for Cement Stabilization of Well-Graded Granular Materials (17)

Sieve Size	
passing No. 4	minimum of 55 percent
passing No. 10	minimum of 35 percent
passing No. 10, retained No. 200	minimum of 25 percent

Criteria for Bituminous Stabilization

Asphalt cement, cutback asphalt and emulsified asphalt are the most commonly used bituminous soil stabilization binders. Current design and construction trends, particularly in the state highway departments, have indicated that stabilization of base courses with asphalt cements is by far the most popular form of bituminous stabilization (18). In general, those materials which are most effectively stabilized with asphalt cement have lower percentages of fines than those materials which have been stabilized with cutback asphalts and emulsion.

Some of the earliest criteria for bituminous stabilization were developed by the Highway Research Board Committee on Soil-Bituminous Roads. These criteria were revised and publised by Winterkorn (19) and appear in Table 4. The American Road Builders Association (20) made similar recommendations and these are shown in Table 5.

The Asphalt Institute in a 1965 publication (21) offered guidelines for grading and plasticity requirements for bituminous base courses as follows:

1. Less than 25 percent passing No. 200 sieve,
2. Sand equivalent less than 25 and
3. Plasticity Index less than 6.

Table 4. Types of Soil Bitumen and Characteristics of Soils Empirically Found Suitable for Their Manufacture (19)

Sieve Analysis	Soil Bitumen,† %	Sand Bitumen, %	Waterproofed Granular Stabilization, %		
Passing:			A	B	C
1 1/2-in.	...	...	100		
1-in.	‡	...	80-100	100	
3/4-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§ ||	8-12	10-16	13-30
		Characteristics of Fraction Passing No. 40 Sieve			
Liquid limit	<40	...	...	...	...
Plasticity index	<18	...	<10; <15	<10; <15	<10; <15¶
Field moisture equiv.	...	<20§	...	...	...
Linear shrinkage	...	<5§	...	...	...

† Proper or general.

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

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

|| A certain percentage of -200 or filler material is indirectly required to pass supplementary stability test.

¶ Values between 10 and 15 permitted in certain cases.

1 in = 2.54 X 10^{-2} mm.

A later version of this manual (22) recommends that the sand-equivalent test (ASTM D-2419 of AASHTO T176) be used to detect that presence of excessive quantities of clay-type fines. In general, materials with a sand-equivalent above 30 can be stabilized successfully with asphalt. The chances of successful stabilization with 20-30 sand-equivalent material depends on the ability of the asphalt and soil aggregate to be mixed and the particles effectively waterproofed. Stabilization of materials such as clay-gravels with sand equivalent of less than 20 generally has not been successful.

Herrin (23) has presented and revised (24) a table (Table 6) recommending suitable soils for stabilization by bituminous materials. Contained in this table are recommendations on the suitability of various soils with certain percentages of minus No. 200 material, and certain liquid limit and plasticity index ranges.

Certain limits have been developed by the Asphalt Institute's Pacific Coast Division, Chevron Asphalt Company, and Douglas Oil Company for emulsion-treated materials. The requirements recommended by the Asphalt Institute (25) (Table 7) suggest that the percent minus No. 200 should be in a range of 3-15 percent, the plasticity index should be less than 6, and the product of the plasticity index and the percent passing the No. 200 sieve should not exceed 60. The Chevron Asphalt Company (26) has presented criteria (Table 8) which indicate that the California Sand Equivalent (SE) test should be used as a measure of the plasticity requirements for the soil and have a minimum value of 30. Up to 25 percent passing the No. 200 sieve is allowed for some materials such as silty sands.

Dunning and Turner (27) of the Douglas Oil Company have presented guidelines for emulsion stabilization as shown in Table 9.

Materials Research and Development, Inc. of Oakland, California, has published a guide for stabilization for the U.S. Navy in which criteria recommended by the Asphalt Institute and Chevron Asphalt Company have been utilized (28). This guide recommends that the maximum percent passing the No. 200 sieve should be less than 25 percent, the plasticity index less than 6, Sand Equivalent less than 30, and the product of the plasticity index and the percent passing the No. 200 sieve less than 72 in all cases. These criteria apply to both cutback asphalt and emulsified asphalts used as soil stabilizers. The grading requirements suggested for sands and semi-processed materials are identical to those recommended by Chevron Asphalt Company (Table 8).

Table 5. Grading and Plasticity Requirements for Soil-Bitumen Mixtures (20)

Sieve Size	Percent Passing
No. 40	50-100
No. 200	0-35
Atterberg Limits	
LL	30
PI	10

Table 6. Engineering Properties of Materials Suitable for Bituminous Stabilization (23)

% Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel-Bitumen
1-1/2"			100
1"	100		
3/4"			60-100
No. 4	50-100	50-100	35-100
10	40-100		
40		35-100	13-50
100			8-35
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

(a) Includes slight modifications later made by Herrin. 1 in. = 2.54 X 10^{-2} mm.

Table 7. Grading, Plasticity, and Abrasion Requirements for Soils Suitable for Emulsified Asphalt-Treated Base Course (25)

Sieve Size	Percent Passing by Weight		
	2 inch maximum	1-1/2 inch maximum	3/4 inch maximum
2-1/2 inch	100		
2 inch	90-100	100	
1-1/2 inch		90-100	
1 inch			100
3/4 inch	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 Los Angeles Abrasion Machine	50 percent maximum

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

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1 in. = 2.54 X 10^{-2} mm.

Table 8. Typical Aggregates Suitable for Treatment** with Asphalts (26)

Category	ASTM Test Method	Processed* Dense Graded Aggregates	Poorly Graded	Well Graded	Silty Sands	Semi-Processed Crusher, Pit or Bank Run Aggregates
Gradation: 1-1/2"		100				100
% Passing 1"		90-100				80-100
3/4"		65-90				-
1/2"		-	100	100	100	-
# 4		30-60	75-100	75-100	75-100	25-85
16	C-136	15-30	-	35-75	-	-
50		7-25	-	15-30	-	-
100		5-18	-	-	15-65	-
200		4-12	0-12	5-12	12-25	3-15
Sand Equivalent, %	D-2419	30 min.	30 min.	30 min.	30 min.	30 min.
Plasticity Index	D-424	-	NP	NP	-	-
Untreated Resistance R Value		78 min.	60 min.	60 min.	60 min.	60 min.
Loss in Los Angeles Rattler (after 500 revolutions)	C-131	50 max.	-	-	-	60 max.

*Must have at least 25% Crush Count

**See AASHO T-174, T-175, and T-176.

1 in. = 2.54×10^{-2} mm.

Table 9. Guidelines for Emulsified Asphalt Stabilization (27)

	Good	Fair	Poor
% passing No. 200 sieve	3-20	0-3, 20-30	>30
Sand Equivalent	>25	15-25	<15
Plasticity index	<5	5-7	>7

State transportation agency gradation requirements for soils suitable for stabilization with asphalt cement in hot central plant operations have been recently reviewed and available information is contained in Reference 18. 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. Those state agencies that specify asphalt cement stabilized base materials normally will allow a larger maximum size of aggregate and a less restrictive grading bond.

Air Force recommendations for gradings of materials suitable for asphalt cement-treated base course are shown in Table 10 (29). Although the manual specifically recommends Gradations 6, 7, 8, and 9, it is believed that all gradations are practical, provided they are economically feasible.

According to a University of Illinois study (30), materials that are suitable for asphalt treatment include AASHO-classified A-1-2,A-1-b, A-2-4, A-2-6, A-3, A-4, and low plasticity A-6 soils, and soils classified by the Unified Classification System as SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, SC, SM-SC, GW, GP, GW-GM, GP-SM, GW-GC, GP-GC, GM, GC,-and GM-GC provided certain plasticity and grading requirements are met.

Criteria for Fly Ash Stabilization

Although fly ashes have been utilized for soil stabilization purposes in the United States since 1950, little definitive information is available which identifies soils which are most suitable for stabilization with fly ash, lime-fly ash, or cement-fly ash mixtures.

Fly ashes are normally used in stabilization operations to act as pozzolans and/or as fillers to reduce air voids in naturally occurring or blended aggregate systems. Since the particle size of the fly ash is normally larger than the voids in fine grained soils, the role as a fill-

Table 10. Aggregate Gradation Specification Limits for Bituminous Pavements (29).

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		
Surface Course															
	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	---	---						

1 in. = 2.54 X 10^{-2} mm.

er is not appropriate for use in fine grained soils. Thus the only role for the fly ash in stabilization of fine grained soils is that of a pozzolan. Most clays (but not all clays) are pozzolanic in nature and thus do not require additional pozzolans. Thus, silts are generally considered the most suitable fine grained soil type for treatment with lime-fly ash or cement-fly ash mixtures.

Aggregates which have been successfully utilized in lime-fly ash mixtures include a wide range of types and gradations, including sands, gravels, crushed stones, and several types of slag. Lime-fly ash aggregate mixtures are often more economical to use than lime-fly ash fine grained soil mixtures. In addition these mixtures coarser systems have greater resistance to frost action.

It should be pointed out that some fly ashes that are high in calcium oxide can be used with fine grained soils to form acceptable stabilized materials. These fly ashes are normally obtained from power plants itilizing Western United States coals.

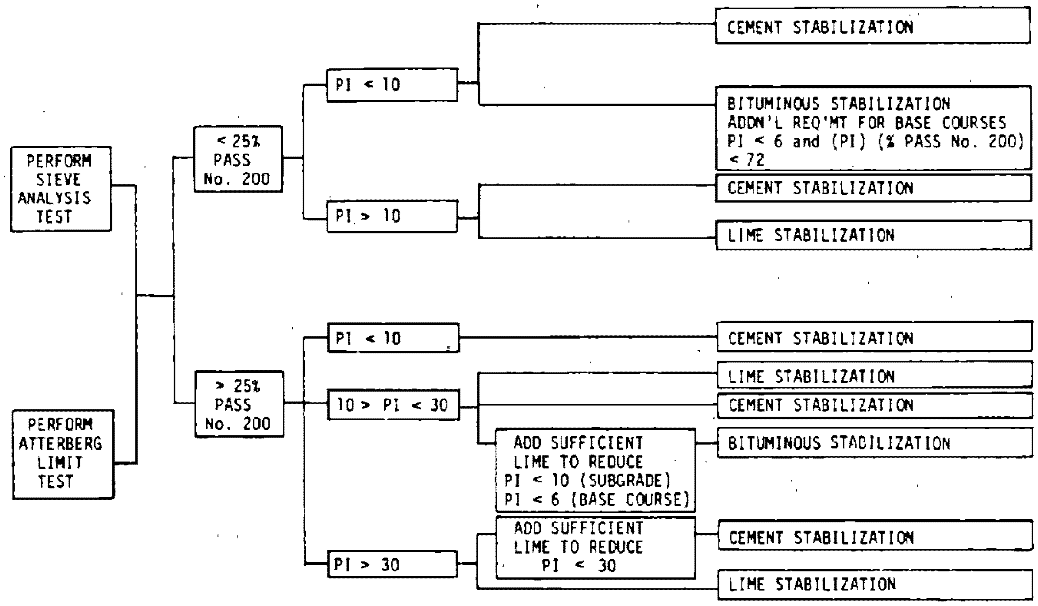

Figure 7. Selection of Stabilizer (31)

Combination Stabilizers

Combination stabilizers discussed in this section include lime-cement and lime-asphalt.

Robnett and Thompson (30) have reviewed the literature associated with combination stabilizers and have suggested that AASHO classified A-6 and A-7 soils and certain A-4 and A-5 soils can be economically treated.

The main purpose cited for using combination stabilizers is to reduce plasticity and increase workability so the soil can be inimately mixed and effectively stabilized. As noted, lime is the pretreatment stabilizer followed by cement or asphalt.

The advantage of using lime in certain asphalt stabilization operations is to prevent stripping of asphalts from certain aggregates in the presence of water. In addition, lime can be used to reduce the plasticity index and thus provide better mixing and coating.

Portland cement has been utilized in emulsion stabilization operations to help control emulsion break.

SUMMARY OF CRITERIA FOR SELECTING STABILIZING AGENTS

Criteria have been presented which represent a wide range of opinion as to the types of soils that can be stabilized by certain stabilizers. Most published information gives reference to soils classified either by the AASHO or Unified Soil Classification systems; however, the authors feel that a more appropriate separation of soils for stabilization can be made utilizing Atterberg Limits and sieve analysis. It should be remembered that both Atterberg Limits and sieve analysis are relatively easy tests to perform in the laboratory and both are a necessary input for the AASHO and Unified Soil systems. Figure 7 presents guidelines that can be used by the engineer to select the most suitable stabilizer for a given soil (31). Once the stabilizer is selected, detailed laboratory tests should be performed as outlined in those chapters associated with the individual stabilizers.

Once the type or types of stabilizers have been selected for a particular soil, the engineer should be aware of certain climatic limitations that may restrict the use of the stabilizer. In addition, safety considerations should be understood by the engineer prior to the selection of the stabilizer. General climatic limitations and construction safety precautions are given on Table 11.

Table 11. 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 at 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 at all times
Asphalt	Air temperature should be above 32 F (0 C) when using emulsions Air temperatures should be 40 F (5 C) and rising when placing thin lifts (1-inch) 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 350 F (175 C)

1 in. = 2.54×10^{-2} mm.

III LIME STABILIZATION

INTRODUCTION

Lime is one of the oldest soil stabilizing agents known to man. Lime stabilization was used as a roadway stabilizer by the early Romans as well as other early civilizations.

Lime stabilization of clayey base materials was used in Texas in the early 1940's and has been used by more than 40 states. Twenty-eight states have used lime for Interstate highway construction. Lime has also been used as a soil stabilizer for large municipal airport runways, military runways and roads, county and municipal roads and streets, and for parking areas.

TYPES OF LIME

In general, the term lime refers to oxides and hydroxides of calcium and magnesium, but not to carbonates. There are various types of lime commercially available. Calcitic quicklime (CaO) and dolomitic quicklime ($CaO + MgO$) are produced by calcining 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, $Ca(OH)_2 + Mg(OH)_2$. Reference 32 further defined the types of lime and lime production processes. Typical properties of commercial varieties of quicklime and hydrated limes are summarized in Table 12.

Various forms of lime, including products with varying degrees of purity, have been successfully utilized as a soil stabilizing agent for many years. However, the most commonly used products are hydrated high calcium lime $Ca(OH)_2$, monohydrated dolomitic lime $Ca(OH)_2$. MgO, calcitic quicklime CaO, and dolomitic quicklime $CaO \cdot MgO$. The use of quicklime for soil stabilization has increased during the past few years; in the United States it now accounts for more than 10 percent of the total stabilization lime, while in Europe quicklime is the major type used.

By-product lime also provides a source of lime that may be suitable for use in stabilization. This type of lime is usually available from various manufacturing processes. Two types of by-product limes commonly available are: (a) that collected from the draft of the calcining process in lime production operations (flue dust), and (b) the by-product of acetylene gas production from calcium carbide. By-product lime may be a very economical source of lime; however, these limes may be nonuniform in quality.

Table 12. Properties of Commercial Limes*

(a) Quicklime

Constituent (percent)	High Calcium		Dolomitic	
CaO	92.25 - 98.00		55.70 - 57.50	
MgO	0.30 - 2.50		37.60 - 40.80	
CO_2	0.40 - 1.50		0.40 - 1.50	
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	
Specific Gravity	3.2 - 3.4		3.2 - 3.4	
Specific heat at 100 F (38 C)	Btu/lb 0.19	(J/kg) (440)	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) (675)	Btu/lb 0.29	(J/kg) (675)	Btu/lb 0.29	(J/kg) (675)
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)

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*Data taken from "Chemical Lime Facts," Bulletin 214 (3rd ed.), National Lime Association (1973).

Although many by-product limes may be similar to virgin limes in terms of chemical composition, other important properties may be considerably different. For example, commercial hydrates generally are more finely divided and have higher specific surfaces than carbide limes.

A new form of lime for stabilization has recently been developed in the Chicago area. The material is a by-product hydrate produced by hydrating a mixture of flue dust and normal quicklime. Although the by-product hydrate is not chemically equivalent to normal commercial hydrated lime, it has been successfully used in soil stabilization (33). By-product hydrate is less expensive and more readily available in the Chicago area.

There is some concern as to whether calcitic lime, $Ca(OH)_2$, or monohydrated dolomitic lime, $Ca(OH)_2 + MgO$, is the more effective lime for soil stabilization. Studies by Thompson (34) and the Portland Cement Association (35) have shown that high calcium limes are generally more effective for modifying soil plasticity. Dolomitic limes produced higher cured strength in the Illinios study but the PCA investigation indicated that, "Most soils do not respond preferentially to dolomitic monohydrate or hydrated calcitic lime stabilizations for strength improvement." It can probably be concluded that either high-calcium or monohydrated dolomitic lime is, in general, satisfactory for use in soil stabilization.

Most types of lime (exclusively of dehydrated dolomitic, $Ca(OH)_2 \cdot Mb(OH)_2$) 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 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 (primarily particle size) properties are normally the major factors considered in a lime specification.

AASHTO M 216 is an example of a specification directed to the use of lime for soil stabilization. Many state and agency specifications incorporate ASTM C 207 (Type N) or a modified version of ASTM C 207 in their own specifications. Note that ASTM C 207 is entitled "Hydrated Lime for Masonry Purposes". ASTM has thus far not developed a lime specification for soil stabilization.

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.

SOIL-LIME REACTIONS

The addition of lime to a fine-grained soil initiates several reactions. Cation exchange and flocculation-agglomeration reactions take place rapidly and produce immediate changes in soil plasticity, workability, and the immediate uncured strength and load-deformation properties. Depending on the characteristics of the soil being stabilized, a soil-lime pozzolanic reaction may occur. The pozzolanic reaction results in the formation of various cementing agents which increase mixture strength and durability. Pozzolanic reactions are time dependent; therefore, strength development is gradual but continuous for long periods of time, amounting to several years in some instances. Temperature also affects the pozzolanic reaction. Termperatures less than 55 to 60 F (13 to 16 C) retard the reaction and higher temperatures accelerate the reaction (36).

Cation Exchange and Flocculation-Agglomeration

Practically all fine-grained soils display cation exchange and flocculation-agglomeration reactions when treated with lime. The reactions occur quite rapidly when soil and lime are inimately mixed.

The general order of replaceability of the common cations associated with soils is given by the lyotropic series, Na+ <K^+ <Ca^{++} <Mg^{++} (37). Cations tend to replace cations to the left in the series and monovalent cations are usually replaceable by multivalent cations. The addition of lime to a soil in sufficient quantity supplies an excess of Ca^{++} and cation will occur, with Ca^{++} replacing dissimilar cations from the exchange complex of the soil. In some cases, the exchange complex may be Ca^{++} - saturated before the lime addition, and cation exchange does not take place or is minimized.

Flocculation and agglomeration produce an apparent change in texture, with the clay particles "clumping" together into larger-sized "aggregates". According to Herzog and Mitchell (38), the flocculation and aggomeration 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. Diamond and Kinter (39) suggested that the rapid formation of calcium aluminate hydrate cementing materials is significant in the development of flocculation-agglomeration tendencies in soil-lime mixtures.

Soil-Lime Pozzolanic Reaction

The reactions between lime, water, and various sources of soil silica and alumina to form cementing-type materials are referred to as soil-lime pozzolanic reactions. 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 compared to the pH of natural soils. The solubilities of silica and alumina are greatly increased at elevated pH levels (40).

Studies have shown that the soil-lime reaction products are forms of hydrated calcium silicates and hydrated calcium aluminates. A wide variety of hydrate forms can be obtained, depending on reaction conditions, curing time, and temperature.

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 (41) has termed reactive those soils that react with lime to produce substantial strength increase, i.e., greater than 50 psi (345 kPa^2) following 28-day curing at 73 F (22.8 C). He has called non-reactive those soils that display limited pozzolanic reactivity, less than 50 psi (345 kPa^2) strength increase.

Certain soil properties and characteristics influence the lime-reactivity of a soil, i.e., the ability of the soil to react with lime to produce cementitious materials. These include soil pH, organic carbon content, natural drainage, presence of excessive quantities of exchangeable sodium, clay mineralogy, degree of weathering, presence of carbonates, extractable iron, silica-sesquioxide ratio, and silica-alumina ration. Detailed summaries concerning the effects of soil properties on lime reactivity are contained in references 41, 42, and 43. It is emphasized that the main factors controlling the development of cementitious materials in a lime-treated soil are the inherent properties and characteristics of the soil. If a soil is non-reactive, extensive pozzolanic strength development will not be achieved regardless of lime type, lime percentage, or curing conditions of time and temperature.

Carbonation

Lime carbonation is an undesirable reaction which may also occur in soil-lime (lime reacts with carbon dioxide to form a carbonate, $CaO + CO_2 \rightarrow CaCO_3$). Construction should be carried out in such a fashion that lime carbonation is minimized. Major actions to avoid are long exposure of the lime 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 rubber tired roller.

More extensive and detailed background information on basic soil-lime reactions can be found in papers written by Diamond and Kinter (39) and a comprehensive publication by Stocker (44).

Summary

Soil-Lime reactions are complex and not completely understood at this time. However, sufficient basic understanding and successful field experience are available to provide the basis of an adequate technology for successfully utilizing soil-lime stabilization under a wide variety of conditions.

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 respond most favorably. A minimum clay content (<2μ) 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 lay.

Suitable stabiliation of organic soils may be difficult with normal lime contents.

TYPICAL PROPERTIES OF LIME-STABILIZED SOILS

In general, when mixed with lime all fine-grained soils exhibit decreased plasticity, improved workability, and reduced volume change characteristics; 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. Soil type, lime type, lime percentage, and curing conditions (time, temperature, moisture) are the most important variables.

Generally only lime reactive soils (those that display a significant compressive strength increase) are utilized as structural paving layers. Cured lime-treated reactive soils are approximately termed "cemented materials". Thus, such engineering properties as strength, stress-strain behavior, and durability are of major interest. These properties will be considered in detail.

Lime treatment also has an immediate effect on pertinent soil properties. Immediate effects are achieved without curing and are of interest primarily during the construction stage.

Uncured Mixtures

Plasticity. Substantial reduction in plasticity (reduced PI, increased shrinkage limit) is effected by lime treatment and, in some cases, the soil may become nonplastic. Generally, high initial PI and clay content soils require greater quantities of lime for achieving the nonplastic condition, if it can be achieved at all. The first increments of lime are generally most effective in reducing plasticity of the lime-treated soil.

The silty and friable texture of the treated soil causes a marked increase in workability. The improved level of workability expedites subsequent manipulation and placement of the treated soil.

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. Maximum dry density reductions of 3-5 pcf and optimum water content increases of 2-4 percent are common. Figure 8 illustrates the effect of lime on the compaction characteristics of a CL soil (AASHTO T-99 compaction).

If a mixture is allowed to cure and thus gain strength in a loose state prior to compaction, further reductions in maximum dry density and increases in optimum moisture content may be noted as compared to untreated soil. It is therefore important that the appropriate moisture-density relation be utilized for field control purposes.

Swell Potential. Soil swell potential and swelling pressures are normally significantly reduced by lime treatment. The reduced swell characteristics are generally attributed to 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 (45). Mitchell and Raad (46), in considering additive treatments for swell control, concluded that lime continues to be the most effective additive for stabilization of expansive soils.

