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16. Abstract

Volume 1 and 3 of this report are guide booklets for administrators and road builders respectively. Volume 4 of this report documents the use and cost-benefits of four stabilizaton treatments, i.e., lime, asphalt, cement, and lime-fly ash in the construction of low-volume roads.

Volume 2, contained ~~herein~~, is a guide booklet for the road engineers to provide assistance in evaluating and designing stabilized soil pavements for low-volume roads. The information about specific factors such as traffic, soil, climate, stabilizer selection, unique design procedures, construction equipment and quality control is presented. The procedures for the evaluation and selection of a compatible stabilizer, the stabilizer's application rate and specific construction requirements are outlined. The five major construction steps: soil preparation, stabilizer application, pulverization and mixing, compaction, and curing are detailed.

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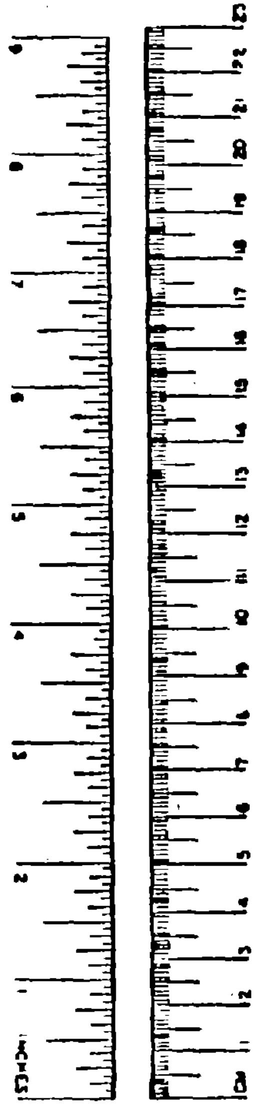
NOTICE

METRIC CONVERSION FACTORS

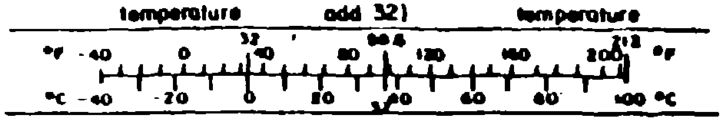
APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
AREA				
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.6	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
MASS (weight)				
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons(2000 lb)	0.9	tonnes	t
VOLUME				
tsp	teaspoons	5	milliliters	ml
tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
TEMPERATURE (exact)				
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
AREA				
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares(10,000m^2)	2.5	acres	
MASS (weight)				
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000kg)	1.1	short tons	
VOLUME				
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	36	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
TEMPERATURE (exact)				
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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INTRODUCTION

This publication was written to assist technical personnel associated with road jurisdictions at the local level in evaluating and designing stabilized soil pavements for low-volume roads. The average county or local government technical staff has a multitude of engineering, financial, and "public relations" tasks, in addition to keeping abreast of road technology. Generally they are not updated on the "state-of-the-art" of soils stabilization. However their background commonly includes an awareness of standard design procedures, materials tests, construction methods, and cost analyses. They need more information about specific factors such as traffic, soils, climate, stabilizer selection, unique design procedures, construction equipment, and quality control.

The booklet offers assistance in the identification and evaluation of each of these factors. It outlines the proper procedures to follow in applying the results of the evaluation process to the selection of a compatible stabilizer, the stabilizer's application rate, and specific construction requirements. The economic analysis procedure is documented in Volume 4 - Cost-Benefit Analysis in this four volume series. Engineers who need or want further data about soils stabilization will find more information in Volume 1 - Executive Summary, and Volume 3 - Road Builder's Guide, published together with this booklet. A more technical treatment of soil stabilization can be found in another recently published FHWA two volume document titled: Soils Stabilization in Pavement Structures - A User's Manual (1) (2).

The definition of low-volume roads, as adopted in this booklet, is stated in Figure 1. Low Volume Roads Definition, on page 38. Low-volume roads are for the use of the people living or working in the local area; the roads carry only the types of vehicles normally used in the local area; and the roads are usable and safe through-out the year, at slower speeds and on a less smooth surface than required on high-volume highways.

This booklet first reviews the standard procedures for the design of pavement thicknesses necessary to provde the serviceability requirements for high-volume, high-speed, all-type vehicle traffic. However, the serviceability requirements for the low-volume roads defined above are for low-speed, particular-type vehicle traffic; therefore current design standards are not suitable for determining soil stabilization requirements for these low-volume roads. Accordingly, low-volume roads are generally constructed with a roadbed thickness of six inches for most common vehicle traffic, a thickness based mainly on the mixing capacity of normal construction equipment. This booklet offers guidance for determining thicker roadbeds when more extreme vehicle traffic predominates, such as roads to quarries and warehouses.

Most low-volume roads will last indefinitely when properly planned and routinely maintained on a realistic schedule. However when routine maintenance becomes excessive, some corrective action based on an engineering evaluation is required. The first step to improve such roads is always drainage oriented, as insufficient roadway drainage will negate any other improvements that may be attempted.

For many low-volume roads, soil stabilization will provide and retain the necessary serviceability requirements with much less maintenance. Granular soils can be stabilized in a plant or after placement on the roadway. Frequently existing roadbed soils are stabilized in place. The stabilization of suitable in situ soils is often less expensive than hauling in better material, particularly when the elevation and width of the existing roadway must be retained. The thickness of the stabilized soil layer depends on the volume and type of traffic, the type of soil to be stabilized, the nature of the subgrade, and on climatic factors such as temperature and precipitation.