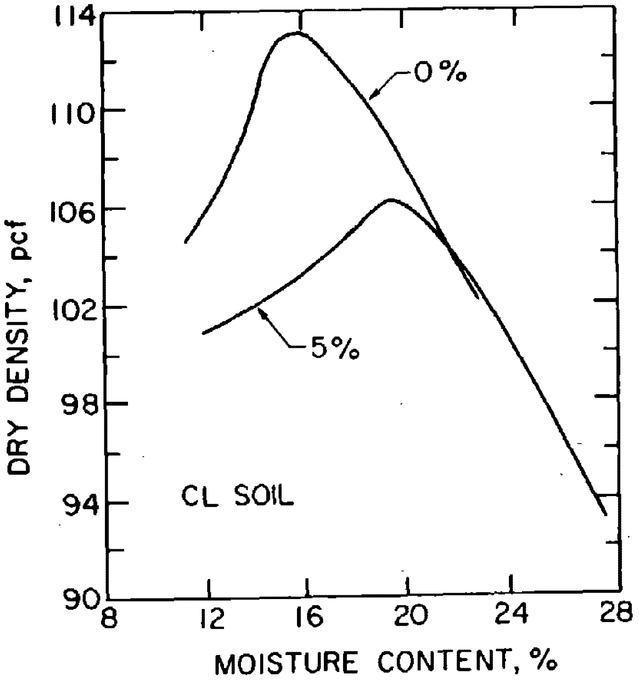
Strength and Deformation Properties. Lime treatment of fine-grained cohesive soils produces immediate improvements in strength and deformation properties of "uncured" soil-lime mixtures. These immediate benefits can be characterized in terms of shear strength, CBR, cone index, static-compressive modulus of elasticity, and resilient modulus.

Typical moisture content - CBR relations for an uncured soil-lime mixture and the natural soil are shown in Figure 9. The compactive effort as AASHTO T-99.

Robnett and Thompson (47) characterized the immediate effects of lime treatment on the resilient behavior of fine-grained cohesive soils. Figure 10 illustrates the improvements obtained.

Some agencies, California for example, use stabilometer data to characterize uncured soil-lime mixtures. R-values in excess of 60 to 80 are typically obtained. A recent California Study (48) indicated that the R-value of an uncured soil-lime mixture is not necessarily indicative of the ultimate or cured structural value of lime-treated materials.

It is apparent that the immediate strengthening effects of lime treatment are substantial. As curing progresses and the soil-lime pozzolanic reaction proceeds, the soil-lime mixture will develop much higher levels of strength and stiffness characteristics.



1 pcf = 1.602 X 10' kg/mm.^{3}

Figure 8. Moisture-Density Relations for a Natural and 5% Lime-Treated CL Soil (AASHTO T99 Compaction).

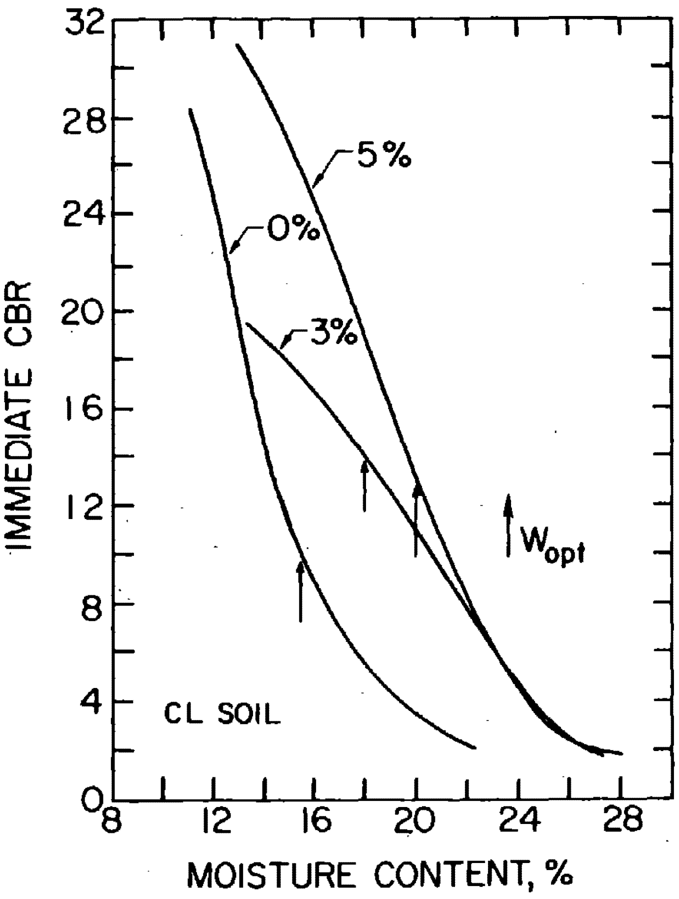


Figure 9. CBR-Moisture Content Relations for Natural and Lime Treated (3%,%%) CL Soil (AASHTO T99 Compaction).

Cured Mixtures

The strength and deformation properties of lime-treated soils are dependent on many variables. Soil type, lime type, lime percentage, compacted density, and curing conditions (time-temperature) are the most important. The properties of a lime-treated soil are, therefore, not "Static values" but will vary in response to changes in the variables listed above.

Strength Properties.

Unconfined Compression. The unconfined compression test is a simple and effective test for evaluation of the properties of treated soils.

Soil-lime mixture strength is measured by cured mixture strength minus strength of natural soil, and varies substantially (49, 50). Soil-lime mixture strength increases for Illinois soils cured 28 days at 73 F (23 C) extend up to approximately 265 psi (1724 kPa) with many soils displaying increases greater than 100 psi (690 kPa). 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).

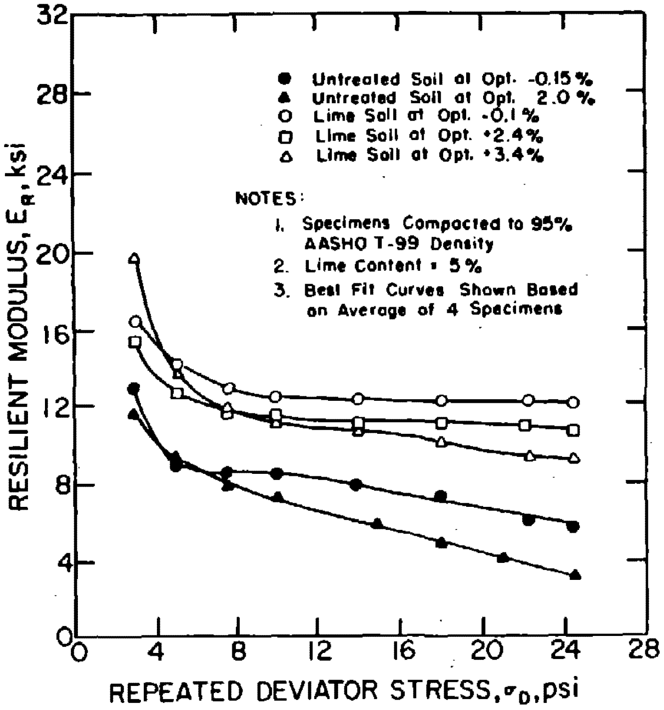
Extensive California test data (51) indicate a wide range of strength increases for 5 percent lime treatment and 6-month curing (ambient lab temperature, sealed curing to preserve moisture content). The maximum strength increase achieved was 770 psi (5300 kPa), and 30 of the 41 soils evaluated developed compressive strength increases in excess of 100 psi (690 kPa). Field data indicate that with some soil-lime mixtures, strength continues to increase with time 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-pozzolanic reaction has proceeded (50). Substantial strength increase indicates that the soil is reactive with lime and can be stabilized to produce a quality paving material.

Shear Strength. Some cured soil-lime mixtures (stabilized reactive soils) are cemented type materials. The major effect of lime on the shear strength of a reactive fine-grained soil is to produce a substantial increase in cohesion with some minor increase in friction angle (ϕ). At the low confining pressures normally considered to exist in a flexible pavement structure, the cohesion increase is of the greatest significance.

Substantial shear strengths can easily be developed in cured lime-soil mixtures. It has been demonstrated (52) that if high quality soil- lime mixtures are used in typical flexible pavement structures, the strengths are adequate to prevent shear failure. Shear type failures have not been noted and/or reported to any extent for field service conditions.

For typical lime reactive Illinois soils, the angle of shearing resistance for cured soil-lime mixtures ranged from 25° to 35° (52). The cohesion of the mixtures was substantially increased compared to the natural soils,



1 psi = 6.89 X 10^3 Pa

1 ksi = 6.89 X 10^6 Pa

Figure 10. Effect of Lime Treatment and Variable Compaction Moisture on Resilient response of Flanagan B Soil.

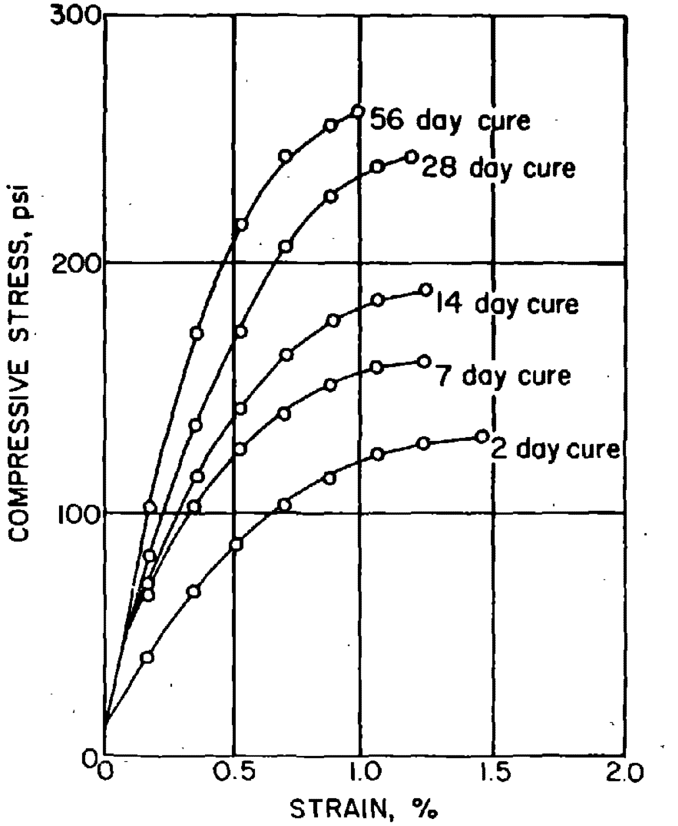


Figure 11. Compressive Stress-Strain Relations for Cured Soil-Lime Mixtures. (Goose Lake Clay + 4% Lime) (55).

and cohesion continued to increase with increased compressive strength. Cohesion can be predicted based on unconfined compressive strength results. (32).

Cohesion (psi) = 9 + 0.29 Unconfined Strength (psi)

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 possession substantial resistance to tensile stresses. Two test procedures, split-tensile and flexural, have been used for evaluating soil-lime mixture tensile strength.

Split-tensile strength display large variation depending on the soil-lime mixture and curing conditions. According to Thompson's study (53), the ratio of split-tensile strength to unconfined compressive strength of the mixtures is approximately 0.13.

The most common method used for evaluating the tensile strength of highway materials is the flexural test. A realistic estimate of flexural strength (modulus of rupture) is 25 percent of the unconfined compressive strength of the cured soil-lime mixture (45).

California Bearing Ratio (CBR). CBR testing procedures have been extensively used to evaluate the strength of lime stabilized soils. Many agencies have arbitrarily adopted this technique because of their familiarity with the test. In reality, CBR is not appropriate for characterizing the strength of soil-lime mixtures under all conditions.

The CBR values for any cured soil-lime mixtures are quite large and definitely indicate 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. If extensive pozzolanic cementing action has not developed due to either lack of curing time or nonreactivity of the treated soil, the CBR value may serve as a general strength indicator.

Fatigue Strength. It has been shown (54) that for typical highway pavement loading conditions flexural strength, not the shear strength, of reactive soil-lime mixtures will probably be the limiting factor in their applications as subbase and base courses. Therefore flexural fatigue is an important consideration in the evaluation of lime-soil mixtures. The flexural fatigue 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 laboratory fatigue strengths (at 5 million stress repetitions) of soil-lime mixtures in the Illinois study (54) varied from 41 to 66 percen tof 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 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 flexural strength) will decrease and the fatigue life of the mixture will increase.

Deformation Properties

Stress-strain properties are essential for properly analyzing the behavior 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 11 (55). The failure stress is increased and the ultimate strain is decreased for soil-lime mixtures to the natural soil. Soil-lime mixtures tested in triaxial compression are strain sensitive and the ultimate strain (for maximum compressive stress) is approximately 1 percent, regardless of the soil type or curing period (52). It has been demonstrated that the compressive modulus of elasticity (for 15 psi [103.4 kPa] confining pressure) can be estimated from the unconfined compressive strength of the soil-lime mixture according to the following relation (52):

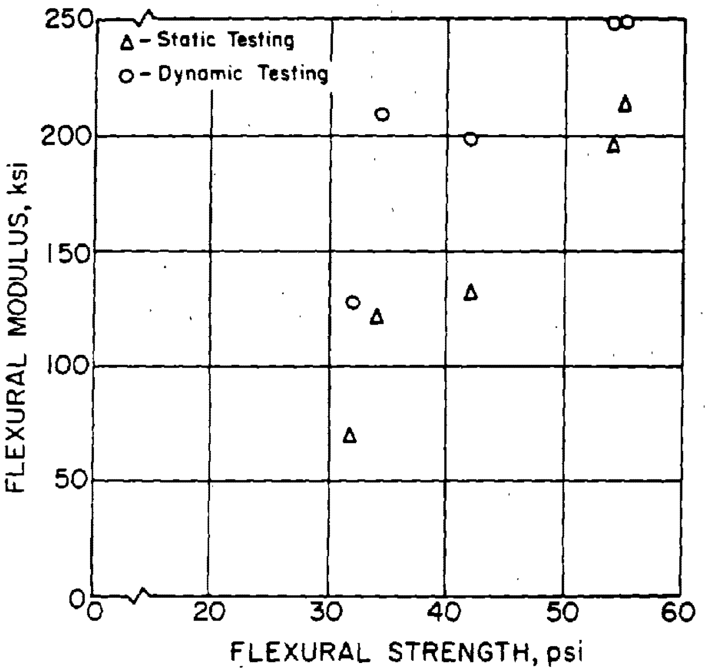
$$E(\text{ksi}) = 9.98 + 0.124 \text{ Unconfined Compressive Strength (psi)}$$

Repeated or dynamic compressive loading data for soil-lime mixtures are limited. Suddath and Thompson (55) evaluated the resilient moduli for a 4 percent lime-treated Goose Lake Clay mixture following curing periods of 2, 7, 14, and 28 days. The cured static compressive strengths ranged from about 125 psi (862 kPa) to 250 psi (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 psl (483,000 kPa) to 125,000 psl (862,000 kpa). In contract, static modulus of deformation data for the same mixtures were substantially lower and varied from approximately 16,000 psl (110,000 kPa) to 45,000 psl (310,000 kPa).

Flexural Moduli. For soil-lime pavement layers possessing high shear strength, the flexural stresses in the lime-soil mixture may be the controlling design factor. In view of this fact, flexural moduli of elasticity (static loading) have been evaluated for typical cured soil-lime mixtures (45). Flexural modulus and flexural strength increase with extended curing.

A recent study (55) considered the dynamic flexural moduli of elasticity for cured lime-treated Goose Lake Clay mixtures. Curing conditions ranged from 2 to 56 days @ 73 F (23 C). Dynamic and static flexural moduli were determined at approximately 50 percent of the ultimate flexural strength of the mixture. Figure 12 is a plot of flexural modulus versus flexural strength. Increase curing and strength development is accompanied by a corresponding increase in flexural modulus.

Poisson's Ratio. Only limited data are available for cured soil-lime mixtures (45, 55). Reported values at low stress levels (less than 50 percent of the ultimate compressive strength) are generally in the range of 0.1 to 0.2. At higher stress levels, Poisson's ratio may be closer to the 0.2 - 0.3 range. A value of 0.15 to 0.20 is reasonable for analysis purposes.



1 psi = 6.89 X 10^3 Pa

1 ksi = 6.89 X 10^6 Pa

Figure 12. Flexural Modulus - Flexural Strength Relationships for Cured Soil-Lime Mixture (Goose Lake Clay + 4% Lime) (55).

Summary. Since the properties 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 due to field curing effects. It may be more desirable to use unconfined compressive strength or other simple tests for evaluating the quality of the mixtures and estimate other pertinent mixture properties utitizing previously developed correlations.

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. Limited data (56) for four typical Illinois soils indicated that lime treatment decreased shrinkage potential. 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 indicated that for typical field service conditions shrinkage of cured soil-lime mixtures will not be extensive. A comprehensive California study (48) of the field performance of lime-treated roadways showed that only 20 percent of the pavements with soil-lime bases showed evidence of shrinkage cracking which ultimately reflected through the surface course.

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 only thin surface courses or only surface treatments.

Moisture Effects. Prolonged exposure to water produces only slight detrimental effects, and the ratio of soaked to unsoaked compressive strength of the soil-lime mixtures is quite high (57), on the order of .7 to .85. For the mixtures included in the Illinois study (57), the soaked specimens seldom achieved 100 percent saturation and, in most cases, the degree of saturation was in the range of 90-95 percent. Similar response to soaking has been noted in extensive studies conducted by the Road Research Laboratory, England (58).

Freeze-Thaw Effects. 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 lime 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 (59) have demonstrated that the most expeditious way of characterizing the field temperature environment is through the use of a theoretical heat flow model. Their data indicated that for Illinois the dominate frost action form was cyclic freeze-thaw and not heaving.

Cyclic Freeze-Thaw. In zones where freezing temeratures occur, freeze-thaw damage may be incurred in the soil-lime mixtures. The damage is generally characterized by volume increase and strength reduction (60).

Initial unconfined compressive strength (0 freeze-thaw cycles) is a good indicator of freeze-thaw resistance. Recent freeze-thaw durability studies (61) of several different types of "cementitious stabilized materials" (soil-lime,soil-cement, lime-fly ash) have confirmed the concept that initial compressive strength of the cured stabilized mixture can be used to predict the cyclic freeze-thaw resistance of stabilized soils. Factors influencing strength development (curing time, density, additive content, etc.) influence cyclic freeze-thaw resistance in the same fashion.

It has been shown that some soil-lime mixtures display autogenous healing properties (62). 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, since autogenous healing during favorable curing conditions would serve to restore the stability of the material. Confirming field data on autogenous healing have been presented by McDonald (63).

Frost Heaving Action. In the past, little consideration has been given to the heaving resistance of soil-lime mixtures. The general philosophy regarding heaving potential of "cemented systems" has been founded on the assumption that if a certain critical strength level is achieved, the tensile strength of the stabilized material is sifficient to withstand the heaving pressures generated, thus limited the heave potential to tolerable values. British experience with the Road Research Laboratory Heave Test (65, 65) 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." Many American and European mix design procedures for soil-cement suggest minimum 7-day cured strength of 250-400 psi (1723-2758 kPa) which supposedly would result in "durable" materials with good resistance to heaving.

Dempsey and Thompson (66) demonstrated that sustained freezing of a soil-lime mixture does not effect a strength decrease. It should be noted though that the test conditions provided for the whole specimen to be forzen, 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 (67). 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 (64) for non-frost-susceptible materials is a heave of less than about 8 percent.

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 (64) conclusively demonstrated that is sufficient strength is developed in the cement-stabilized material, the material will not heave excessively (<8%).

Summary. 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, the "residual strength" of the stabilized materials is adequate to meet field service requirements. Durability considerations must be taken into account in establishing the mix composition and selecting engineering properties for use in pavement design.

SELECTION OF LIME CONTENT

The major objective of the mixture design process is to establish 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. Because of the many varied applications of lime treatment of soils, several mixture design procedures have been developed. The general principle of soil-lime mixture design is that the mixture should provide satisfactory performance when constructed in a desired position in the pavement structure. It is apparent that a wide range of soil-lime mixtures of varying quality can be successfully used to accomplish differing lime treatment objectives. Generally, 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 base and subbase applications, acceptable soil-lime mixtures may not be produced regardless of the lime percentage used to treat certain soils.

Approximate Quantities

Most fine-grained soils can be effectively stabilized (relative to a predetermined stabilization objective) with 3010 percent (dry weight of soil basis) lime. Under normal field construction conditions, 2-3 percent lime is the minimum quantity that can be effectively distributed and mixed with a fine-grained soil.

A quick test for determination of lime content has been developed by Eades and Grim (68) and is suggested for use when detailed testing cannot be performed.

Detailed Testing

The basic components of a mixture design procedure generally are:

1. method for preparing the soil-lime mixture,
2. specimen preparation,
3. procedure for preparing and curing specimens,
4. testing procedures for evaluating a selected property or properties of the cured soil-lime mixtures, and
5. appropriate criteria for establishing the design lime content.

A comprehensive summary (from reference 69) of various "strength based" mixture design procedures is presented in Appendix A.

General comments concerning the various aspects of mixture design are presented below.

Mixture Preparation. Lime contents generally are specified as a percentage of the dry weight of soil, although a few agencies specify on a volume basis. Soil-lime mixtures are normally prepared first by dry mixing the proper amounts of soil and lime and then blending the required amount of water into the mixture. In most procedures, mixtures are prepared at or near optimum moisture content as determined by AASHTO T-99, T-180, or T-212.

Frequently the soil-lime mixture is allowed to mellow one hour or some other designated time prior to preparing test specimens.

ASTM D3551-76 (Laboratory preparation of Soil-Lime Mixtures Using a Mechanical Mixer) is a good general guide. AASHTO T220-66 (Determination of the Strength of Soil-Lime Mixtures) also describes a mixture preparation procedure.

Specimen Preparation. Strength test specimens are generally cylindrically shaped. Diameter and heights vary substantially, ranging from 1.4 incles (35.6 mm.) in diameter by 2.8 inches (71.1 mm.) high to 6 inches (152 mm.) in diameter by 8 inches (203 mm.) in height. Since the length to diameter ratios (l/d ratios) vary, it is recommended that compressive strength values be corrected to an l/d ratio of 2 for comparison purposes.

ASTM standard method of test C42-68 can be used for this correction.

The density of the compacted specimens must be carefully controlled because the strength of a cured soil-lime mixture is greatly influenced by density, and small density variations may make it difficult to accurately evaluate the effect of other variables such as lime percentage and curing conditions. Thus, the compactive effort should always be specified since some test methods specify AASHTO T-99 or the equivalent and other procedures specify AASHTO T-180 or T-212 or some other value.

Curing conditions. Time, temperature, and moisture conditions during the curing period vary significantly. Some agencies cure at room temperatures, while other cure at elevated temperatures, e. g., 48 hours at 120 F (48.9 C). Normally elevated temperature curing is of shorter duration than ambient temperature curing. Many procedures specify that the specimens should be cured in a sealed condition while other procedures (AASHTO T-220) require a moist curing cycle followed by a drying and capillary wetting cycle. In some procedures no curing period is required. A summary of various current curing conditions is presented in Table 13.

Laboratory curing conditions should, to some degree, be correlated with field conditions. For freeze-thaw zones, the first winter's exposure is most critical. Thus it is important to approximate the "field strength" of the mixture prior to the beginning of the winter.

The great disparity in curing conditions makes it very difficult to compare the results obtained from different testing methods. Thus, mixture quality criteria developed for a particular test procedure should not be arbitrarily adopted for analyzing test results obtained from a different test method.

Testing Procedures. Soil-lime specimen testing procedures usually involve conventional tests. For example, California Bearing Ratio (AASHTO T-193) and R-Value (AASHTO T190) are used for many different types of materials. AASHTO T-220-66 (Determination of the Strength of Soil-Lime Mixtures) is the only "Standard Procedure" available. The various mixture design procedures described in Appendix E contain testing instructions.

There is probably more variation in unconfined compression testing that in any other procedure. Thus, details concerning specimen size, rate of loading, Etc., should be specified in the description of any test method which is not standardized.

Mixture Design 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., invironmental factors, wheel loading considerations, design life, etc. It is thus 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. For soil-lime mixtures utilized in structural layer applications, minimum strength requirements are generally specified. Design lime content is normally that percentage which produces the required strength for given curing conditions.

Most current minimum strength criteria are specified in terms of compressive strength. The minimum strength requirements generally are higher for base materials than for subbase materials since stress and durability conditions differ for various depths in the pavement structure. Typical strength requirements are summarized in Table 13.

Although the mixture design procedures described in Appendix E do not require a "durability test," soil-lime mixture durability is a major consideration. Mixture durability is discussed earlier in this report. Minimun compressive strength requirements can be used to ensure mixture durability.

Table 13. Typical Specimen Curing and Strength Requirements

Agency	Specimen Curing	Minimum Strength Requirements, kPa	
		Base Course	Subbase
California DOT	Mixture is "loose cured" 24 hours prior to compaction.		
Illinois DOT	48 hours @ 120 F (48.9 C)	150	100
Louisiana DOT	7 day moist room, 8 hours air drying @ 140 F (60 C), 8 hours cooling, and 10 day capillary soaking at a confining pressure of 1 psi (AASHTO T-212)	100	50
Texas State Department of Highways and Public Transportation	AASHTO T220	100	50
Virginia Department of Highways and Transportation	72 hours @ 120 F (48.9 C)	Not Used	150

1 psi = 6.89 X 10^3 Pa

Mixture design criteria can be validated only on the basis of actual field performance. McDowell's extensive publications (references 70 through 75) concerning Texas experiences, Anday's summary (76) of Virginia projects, McDonald's recent reports (63, 77, 78, 79), a Corps of Engineer Study (80), Kelley's recent field study (81) and California's comprehensive field investigation (48, 82) are examples of extensive validation activities for widely separated geographic areas with drastically different climatic conditions.

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 apsects of the problem before adopting any criteria. As a starting point it is suggested that test methods and criteria as given in Appendix A be utilized.

IV. LIME-FLY ASH STABILIZATION

INTRODUCTION

The use of lime-fly ash-aggregate mixtures in road construction has increased steadily 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 in acceptance to the point where it is now in the specifications as an accepted material for base and subbases in a number of states and has been approved for use by both the Federal Highway Administration and the Federal Aviation Administration. 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. Several million tons of lime-fly ash-aggregate mixtures are currently used on an annaul basis in pavement construction.

In-service pavements built with lime-fly ash-aggregate mixtures, or lime-and-cement-fly ash-aggregate mixtures, have base thicknesses which range from 4 to 5 inches (100 to 125 mm.) for light duty parking facilities to 6 to 10 inches (150 to 250 mm.) for medium and heavy duty pavements, and up to 30 inches (750 mm.) for the very heavy duty pavements. A wearing surface is always used with these materials--either a bituminous surface treatment for light duty pavements, or asphalt concrete ranging from 2 to 6 inches (50 to 150 mm.) in thickness for medium and heavy duty pavements.

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 the 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 which is handled by a number of methods.

The characteristics of 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.

Collected fly ash can be stored either in the dry state in silos or other protected storage bins or in a dampened or conditioned state referred to as conditioned fly ash. Fly ash is sometimes 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 ashes wich contain 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 use 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 usually will segregate by particle size and may react chemically and thus will not produce as uniform and desired a product as fly ashes 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 C593 "Fly Ash and other Pozzolans for Use with Lime." Conditioned fly ashes which take on an initial set must be crushed before use, but are usually not crushed to the same degree of fineness as dry fly ashes. While it has been shown that the amount of minus 200 material in the fly ash is a most critical factor in the reactivity of the fly ash, most specifications for crushed fly ash specify a fineness in the following range for production control:

Sieve Size	Percent Passing
1/2 inch (12.2 mm.)	100
3/8 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 (passing the #200 sieve).

SOIL, LIME-FLY ASH REACTIONS

Reactions which occur in lime-fly ash-water systems are quite complex. Several studies provide 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 (83) 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. Low-sulfate sulfoaluminates may also be formed under favorable conditions.

The amorphus glassy component in the fly ash is believed to provide 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 (CaOH 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 ashes 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 (84 to 87) have established some basic properties of fly ash which are at least indicative of the fly ash reactivity. These include:

1. Increased percentage of fly ash passing the No. 325 (45-μm) sieve (84) or increase surface area (85, 86),
2. Increased SiO_2 (2, 3), $Si)_2 + R_2O_3$ (2), and $SiO_2 + Al_2O_3$ (85) contents ($R = Ca^{++}$ or Mg^{++}),
3. Low carbon content (84) or low loss on ignition (86) and
4. Increased alkaline contents (86).

Minnick, et. al. (87) emphasize 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."

In addition to the primary reaction between the 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 (See Chapter III on Lime Stabilization).

Cation exchange and flocculation-agglomeration reactions take place quite rapidly and couse 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.

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.

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 the 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 III). 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 (88 to 95). The resulting mixtures, if designed to be economically competitive with other methods of construction, are usually not as high quality as the lime-fly ash-aggregate mixtures, in part because of the initial lack of mechanical stability in the unstabilized soils and the greater tendency towards frost-susceptibility in 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:

1. Base course for secondary roads, parking lots, etc., where heavy traffic loads are not anticipated,

2. Subbase beneath conventional pavements, and

3. Subgrade improvement 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 4, pages 92 - 121.

Because of the number of variable 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 (61, 84, 85, 86, 98, 99, 100, 101, 102). Aggregates should be of such gradation 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, free from soft or disintegrated pieces.

Fine-grained aggregate mixtures have generally produced materials of greated durability than coarser-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 coarser-grained aggregates. The key to the ultimate strength development lies in the lime-fly ash matrix rather than the aggregate. Typical aggregate gradations which have been specified for use in lime-fly ash-aggregate mixtures are shown in Table 14. Some optimum gradation ranges for well-graded aggregates are given in Table 15. Other typical requirements for aggregates appear in Table 16.

Table 14. Typical Aggregate Specifications for Lime-Fly Ash Mixtures

Sieve	% Passing			
	Illinois	Pennsylvania		Ohio
2" (50.0 mm)	---	100	100	100
1 1/2" (38.1 mm)	100	---	---	---
1" (25.0 mm)	90-100	---	---	---
3/4" (19.0 mm)	---	52-100	70-100	---
1/2" (12.5 mm)	60-100	---	---	50-85
3/8" (9.5 mm)	---	36-70	58-100	---
No. 4 (4.75 mm)	40-70	24-50	45-80	35-60
No. 8 (2.36 mm)	---	---	---	15-45
No. 16 (1.18 mm)	---	10-30	25-50	10-35
No. 40 (425 μm)	0-25	---	---	---
No. 50 (300 μm)	---	---	---	3-18
No. 100 (150 μm)	---	---	6-20	---
No. 200 (75 μm)	0-10 (Gravel) 0-15 (Crushed Stone & Slag)	0-10	---	1-7

Table 15. General Requirements for Gradation of Aggregate for the Plant-Mix Base Course

Sieve designation (square openings)	Percentage by weight passing sieves A	B	C
2" (50.0 mm)	100	-	-
1-1/2" (38.1 mm)	-	100	-
1" (25.0 mm)	55-85	70-95	100
3/4" (19.0 mm)	50-80	55-85	70-100
No. 4 (4.75 mm)	40-60	40-60	40-65
No. 40 (425 μm)	10-30	10-30	15-30
No. 200 (75 μm)	5-15	5-15	5-15

Table 16. Other Typical Requirements for Aggregates

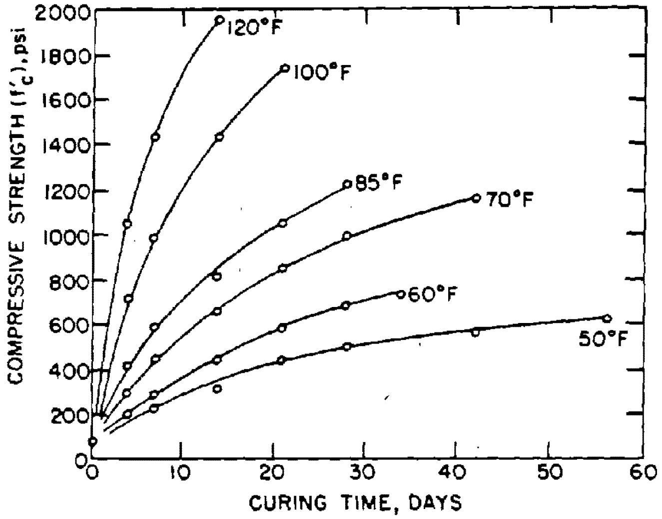
Property	Illinois	Pennsylvania	Ohio	FAA
Sodium Sulfate Soundness (AASHTO-T104)	<25%	<20%	<15%	<12%
Los Angeles Abrasion (AASHTO-T96)	<45%	<55%	---	---
Plasticity Index	<9	<6	---	<6
Liquid Limit	---	<25	---	<25

TYPICAL PROPERTIES OF LIME-FLY ASH STABILIZED SOILS

Pozzolanic reactions from which LFA mixtures derive 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 III, with details on how characteristics of the fly ash itself affect the reaction.

For an LFA mixture to develop its maximum possible strength, the ingredients must be thoroughly mixed. The time required to achieve a uniformly belnded product depends upon the type and efficiency of the available mixing equipment, mixture proportions, and, to some extent, on the ingredients themselves.

Curing conditions have a profound infludence on the properties of LFA mixtures. Both curing time and temperature greatly affect the strength and durability of "hardened" mixtures. (Figure 13).

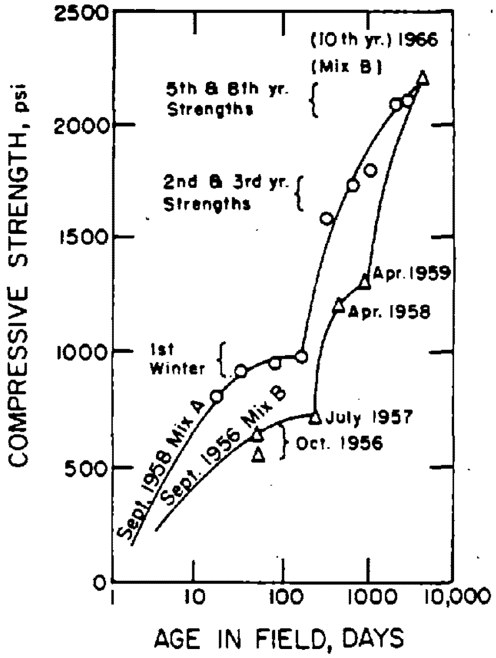


1 psi = 6.89 x 10^3 Pa °F = 5/9 (°C-32)

Figure 13. The effects of curing time on the strength of selected LCFA mixtures at varying curing temperatures.

Because of the combined effects of time and temperature on the strength development of the LFA mixtures, 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. The degree-day concept is detailed in Reference 105.

While curing at low temperatures retard the reaction process of LFA mixtures and almost entirely stop the reaction below 40 F (4 C), reduced temperatures or even freezing of the mixtures have no apparent permanent detrimental effect on the chimical properties of the constituents (26, 24). Although these materials may be subjected to a significant number of freeze-thaw cycles in the field during the winter months, increases in strength are again developed with rising temperatures during the subsequent spring and summer months (Figure 14).



1 psi = 6.89 x 10^3 Pa

Figure 14. Age-strength relationships of selected lime-fly ash-aggregate mixtures from field cores.

Under acceptable curing conditions, chemical reactions in LFA mixtures continue as long as sufficient lime and fly ash are available to react. Cores taken from pavements over a 10-year period indicate a continuing development in the strength of the mixture with time (Figure 14). This continuing reaction process can manifest itself in a phenomenon called autogenous healing, which is one of the properties of LFA mixtures (104, 106, 107) and is discussed later is this section. There are a number of recorded cases where distressed areas caused by improper loading of LFA pavements during early life have actually 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.

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 hydration of portland cement complements the slower strength development associated with some lime-ash reactions (12, 102, 108, 109).

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

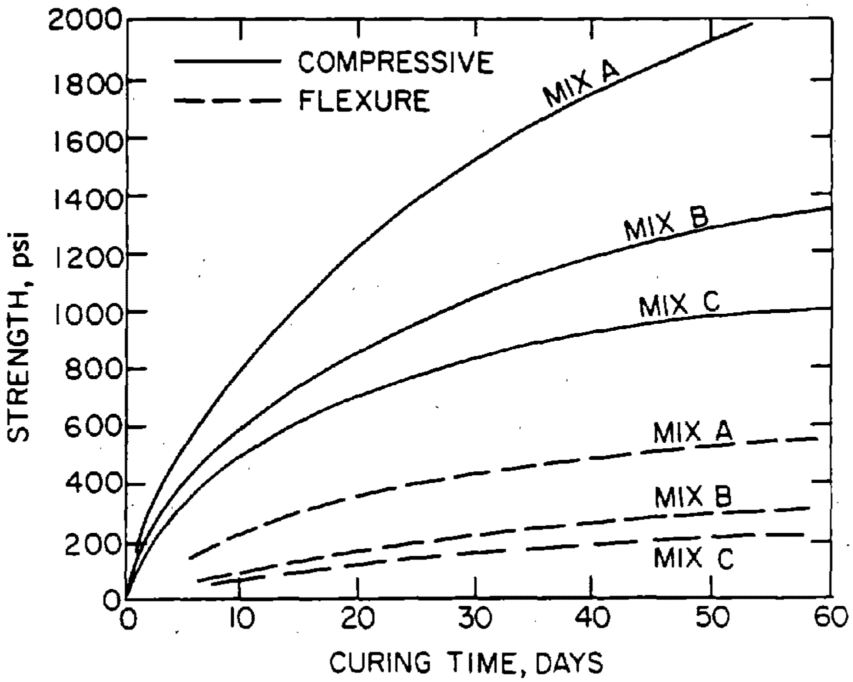
Properly designed mixtures compacted to a high relative density and properly cured may ultimately develop compressive strengths well in excess of 3,000 psi (20680 kPa). Materials cured for seven days at 100 F (38 C) normally develop compressive strengths in the range of 500 to 1,000 psi (3450-6890 kPa). These same materials are likely to develop compressive strengths in excess of 1,500 psi (10,340 kPa) after one or two years of service (Figure 14).

Flexural Strength

LFA mixtures are significantly stronger in compression than in tension. Thus, the tensile strength is a critical indicator of quality. 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 20 percent of the compressive strength is a good, conservative, engineering estimate of the flexural strength of LFA mixtures (96) (Figure 15).

Durability

Durability is a measure of a material's ability to perform in an unfavorable environment. Properly designed LFA mixtures can be produced to meet durability criteria for high quality base materials. Several methods for evaluating the durability of LFA mixtures have been developed (59, 61, 84, 96, 97, 110, 111). These include a strength loss upon vacuum saturation (84, 85, ASTM C593), a minimum strength criterion (114), and a weight loss on freeze-thaw (96, 97).

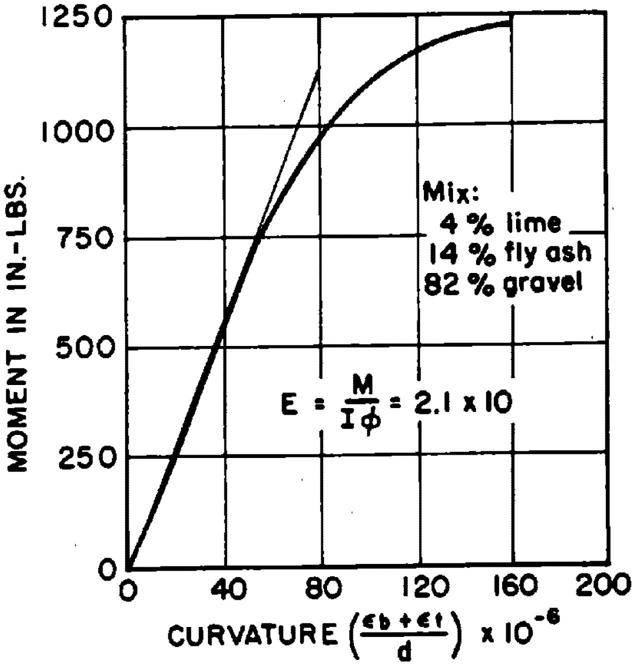


1 psi = 6.89 x 10^3 Pa

Figure 15. Relationship between flexural and compressive strength of LFA mixes cured at ambient temperature.

Bending Resistance

The stiffness of LFA mixtures is usually expressed in terms of their moduli of elasticity (E). Typical E values for LFA mixtures range from 0.5 x 10^6 to 2.5 x 10^6 psi (3.4 x 10^6 - 17.2 x 10^6 kPa). Specific values depend on whether a tangent modulus or secont modulus is used (Figure 16) as the relationship between stress and strain is non-linear.



1 in-lb = 1.13 x 10^{-1}Nm

Figure 16. Moment-Curvature Relationship for Lime-Fly Ash-Aggregate Mixture (15).

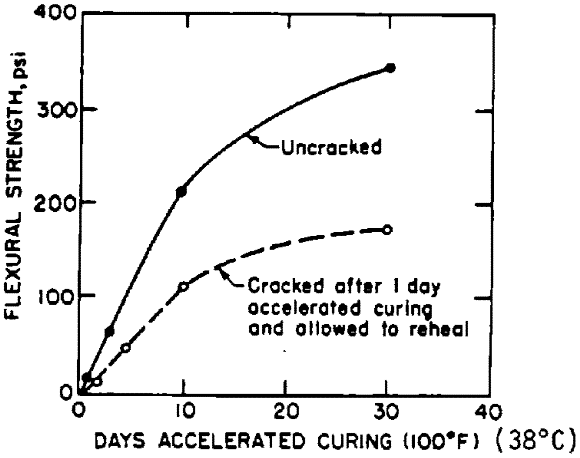
The expected range of E values for a specific LFA mixture is a function of several factors, in particular, aggregate characteristics (Particle hardness and gradation), degree of compaction, and extent and type of curing of the mixture (96, 98, 112, 115, 116).

Autogenous Healing

A beneficial characteristic of LFA mixtures is their inherent ability to heal or re-cement across cracks by a self-generating mechanism (Figure 17). This phenomenon as autogenous healing. The degree to which autogenous healing occurs is dependent upon many factors, including:

1. the age at which the mixture cracks,
2. the degree of contact of the fractured surfaces,
3. the curing conditions,
4. the availability of reaction products (lime and fly ash), and
5. moisture conditions.

Because of the autogenous healing property, LFA mixtures are less susceptible to deterioration under repeated loading and are more resistant to attacks by the elements than other materials which do not possess this property (59, 61, 84, 96, 107, 113, 114).



1 psi = 6.89 x 10^3 Pa

Figure 17. Effects of fracture and re-molding on strength of lime-fly ash-aggregate mixtures.

Fatigue

Like all paving materials, lime-fly ash-aggregate mixtures can fail under repeated loading at stress levels considerably less than the ultimate stress required to cause failure in a single load application (Figure 18). Because of autogenous healing characteristics, however, LFA mixtures are less susceptible to failure by fatigue than most other paving materials. This is due to the healing process which provides a greater curing effect than the damage being caused by the repeated loads (96, 115).

Unless fatigue failure occurs during the first few days of loading, it is not normally a factor in the performance of these pavements.

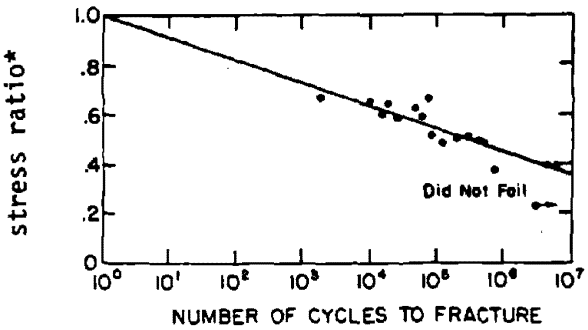


Figure 18. Relationship between maximum stress level and logarithm of number of cycles to fracture (zero minimum stress level).

* Ratio of applied repeated stress to modulus of rupture

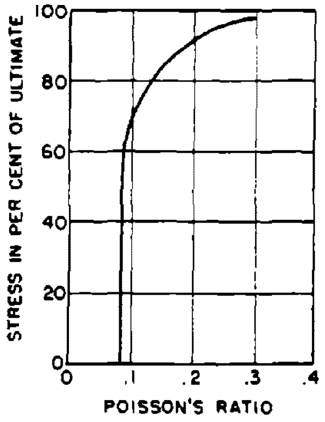


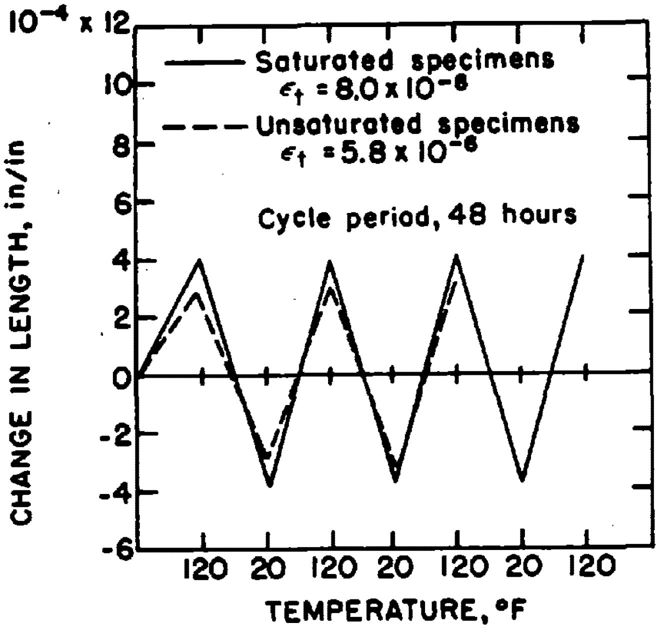
Figure 19. Effect of stress level on Poisson's ratio for a lime-fly ash-gravel mixture (96).

Poisson's Ratio

The Poisson's ratio of LFA material usually varies somewhat with the intensity of the applied stress. For most mixtures, however, 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 (96, 104) (Figure 19). For most calculations, Poisson's ratio for LFA mixtures can be taken as between 0.10 and 0.15 without appreciable error.

Coefficient of Thermal Expansion

Hardened LFA materials, like all stabilized paving materials, are subject ot 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 material (Figure 20). Typical values for the coefficient are about the same as for concrete at the same moisture content (approximately 6 X 10 x 10^{-6} inches per inch per degree Fahrenheit)) (96, 100, 104, 106).



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

Figure 20. Length change of cured LFA specimens with change in temperature.

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 vary from a low of 8 percent to a high of 36 percent (61, 86, 103, 104). Typical proportions are 2-1/2 to 4 percent lime and 10 to 15 percent fly ash. In some instances, small quantities (0.5 to 1.5 percent) of Type 1 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 Federal Aviation Administration as shown in Table 17. The Pennsylvania Department of Transportation has a durability requirement, but not a strength requirement. ASTM C 593 specifies a minimum cured compressive strength, and the vacuum saturation strength durability requirement to be incorporated into ASTM C 593 specifies a minimum vacuum saturation strength of 400 psi (2760 kPa) and replaces the maximum weight loss criteria previously specified. The Illinois Department of Transportation is also currently considering a vacuum saturation strength requirement.

Table 17. Specified Design Criteria for LFA and LCFA Mixtures

Agency	Minimum Comparessive Strength psi	Maximum Weight Loss[1] %
ASTM C 593	400[2]	-
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.

[2] Minimum compressive strength after vacuum saturation test, and no weight loss. Criteria has been approved by ASTM but had not been published at the time of this report.