There are many stabilizing agents in use throughout the country today. This booklet deals with only a few of the common stabilizers: lime, cement, and asphalt; and one typical combination stabilizer, lime-fly ash. This approach was chosen to reduce the subject matter to a manageable amount. Much of the information presented holds true for other stabilizers which may also be suitable. An array of technical information is available about all stabilizing agents from the various stabilizing agent suppliers, their industry associations, other local and county technical staffs, and state and federal agencies.

The evaluation of both the impact of the various technical factors on each other and their combined ability to properly service the needs for the general public constitutes a yet imperfect exercise called pavement structural design. This booklet will address each of the design considerations in order, beginning with a review of current design practices. It continues with information about traffic volumes and types, soils evaluations, climatic constraints, and stabilizer selection. Design methods for bituminous, portland cement, lime, and lime-fly ash are then discussed in order. The last section covers the five major construction steps: soil preparation, stabilizer application, pulverizing and mixing, compaction, and curing, in order, with comments about the individual stabilizers as required.

DESIGN CONSIDERATIONS

Proper pavement structural design takes into account:

1. the total projected traffic loadings that will occur during the design life of the pavement;

2. the characteristics of the soil under the pavement;

3. the environmental conditions (temperature and precipitation) under which pavement must function; and

4. the construction materials available.

Practical pavement structural design must also consider the economic, construction, and maintenance capabilities of the road building organization involved. Very few new roads are being built. Most local and county governments have been forced to maximize and budget their low-volume road resources by stressing maintenance of existing roads, improving trouble spots as they occur, and conducting modest upgrading programs to meet the anticipated needs of the public they serve.

Structural road failure occurs when the pavement will no longer support the axle loadings that are applied to it. In this booklet road failure is deemed to occur when excessive maintenance costs are required to keep the road operational. Excessive maintenance costs are different for each individual road and for each type of improvement. The cost of improving in place material may be less than the cost of using gravel imported from a distant pit, while the cost of laying a plant mix pavement may be even greater. Therefore, the acceptable cost of an increased maintenance effort increases as the costs of the available improvement choices increase.

Pavement structure design charts are based on the functional failure of the pavement's surface course, i.e., the pavement's serviceability drops below a predefined value as measured by roughness. Serviceability is defined as the pavement's ability to serve high-speed, high-volume automobile and truck traffic (3). This definition is inappropriate for the type of roads defined in Figure 1., on page 38. A serviceability criterion, per se, is unlikely to govern low-volume road reconstruction activities, especially in tight economic times. It does, however, precipitate the potential of excessive maintenance costs.

Pavement design thickness considerations are usually based on stress distribution (AASHTO Interim Guide, Multilayered elastic analysis, etc.) which determines pavement component layer thickness. Layer thicknesses are also controlled by minimum and maximum practical construction depths. Often low-volume road bases and subbases are built to a minimum practical construction compacted depth of six inches to allow for imperfect quality control. However, the projected traffic must be evaluated during the decision making process to determine if the loadings exceed the design parameters of a six-inch thickness. In such cases, standard serviceability tables are often used, because no better design tools exists. Reference 3 should be consulted by engineers wishing to use these tables as it contains complete instructions for their use.

Design tables indicate that many coarse grained soils in moderate environmental areas can be adequately strengthened by the addition of a stabilizing admixture to the top six inches. If the actual pavement life proves to be shorter than the desired design life, such stabilization is often considered as stage construction, which in an economy with increasingly high discounted future costs and benefits, is more economical than initial overdesign. Since low-volume road design life is much more difficult to anticipate than major highway design life, satisfactory performance under immediately foreseeable circumstances, rather than undocumented future traffic projections, should be the criteria for evaluating soil stabilization. The implications of traffic volume assessment are described in the next section.

TRAFFIC

The traffic values used in standard design nomographs, also located in Reference 3, for pavement design are not the normal vehicles per day traffic values found by using automatic traffic counters. Design nomographs use traffic volumes that are adjusted for the various types of vehicles (cars, buses, trucks), which make up the traffic stream on the specific road under consideration. This combination of vehicles is adjusted to fit into the design tables by expressing the axle loadings of each vehicle as equivalent 18,000 pound (18-Kip) single axle load applications.

The lowest total equivalent 18-Kip single axle load applications found on standard flexible pavement design nomographs is 50,000. These nomographs also have average daily traffic (ADT) values for a 20 year analysis period. Optimum stage construction for low volume roads is achieved, however, by using a staging interval that results in the least cost. That interval is generally 10 years or less depending on anticipated traffic growth and subgrade strength (4). Consequently, any projected ADT values should be converted to total equivalent axle loads for the anticipated design period before using the design tables developed for this booklet.

Design nomographs are based on total equivalent axle loadings on a single lane of pavement. A common assumption made for low-volume roads is that half the traffic travels each way unless available data indicates a different value should be used (e.g., empty trucks go one way and return loaded on the opposite lane).

Equivalent axle loading ADT'S can be quite misleading to the engineer who commonly deals with the vehicles per day concept of traffic counting. A value called the Traffic Equivalent Factor (3) compares the effect of various axle loads to the effect of the design axle 18-Kip load on pavement serviceability. For example, a 2-Kip axle loading has a Traffic Equivalence Factor of 0.0002. This means that 5000 2-Kip axle loadings have the same effect on pavement serviceability as a single 18- Kip standard axle loading. Table 1: Review of Axle Load Equivalences, Flexible Pavement, on pages 61 and 62, equates various axle loads to familiar vehicles. Normally vehicles with a 10,000 pound gross vehicle weight (GVW) or less are not considered in pavement design.

A vehicle such as a 5 cubic yard single rear axle dump truck (GVW 27,500 lbs.) commonly has a 18-Kip loading on its rear axle when fully loaded. Its front axle loading would therefore be 9.5-Kips. The entire vehicle (both axles) will have an Axle Load Equivalence Factor of less than 1.09. The same loaded truck with tandem rear axles however would be rated as less than 0.2 18-Kip single axles with the greater axle loading occuring on the front axle. The minimum design value of 50,000 total equivalent 18-Kip single axle load applications found on the standard serviceability nomographs therefore represents approximately 45,900 loaded single rear axle 5 yard dump trucks going in each direction on a two lane road. The relationship of this volume of loaded 5 yard dump trucks to ADT values for several different time spans based on reaching a serviceability rating value of 2.0, (the value used in nomographs for low-volume roads) is as follows:

Time	ADT per Lane
10 years	13
5 years	25
4 years	31
3 years	42
2 years	63
1 year	125

The same 50,000 total equivalent 18-Kip single axle load applications represents at 10 year ADT of 34,000 empty pickup trucks per lane.

Normal traffic found on most low-volume roads obviously falls well below the total equivalent axle loadings affecting thickness design as indicated in design nomographs. Traffic evaluations should start with an investigation of the extent of heavy axle loads likely to use the road under consideration. Unless such a study indicates a specific source of heavy vehicle traffic, random truck traffic will probably not significantly affect the design life of a six inch stabilized soil layer. If a steady source of heavy traffic is identified, it should of course be factored into a design thickness determination along with the strength of the soil under the pavement described in the next section.

SOILS

The flexible pavement design nomographs shown in Reference 3 also include a Soil Support Value (SSV). This value is included because the performance of a pavement structure is directly related to the physical properties and condition of the roadbed soils. There is no direct test to determine the SSV but different design agencies have attempted to establish correlations between soil classification tests and the soil support value. There are also different soil classifications in use throughout the country. This stabilization booklet will use the Unified Soil Classification System to define soil types so that the SSV can be estimated.

The Unified Soil Classification System identifies soils according to their textural and plasticity (moldability) qualities. In this system soils are divided into three major groupings: coarse-grained soils (more than 50% of the material passing the 3-inch, or 77-mm, sieve is retained on the No. 200 sieve); fine grained soils, (50% or more of the above passes the No. 200 sieve); and highly organic soils. Coarse-grained soils are further subdivided into gravels (G) and sands (S). Gravels have 50% or more of the coarse fraction (that portion passing the 3-inch sieve and retained on the No. 200 sieve) retained on the No. 4 sieve, while sands have more than 50% of the coarse fraction passing the No. 4 sieve. The coarse-grained soils are further subdivided depending on the type and amount of fines (material passing the No. 200 sieve) present and the shape of the grain-size distribution curve.

Fine-grained soils are subdivided into silts (M) and clays (C) depending on their liquid limit and plasticity index. These groups have secondary divisions based on whether the soils have a relatively low (L) or high (H) liquid limit (i.e. greater than 50). Figure 2: Unified Soil Classification System, on pages 39 thru 43, described the 15 soil groupings that make up the Unified Soil Classification System while Figure 3: Laboratory Identification Procedure, on page 44, outlines the laboratory identification procedure used to classify these soils.

The various groupings within the Unified Soil Classification System also identify the performance of individual soils as engineering construction materials. Figure 4: Soil Characteristics, on pages 45 and 46, shows the engineering characteristics of each of the 15 soil groupings and the range of CBR values that can be expected from each soil.

Stability of the clean, well-graded, coarse-grained soils is due to their mechanical resistance to lateral flow. Water has little effect on the internal friction or volume change of these larger sized particles. In fine-grained materials (silts and clays) stability is very moisture dependent. Unfortunately, in the mixtures of soil found in nature, relatively small amounts (as little as 5 to 12% by weight) of fine-grained materials has a pronounced effect on the stability of the coarse-grained materials has a pronounced effect on the stability of the coarse-grained materials in the soil matrix. To reduce the effect of the fine-grained material, stabilizing agents can be used to obtain and maintain desired moisture content, to increase cohesion, to produce a cementing action, and to act as a water-proofing material. However the soil support value (SSV) used in design nomographs is based on the field CBR value of the untreated material under the stabilized layer or in the soil base course if stabilization is not required. By interpolation, the SSV for each of the soils in Figure 4, be assumed as:

Soil Code	CBR Range	SSV Range
GW	60-80	8.0-8.5
GP	25-60	6.5-8.0
GMd*	40-80	7.2-8.5
GMu*	20-40	6.2-7.2
GC	20-40	6.2-7.2
SW	20-40	6.2-7.2
SP	20-40	6.2-7.2
SMd*	20-40	6.2-7.2
SMu*	10-20	5.2-6.2
SC	10-20	5.2-6.2
ML	5-15	4.0-5.8

CL	5-15	4.0-5.8
OL	4-8	3.5-4.8
CH	3-5	3.0-4.0
OH	3-5	3.0-4.0

*Suffix d used when liquid limit is 25 or less and plasticity index is 5 or less; the suffix u is used otherwise.