1 psi = 6.89×10^3 Pa

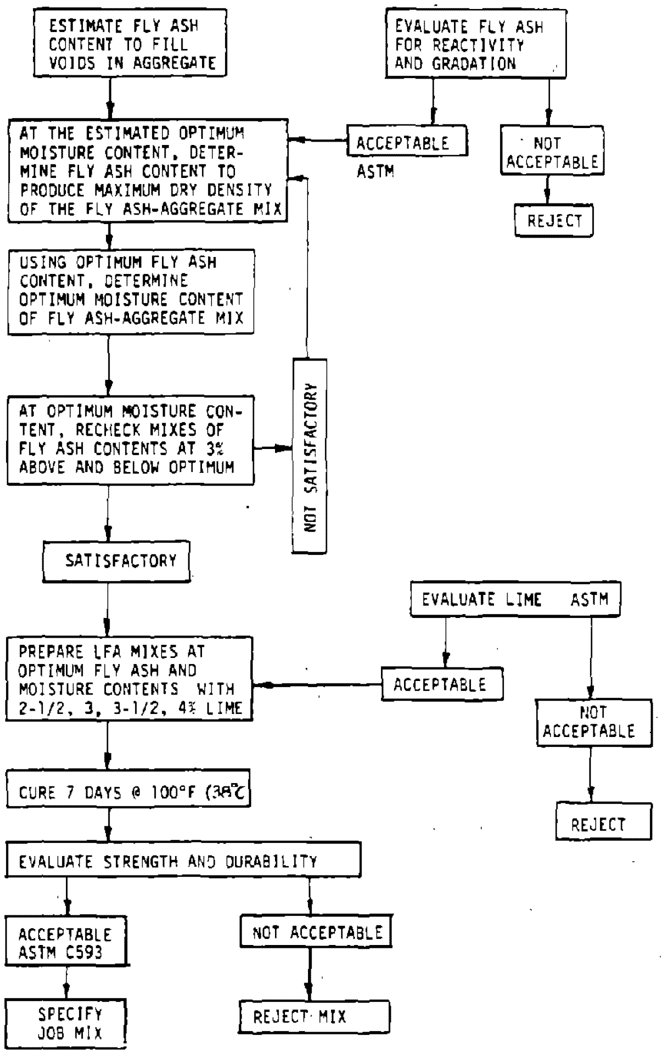


Figure 21. Mixture Design Flow Diagram.

Thompson and Dempsey (61) advocate the use of the residual strength approach for establishing freeze-thaw durability criteria. The approach emphasizes that a sliding scale of quality should be specified, depending on the field service conditions anticipated for the mixture. For example, little freeze-thaw action occurs in an LFA base course in southern Illinois, but many freeze-thaw cycles occur in a base course constructed in Chicago. In fact, it has been proposed that Illinois be divided into three separate zones for the purpose of establishing stabilized mixture 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 21. The design mixture must: (1) possess adequate strength and durability for its designated use, (2) be easily placed and compacted, and (3) 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 a 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 (97, 102).

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 number 4 seive. Only if there is sufficient matrix material to "float" the coaser aggregate fraction is it possible to achieve a high compacted density which is essential to good strength and durability of the mixture (97). 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 (97). Also, sand aggregates with single-sized particles and sands devoid of minus 200-sized particles may require high fly ash content to serve as filler or void reducer as well as a pozzolan in the mixture (97, 102, 108, 116).

Figures 22 and 23 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 (2-3 percentage points) of that required for maximum density. As indicated earlier, poorly graded materials, such as the Plainfield sand in Figure 23, require a higher lime plus fly ash content because of the volume of voids to be filled.

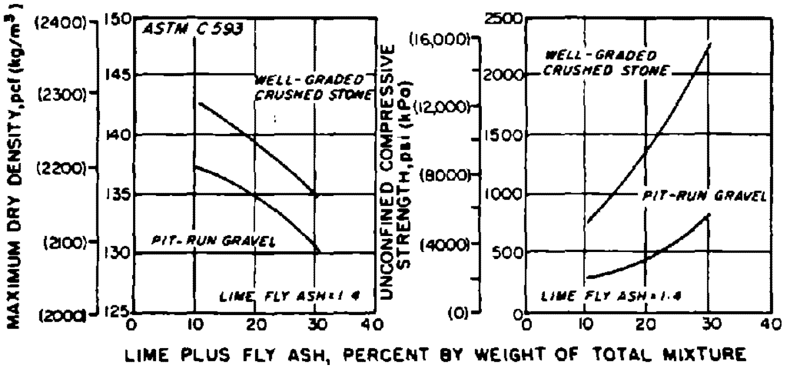
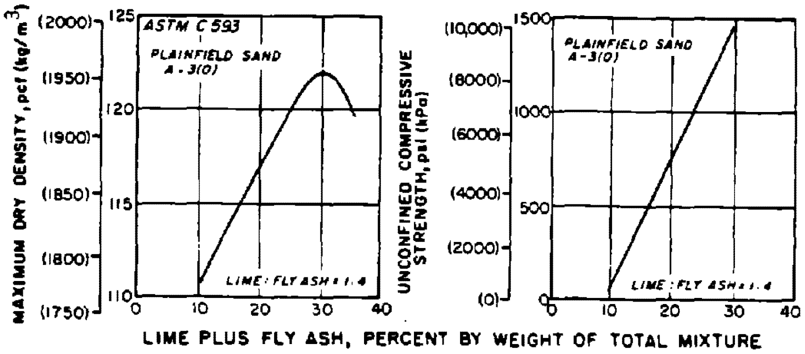


Figure 22. Variation of maximum density and unconfined compressive strength with lime plus fly ash content for coarse-grained aggregates.



1 pcf = 1.602 x 20^1 kg/mm^2 1 psi = 6.89 x 10^3 Pa

Figure 23. Variation of maximum density and unconfined compressive strength with lime plus fly ash content for fine-grained aggregate.

The proper proportions of lime to fly ash, or lime and 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 desired in the mixture. Lime to fly ash ratios of from 1:2 to 1:7 have been evaluated and found acceptable (116), but most mixtures have a ratio of about 1:3 or 1:4 for reasons of economy and quality.

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 at 100 F or 38 C), and testing for strength, durability, and rate of strength development. This latter test requires curing at various temperatures for varying time periods. Durability requirements for these materials are given in ASTM C 593, as modified in 1976.

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 1/2 percentage points (117).

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 18 have provided highly serviceable mixtures for normal construction operations.

Table 18. 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"	100	100	100	100
3/4"	90 - 100	90 - 100	100	100
1/2"	60 - 85	60 - 85	100	100
#4	50 - 75	50 - 75	90 - 100	90 - 100
#40	10 - 20	7 - 15	20 - 40	10 - 40
#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.

1 in. = 2.54 X 10^{-2} mm.

Laboratory Testing Program

Moisture-Density Relationship. Moisture-density tests are conducted in the usual manner as described in AASHTO T 180-74 with the exception of the compactive effort used. In Table 19, several of the different compactive efforts in common use are summarized. In each case, 4.0-inch (102 mm) diameter by 4.6-inch (117 mm) high, 1/30 cubic foot (9.4×10^{-4} m^3) molds are used.

Table 19. Specified Compactive Efforts for LFA and LCFA Mixtures

Agency	Procedure Designation	Compactive Effort[1]
	ASTM C 593	10/18/3/25
Illinois Department of Transportation	-	10/18/3/25
Ohio Department of Transportation	ASTM C 593	10/18/3/25
Pennsylvania Department of Transportation	PTM 106	5.5/12/3/25
Federal Aviation Administration	FAA T 611	10/18/5/25

[1] Hammer weight (lbs)/height of drop (inch)/no. of layers/blows per layer.

1 lb mm = 4.536×10^{-1} Kg

1 in. = 2.54×10^{-2} mm

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.

Compressive Strength Tests. Standard Proctor-sized specimens (4.0 inch [117 mm[high) are most commonly used to evaluate the compressive strength of cured LFA or LCFA mixtures. Aggregate particles larger than 3/4 inch (19 mm) are normally scalped from the aggregates and discarded. For fine-grained aggregate mixtures, such as those containing fine sand, 2-inch (51 mm) diameter by 4-inch (102 mm) high specimens have also been used, but there is difficulty in correlating the results from the two sizes of specimens.

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 date. The standard curing conditions for these materials are seven days and 100 F (38 C). But 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 105.

Durability Tests. Predominatly three procedures have been utilized for evaluating the freeze-thaw durability of LFA and LCFA mixtures. The freeze-thaw brushing procedure formerly included in ASTM C 593 is basically modeled after the soil-cement procedure (AASHTO T 136-70). Thompson and Dempsey (61) indicated that the temperature conditions utilized in the ASTM C 593 procedure are unrealistic and do not simulate field conditions. The "weight loss" factor determined in the ASTM procedure has no physical significance in terms of basic engineering properties (strength, stiffness, etc.)

Dempsey and Thompson (84) developed automatic freeze-thaw testing equipment which accurately simulates field conditions. 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 84.

The vacuum saturation test procedure proposed by Dempsey and Thompson (85) is a rapid technique (approximately one hour). The justification for using the vacuum saturation procedure is the excellent correlation between the compressive strengths of vacuum saturation specimens and freeze-thaw (Dempsey-Thompson technique) specimens. Details of the procedure are presented in References (85, 117). The revision of ASTM C 593 currently approved includes the use of the vacuum saturation procedure for durability evaluation purposes.

V. CEMENT STABILIZATION

INTRODUCTION

Since 1915, when a street in Sarasota, Florida, was constructed using a mixture of shells, sand, and portland cement mixed with a plow and compacted, cement treatment has become one of the most widely used forms of soil stabilization for highways. A one-and-one-half mile section of soil-cement base constructed near Johnsonville, South Carolina, is considered the first engineered soil-cement road and 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 square yards. Roads were build 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 salvage and reconstruction of old pavements and for pavements to support heavy industrial vehicles.

Many in-service, cement-stabilized soil pavements are 6 inches (150 mm) thick. This thickness is generally adequate for secondary roads and residential streets. Soil-cement thickness of 7-8 inches (175-200 mm) is widely used for primary roads and high traffic volume secondary roads. Although soil-cement pavements with thicknesses of 9 inches (225 mm) or greater have not been common in the past, considerably greater thicknesses are now being used for pavements to support heavy industrial vehicles (118).

TYPES OF PORTLAND CEMENT

Portland cement is an energy-rich anhydrous tricalcium silicate (C_3S)* with excess lime. Unhydrated cements contain a range of particle sizes, with an average particle diameter of the order of 10μm (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. Calcium silicate hydrate (CSH)*, termed tobermorite, is the predominant cementing compound in hydrated portland cement.

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

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* C = CaO, S = SiO_2, H= H_2O, A= Al_2O_3

the Type I cements as greater sulfate resistance is obtained while the cost is often the same. This greater sulfate resistance is achieved by limiting the tricalcium aluminate (C_3A)* content to 8 percent. High early strength cement (Type III has been found to give a higher strength in some soils. Type III cement has a finer particle size and a different compound composition than do the other cement types. Maximum contents of K_2O and Na_2O may be specified in any cement type to limit alkali-aggregate reactions, if necessary. Chemical physical property specifications for portland cement can be found in ASTM C150.

SOIL-CEMENT REACTIONS

Cement stabilization resembles lime stabilization in many ways, except with cement, pozzolanic material is present in the cement initially and need not be derived from the soil itself. In predominately 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 (CSH).

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 initally may have a beneficial effect.

SOILS SUITABLE FOR CEMENT STABILIZATION

A wide range of soil types may be stabilized using portland cement. The greatest effectiveness and economy in highway construction in comparison to other stabilizers, however, is with sands, sandy and silty soils, and clayey soils of low to medium plasticity. If the plasticity index exceeds about 30 percent, cement becomes difficult to mix with the soil. If cement stabilization is to be used for highly plastic soils, then lime may be added first to reduce the plasticity index and improve workability prior to addition of cement.

A soil may be acid, neutral, or alkaline and still respond well to cement treatment. Although certain types of orgainic matter, such as undecomposed vegetation, may not influence stabilization adversely, organic compounds of lower molecular weight, such as necleic 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)

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

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 (119).

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 (120, 121) it is known that the resistance to sulfate attack differs for cement-treated coarse-grained and fine-grained soils and is a fundtion of sulfate concentrations. Sulfate-clay reactions can cause deterioration of fine-grained soil-cement(121). On the other hand, granular soil-cements do not appear susceptible to sulfate attack and, in fact, in some cases the presence of small amounts of sulfate in the soil at the time of mixing with the cement may even be beneficial. The use of sulfate-resistant cement 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.

TYPICAL PROPERTIES OF CEMENT-STABILIZED SOILS

For many applications soil-cements and cement-treated soils can be divided into two groups: granular and fine-grained. Granular soil-cements are made using the coarser-grained cohesionless soil types, i.e., A-1, A-2, and A-3 soils according to the AASHTO classification system and the (G-) and (S-) soils according to the Unified Soil Classification System. Fine-grained soil-cements are made using cohesive soils, i.e., AASHTO class A-4, A-5, A-6, and A-7 soils, corresponding to the (C-) and (M-) soils in the Unified system.

The properties of cement-treated soils 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 quantitative 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. The unconfined compressive strength is such a property.

In general, for a given cement content, the higher the density the higher the strength of cohesionless soil and cement mixtures. Both water content at compaction and compaction method may be important in cohesive soil and cement mixtures (122).

Compaction Characteristics. Cement addition to a soil generally causes some change in both the optimum water content and maximum dry density for a given compactive effort. Often 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 may also be important because it influences the packing of particles.

A delay between mixing and compaction leads to a decrease in both density and strength for a fixed compactive effort (123). 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 a few tens to more than 2000 psi (1 to more than 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 24 shows that a limear relationship can be used to approximate the relationship 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 25. The relationship between strength and curing time for a given soil and cement mixture can be given by:

$$(UC)_d = (UC)_{d_0} + K \log (3/d_0)$$

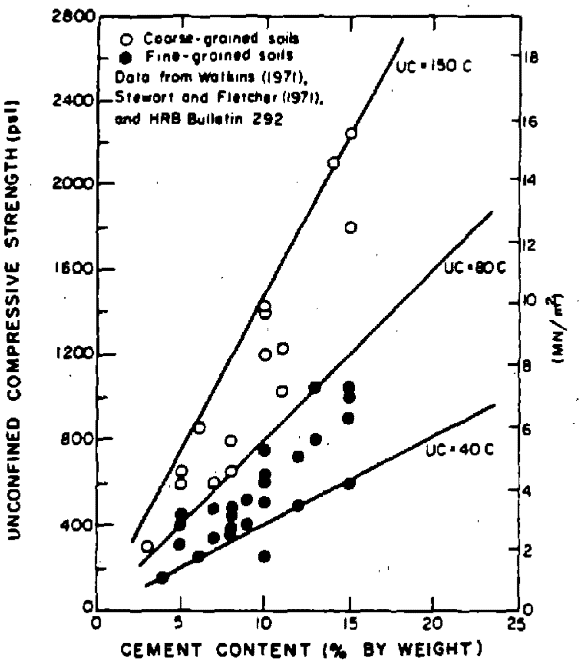
Where $(UC)_d$ = unconfined compressive strength at an age of d days, in psi

$(UC)_{d_0}$ = unconfined compressive strength at an age of d_0 days, in psi

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

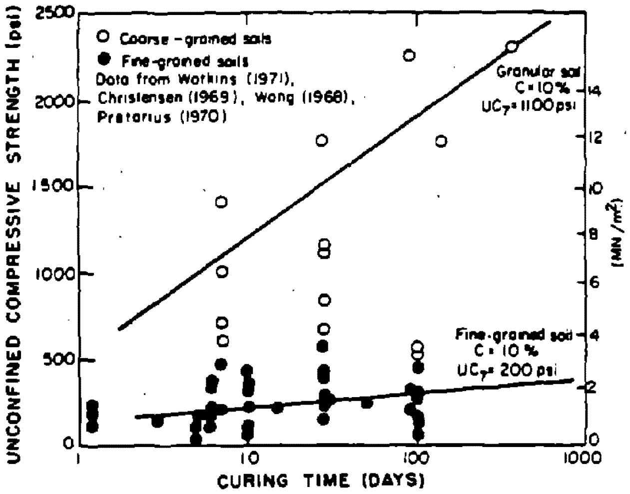
C = cement content, in percent by weight.

The 28-day strength was found to be 1.7 times the 7-day strength by Dunlap et al. (119), and 1.4 time the 7-day strength is suggested by Williams (124). A value of 1.5 times the 7-day strength would seem a reasonable value for estimating purposes.



1 psi = 6.89×10^3 Pa = 6.89×10^{-3} MN/mm^2

Figure 24. Relation between cement content and unconfined compressive strength for soil and cement mixtures (Equations give strength in psi).



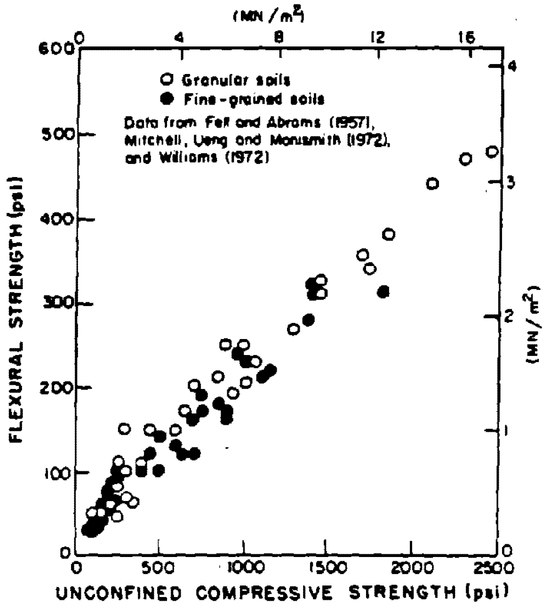
1 psi = 6.89×10^3 Pa = 6.89×10^{-3} MN/mm^2

Figure 25. The effect of curing time on the unconfined compressive strength of some soil and cement mixtures

Tensile Strength. Flexural beam tests, direct tension tests, and the split tension (Brazilian) test have all been used to evaluate the tensile strengh. The results of several studies have indicated that the flexural strength is about 1/5 to 1/3 of the unconfined compressive strength. Data for some soils are shown in Figure 26. In low-strength mixtures, the flexural strength is a greater proportion of the compressive strength (up to 1/3) than in high-strength mixtures (down to less than 1/5). A good approximation for the flexural strength f is:

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

Where UC is the unconfined compressive strength.



$1\ \text{psi} = 6.89 \times 10^{3}\ \text{Pa} = 6.89 \times 10^{-3}\ \text{MN/mm}^{2}$

Figure 26. The relation between unconfined compresive strength and flexural strength of soil and cement mixtures.

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. (125) 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 f_t is given by:

$$f_t = \frac{2P}{\pi DL}$$

where P = load at failure

d = cylinder diameter

L = cylinder length

Griffith crack theory (126, 127) has been found useful for characterizing the strength of cement-treated soils uncer various combinations of major (σ_1) and minor (σ_3) principal stresses (128). Normalized strength data (failure stresses divided by the unconfined compressive strength) for several soils are summarized in Figure 27. With this figure and a knowledge of the unconfined compressive strength, principal stress combinations causing failure can be estimated directly.

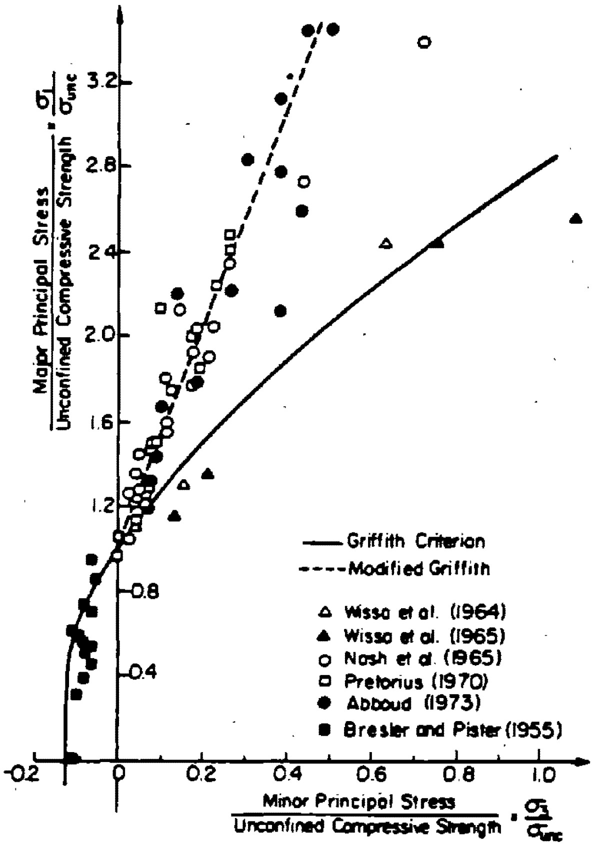
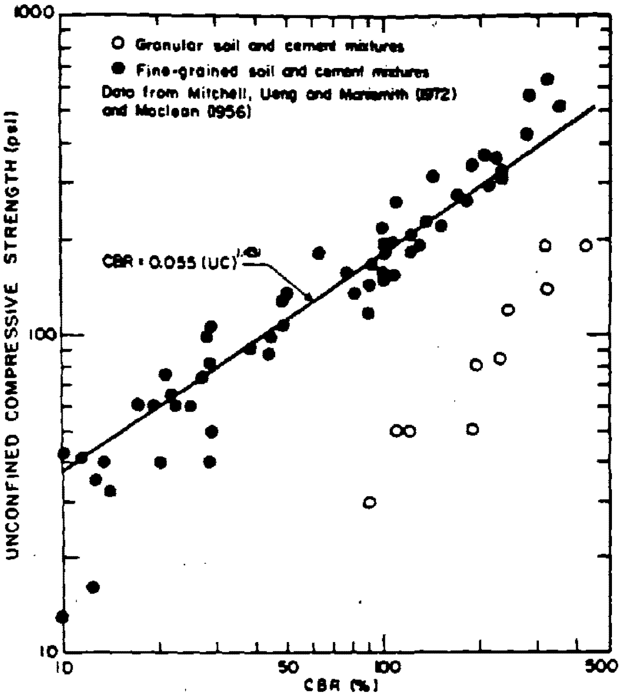


Figure 27. Failure envelope for cement-treated soils (132)

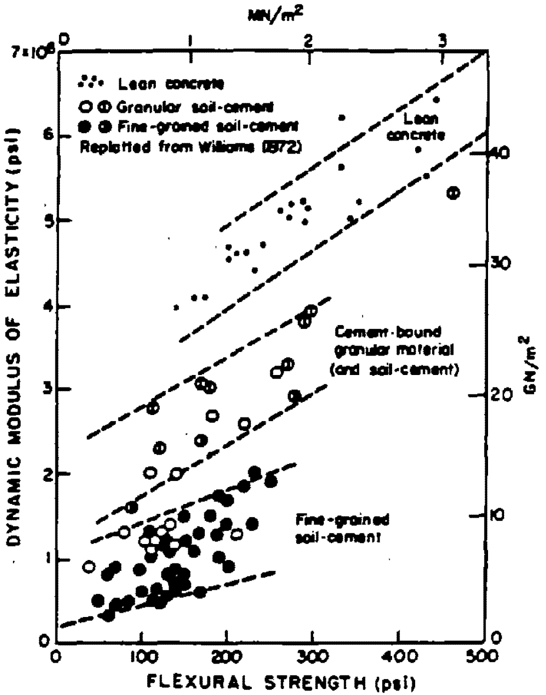
California Bearing Ratio (CBR). The relationship between unconfined compressive strength and CBR for some granular and fine-grained soil and cement mixtures is shown in Figure 28. The difference between the relationships for fine-grained and granular-treated soils probably results from the uncertainly 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 28 can be interpreted as a strength index only.



1 psi = 6.89 x 10^3 Pa

Figure 28. The relation between CBR and the unconfined compressive strength of soil and cement mixtures.

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 as 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 29. 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 (124).



1 psi = 6.89 x 10^3 Pa = 6.89 x 10^{-6} GN/mm^2

Figure 29. Relationship between flexural strength and dynamic modulus of eleasticity for different cement-treated materials

Figure 30 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 (129).

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

$_{RC}$ can be extressed by:

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

where UC = unconfined compressive strength (psi)

$(\sigma_1 - \sigma_3)$ = deviator stress in psi

σ_3 = confining pressure in psi

K_c = material constant

k_1 = 0.2 to 0.6

k_2 = 0.25 to 0.7

$n = 1.0 + 0.18\ C$

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 $(\sigma_1 - \sigma_3)$.

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

$$M_{RF} = K_F(10)^m \cdot UC$$

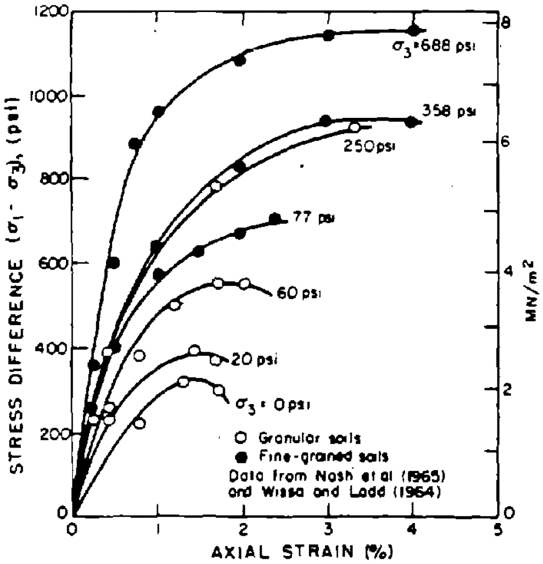
where K_F = material constant

UC = unconfined compressive strength in psi

$m = 0.04(10)^{-.186C}$

C = cement content in percent by weight

<u>Poisson's Ratio</u>. 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.



$$1 \text{ psi} = 6.89 \times 10^{3} \text{ Pa} = 6.89 \times 10^{-3} \text{ MN/mm}^{2}$$

Figure 30. Typical stress-strain behavior for soil and cement mixtures.

Fatigue Behavior. Cement-treated soils are susceptible to fatigue failure after repeated applications of stresses greater than some limiting value. 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:

1. Fatigue life is shorter under repeated direct tensile stresses than in compression.

2. Flexural fatigue is unlikely for repeated stress levels less than 50 percen of the flexural strength.

3. The flexural fatigue of soil-cement can be related to radius of curvature (130, 131) according to:

$$R_c/R = aN^{-b}$$

where R_c = critical radius of curvature, i.e. the radius of curvature causing failure under static loading

R = radius of curvature leading to failure under N load applications

$$a = \frac{h^{3/2}}{2.1h - 1}$$

h = slab thickness

b = 0.025 for granular soil-cements and 0.050 for fine-grained soil cements

N = number of load applications

4. 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. (132). A stress factor F is defined as:

$$F = \frac{(\sigma_1 - \sigma_3)^2}{8(\sigma_1 + \sigma_3)} \quad \text{for } \sigma_1 + 3\sigma_3 > 0$$

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

for compression positive.

Repeated tensile stresses cause a progressive decrease in tensile strength from its initial value T.. 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 31. The two curves shown pertain to different times after treatement.

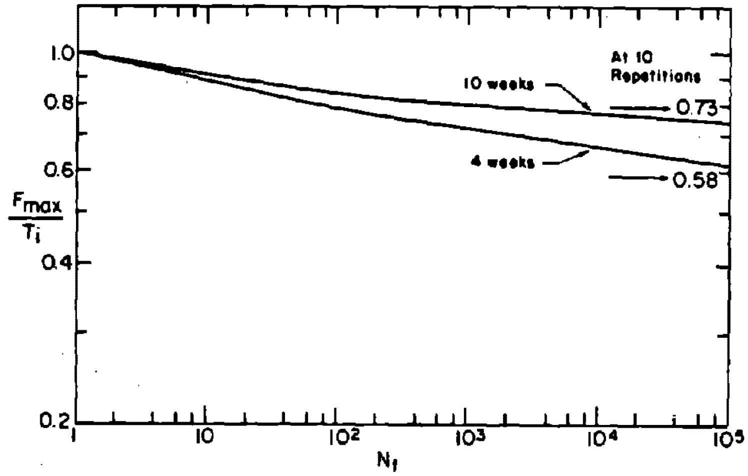


Figure 31. Suggested fatigue failure criteria for cement-treated soils (132).

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 the cracks to be from 1/8 to 1/4 inch (3 to 6 mm) wide 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 consequent to loss of support

Summary. Table 20 provides a general summary of the properties of cement-stabilized soils. The numerical values indicated are typical for usual conditions. Final design values in any case should be based, whenever possible, on carefully conducted tests in which the anitcipated field conditions are simulated as closely as possible.

SELECTION OF CEMENT CONTENT

Approximate Quantities

Table 21 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, and mild exposure conditions, satisfactory stabilization may be achieved using lower cement contents.

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 32 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.

Table 21 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 (15) and by the following ASTM Test Standards, which are approved also as American National Standards by the American National

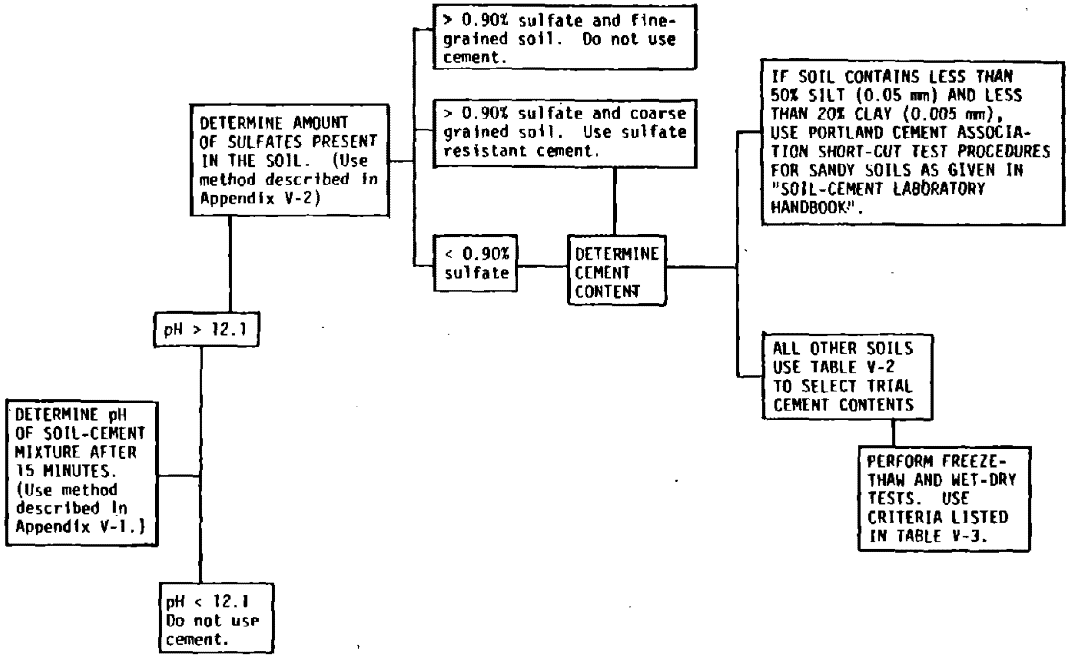


Figure 32. Subsystem for nonexpedient base course stabilization with cement (adapted from reference 119).

Table 20. Summary of the Properties of Cement-Stabilized Soil.

(UC = unconfined compressive strength; C = cement content, percent by weight)

Property	Granular Soils	Fine-Grained Soils	Notes
Density	1.6 - 2.2 t/m^2	1.4 - 2.0 t/m^3	May be higher or lower than untreated soil. Delay between mixing and compaction causes density reduction
Unconfined Compressive Strength	UC = (90 to 150) C UC = (0.5 to 1.0) C $(UC)_d = (UC)_{d_o} + K \log (d/d_o0)$ K = 70 C (psi) K = 0.5 C (MN/m^2)	UC = (40 to 80) C UC = (0.3 to 0.6) C K = 10 C (psi) K = 0.7 C (MN/m^2)	UC in psi UC in MN/m^2 d = age (days) ($d > d_o$) $(UC)_{d_o}$ = UC strength at age of d_o days
Cohesion	To a few hundred psi c = 7.0 + 0.225 (UC) psi To a few MN/m^2 c = 0.05 + 0.225 (UC MN/m^2)	To a few hundred psi To a few MN/m^2	Depends on C, d
Friction Angle	45-45°	30-40°	May decrease at high confining pressures
Flexural and Tensile Strength	Tensile Strength = $(\frac{1}{5}$ to $\frac{1}{3})$ compressive strength		Need 1-3% cement to develop
Strength under Combined Stress States	$(\sigma_1 - \sigma_3)^2 = UC(\sigma_1+\sigma_3)$ for $\sigma_3/UC < 0.1$ $\sigma_1 = UC + 5\sigma_c$ for $\sigma_3/UC > 0.1$ (compression positives)		Relationships developed using Griffith crack theory
CBR	$CBR = 0.55 (UC)^{1.431}$		UC in psi
Modulus-Compression	$1 \times 10^6 - 5 \times 10^6$ psi 7 - 35 GN/m^2 $E_t = [1 - \frac{0.75(1 - \sin\phi) (\sigma_1 - \sigma_3)}{2 c \cos\phi + 2\sigma_3 \sin\phi}]^2 E_i$ $E_i = Kp_a(\frac{\sigma_3}{p_a})^n$	$10^5 - 10^6$ psi 0.7 - 7 GN/m^2	Depends on stress level E_i = initial tangent modules E_t = tangent modulus σ_3 = confining pressure p_a = atmospheric pressure n = 0.1 - 0.5 K = 1000 - 10,000
Modulus - Tension and Flexure	Same order of magnitude as in compression		$E_c > E_t$ (usually)
Resilient Modulus - Compression	$M_{RC} = K_c(\sigma_1 - \sigma_3)^{-k_1}(\sigma_3)^{k_2}(UC)^n$		k_1 = 0.2 to 0.6 k_2 = 0.25 to 0.7 n = 1.0 + 0.18 C
Resilient Modulus - Flexure	$M_{RF} = K_f(10)^{m \cdot UC}$		$m = 0.04(10)^{-.186C}$ Effect of confining pressure not known
Fatigue Behavior	No fatigue for $F/T_i < 0.50$ T_i = initial tensile strength $F = \frac{(\sigma_1 - \sigma_3)^2}{8(\sigma_1 + \sigma_3)}$ for $\sigma_1 + 3\sigma_3 > 0$ $F = - \sigma_3$ $\sigma_1 + 3\sigma_3 < 0$		
Poisson's Ratio	0.1 - 0.2	0.15 - 0.35	
Shrinkage	A few tenths of 1%	Up to 1%	Shrinkage cracks generally inevitable
Thermal Properties			
(a) Conductivity	k = 0.6 k = 1.0	k = 0.3 k = 0.55	BTU - ft/hr · ft^2 · °F w/m · °K
(b) Heat Capacity	C = 0.82 C = 8.40		BTU/lb · °F J/kg · °K
(c) Thermal Expansion	$\varepsilon = 5 \times 10^{-6}$ $\varepsilon = 9 \times 10^{-6}$		$°F^{-1}$ $°C^{-1}$

1 psi = 6.89×10^3 PA = 6.89×10^{-3} MN/mm^2

Table 21. Cement Requirements for Various Soils (133)

AASHO Soil Classification	Unified Soil Classification*	Usual Range in cement requirement**		Estimated cement content and that used in moisture-density test, percent by weight	Cement contents for wet-dry and freeze-thaw tests, percent by weight
		percent by vol.	percent by wt.		
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

**for most A horizon soils the cement should be increased 4 percentage points, if the soil is dark grey to grey, and 6 percentage points if the soil is black

Standards Institute:

ASTM D558-57 (Reapproved 1976): Test for Moisture-Density Relations of Soil-Cement Mistures.

ASTM D559-57 (Reapproved 1976): Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures.

ASTM D560-57 (Reapproved 1976): Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures.

ASTM D1632-63 (Reapproved 1974): Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory.

ASTM D1633-63 (Reapproved 1974): Test for Compression Strength of Molded Soil-Cement Cyliners.

ASTM D2901-70 (Reapproved 1975): Test for Cement Content of Freshly Mixed Soil-Cement.

Criteria for satisfactory performance of soil-cement in the durability tests are listed in Table 22. 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 a 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 1-12 inches [25 to 40 mm]) asphaltic concrete surfacing 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).

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.

Table 22. Criteria for Soil-Cement as Indicated by Wet-Dry and Freeze-Thaw Durability Tests

AASHTO Soil Group	Unified Soil Group	Max. 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

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

Additional Criteria:

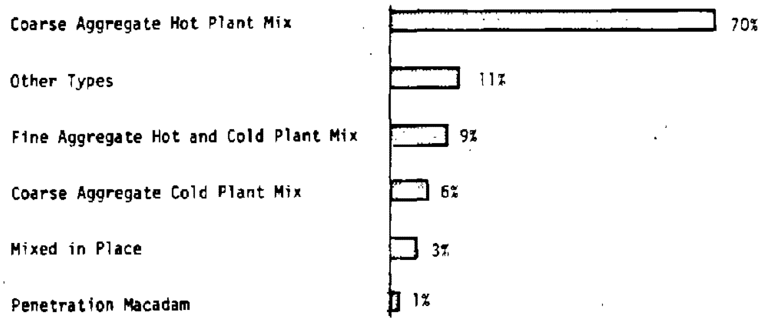
1. Maximum volume changes during durability test should be less than 2 percent of the initial volume.
2. Maximum water content during the test should be less than the quantity required to saturate the sample at the time of molding.
3. Compressive strength should increase with age of specimen.

VI. ASPHALT STABILIZATION

INTRODUCTION

Asphalt is one of the oldest adhesives known to man. As early as 3800 B.C. asphalt was being used as mortor for building stones and paving blocks. The use of asphalt for streets in the United States began in 1870 with the laying of a stretch of pavement in Newark, New Jersey. Soil and sand-asphalt stabilization projects were places in Florida, Oklahoma and South Carolina in 1930. Since 1930 a significant portion of low traffic volume roads have utilized mixed-in-place asphalt stabilization. In the last 20 years hot, central plant operations have been the major type of asphalt stabilization used by state agencies (Figure 33) (134). Some of the reasons for utilizing asphalt stabilization are listed below:

1. Waterproofing fine-grained subgrade soils,
2. Construction expediency,
3. Upgrading of marginal materials,
4. Reduction of pavement layer thickness to conserve materials, reduce cost and conserve energy,
5. Provide temporary and permanent wearing surfaces and
6. Reduce dusting.



COMBINED TOTAL ALL TYPES = 36,796,496 TONS

1 ton = 9.07×10^2 Kg

Figure 33. Bituminous Bound Base Courses, Practice in United States, All States Reporting.

Alaska only state not using this type construction.

TYPES OF ASPHALT

Asphalts most commonly used are refined from petroleum. Asphalt cement is the basic refined material and is the hard, high-molecular, weight fraction of crude oil. Asphalt cement at ambient temperatures is a semi-solid. Liquid asphalt products are most often derived from asphalt cement by blending petroleum distillates to form cutbacks or by emulsifying with water to form emulsified asphalts.

Asphalt Cements

Asphalt cements are graded on the basis of consistency or viscosity. Three different techniques are utilized to grade asphalts on this basis: penetration at 77 F (25 C) of original asphalt, viscosity at 140 F (60 C) of original asphalt, and viscosity at 140 F of laboratory-aged asphalt. Specifications have been developed by AASHTO, ASTM, and a West Coast User Producer group (Table 23). 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.

Asphalt cements must be heated to obtain a mixing and spraying consistency. Asphalt cements are normally used in central plants with heated aggretates; however, soft asphalt cements have been mixed in-place and some hard asphalts have been used in foaming operations in-place. The curing or setting time of mixtures utilizing asphalt cements occurs as the heat required for mixing, laydown, and compaction dissipates.

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, and
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, on RC-70 is a Rapid Curing cutback with a viscosity at 140 F (60 C) between 70 and 140 centistokes.

It is usually desirable to heat cutbacks to aid distribution and mixing. Partial curing is usually necessary after mixing and prior to compaction. Most cutbacks are used for in-place operations.

Table 23. Asphalt Specifications

Material		Specification	
		AASHTO	ASTM
Asphalt Cement	Penetration basis	M20	D946
	Viscosity basis	M226	D3381
Cutback	Rapid curing	M81	D2028
	Medium curing	M82	D2027
	Slow curing	M141	D2026
Emulsion	Anionic	M140	D977
	Cationic	M208	D2397

Emulsified Asphalts

Emulisified asphalts are mixtures of asphalt cement, water, and an emulsifying agent. Anionic emulsions are manufactured with anionic (negatively charged) emulsifying chemicals. Cationic emulsions are manufactured with cationic (positively charged) emulsifying chemicals. The type and amount of emulsifying agent will determine to a large degree the setting characteristics of the asphalt emulsion. Rapid-setting (RS), medium-setting (MS), and slow-setting (SS) anionic and cationic emulsions are manufactured. 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. Characteristics of the asphalt cement utilized to manufacture the emulsion and viscosity of the asphalt emulsion are utilized to define the grade. For example, a major difference between the CRS-1 and CRS-2 (cationic rapid-setting emulsions) is the viscosity of the emulsion, while the major difference between CMS-2 and CMS-sh (cationic medium-setting emulsions) is the penetration of the base asphalt cement. It should be noted that a wide number of asphalt suppliers use company terminology to describe emulsions.

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. Selections of the type of asphalt for a given stabilization use is discussed later. In general asphalt cements are listed 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.

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 a waterproofing phenomenon. 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. In addition, asphalt stabilization can improve durability characteristics. Since the soil particles or aggregates are coated with water-repelling asphalt film, the soil is resistant to the detrimental effects of water such as volume change due to alternating wet-dry and 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, elastic modulus, etc., in the presence of water.

The second mechanism has been identified as adhesion. The aggregate particles adhere to the asphalt and thus the asphalt acts as a binder or cement. The cementing effect increases shear strength by increasing cohesion. 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 stregth, 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, resulting in a strengh loss of the subgrade materials. In surface course applications, the asphalt binder has the capability of eliminating or reducing the occurrence of raveling, rutting, washboarding, loss of fines, etc., under traffic.

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 percent 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 are shown on Table 24 (23). As noted on this table, the amount of material passing the No. 200 sieve should be less than 25 percent. In addition, the plastic index 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.

Table 24. Engineering Properties of Materials Suitable for Bituminous Stabilization (23).

% Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel Bitumen
1-12"			100
1"	100		
3/4"			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 & 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.

1 in. - 2.54 X 10^{-2} mm.

Coarse Grained Soils

Cohesionless soils (plasticity index less than 6) suitable for asphalt stabilization are shown on Table 24 and identified as sand-bitumen and sand-gravel-bitumen. In addition, 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, etc., than the more one-sized sand-asphalt mixtures.

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: (1) stability, (2) durability, (3) fatigue behavior, (4) tensile behavior, (5) stiffness, (6) flexibility, and (7) workability. Few tests have been developed to indicate the flexibility and workability of bituminous stabilized materials. Elongation and certain tensile tests are attempts to measure flexibility, while gradation limits and compaction tests have been used to control workability.

Stability, durability, fatigue behavior, tensile properties, and stiffness 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: (1) type of asphalt, (2) type and gradation of the aggregate, (3) density of the compacted mixture, and (4) curing and/or aging conditions.

Stability

Specifications and criteria for bituminous-stabilized soils and aggregates 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.

A recent survey of state practices published by the Highway Research Board (134) indicates that the most widely used stability tests are the Hveem, Marshall, and unconfined compression test. Other tests used for stability type determinations include Hubbard-Field, triaxial compression, repeated load triaxial, and various penetration type tests including the California Bearing Ratio, the Iowa Bearing Value, and the Florida Bearing Value.

Typical criteria and hence typical values for Hveem, Marshall, and unconfined compressive strength are shown in Table 25. Methods of sample preparation, test temperatures, and curing conditions prior to testing vary from state to state. Most of the criteria presently utilized were originally developed for surface courses and adapted to base course design.

Table 25. Design Methods and Criteria for Asphalt-Stabilized Base Courses (134)

A. Hveem Method

STATE	STABILITY	PERCENT AIR VOIDS	PERCENT VOIDS FILLED WITH ASLPHALT	COHESIOMETER
California	35 Minimum	4-6	80-85	300 minimum
Colorado	30-45	3-5	75	
Hawaii	35 Minimum	5-10		
Nevada	30-37 min.	3-5		
Oklahoma	35 Minimum	8 Maximum		
Oregon	30 Minimum	10 Maximum		150 minimum
Texas	30 Minimum			
Washington	20 Minimum			50 minimum

B. Marshall Method

STATE	STABILITY, LBS.	FLOW, 0.01 INCH	PERCENT AIR VOIDS	PERCENT VOIDS FILLED WITH ASPHALT
District of Columbia	750 Minimum	8-16	3-8	65-75
Georgia	1800 Minimum	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 Maximum	5-7	50-70
New Jersey	1100-1500	6-18	3-7	
N. Carolina	800	7-14	3-8	
N. Dakota	400 Minimum	8-18	3-5	
Pennsylvania	700 Minimum	6-16		60-85
Rhode Island	750 Minimum		3-8	
S. Carolina	1200-3000	6-12		
S. Dakota		8-18	3-5	
Wyoming	1000 Minimum			

C. Unconfined Compressive Strength

STATE	LOAD, PSI	PERCENT AIR VOIDS	PERCENT VOIDS FILLED WITH ASPHALT
Colorado	200-400	3-5	80-85
Oregon	150 Minimum		

1 in = 2.54 X 10^{-2} mm; 1 lb_f = 4.448 N; 1 psi = 6.89 X 10^3 Pa

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 retained strength (70 percent) or a minimum stability after soaking.

Freeze-thaw and wet-dry durability-type tests for asphalt-stabilized mixtures are nearly non-existent. The water saturation test coupled with freezing and thawing developed by Lottman is an exception (135).