Unfortunately, the soil under the roadway does not have a constant strength or Soil Support Value. The above values are inferred from laboratory conditions, while the soil under the road is subjected to changing conditions every day. These changes are accounted for, in pavement structural design, by a climatic modifier.

ENVIRONMENTAL FACTORS

Water is the most important environmental factor affecting road structural strength. The first step in upgrading any roadbed is the improvement of all inadequate drainage facilities. Drainage improvement falls into two separate activities:

1) Removal of precipitation from the road surface as quickly as possible by means of a proper crown and adequate ditches (or an adequate closed drainage system with catchbasins).

2) Prevention of water penetration into the roadbed material by means of a water resistant surface material, proper culverts, interceptor drainage, or subdrains.

In some cases drainage improvements will eliminate the need for soils stabilization and in all cases adequate drainage is necessary for the successful soil stabilization.

Frost damage, the deadly enemy of roads in many areas of the country, is a factor of water, soil type, and temperature variations. Figure 5: Six Climatic Regions in the U.S., on pages 47 and 48, shows that the United States can be divided into six climatic regions comprising three areas of frost conditions, each being further divided into wet and dry areas. The northern most areas (III & VI) suffer severe winters having extremely low temperatures with a high potential for subgrade frost damage; the middle areas (II and V) have moderate winters with a high potential for freeze-thaw activity extending deep into the roadway throughout the winter; and the remaining zones (I and IV) experience mild winters with some surface freeze-thaw cycling in the northern sections and high temperature stability problems over the entire zones in the summer.

These division lines correspond to the weather bureau's design freezing index values of 500 and 50. Zones V and VI are not as homogenous as the other four zones. The proper frost zone designation for any area can be determined by contacting the local weather bureau office.

The north-south dividing line between the wet and dry areas is based on the concept of potential evapotranspiration, or the amount of moisture that would leave the soil through evaporation and transpiration if there were an unlimited supply of water to the soil system (8). East of the dividing line there is more precipitation than potential moisture loss while west of the division line there is less precipitation than potential moisture loss. The west coast states unfortunately do not fall neatly into this wet-dry classification and care must be exercised to prevent being misled by mapping simplifications. Again, the weather bureau may be able to supply the Thornthwaite Index for your area.

Frost-susceptibility in soils is dependent to a large degree on the size of the voids contained within the soil (due to capillary action). Most inorganic soils containing 3% or more of grains finer than 0.02 mm in diameter by weight are frost susceptible in road pavement structures (A 200 mesh sieve has 0.075 mm holes). Table 2: Frost Design Soil Classification, on page 63, lists typical soil types that are frost susceptible. These soils are listed by frost groups. Group F1 is the least susceptible to frost action. Group F4 soils are of especially high frost susceptibility (9).

In order to have detrimental effects from frost action in subsurface soils (nonuniform heave as a result of ice segregation and a period of weakness when melting ice supersaturates parts of the road pavement structure), not only must the soil be frost susceptible, but also the freezing temperatures must penetrate the soil. A source of water must also be available (to become ice). Water sources include high groundwater tables, infiltration, an aquifer, or water held within the voids of fine-grained soils.

Roads in zones II and V of Figure 5, on page 46, often suffer more damage from frost than roads in hard freeze areas because damages occur during the repeated thawing cycles. Road damages can be reduced by replacing the frost-susceptible soils, closing roads to traffic during thaws, or by stabilizing the soil to change its the frost-susceptibility characteristics. Unfortunately, stabilization does not change the frost-susceptibility of of the natural soil below the stabilized layer. Ideally the soil should be stabilized for the full depth of frost penetration.

Design nomographs used for the design of pavement structures attempt to account for environmental differences throughout the country by introducing Regional Factors. The adjustment for climatic and environmental conditions is made in the design nomographs by means of a separate modifying scale. Figure 6: Contours of Equal Regional Factors, on page 49, shows contours of equal Regional Factors. Figure 7: Regional Factor "R", on pages 50 and 51, describes the U.S. Forest Service approach to determine Regional Factors. State Highway Agencies are also sources for appropriate Regional Factors.

Once a Regional Factor has been selected, it can be applied, along with the traffic volume (in equivalent axle loads) and the assumed subgrade strength, to an investigation of pavement thickness. The method for determining pavement thickness is described below, not as an exact art, but as a means to define a range of values that indicate a particular low-volume road pavement thickness will suffice.

PAVEMENT THICKNESS

Pavement design nomographs are intended for use with various combinations of terminal serviceability indexes, total equivalent 18-kip single-axle loads, soil support values and Regional Factors. The terminal serviceability index for low-volume roads is assumed to be a value of two in a range of values from 0 to 5 (five being the best). The terminal serviceability index represents the lowest serviceability that will be tolerated on a high-speed high-volume road before resurfacing or reconstruction is warranted. This criterion is not applicable to low-volume, low-speed roads since it is based on economic evaluations including, among other things, road user's time savings for large numbers of people using the road.

Pavement design nomographs therefore have an input of the four variables: the terminal serviceability index, soil support value, equivalent axle loads, and regional factor. The nomographs output is an abstract number called the weighted

Structural Number (SN). The SN expresses the required structural strength of the pavement. It must be converted to actual thickness of surfacing, base and subbase using layer coefficients representing the relative strength of the material to be used for each layer. The SN is a total value. The layer coefficient is a value assigned to one inch of a material. Various combinations of materials can be evaluated by multiplying their layer coefficients by their proposed thicknesses. The sum of the various layers must be equal to or greater than the weighted structural number for proper design. The seal coat surfacing required to protect asphalt, lime, cement, or lime-fly ash stabilized materials is not considered as a structural component. This approach also holds true for gravel roads that are seal coated.

Various researchers have proposed different structural layer coefficients for various materials. These are all based on results from the American Assoication of State Highway Officials (AASHO, now AASHTO) road test, This test determined the following layer coefficients (3):

Asphaltic concrete surface course	0.44
Crushed stone base course	0.14
Sandy gravel subbase course	0.11

The layer coefficients assumed in this booklet are:

Asphalt-soil roadmix surface	0.20
Asphalt plantmix surface	0.40
Cement-treated or soil cement soil-aggregate base	0.20
Lime-treated soil-aggregate base	0.20
Lime-Fly ash soil-aggregate base	0.25

However, coefficients for many other types of layers are used in design (see Reference 1).

The standard serviceability nomographs for pavement design do not extend into the low traffic volumes found on many of the roads that are suitable for stabilization. Therefore Table 3: Total Equivalent 18-Kip Single Axle Load Applications per Lane, on pages 64 and 65, was developed for this text. The table is formatted as follows:

1) The left hand column lists the lowest Soil Support Values for each subgrade soil code (see Pages 10 and 11). Therefore the table values represent the minimum total equivalent 18-Kip single axle load applications which will reduce the terminal serviceability index to a 2.0 value, this is NOT structural failure (see page 4).

2) The top horizontal row lists the regional factors shown in Figure 6: Contours of Equal Regional Factors, on page 49.

3) The body of the table represents the total equipment 18-Kip single load applications per lane. For a two lane road these values must be doubled.

4) A description of the sections of Table 3, on page 64, follows:

Table 3a, SN = 0.84, indicates the minimum traffic capabilities of 6-inches of crushed stone base with a layer coefficient of 0.14;

Table 3b, SN = 1.20, indicates the minimum traffic capabilities of 6-inches stablilized material with a layer coefficient of 0.20, or 8-inches of 0.15 layer coefficient materials;

Table 3c, SN = 1.60 indicates the minimum traffic capabilities of 8-inches of stabilized material with a layer coefficient of 0.20;

Table 3d, SN = 2.00 indicates the minimum traffic capabilities of 10-inches of stabilized material with a layer coefficient of 0.20 on poor sands and on fine grained materials;

Table 3e, SN = 2.40 indicates the minimum traffic capabilities of 12-inches of stabilized material with a layer coefficient fo 0.20 on fine grained materials;

Other layer coefficients which total the same weighted Structural Number (SN) have the same traffic values.

These tables are not precise, they represent the compilation of a set of variable factors and considerable judgement. Their purpose is to indicate reasonable expectations under the assumed conditions. Any inference drawn from Table 3: <u>Total Equivalent 18-Kip Single Axle Load Applications per Lane</u>, that all stabilizers give the same structural value in all cases is not correct. Table 3 is drawn from conservative average values but, in actual practice, materials and construction methods influence the strength of the stabilized layer to some degree. Any inference that stabilizers can be substituted one for another because Table 3 groups their strength factors together is even less correct. Stabilizer selection depends on the soil to be stabilized. Incorrect stabilizer selection leads to a more costly pavement and can result in failure of the stabilized layer in extreme circumstances. The next section deals in detail with correct stabilizer selection.

STABILIZER SELECTION

The stabilizers considered in this booklet do not react equally well will each soil classification. However, there is considerable overlap in the ability of each stabilizer to react with specific soils. A few soils can be stabilized with any of the stabilizers under consideration, while other soils are best suited to specific stabilizers. This can lead to apparently conflicting advice from sales persons who have vested interests in particular products. Most sales presentations are correct in their assertions that a particular product will stabilize specific soils. In practice, in spite of the overlapping capabilities, certain stabilizers are better suited to certain soils because the reaction is more complete or less quantity of that type of stabilizer is required. However, stabilizing agent cost considerations and construction equipment availability may favor the use of a stabilizer other than that which requires the least quantity.

Stabilizers must be selected for the soil to be stabilized, not the subgrade material under the stabilized layer. Table 3, on page 64, is based on the Soil Support Value of the subgrade on which the stabilized base course is placed. The material actually stabilized can be either in-place material or an imported material. If the material is imported the stabilization procedure must be evaluated for that material. If the material is in-place but different from the subgrade material, the evaluation must be made for the layer of in-place material to be stabilized.

Figure 8: Selection of Stabilizers, on page 52, was developed from the U.S. Air Force Soils Stabilization Index System (12). Figure 8 is based on the sieve analysis and Atterberg Limits described earlier in the Laboratory classification for

for the Unifed Soil Classification System. Table 4: Stabilizing Agents/Soils Classifications, on pages 66 and 67, shows applicable soils types by stabilizer classification.

Stabilization can be applied to the subgrade to provide support for the equipment constructing the rest of the pavement structure, to limit the expansive capabilities of the subgrade soil, or to counter frost heaves. It can be applied to sub-base or base course material to provide structural strength. Base course stabilization can consist of stabilizing in-situ materials; in-place aggregate surfacing materials, or a combination of both in-situ and existing surfacing materials; or an imported soil. The material can be stabilized in-place or in a mixing plant. The most economical soil stabilization is in-place stabilization, however quality control is higher using plant mix and consequently the stabilized material is more homogenous and usually stronger. The added quality is not always necessary for low-volume traffic. In-place stabilization has the additional benefit of maintaining existing roadway elevations and widths without the extra expense of removing the existing material.