Fatigue Behavior

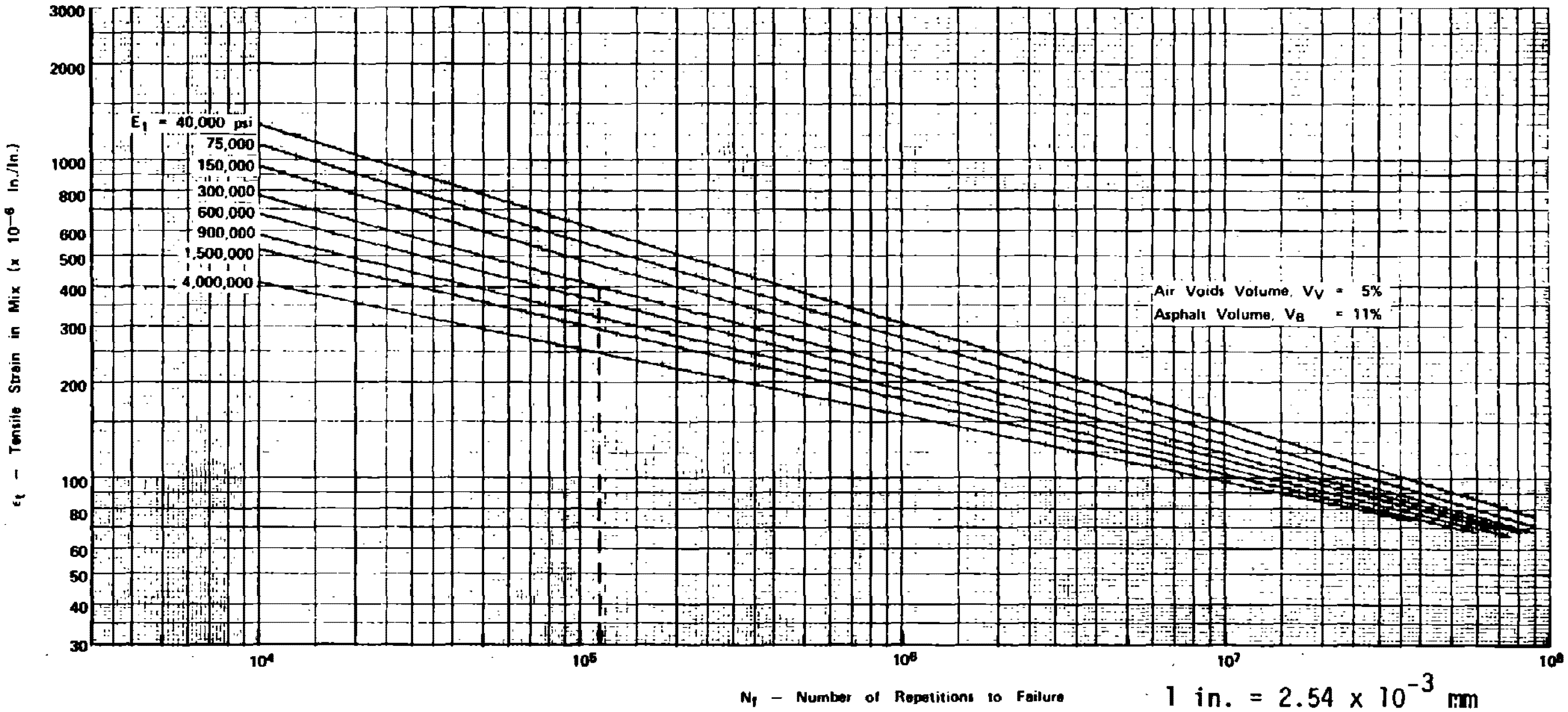
The fatigue behavior of bituminous-stabilized materials has been reviewed by Witczak (131), Epps and Monismith (137), Pell (138), and Santucci (139). These reviews indicate the relative importance of asphalt type, aggregate gradation, aggregate type, air void content, and other mixture variables. The criteria offered by Santucci for asphalt concrete emulsion mixtures, and cement-modified emulsion mixtures appear to be typical of asphalt-stabilized materials (Figures 34 and 35).

Tensile Properties

A wide variety of tensile tests, including direct tension, indirect tension, dumbbell, and "dornprobe," have been performed on asphalt-stabilized mixtures. The most popular test at present appears to be the indirect tension or splitting tensile test. This test has been utilized by the University of Texas, University of California, University of Alberta, Texas A & M University, University of Idaho, Virginia Highway Research Council, 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, and temperature. Typical values obtained under conditions simulating highway loadings are on the order of 100 to 800 psi (690 to 5,500 kPa).

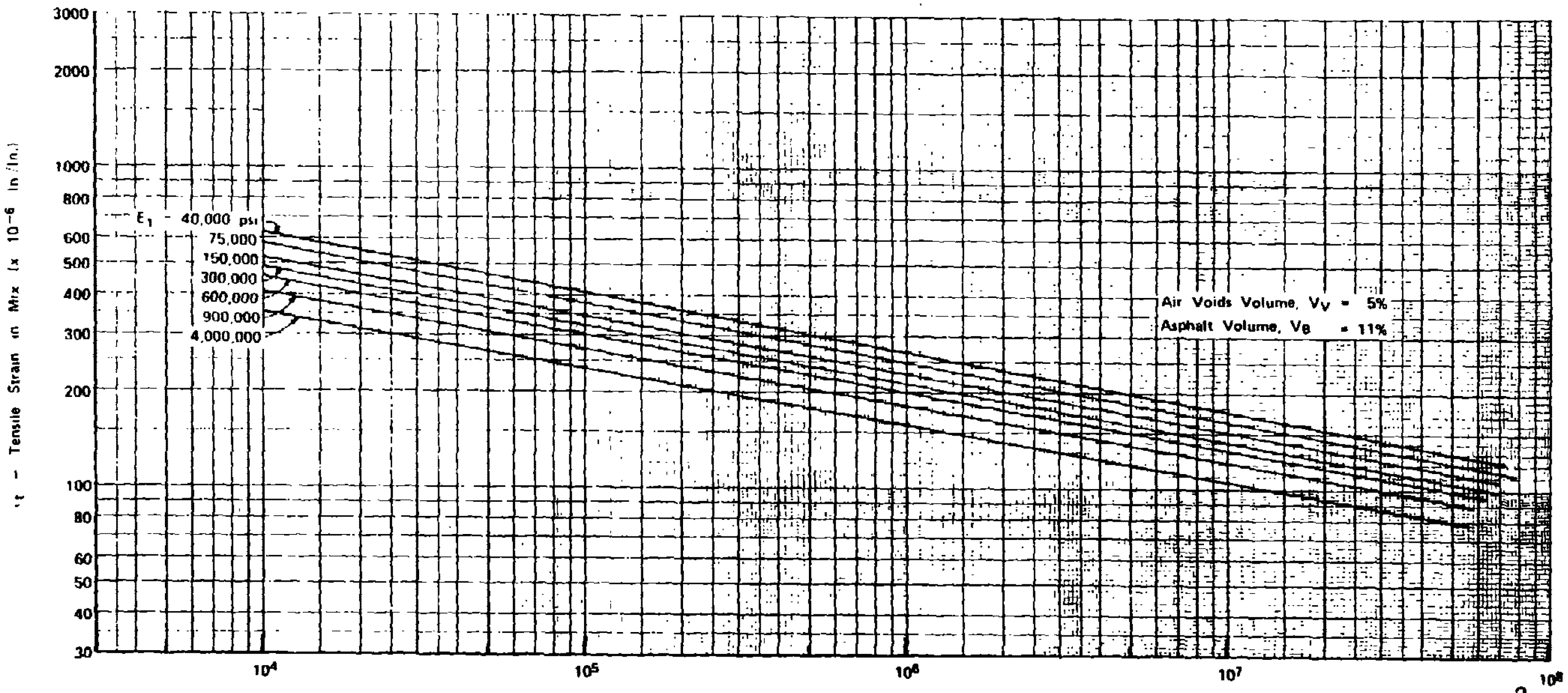
Stiffness

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. Figure 36 indicates the wide range of this property as a function of temperature and time for an asphalt-stabilized aggregate. Values typical of highway loading conditions are on the order of 200,000 to 800,000 psi (1,380,000 to 5,510,000 kPa). Figure 37 indicates ranges in resilient modulus for a wide range of asphalt-stabilized materials tested at 0.1 sec. loading and at 73 F (23 C) (140). Resilient modulus is defined as the ratio of the applied repeated stress to the recoverable strain and closely approximates stiffness.



1 in. = 2.54 x 10^{-3} mm

Figure 34. Fatigue criteria for asphalt and emulsion mixes (139).



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

Figure 35. Fatigue criteria for cement-modified asphalt emulsion mixes (139).

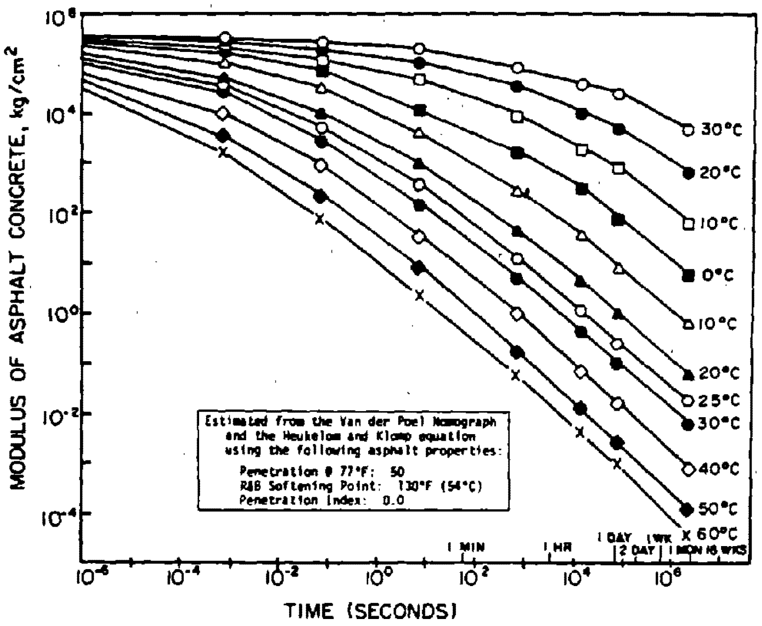
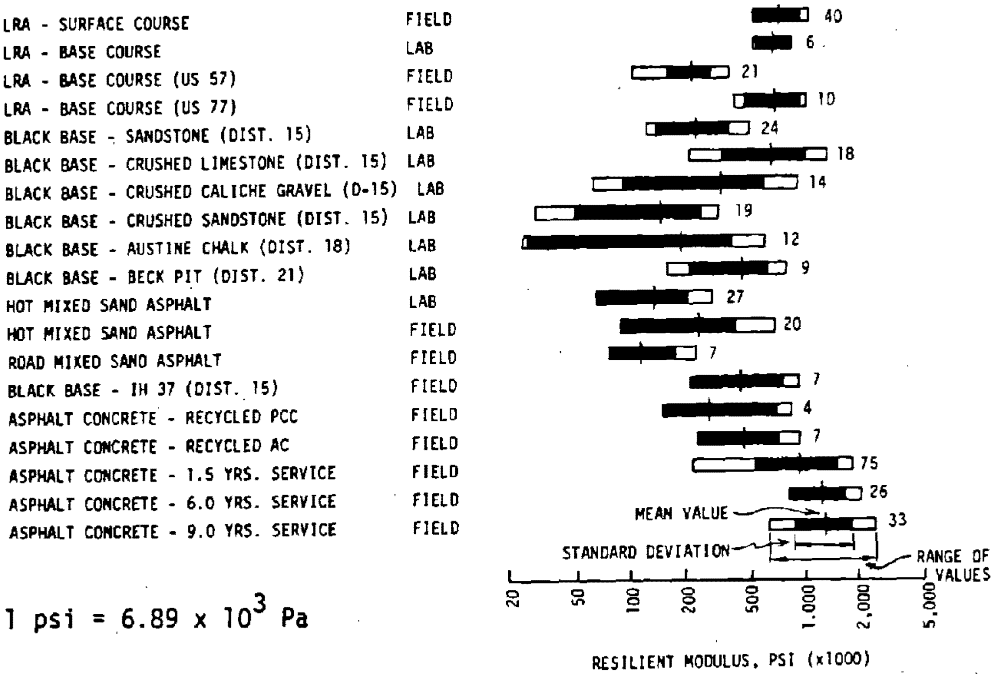


Figure 36. Asphalt concrete modulus as a function of Temperature and time of loading



1 psi = 6.89 x 10^3 Pa

Figure 37. Ranges of resilient moduli at 73 F (23 C) for various types of asphalt aggregate mixture (140).

Values of stiffness can be obtained from a number of testing techniques including repeated load tests. Perhaps the most common apparatus presently utilized is that developed by Schmidt (141) and used extensively by Chevron, U.S.A. as well as a number of other agencies.

Summary

Presently-used mixture design procedures are based primarily on the use of stability and curability tests. Fatigue behavior, tensile properties, and stiffness parameters are utilized for pavement structural design purposes but are not commonly utilized for establishing binder contents.

SELECTION OF ASPHALT TYPE AND ASPHALT CONTENT

Selection of the type and amount of asphalt for a particular use is influenced by several considerations. Some of the major factors are discussed below.

Method of Construction. The basic types of construction include central plant (both hot and cold operations) 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. Some warm central plant operations have utilized emulsions.

Construction Equipment. Central plants are typically batch or continuous in operation. The drum-mixer continuous plants are the most popular plants presently marketed. The continuous plants utilizing pugmills for mixing are often used for cold mixing operations. In-place equipment has various degrees 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. Examples of suitable types of construction can be found in Volume 1, Chapter 4.

Pavement Layer. Asphalt-stabilized materials used as surface courses, base courses, or subbases may require different types and quantities of asphalts. Asphalt cements are popular binders for surface courses while emulsions and mixed-in-place operations are utilized extensively for subbase and base course construction.

Loading and Environmental 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) should input to the selection of the type and grade of asphalt.

Aggregates. The gradation, surface texture, absorption, and soundness of the aggregate will to some extent control the selection of the asphalt grade.

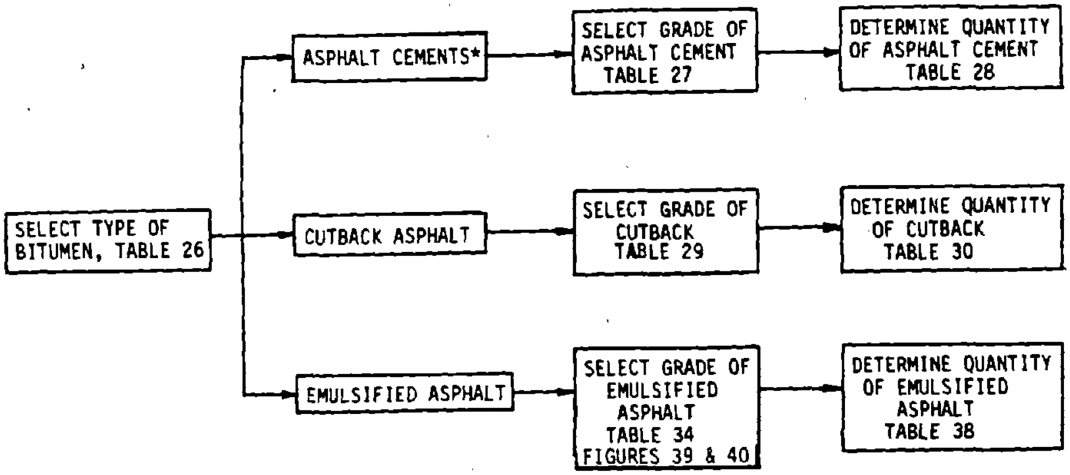
The method of construction and the equipment available will determine in general the type of asphalt (asphalt cement, cutback, or emulsion). The grade selected, including its viscosity and setting or curing characteristics, will be influenced by the gradation and the amount of fine particles in the aggregate, the climatic conditions during and after construction, the type of mixing equipment, and, to a degree, the magnitude of loads expected on the pavement. In general, asphalt cements will normally be used with hot central plant operations, and emulsions will be used with mixed-in-place operations and some cold or warm central plant operations. The use of cutbacks is discouraged due to problems with air quality, safety, and the alternate use of cutter stocks for more important purposes.

Approximate Asphalt Quantities

References 11 and 145 and their associated Tables 26 - 31 and Figures 38 - 40 can be used as guides for selecting the type of asphalt, the grade of asphalt, and an approximate quantity of asphalt. It should be realized that these tables and figures result in suggested types, grades, and quantities. The exact quantity and perhaps the grade should be based on a laboratory testing program as described below. Estimates of asphalt quantities can be obtained by performing the test and calculations associated with the California CKE procedure as outlines in Reference 142.

Table 26. Selection of a Suitable Type of Bitumen for Soil Stabilization Purposes

Sand Bitumen	Soil Bitumen	Crushed Stones and Sand-Gravel Bitumen
Hot Mix: Tables 27, 28, 31		Hot Mix: Tables 27, 28, 31
Cold Mix: Cutbacks See Tables 29, 30	Cold Mix: Cutbacks See Tables 29, 30	Cold Mix: Cutbacks See Tables 29, 30
Emulsions See Table 34 See Figures 29 and 40 to determine if a cationic or anionic emulsion should be used	Emulsions See Table 34 See Figures 39 and 40 to determine if a cationic or anionic emulsion should be used	Emulsions See Table 34 See Figure 40 to determine if a cationic or anionic emulsion should be used



*Hard asphalt cements are preferred in hot climates.

Figure 38. Subsystem for Expedient Base Course Stabilization with Bituminous Materials.
Selection of type, grade, and quantity asphalt for stabilization purposes.

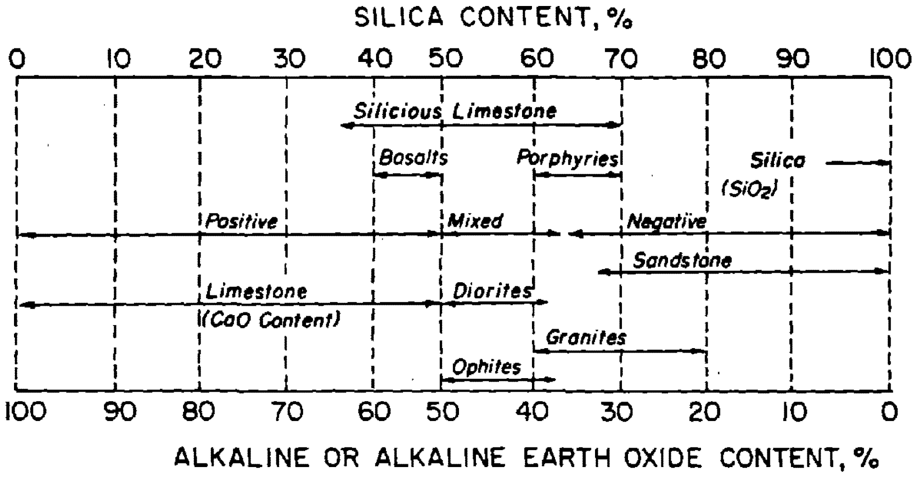


Figure 39. Classification of Aggregates (151)

Table 27. Recommendations for Selection of Paving Asphalt (142)

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."

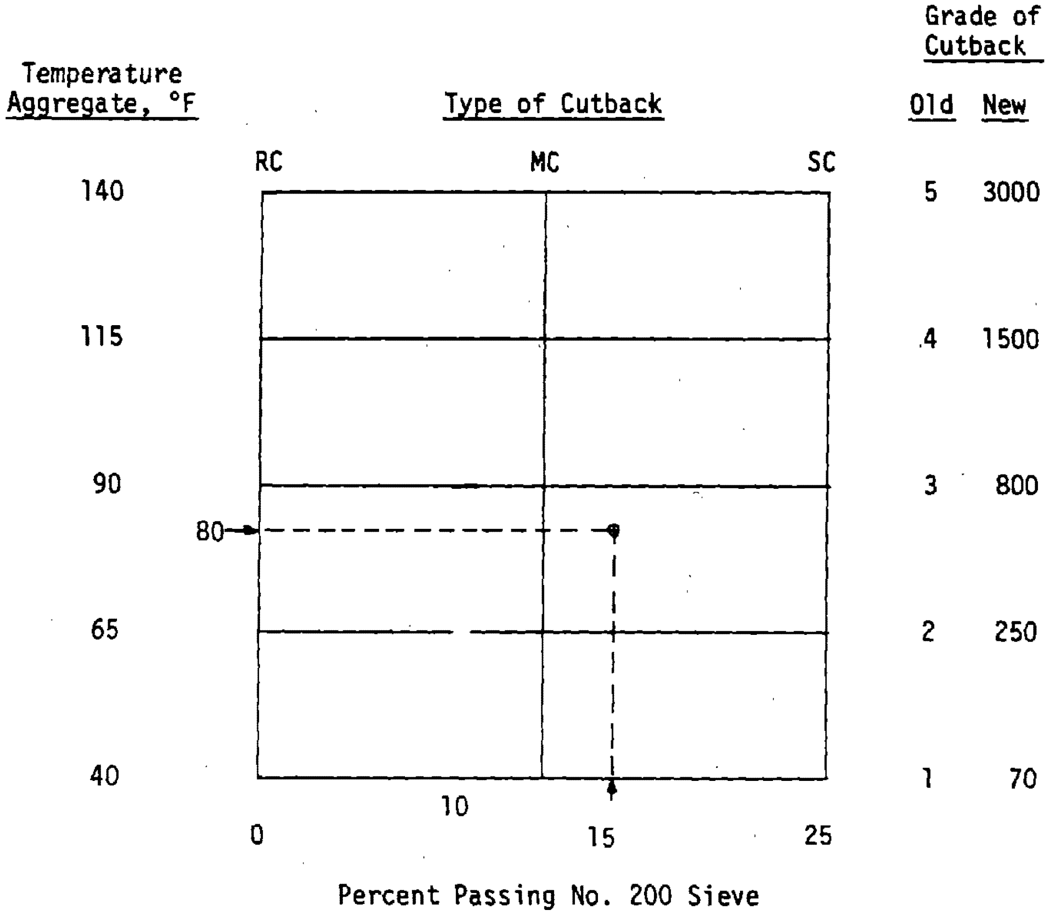
*As per U.S. Weather Bureau climatological reports.

Table 28. 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 judgment.

Table 29. Selection of Type of Cutback for Stabilization (28)



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

Table 30. Determination of Quantity of Cutback Asphalt (146)

$$p = 0.02\,(a) + 0.07\,(b) + 0.15\,(c) + 0.20\,(d)$$

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 30 and utilizing the following equation:

$$\text{percent cutback} = \frac{\text{percent residual asphalt } (p)}{(100 - \text{percent solvent})} \times 100$$

Table 31. Cessation Requirements* (150)

	RECOMMENDED MINIMUM LAYDOWN TEMPERATURE					
BASE TEMP.	1/2"	3/4"	1"	1-1/2"	2"	3" AND GREATER
20-32	-	-	-	-	-	285[1]
+32-40	-	-	-	305	295	280
+40-50	-	-	310	300	285	275
+50-60	-	310	300	295	280	270
+60-70	310	300	290	285	275	265
+70-80	300	290	285	280	270	265
+80-90	290	280	275	270	265	260
+90	280	275	270	265	260	255
Rolling time, min.	4	6	8	12	15	15

[1] Increase by 15° when placement is on base or subbase containing frozen moisture.

*Laydown temperatures below which asphalt paving operation should be ceased.

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

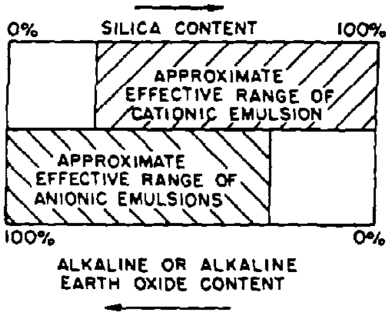
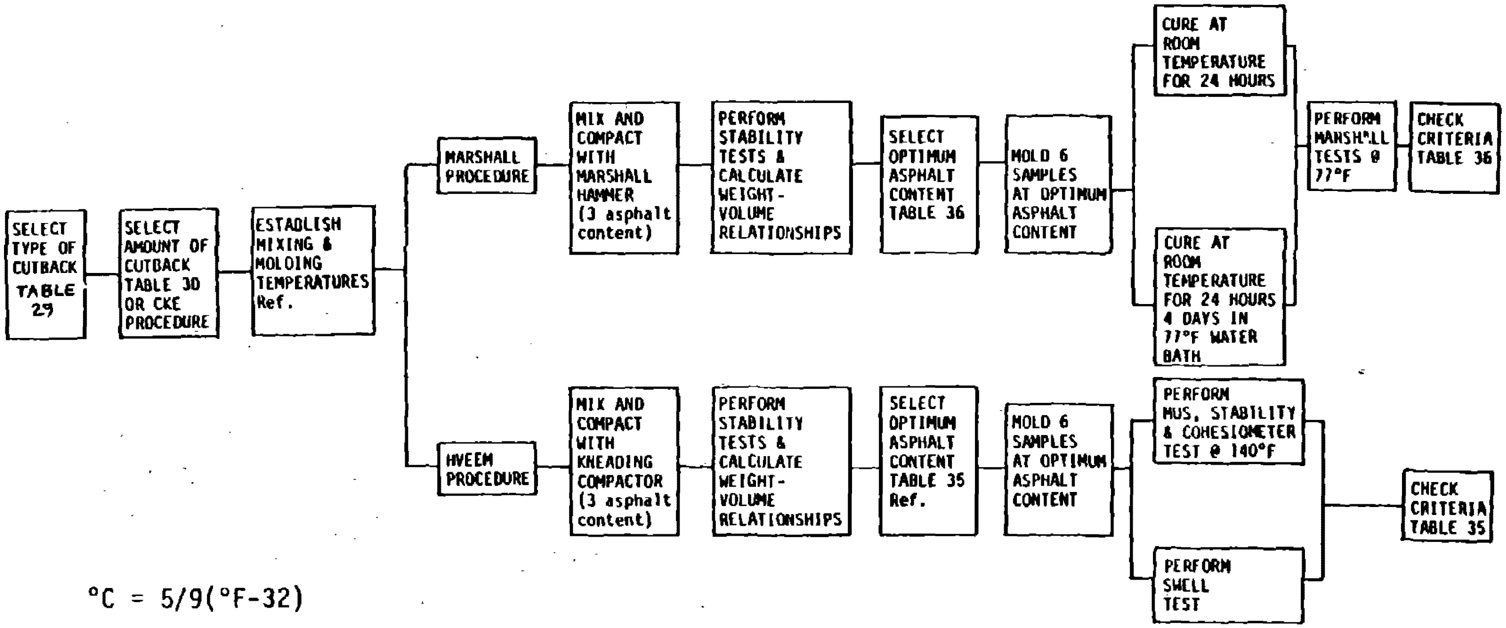


Figure 40. Approximate Effective Range of Cationic and Anionic Emulsions on Various Types of Aggregates (151).