A base course of stabilized material requires a surface treatment to prevent ravelling and water intrusion. The surface treatment does not have to add additional strength to the pavement structure if the base course has sufficient strength. Table 3, on page 64, is based on the strengths that can be achieved by careful in-place soil stabilization with no additional strength contributed from the surface treatment. Unstabilized gravel base course material of the type evaluated in Table 3a, on page 64, is granular material that passes exacting specifications.

This type of gravel (i.e., properly graded with less than 5% material passing the No. 200 sieve, etc.) rarely occurs in nature. If pit material is being used as a base course it must be assumed that it does not meet gravel base course specifications. In all probability it contains too many fines which will retain water under any surfacing treatment and become weaker than the base course material evaluated in Table 3a. However, pit material may be well suited for stabilization purposes.

Table 4: Stabilizing Agents/Soil Classifications, on page 66, indicates the types of stabilizing agents most commonly recommended for various soil types. Lime-fly ash stabilization is not included in Table 4 because it involves more variables, namely the type of fly ash used, proportional quantities of lime and fly ash used, and the distance to the source of the fly ash. Lime-fly ash applications are most suited for stabilization of aggregate base material and mixtures of aggregate base and subgrade soils, which are the same soils listed in Table 4, for bituminous stabilizers. Fly ash is a plentiful and generally inexpensive waste product of coal-burning power plants, therefore if an area has ready access to sources of good quality fly ash, the cost of such stabilization may compete favorably with the current costs of bituminous stabilization for specific load bearing capacity requirements.

The selection of the appropriate stabilizing agent for a specific soil type using the cook book approach above does not indicate the amount of agent required, and unfortunately, does not guarantee the selected agent will react properly with a specific soil being evaulated. Therefore additional evaluation must be undertaken during the actual design of the stabilized soil mix.

Each stabilizer has its own climatic limitations which may restrict its use in particular areas, Table 5: Climatic Limitations and Construction Safety Precautions, on page 68, list these limitations and the safety considerations which should be evaluated both during the stabilizer selection process and during the design procedures described below.

DESIGN METHODS

The design of soil stabilization mixes can be quite involved. It is detailed in previous Federal Highway Administration publications: (References 1 and 2). Each stabilizing agent considered here is also represented by an industry association that publishes design manuals offering technical advice for design procedures. These associations employ field representatives to provide technical assistance as do many individual stabilizing agent manufacturers and some distributors. The four major industry associations are:

- The Asphalt Institute
  Asphalt Institute Building
  College Park, Maryland 20740

- Portland Cement Association
  5420 Old Orchard Road
  Skokie, Illinois 60077

- National Lime Association
  3601 N. Fairfax Drive
  Arlington, Virginia 22201

- The National Ash Association, Inc.
  1819 H Street, N.W.
  Washington, D.C. 20006

This booklet will not attempt to describe the complete design procedure for each or any of the design techniques generally accepted within the engineering community. Some general guidelines are provided however to facilitate the use of the accepted design procedures.

Bituminous Stabilization

Bituminous stabilization works best on granular soils. The U.S. Navy (13) recommends that the granular material to be stabilized with asphalt for a base course should contain less than 25% material passing the No. 200 sieve, and should have a plasticity index of less than 6. In addition, the product of the plasticity index and percent passing the No. 200 sieve should be less than 72. These criteria apply to both cutback asphalt and emulsified asphalts used as soil stabilizers.

All bituminous stabilization designs are based on the weight of the asphaltic cement content of the asphaltic cutback or emulsion. The design percentages are by weight of dry aggregate as are weights given in most soils tables such as Figure 4, on page 45, of this booklet. An indication of the percentages of asphaltic residue (by weight) is shown in Table 6: Selection of Asphalt Cement Content, on page 69.

Figure 9: Chart for Determining Cutback Asphalt Requirement, on page 53, is a nomograph published by the U.S. Navy as is Table 7: Emulsified Asphalt Requirements, on page 70. Both give the required asphalt content as a function of aggregate gradation. Each should be modified for porous material such as coral or slag. The total asphalt content should include one fourth more asphalt for whatever portion (P) of the material is porous. For example, if the indicated asphalt content is 5% and 40% of the material is porous (P = 0.4), the total asphalt content = (1+0.25x0.4)x(5.0) = 5.5%. In addition the quantity of emulsified asphaltic residue indicated in Table 7, on page 70, should be increased by 20% for work to be done in localities subject to seasonal severe freezing-thaws.

Figure 10: Determination of Quantity of Cutback Asphalt, on page 54, contains a formula for determing asphaltic residue quantities (14).

Cutback asphalts are made by thinning asphaltic cement with special solvents or oils which begin to evaporate when the cutback is distributed on the soil or aggregate. Rapid-curing cutback asphalts (RC) are made with highly volatile solvents (e.g. naphtha) which evaporate rapidly. Medium-curing cutgack asphalts (MC) are made with slowly volatile solvents (e.g. kerosene), and slow-curing cutback asphalts (SC) are made with relatively non-violatile oils. Slow-curing asphalts are sometimes called road oils. It should be noted that many areas prohibit the use of cutback asphalts for road construction, because of environmental and energy concerns.

There are six viscosity (degree of fluidity) grades of cutback asphalts. The grades used to be known as 0 thru 5, but new grades (which are roughtly comparable) are listed by the lower limit of the viscosity range for each grade. The new grades are 30, 70, 250, 800, 1500, and 3000; however the 1500 grade is not currently being produced. The new designation represents the lower limit of the viscosity range for the grade of cutback while the upper limit of each grade is twice the lower limit (e.g. MC 250 is a Medium-Curing cutback with a viscosity at 140°F (60°C) between 250 and 500 centistrokes. A lower grade number means the cutback is more fluid.

Choice of the proper cutback asphalt grade is related to the fineness of the soils particles and the temperature of the aggregate. Figure 11: Selection of Type of Cutback for Stabilization, on page 55, can be used for this determination. Once that determination is made, Table 8: Asphalt Cutback Composition, on page 71, can be used to determine the solvent percentage in the cutback. The percentage of residual asphalt required, found in either Figure 9 or 10, (on page 53 or page 54), can be converted to the total percent of cutback using the following formula:

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

Emulsified asphalts used for soil stabilization consist of from 57% to 65% asphalt. The exact amount should be confirmed by the supplier. Table 9: Selection of Type of Emulsified Asphalt for Stabilization, on page 71, permits emulsion selection based on the percentage of soil passing a No. 200 sieve and the relative water content of the soil. Figure 12: Approximate Effective Range of Cationic and Anionic Emulsions on Various Types of Aggregates, on page 56, and Figure 13: Classification of Aggregates, on page 57, will assist in the choice of emulsified aspahlt. In many cases however, only one type (Catonic or Anionic) is available from a local supplier. This is usually the type most appropriate for local aggregates and conditions, but this fact should be confirmed by contacting the State Highway Department if the information in this booklet indicates it is not suitable for the soil being evaluated.

Portland Cement Stabilization

Soil stabilization with portland cement can be divided into two categories:

1) Soil-cement which is a hardened material formed by curing a mechanically compacted high density mixture of pulverized soil and measured amounts of portland cement and water. It contains sufficient cement to pass specified durability tests. This implies a major improvement in strength.

2) Cement-modified soil which is an unhardened or semi-hardened mixture of pulverized soil, portland cement and water. This denotes an improvement in engineering behavior without appreciable gains in strength. Cement-modified soil has significantly smaller cement content than soil-cement. Fine grained cement-modified soils are not suitable for base material for low-volume roads.

Several types of cement have been used successfully for cement stabilization. Normal portland cement (Type 1) and air-entrained cement (Type 1A) were used in the past, giving about the same results; but currently Type II cement is favored because of the greater sulfate resistance obtained at approximately the same cost.

Potable water is normally used for cement stabilization, although sea water has given good results. Brakish water, such as swamp water, contains organic materials which interfere with the hardening process. If a local water source is to be used, all laboratory testing should be done on samples that are made from the local water.

A wide range of soils are suitable for cement stabilization. Table 10: Cement Requirements for Various Soils, on page 72, shows the usual range in cement requirements for soil-cement stabilization of different soils in both percent by volume and percent by weight, along with the suggested cement contents to be used in the appropriate moisture-density test and in the wet-dry and freeze-thaw tests. However, well-graded granular materials that possess sufficient fines to produce a floating aggregate matrix have given the best results.

The Air Force (12) has established the following criteria for soils most suitable for portland cement stabilization:

1) P.I. less than 30 for sandy materials.

2) P.I. less than 20 and L.L. less than 40 for fine grained soils.

3) It is desirable to have a minimum of 45% by weight passing the No. 4 sieve in gravel type soils. In addition, the P.I. of the soil should not exceed the number indicated in the following equation:

$$20 + \frac{50 - \text{Fines Content (passing No. 200 sieve)}}{4}$$

Certain types of organic matter, such as undecomposed vegetation may not influence portland cement stabilization adversely but may require additional cement (see foot note on Table 10), on page 72. Other organic compounds may act as hydration retarders and reduce strength. The test for this type of soil problem is conducted by mixing a 10:1 combination (by weight) of soil and cement and testing the pH 15 minutes after mixing. If the pH is at least 12.1 it is probable that organics, if present, will not interfere with normal hardening. pH meters reading up to a pH value of 14 are available from soil testing equipment suppliers and agricultural school equipment suppliers. Item 6 of Figure 14, on page 58, has more details of the proper type and use of the pH meter. The pH meters sold through mail order catalogues for testing flower pot soil are not suitable. pH testing is also required for lime stabilization design as indicated in the next section.

Portland cement stabilization has also been successfully used on a number of miscellaneous materials such as caliche, chert, cinders, shale, etc. The procedure for testing miscellaneous materials is the same as that used for regular soils. Table 11: Average Cement Requirements of Miscellaneous Materials, on Page 73, gives the same information for an assortment of unusual materials as is shown in Table 10, on page 72, for the standard soil classifications.

Shrinkage is a natural characteristic of soil-cement. Shrinkage cracks and reflective cracks in the surface treatment are not the result of structural failure(16). They can be minimized by using a granular soil with a minimum clay content; by compacting the stabilized material at close to the optimum moisture; by using proper subgrade compaction controls with expansive clay subgrades; and by using the highest penetration (softest) residue asphaltic cement commensurate with adequate stability for the ensuing surface treatment. Delaying the final surface treatment as long as possible further reduces reflective cracks. The inclusion of on untreated granular layer between the stabilized base course and the surface treatment will also minimize and delay reflective cracking. Note however that shrinkage cracks are usually transverse with a fairly regular pattern and some longitudinal cracking near the center-line. Wheelpath "alligator" cracking on the other hand is an indication of inadequate design and structural failure.