Detailed Testing

Testing techniques commonly utilized to select asphalt content are the Hveem and Marshall Stability tests. ASTM, AASHTO, the Asphalt Institute (143), Chevron Asphalt (144), and the U.S. Air Force (145) have standardized procedures and established mixture-design acceptance criteria. Hveen mixture-design criteria exist for asphalt cement, cutback, and emulsion-stabilized materials. Marshall criteria presently exist for asphalt cement and cutback-stabilized systems. The University of Illinois and Purdue University are presently developing procedures for emulsion-stabilized materials based on Marshall testing (149). Methods of mixture design are detailed below.

Asphalt Cement. Material stabilized with asphalt cement may employ the standard ASTM and AASHTO, 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 are shown on Table 25. 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 on Tables 32 and 33 for Hveem and Marshall test procedures respectively. A standard curing procedure is not required as part of this design method.



°C = 5/9(°F-32)

Figure 41. Cutback asphalt mixture design methods (146)

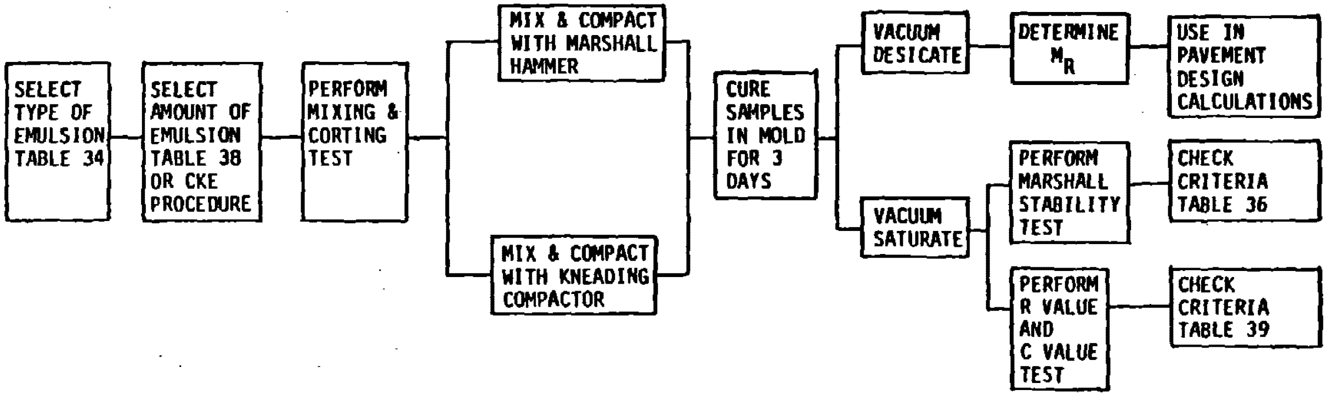


Figure 42. Emulsified Asphalt Mixture Design Method.

Cutback Asphalts. Design procedure for mixtures stabilized with cutbacks have been standardized by the Asphalt Institute (146). A summary outline of the design methods for both Hveem and Marshall test procedures are shown on Figure 41. Details of the mixture fabrication, curing, and testing procedures can be found in reference 146. Design criteria are shown on Tables 35, 36, and 37. 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 Asphalts. Design procedures for mixtures stabilized with emulsions have been standardized by Chevron, U.S.A., for Hveem testing procedures. Emulsion mixture-design methods based on Marshall test methods are under development at the University of Illinois (147), Purdue University (148), and ARMAC. Unfortunately, the developed methods have not been standardized by ASTM or AASHTO. A summay outline of suggested design methods for both the Hveem and Marshall test procedures are shown on Figure 42. The suggested Hveem procedure is based on procedures advanced by Chevron, U.S.A. (144), while suggested Marshall mixture-design procedures are based on Chevron mixing and curing procedures but Marshall compaction and Marshall testing procedures. Design criteria are those advanced by Chevron for the Hveem procedures (Table 39) and those advanced by the Asphalt Institute (for cutback asphalts) for use with Marshall testing techniques (Table 36). Details of the mixture fabrication, curing, and testing techniques can be found in references 142 and 149. The critical elements of the suggested procedure are the moisture content during mixing and 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. In addition, the Resistance value and Cohesiometer Value are obtained rather than the "S" value in the Hveem method.

Example Problem

The use of Tables 26 to 39 and Figures 38 to 42 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 and the following results were obtained:

Percent passing 3/8 inch 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

Table 32. Hveem Design Criteria (143)

TEST CATEGORY	Heavy		Medium		Light	
TEST PROPERTY	min.	max.	min.	max.	min.	max.
STABILOMETER VALUE	37	-	35	-	30	-
SWELL	less than 0.030 inch (0.762 mm)					

NOTES:

1. Although not a routine part of this design method, an effort is made to provide a minimum percent air voids of approximately 4 percent.

2. All criteria, and not stability value alone, must be considered in designing an asphalt paving mix.

3. 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 33. Marshall Design Criteria (143)

TRAFFIC CATEGORY NO. OF COMPACTION BLOWS EACH END OF SPECIMEN	HEAVY 75		MEDIUM 50		LIGHT 35	
Test Property	Min.	Max.	Min.	Max.	Min.	Max.
Stability, all mixtures, lb	750	-	500	-	500	-
(N)	(3,336)		(2,224)		(2,224)	
Flow, all mixtures, 0.01 in. (0.25 mm)	8	16	8	18	8	20
Percent air voids						
Surfacing or Leveling	3	5	3	5	3	5
Base	3	8	3	8	3	8
Percent voids in mineral aggregate			See Table below			

NOTES:

1. Laboratory compactive efforts should closely approach the maximum density obtained in the pavement under traffic.
2. The flow value refers to the point where the load begins to decrease.
3. The portion of the asphalt cement lost by absorption into the aggregate particles must be allowed for when calculating percent air voids.
4. Percent voids in the mineral aggregate is not to be calculated on the bases of the ASTM bulk specific gravity for the aggregate.
5. All criteria, and not stability value alone, must be considered in deisgning an asphalt paving mix.
6. 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 (37.8°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.

$1\ \mathrm{lb}_f = 4.448\ \mathrm{N}$

$1\ \mathrm{in} = 2.54 \times 10^{-2}\mathrm{m}$

Table 34. Selection of Emulsified Type (144)

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-2S)	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 39 and 40 can be used as a basis for selecting anionic or cationic emulsions. The geologic type of aggregate is located on Figure 39 and the approximate silica or alkaline earth oxide content determined. These contents are utilized to enter Figure 40 to select the type of emulsion.

Table 35. Suggested Criteria for Cutback Asphalt Mixes (146)

TEST	REQUIREMENT
Stabilometer value	30 min.
Moisture Vapor Susceptibility (Stabilometer value)	20 min.
Swell	0.030 in. max. (0.76 mm) max.

Table 36. Marshall Design Criteria for Paving Mixtures Containing Cutback Asphalt (146)

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)	(See Table 37)	
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 37. Minimum Percent Voids in Mineral Aggregate (146)

USA STANDARD SIEVE NO.	NOMINAL MAXIMUM PARTICLE SIZE, INCHES	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

Example: The minimum allowable voids in the mineral aggregate (VMA) for a 3/4 inch maximum size aggregate gradation is 14 percent.

1 in. = 2.54×10^{-2}m

Table 38. Emulsified Asphalt Requirement (28)

PERCENT PASSING NO. 200	LBS OF EMULSIFIED ASPHALT PER 100 LBS OF DRY AGGREGATE 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	7.5	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	7.4

50 or less

Example: 7.9 percent emulsified asphalt (by dry weight of aggregate) is required for an aggregate with 82 percent passing the No. 10 sieve and 15 percent passing the No. 200 sieve.

1 bm = 4.536 x 10^{-1}Kg

Table 39. Emulsion Mixture Design Criteria
Based on Hveem Stabilometers (142)

CRITERIA	TEST REQUIREMENTS
Resistance R_t - Value After Vacuum Soak - For light and medium traffic, DTN under 100 (*)	70 Min.
For heavy and very heavy traffic, DTN over 100 (*)	78 Min.
Moisture Pick-up percent by Vacuum Soak Procedure	5.0 Max.

(*)See Thickness Design Manual, MS-1, The Asphalt Institute

Table 40. Tentative Lime-Soil Moisture Compresive Strength Requirements

ANTICIPATED USE	STRENGTH REQUIREMENTS FOR VARIOUS ANTICIPATED SERVICE CONDITIONS (b)				
	RESIDUAL STRENGTH REQ., PSI (a)	EXTENDED (8-DAY) SOAKING (PSI)	CYCLIC FREEZE-THAW (e)		
			3 CYCLES (PSI)	7 CYCLES (PSI)	10 CYCLES (PSI)
Modified Subgrade	20	50	50	90 50*	120
Subbase					
Rigid Pavement	20	50	50	90 50*	120
Flexible Pavement					
Thickness of Cover (c)					
10 inches	30	60	60	100 60*	130
8 inches	40	70	70	110 75*	140
5 inches	60	90	90	130 100*	160
Base	100 (d)	130	130	170 150*	200

a) Minimum anticipated strength following first winter exposure.
b) Strength required at termination of field curing (following construction) to provide adequate residual strength.
c) Total pavement thickness overlying the subbase. The requirements are based on the Boussinesq stress distribution. Rigid pavement requirements apply if cemented materials are used as base courses.
d) Flexural strength should be considered in thickness design.
e) Number of freeze-thaw cycles expected in the lime-soil layer during the first winter of service.

*Note: Freeze-thaw strength losses based on 10 psi-cycle except for 7-cycle values indicated by an * which were based on a previously established regression equation.

1 psi = 6.89 x 10^3Pa 1 in. = 2.54 x 10^{-2}m

Material smaller than 0.05 mm (silt and clay combined) = 12
Material smaller than 0.002 mm (clay) = 9
Liquid limit = 20
Plastic index = 9

Table 24 indicates that from a gradation and plasticity standpoint the soil will make a fair soil-asphalt subbase. Table 26 indicates that the soil-asphalt can be stabilized with either a cutback or emulsion. A hot mix operation is discouraged for a soil of this nature.

Cutback Stabilization. Figure 38 indicates that Table 29 can be used to select the type of cutback and Table 30 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 29 indicates that this soil can best be stabilized with a MC-800. The equation contained in Table 30 will allow for the engineer to *estimate* the quantity of cutback. Plotting the results of the soil gradation on aggregate grading charts indicated the following:

$a = 50$, $b = 25$, $c = 10$ and $d = 15$

Thus the percent residual asphalt by weight of dry aggregate = 7.25. 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 on Figure 41 and at asphalt contents of 7.5, 8.5 and 9.5. Results of these tests should be compared with appropriate criteria presented on Tables 35, 36 and 37.

Emulsion Stabilization. Figure 38 indicates that Figures 39 and 40 and Table 34 should be used to select the type and grade of emulsion. Figures 38 and 39 suggest that a cationic emulsion will be best suited for this siliceous soil. Table 34 indicates that a CSS-1 or a CSS-1h emulsions are preferred. A CSS-1h will be selected due to the summer pavement temperatures.

Table 38 estimates that 7.9 percent emulsion will be required for this soil. Marshall or Hveem stability tests should be performed as shown on Figure 42 at emulsion contents of 7, 8 and 9 percent. Results of these tests should be compared with appropriate criteria presented on Table 39.

VII. COMBINATION STABILIZERS

INTRODUCTION

The use of combination stabilizers has not received widespread application in the United States. Most agencies prefer to utilize one stabilizer and avoid the handling and construction requirements of a two-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 or asphalt cannot be mixed successfully 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.

There can be various types of combination stabilizers. However, only four combination stabilizers are considered in this manual. These are:

1. Lime-cement
2. Lime-asphalt
3. Lime-emulsified asphalt
4. Cement-emulsified asphalt

COMBINATION STABILIZER REACTIONS

Lime-Cement Combinations

Combinations of lime and cement often are acceptable expedient stabilizers. Lime can be added to the soil to increase the workability and mixing characteristics of the soil as well as reduce its plasticity. Cement can then be mixed with the soil to provide rapid strength gain.

Details of lime and lime reactions have been covered in Chapter III. 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 50+ percent. Lime also reacts with some silts but normally will not react sandy soils.

Details of cement and cement reactions are discussed in Chapter V. While cement cannot be used alone for heavy clays or highly plastic soils, lime can be first used to initiate cation exchange and flocculation-agglomeration reactions and to produce immediate changes by reducing the plasticity and improving the workability of these soils. Addition of cement then ensures rapid strength development of the mixture. This is especially advantageous when rapid strength gain is required under cooler weather conditions.

Lime-Asphalt Combinations

All asphalt paving materials that are currently being produced may be mixed with some type 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 the effect of moisture may have a significant influence on performance. It is known that the presence of moisture decreases the stiffness or modulus of asphalt mixtures and that this influence is more marked with increased temperature (152).

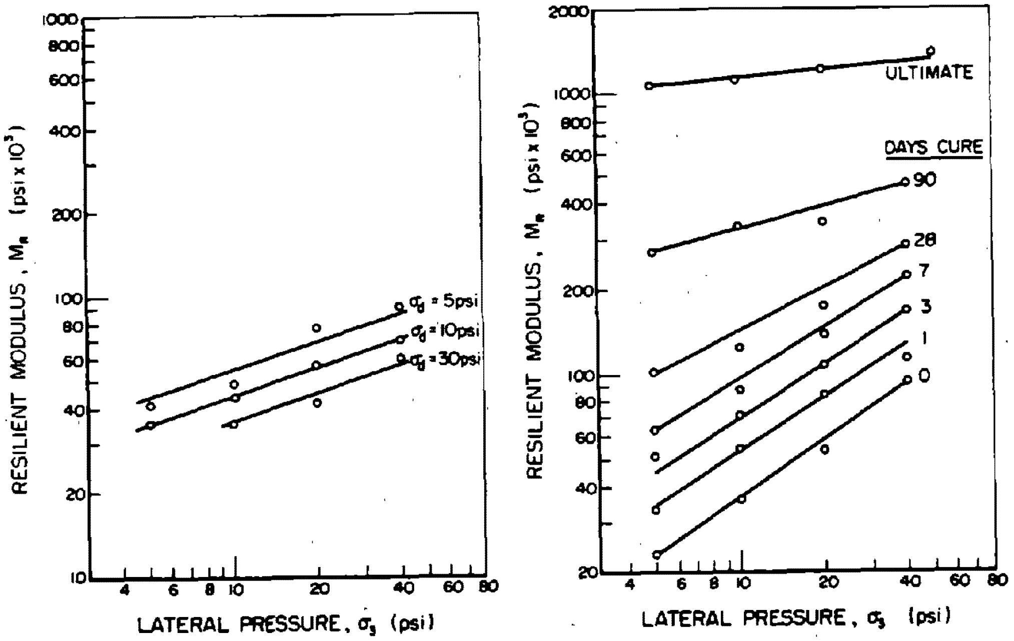
In order to relieve the moisture problem, combinations of lime and asphalt have often been effective. For example, the lime addition may prevent stripping at the asphalt-aggregate interface as well as increase the stability of the mixture. Most notably, lime has been extensively and successfully used to control moisture-induced deterioration of asphalt-treated mixtures.

Lime slurry pretreatment of the soil or aggregate at one percent or more lime level has been quite effective not only in raising the modulus value in some cases but in imparting almost complete water resistance. The mechanism is such that lime in itself can act 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 asphalt. The gain 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 (152). A further observation is that lime improves the workability of some soil-aggregate materials (and hence the other properties noted above) through the pozzolanic action discussed in Chapter III.

Lime-or Cement-Emulsified Asphalt Combinations

Research in emulsified asphalt mixtures has indicated that curing is the key factor contributing to all the drawbacks in the use of emulsified asphalt. The curing or setting of the emulsion-treated material requires loss of water from the mixture. When emulsion-treated base is placed, the curing proceeds rapidly only until the surfacing is laid. Afterwards, the rate of curing levels off, causing delay of the development of full strength. Considerable effort has been expended by researchers in an attempt to promote the curing rate of emulsified asphalt mixtures, such as using elevated mixing temperatures and introduring various additives to the emulsion.

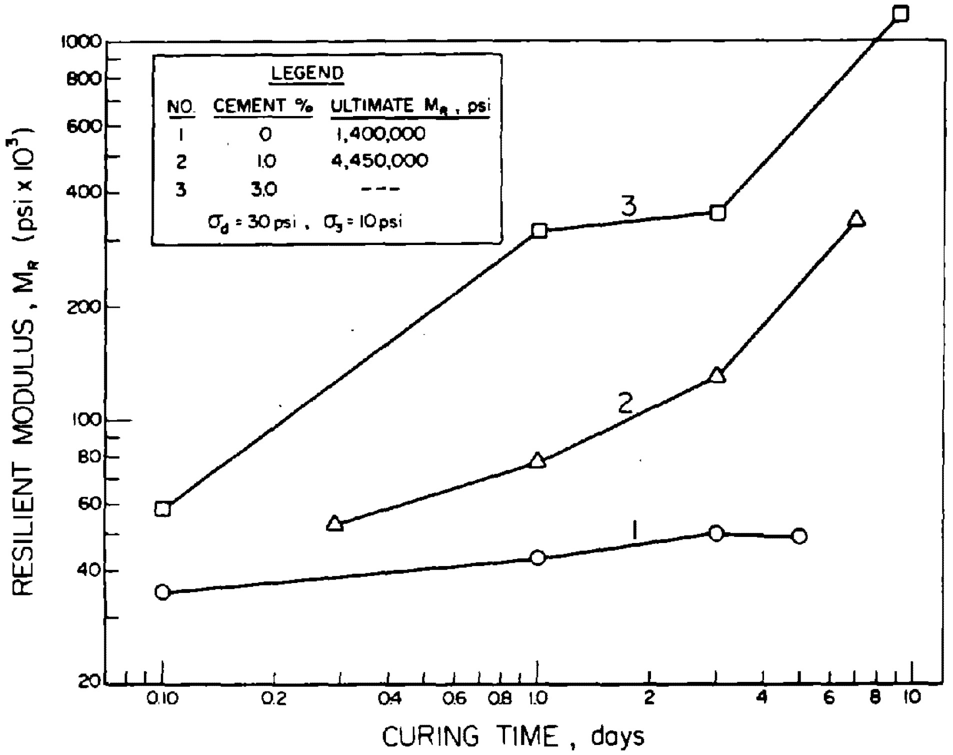
In recent developments, hydrated lime or portland cement has been used to promote curing of the emulsified asphalt-treated materials. Terrel and Wang (153) have shown that the rate of development of strength in emulsified asphalt mixtures on curing is greatly acclerated by cement. Figure 43, from the Terrel and Wang study, shows that when an emulsified asphalt mixture is uncured, it behaves essentially like an untreated granular material (i.e., MR is stress dependent). After varying amount of curing, the material becomes less stress dependent and more like asphalt concrete. Figure 44 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 (152) also demonstrated high moisture resistance of emulsified asphalt mixtures pretreated with lime or cement slurries. 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.



A. NO CURING AFTER COMPACTION.

B. AT INCREASED AGES

Figure 43. Development resilient modulus for SM-K emulsion mixtures at 68°F (153).



1 psi = 6.89 x 10^3 Pa

Figure 44. Resilient modulus vs. curing time for SS-kh mixtures with different cement contents cured at 75°F.

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.

Stabilizer selection procedure can then be based on the percent passing the No. 200 sieve and the Plasticity Index (PI). With the results of the tests, a combination stabilizer can be selected through the process suggested in Figure 45. In general, combination stabilizers are best utilized for soils that have more than 25% passing No. 200 sieve and for Plasticity Index more than 10.

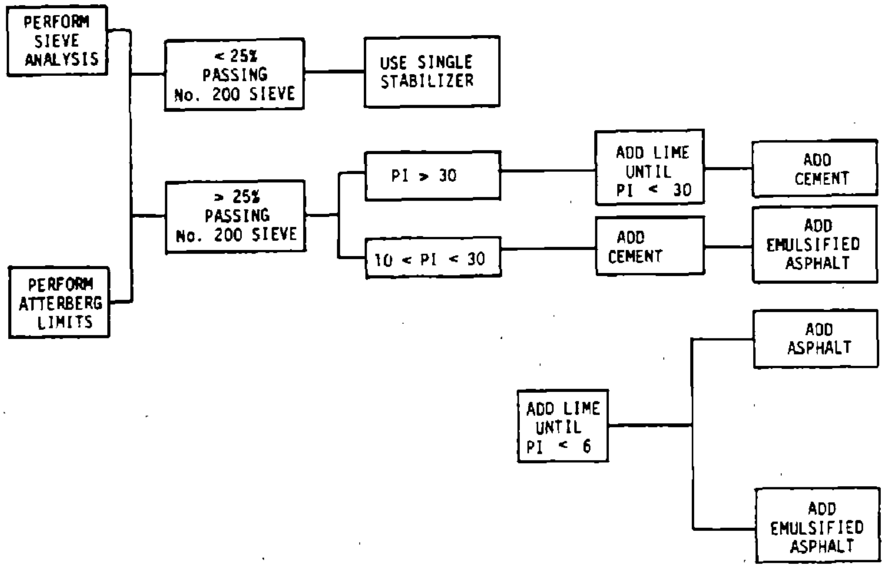


Figure 45. Selection of combination stabilizers.

The various amounts of each individual stabilizer can 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.

Approximate Quantities

Lime-Cement. Since cement cannot be mixed successfully with plastic clays, 1 to 3 percent of lime 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. For the same type of soil condition, more hydrated lime is required than quicklime in the lime-cement mixture.

Lime-Asphalt. Pretreatment of aggregates with at least 1 percent of lime in a slurry form (best used with emulsified asphalt; pulverized lime works best with cutback or asphalt cement) can minimize intrusion of water into asphalt-treated mixtures. This also provides significant strength increase. In general, 1 to 3 percent of lime can be used with 4 to 7 percent 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 a base or subbase has a profound effect on the rate of strength development as well as the ultimate strength level attained. About 1 to 3 percent lime can be combined with 4 to 8 percent emulsified asphalt in the mix.

Cement-Emulsified Asphalt. The addition of small amounts of cement (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 to emulsion of the order of 1 to 5 (based on residual asphalt) appears appropriate to ensure adequate early stiffness without excessive embrittlement.

Detailed Testing

The quantity of stabilizer to be utilized is generally determined by means of laboratory tests, which simulate field conditions of weathering and other durability processes, or by strength tests.

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

The laboratory tests necessary for determining strength and/or durability for the combination stabilizers are described in the following paragraphs.

Lime-Cement. The strength of lime-cement soil mixtures can be evaluated in many ways. The unconfined compression test is the most popular procedure, while flexural and splitting tensile testing for evaluating tensile strength, CBR, and Stabilometer tests are used to a lesser extent. Specimens can be molded in the standard AASHTO compaction cylinder using various amounts of admixture and at optimum water content, and maximum density. The specimen can then be cured in a moist room for a period of 7 days before testing in unconfined compression or in a triaxial testing device.

Factors to be controlled in the strength testing are sample size, compaction procedure, and curing conditions. Specimens 4 inches (102 mm) in diameter by 4.6 to 8 inches (117 to 203 mm) in height have frequently been used. However, 2-inch (51 mm) diameter by 4-inch (102 mm) high specimens may also be used with fine-grained soils. Direct comparison of the strength data developed from specimens of different sizes is difficult. The use of a correlation factor based on length-to-diameter ratio as specified in ASTM C42 should be considered in making such comparisons. Many agencies also use ASTM C593 curing conditions, i.e., 7 days at 100°F (38°C) in a sealed container.

Cyclic freeze-thaw resistance of the lime-cement soil mixture can be determined in several ways. Cyclic freeze-thaw and brushing tests (ASTM C593) can be used for evaluating durability. ASTM C593 criteria require less than 14 percent weight loss following 12 freeze-thaw cycles of the stabilized material, but these criteria could be modified as desired.

The Iowa freeze-thaw test (155) can also be used to evaluate durability of lime-cement stabilized mixtures. The index of resistance R_f is used as a measure of durability where

$$R_f = \frac{100\ P_f}{Pc}$$

in which R_f = Index of Resistance to freeze-thaw

P_f = Unconfined Compressive strength following 28 days moist curing at 71°F (22°C), 24 hours immersion in water, and 10 freeze-thaw cycles.

Pc = Unconfined compressive strength of "control specimen" following 28 days moist curing at 71°F (53°C) and 11 days immersion in water.

For Iowa climatic conditions, it has been proposed that R_f should be greater than 80 percent for stabilized soils.

Stabilized material durability studies have resulted in the development of an automatic freeze-thaw testing procedure that closely simulates field conditions (84). The stabilized mixture is cured 7 days at 100°F (38°F) in accordance with ASTM C593 prior to freeze-thaw testing. The compressive strength after 5 freeze-thaw cycles can be used to characterize the mixture durability. Dempsey and Thompson (85) also proposed a vacuum saturation test procedure which provided a better correlation between the compressive strengths of vacuum saturated specimens and freeze-thaw. Based on this work, ASTM C593 has been revised to incorporate the vacuum saturation testing procedure and the standard freeze-thaw brushing test that was deleted from ASTM C593. Figure 46 shows the relationship between vacuum saturation strength and 5-cycle freeze-thaw strength. This figure can be used to determine the durability of lime-cement stabilized soils.

Lime-Asphalt. The techniques for evaluating lime-asphalt stabilized materials depend upon soil type. For fine-grained materials, absorption tests may be made on the specimens at the desired moisture and density. After the specimens are cured at 100°F (38°C) for a period

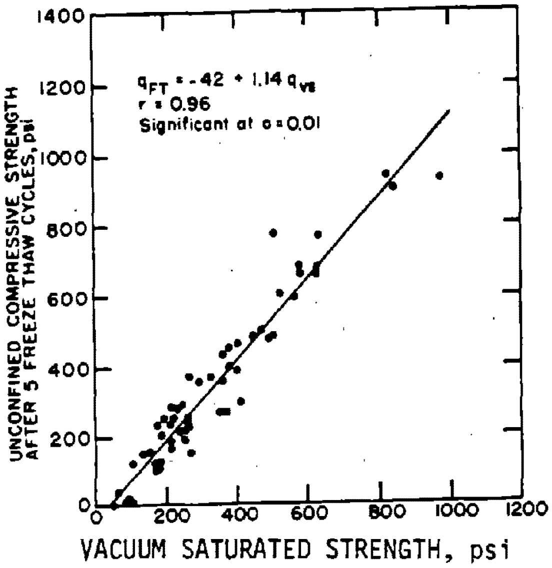


Figure 46. Relationship between vacuum saturation strength and 5-cycle freeze-thaw strength (all data adjusted to equivalent l/d = 2. See ASTM C42-68.)

of at least 7 days, the specimen can be placed upon felt pads or porous stones with the water level maintained at the bottom of the specimen. The purpose of the curing period is to make certain that all of the volatile materials in the asphalt are removed and that the lime has reacted completely with the mixture. The specimens are then weighed before and after saturation and the amount of absorbed water determined. No specific guidelines can be provided at this time, but comparison of absorption of lime-asphalt and asphalt-only specimens will indicate the relative effectiveness of the lime pretreatment. In other words, judgement is largely qualitative.

ASTM Committee D-18 has established a tentative method for evaluating soil-bituminous mixtures that could be adapted for lime-asphalt stabilized fine-grained or sand materials. In this method, the saturated specimens are tested in the Hubbard-Field testing apparatus, rather than unconfined compression.

For lime-asphalt-gravel or lime-asphalt-gravel-sand mixtures, conventional California Bearing Ratio tests and triaxial tests may be used. Tests can be performed on the unstabilized material and then on the mixture; decisions relative to use of the appropriate mixture can be made based on several test results.

Durability tests for lime-asphalt stabilized materials are as outlines for lime-cement stabilized materials.

Lime-or Cement-Emulsified Asphalt. The strength of lime-emulsified asphalt or cement-emulsified asphalt mixes can be determined either by the stabilometer R-value or resilient modulus M_R tests as discussed in earlier chapters.

The R-value test can be used to measure the stability or bearing capacity of the lime-emulsified asphalt or cement-emulsified asphalt mixes at a test temperature of 73° ± 5°F (23°C ± 3°C). An early Resistance-value can be determined on specimens fabricated and cured 24 hours. After a final 7-day cure consisting of 3 days cure in the mold followed by 4 days room temperature vacuum desiccation, the R-value can be measured. To simulate the effect of prolonged exposure to subsurface water on the stabilized mixture, the specimen can be vacuum saturated before final R-value determination. The results can then be compared to those obtained for standard untreated soils and for asphalt concrete.

The Resilient Modulus M_R is a dynamic test response defined as the ratio of the repeated axial deviator stress σ_d to the recovered axial strain ε_a

$$M_R = \frac{\sigma_d}{\varepsilon_a}$$

The test can be conducted in a triaxial device equipped for repetitive loading conditions. Specimen size is normally 4 inches in diameter by 8 inches high (100 to 200 mm). The strain used to calculate the modulus is the recoverable portion of the deformation response (ASTM Test Method D3497-76T).

The M_R of lime- or cement-emulsified asphalt mixtures varies under different conditions, which include the magnitude and duration of deviator stress, magnitude of confining pressure, and cured state. M_R tests can be run on test specimens after 24-hour cure and after 7 days on the specimen which can be assumed to be fully cured. These test values are believed to represent the extremes in strength to be encountered throughout the life of the mixture. The fully-cured material can be tested at two temperatures - 73°F (23°C) and 100°F (38°C) to determine the effect of temperature on mix strength.

The R-value and resilient modulus M_R test are generally run on fully-cured, vacuum-saturated specimens to determine if the stabilized mixture can meet minimum bearing strength requirements when saturated with water.

LIMITATIONS AND PRECAUTIONS

Climatic and/or Construction Limitations

Lime Stabilized soils are relatively slow setting and require some warm weather to harden properly. Cement hydration also ceases when temperatures are near or below freezing. 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 not much 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 beneficial. All lime-cement stabilized bases require a wearing surface of at least a bituminous seal coat. An unprotected lime-cement stabilized base might have 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 hot asphalt stabilized material are being placed, the air temperature should be 40°F 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 laboratory compacted soil samples.

Lime-emulsified asphalt or cement-emulsified asphalt applications 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. A longer breaking or cure time should be anticipated during periods of high humidity. Stabilized material and air temperatures preferably should be above 60°F (16°C). During hot, dry weather conditions, it is advantageous to moisten the soil prior to application of emulsion.

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

Safety Precautions

In using lime, contact of 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.

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APPENDIX A

SUMMARY OF VARIOUS STRENGTH-BASED MIXTURE DESIGN PROCEDURES FOR LIME STABILIZED SOILS

California Procedure

California's current design procedure is based on stabilometer test data developed for mixtures containing various lime percentages. The general procedure is as follows:

1. Soil-lime mixtures are prepared at various lime percentages. The mixture's moisture content is adjusted to approximately optimum (AASHTO T-180) and the moist mixture is loose cured for 24 hours,

2. Stabilometer samples are compacted using the California kneading compactor (California Test Method 301). The compacted specimens are not cured,

3. The compacted specimens are tested using the stabilometer (California Test Method No. 312) to determine the R-value,

4. Depending on the intended use of the mixture, the lime percentage required to develop an R-value in the range of 60 to 80 is determined, and

5. The lime percentage is increased approximately 1 percent to compensate for field construction variability.

Illinois Procedure

The mixture design procedure is based on unconfined compressive strength test data. Specimens with a 2 inch (51 mm) diameter and a 4 inch (102 mm) height of the natural soil and soil-lime mixtures are prepared at optimum moisture content and maximum dry density (AASHTO T-99). The soil-lime specimens, prepared at various lime treatment levels, are cured for 48 hours at 120°F (40°C) prior to testing.

The compressive strength of the soil-lime mixture with 3 percent lime must be at least 50 psi (345 kPa) greater than the compressive strength of the natural soil. The design lime content is designated as the lime percent above which further increases do not produce significant additional strength. For field construction, the lime content is increased 0.5 to 1.0 percent to offset the effects of field variability. Minimum strength requirements are 100 psi (690 kPa) for subbase and 150 psi (1030 kPa) for base course. These minimum strengths relate to AASHTO coefficients of relative strength of 0.12 for subbase materials and 0.11 for base course materials.

Louisiana Procedure

Lime contents for soil-lime mixtures to be used as base or subbase courses are determined in accordance with LDH Designation TR 433-70, "Determining the Minimum Lime Content for Lime-Soil Treatment." Quality requirements, expressed in terms of minimum unconfined compressive strength, are 100 psi (690 kPa) for base course and 50 psi (345 kPa) for subbase courses.

Soil-lime mixtures of various lime contents are prepared at optimum moisture content (LDH TR 418) and specimens 6 inch (152 mm) in diameter and 8 inch (203 mm) in height are compacted to maximum dry density (LDH TR 418).

The curing cycle for the compacted soil-lime mixture is:

1. seven days in moist room,

2. air dried 8 hours at 140°F (60°C),

3. eight hours of cooling, and

4. ten-day capillary soaking at a confining pressure of 1 psi (6.9 kPa), AASHTO T-212 procedure.

Following curing, the specimens are tested in unconfined compression at a rate of 0.15 in./minute (3.81/minute). The minimum lime content providing adequate unconfined compressive strength, i.e., 100 psi (690 kPa) for base, 50 psi (345 kPa) for subbase, is determined from the test data.

South Dakota Procedure

Initial lime requirements are established based on a pH procedure (Test No. SD 128) which is similar to the Eades and Grim Procedure (68). Supplemental strength data are developed by evaluating the CBR of various soil-lime combinations compacted at optimum moisture content (AASHTO T-99) to maximum dry density.

The South Dakota technique (Test No. SK 107) is similar to AASHTO T-193. If, with no curing except for the 96-hour soaking period, the CBR of the soil-lime mixture is 3 to 4 times greater than the CBR of the natural soil, the soil-lime mixture is considered to be of adequate quality for use as a pavement layer (AASHTO coefficient of relative strength = 0.05).

Thompson Procedure

Thompson has developed a mixture design process for soil-lime mixtures. Mixtures which display significant compressive strength increases (50 psi [345 kPa] minimum) can be utilized as base and subbase materials depending on the soil-lime mixture properties and pavement service requirements. Design lime percentage is based on mixture compressive strength data (48 hour curing at 120°F [49°C]) for various lime treatment levels.

Quality criteria for the soil-lime mixtures were established, based on considerations of pavement structural behavior and durability requirements. The soil-lime quality criteria are summarized in Table 40.

The development of the mixture design process and the detailed testing procedures are contained in reference 156.

<u>Texas Procedure</u>

The soil-lime mixture design procedure used by the State Department of Highways and Public Transportation is AASHTO T-220 which provides for the determination of the unconfined compressive strength of soil-lime mixtures. The procedure suggests strength criteria of 100 psi (690 kPa) for base construction and 50 psi (345 kPa) for subbase construction.

Details of the procedure are included in AASHTO T-220; however, a general outline of the procedure is presented below.

1. Based on the grain size and plasticity index data, the lime percentage is selected from a chart.

2. Optimum moisture and maximum dry density of the mixture are determined in accordance with appropriate sections of AASHTO T-212 or Tex-121-E. The compactive effort is 50 blows of a 10 lb (4.54 kg) hammer, with an 18 inch (457 mm) drop.

3. Test specimens 6 inches (152 mm) in diameter and 8 inches (203 mm) in height are compacted at optimum moisture content to maximum dry density.

4. The specimens are placed in a triaxial cell (AASHTO T-212 or Tex-121-E) and cured in the following manner:

   a. seven days at room temperature,

   b. remove cells and dry at a temperature not to exceed 140°F (60°C) for about six hours or until one-third to one-half of the molding moisture has been removed,

   c. cool the specimens for at least 8 hours, and

   d. subject the specimens to capillarity (section 6 of AASHTO T-212 or Tex-121-E) for 10 days.

5. The cured specimens are tested unconfined compression in accordance with sections 7 and 8 of AASHTO T-212 or Tex-121-E.

Virginia Procedure

Virginia's mixture design procedure, VTM-11 Virginia Test Method for Lime Stabilization, is based on the cured compressive strength of soil-lime mixtures stabilized with various amounts of lime. The procedure is summarized below.

1. Proctor-sized specimens at various lime percentages are prepared at approximately optimum moisture content and maximum dry density (AASHTO T-99 compaction test conducted with 6 percent lime).

2. Specimens are cured in sealed containers at high humidity for 72 hours at 120°F (49°C).

3. The soil-lime specimens are tested in unconfined compression using a loading rate of 2,400 pounds/minute (1089 kg/minute) or approximately 19 psi/minute (131 kPa/min).

4. Virginia criteria require a minimum compressive strength of 150 psi (1030 kPa) for soil-lime mixtures tested in accordance with the above procedure.

APPENDIX B

pH TEST ON SOIL-CEMENT MIXTURES (31)

Materials

1. Portland cement to be used for soil stabilization

Apparatus

1. pH meter (the pH meter must be equipped with an electrode having a pH range of 14)
2. 150-ml plastic bottles with screw-top lids
3. 50-ml plastic beakers
4. Distilled water
5. Balance
6. Oven
7. Moisture cans

Procedure

1. Standardize the pH meter with a buffer solution having a pH of 12.00.
2. Weight to the nearest 0.01 gms., representative samples of air-dried soil, passing the No. 40 sieve and equal to 25.0 gms of oven-dried soil.
3. Pour the soil samples into 150-ml plastic bottles with screw-top lids.
4. Add 2.5 gms of the portland cement.
5. Thoroughly mix soil and portland cement.
6. Add sufficient distilled water to make a thick paste. (Caution: too much water will reduce the pH and produce an incorrect result.)
7. Stir the soil-cement and water until thorough blending is achieved.

8. After 15 minutes, transfer part of the paste to a plastic beaker and measure the pH.
9. If the pH is 12.1 or greater, the soil organic matter content should not interfere with the cement stabilizing mechanism.

APPENDIX C

DETERMINATION OF SULFATE IN SOILS
GRAVIMETRIC METHOD (31)

Scope

Applicable to all soil types with the possible exception of soils containing certain organic compounds. This method should permit the detection of as little as 0.05 percent sulfate as SO_4.

Reagents

1. Barium chloride, 10 percent solution of $BaCl_2 \cdot 2H_2O$. (Add 1 ml 2 percent HCl to each 100 ml of solution to prevent formation of carbonate.)
2. Hydrochloric acid, 2 percent solution (0.55 N)
3. Magnesium chloride, 10 percent solution of $MgCl_2 \cdot 6H_2O$
4. Demineralized water
5. Silver nitrate, 0.1 N solution

Apparatus

1. Beaker, 1000 ml
2. Burner and ring stand
3. Filtering flask, 500 ml
4. Buchner funnel, 90 mm
5. Filter paper, Whatman No. 40, 90 mm
6. Filter paper, Whatman No. 42, 90 mm
7. Saranwrap
8. Crucible, ignition, or aluminum foil, heavy grade
9. Analytical balance
10. Aspirator or other vacuum source

Procedure

1. Select a representative sample of air-dried soil weighing approximately 10 gm. Weigh to the nearest 0.01 gm. (Note: When sulfate content is anticipated to be less than 0.1 percent, a sample weighing 20 gm or more may be used.) (The moisture content of the air-dried soil must be known for later determination of dry weight of the soil.)

2. Boil for 1-1/2 hours in beaker with mixture of 300 ml water and 15 ml HCl.

3. Filter thorugh Whatman No. 40 paper, wash with hot water, dilute combined filtrate and washings to 50 ml.

4. Take 100 ml of this solution and add $MgCl_2$ solution until no more precipitate is formed.

5. Filter through Whatman No. 42 paper, wash with hot water, dilute combined filtrate and washings to 200 ml.

6. Heat 100 ml of this solution to boiling and add $BaCl_2$ solution very slowly until no more precipitate is formed. Continue boiling for about 5 minutes and let stand overnight in warm place, covering beaker with Saranwrap.

7. Filter through Whatman No. 42 paper. Wash with hot water until free from chlorides (filtrate should show no precipitate when a drop of $AgNO_3$ solution is added).

8. Dry filter paper in crucible or on sheet of aluminum foil. Ignite paper. Weigh residue on analytical balance as $BaSO_4$.

Calculation

$$\text{Percent } SO_4 = \frac{\text{Weight of residue}}{\text{Oven-dry weight of initial sample}} \times 411.6$$

where

Oven-dry weight of initial sample

$$= \frac{\text{Air-dry weight of initial sample}}{1 + \dfrac{\text{Air-dry moisture content (percent)}}{\text{100 percent}}}$$

Note: If precipitated from cold solution, barium sulfate is so finely dispersed that it cannot be retained when filtering by the above method. Precipitation from a warm, dilute solution will increase crystal size. Due to the adsorption (occlusion) of soluble salts during the precipitation by $BaSO_4$, a small error is introduced. This error can be minimized by permitting the precipitate to digest in a warm, dilute solution for a number of hours. This allows the more soluble small crystals of $BaSO_4$ to dissolve and recrystallize on the larger crystals.

TURBIDIMETRIC METHOD

Reagents

1. Barium chloride crystals (Grind analytical reagent grade barium chloride to pass a 1-mm sieve.)
2. Ammonium acetate solution (0.5N) (Add dilute hydrochloric acid until the solution has a pH of 4.2.)
3. Distilled water

Apparatus

1. Moisture can
2. Oven
3. 200-ml beaker
4. Burner and ring stand
5. Filtering flask
6. Buchner funnel, 90 mm
7. Filter paper, Whatman No. 40, 90 mm
8. Vacuum source
9. Spectrophotometer and standard tubes (Bausch and Lombe Spectronic 20 or equivalent)
10. pH meter

Procedure

1. Take a representative sample of air-dried soil weighing approximately 10 gms, and weight to the nearest 0.01 gms. (The moisture content of the air-dried soil must be known for later determination of dry weight of the soil.)
2. Add the ammonium acetate solution to the soil. (The ratio of soil to solution should be approximately 1:5 by weight.)
3. Boil for about 5 minutes.

4. Filter through Whatman No. 40 filter paper. If the extracting solution is not clear, filter again.

5. Take 10 ml of extracting solution (this may vary, depending on the concentration of sulfate in the solution) and dilute with distilled water to about 40 ml. Add about 0.2 gm of barium chloride crystals and dilute to make the volume exactly equal to 50 ml. Stir for 1 minute.

6. Immediately after the stirring period has ended, pour a portion of the solution into the standard tube and insert the tube into the cell of the spectrophotometer. Measure the turbidity at 30 sec intervals for 4 minutes. Maximum turbidity is usually obtained within 2 minutes and the readings remain constant thereafter for 3-10 minutes. Consider the turbidity to be the maximum reading obtained in the 4 minute interval.

7. Compare the turbidity reading with a standard curve and compute the sulfate concentration (as SO_4) in the original extracting solution. (The standard curve is secured by carrying out the procedure with standard potassium sulfate solutions.)

8. Correction should be made for the apparent turbidity of the samples by running blanks in which no barium chloride is added.

Sample Calculation

Given: Weight of air-dried sample = 10.12 gms

Water content = 9.36 percent

Weight of dry soil = 9.27 gms

Total vlume of extracting solution = 39.1 ml

10 ml of extracting solution was diluted to 50 ml after addition of barium chloride (see step 5). The solution gave a transmission reading of 81.

Calculation

From the standard curve, a transmission reading of 81 corresponds to 16.0 ppm. (See following figure.)

Concentration of original extracting solution = 16.0 x 5 = 80.0 ppm.

$$\text{Percent } SO_4^{--} = \frac{80.0 \times 39.1 \times 100}{1000 \times 1000 \times 9.27} = 0.0338 \text{ percent}$$

Determination of Standard Curve

1. Prepare sulfate solutions of 0, 4, 8, 12, 16, 20, 25, 30, 35, 40, 45, 50 ppm in separate test tubes. The sulfate solution is made from potassium sulfate salt dissolved in 0.5 N ammonium acetate (with pH adjusted to 4.2).
2. Continue Steps 5 and 6 in the procedure as described in Determination of Sulfate in Soil by Turbidimetric Method.
3. Draw standard curve as shown in Figure 50 by plotting transmission readings for known concentrations of sulfate solutions.

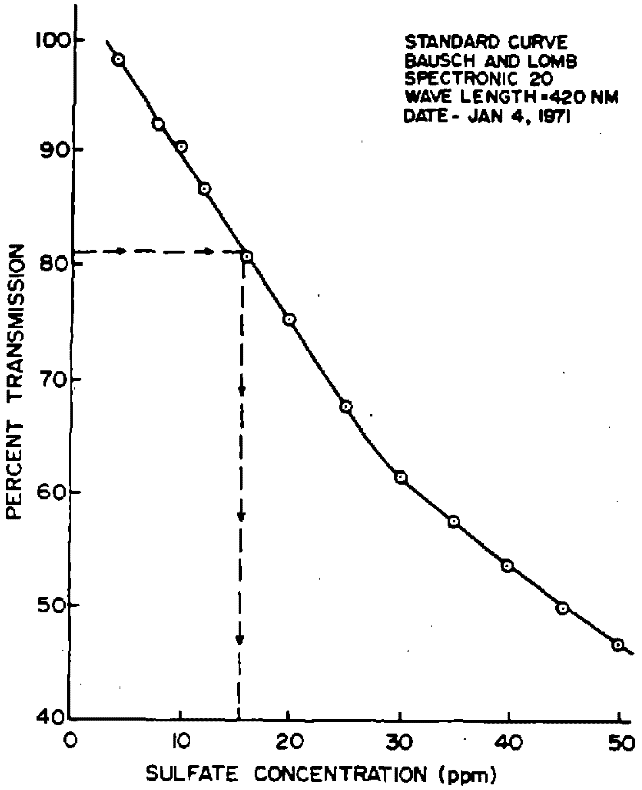

Figure 47. Example standard curve for spectrophotometer.

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