Optimum moisture of a soil-cement mixture is not necessarily the same as the optimum moisture content of the soil alone. In sands and sandy soils, where the surface area is insufficient to absorb the moisture on the surface of the soil particles, most of the moisture is available for cement hydration. At the optimum moisture content (needed to achieve optimum density in the soil), therefore the amount of water required for cement hydration is more than needed, causing a reduction in strength. Clayey soils, however, give maximum strengths at densities slightly above the optimum moisture because of their large surface area.

Lime Stabilization

The designation of lime used for lime stabilization should not be confused with the various types of limestone available, such as lime rock or ground or pulverized limestone fractions. Lime used for stabilization is burnt lime (i.e. either quick or hydrated lime). This type of lime is also derived from limestone as either the oxide or hydroxide of calcium or calcium-magnesium, but it reacts differently from calcium carbonate (limestone) in the soil. It appears that the properties of the soil being stabilized may have a much greater influence on the soil-lime reaction than the lime type or source. In general the use of either high-calcium or mono-hydrated dolomitic lime is satisfactory for soil stabilization.

When lime is added to a fine-grained soil several reactions can begin. Cation exchange and flocculation reactions produce an immediate change in the soils plasticity (i.e. reduced plastic index, increased shrinkage limit), its workability (i.e. increased ease of subsequent manipulation, placement and compaction) and its uncured strength and load-deformation properties. The uncured mixture's swell potential is also significantly reduced, making lime a very effective additive for expansive soils stabilization. The effects of these reactions are very important during the construction phase of stabilization, but they do not improve the soil's strength substantially so they are said to modify the soil.

In certain soils a pozzolanic reaction also occurs. This reaction results in an increased mixture strength and durability that is gradual but continuous for long time periods, sometimes as long as several years. Soils in which this pozzolanic reaction takes place are termed reactive soils. They are soils that react with lime to produce an increase in strength of greater than 50 psi following 28 days of curing at 73°F. The cured effects of lime stabilized of reactive soils vary according to the soil type, lime type, lime percentage, compacted density, and the time- temperature curing conditions. If a soil is non-reactive (less than 50 psi in 28 days), it will not develop pozzolanic strength regardless of the lime type or percentage, or the time-temperature conditions.

All of the above reactions occur only between the fines portion of the soil and lime. Therefore fine grained soils have the most favorable response to lime. A minimum clay content of about 10% and a plasticity index of greater than 10 are acceptable indicators for soils that may be suitable for lime stabilization.

Table 12: Approximate Lime Contents, on page 74, shows the ranges of hydrated lime or quick lime for selected soil types. The ranges in this table are quite large. A closer estimation of the lime content required for stabilizing specific soils can be made using a pH meter of the type mentioned in the section on Portland Cement stabilization. This test, called the Eades and Grim Procedure, results in a lime percentage which is approximately the same as the lime percentage producing maximum compressive strength. The test, which takes about an hour to complete, does not determine whether or not a soil is reactive. It is based on the premise that adding sufficient lime to the soil to insure a pH of 12.4 will sustain the strength-producing, lime-soil pozzolanic reaction. The resulting lime content will stabilize reactive soils but merely modify non reactive soils. The pH test is summarized in Figure 14: Eades and Grim Procedure, on page 58.

Lime-Fly Ash Stabilization

Fly ash consists of very small, separate particles found in stack gas resulting from the burning of coal, lignite or like materials. It is collected by mechanical devices such as cyclonic or bag house collectors or with electrostatic precipitators. Its characteristics are largely determined by the type of coal burned, the type of furnace used, the type of air quality control equipment employed, and the method of handling the fly ash.

Fly ash can be stored dry in protected storage structures, in a dampened state referred to as conditioned fly ash, or in a storage pond. Dry fly ash is chemically and physically stable and will not change with time. Conditioned fly ash may take on a set and consequently require further processing (crushing), but if it does not set up it is also chemically and physically stable. It can be stored indefinitely and used without further processing.

Fly ashes stored in ponds usually segregate by size and may undergo chemical reactions. They are usually not suitable for use in lime-fly ash mixes except as mineral fillers.

Lime-fly ash is most suitable for stabilizing coarse grained materials such as sands, gravels, crushed stone, and several types of slags. Some fine grained soils have also been successfully treated with lime and fly ash, most notably silts. Fly ashes are normally used in lime-fly ash mixes as a pozzolan and as a filler for the voids. The fly ash particle size is normally larger than the voids in fine grained soils so that the stabilization role for fly ash in fine grained soils is solely as a pozzolan.

As indicated in the previous section on lime-stabilization, the pozzolanic reaction increases strength and durability over time. This cementing action is a function of the soil type, the properties of the fly ash, the proportions of the ingredients, the processing technique and the moisture content, field density, and curing conditions. Like lime stabilization (as opposed to lime modification) strength and durability in lime-fly ash stabilized soils increases over a long period of time, even when the curing cycle is interrupted by cold weather.

The relative proportions of lime to fly ash to soil used in stabilization mixes (also termed pozzolanic-aggregate mixtures) varies considerably, in part due to the additional variables induced by the third ingredient, fly ash. Lime content usually varies from 2 to 8 percent, while the fly ash content varies from 8 to 36 percent and the lime to fly ash ratio varies from 1:2 to 1:7. The most typical proportions are 2-1/2 to 4 percent lime and 10 to 15 percent fly ash. The most common ratios of lime to fly ash are 1:3 to 1:4. It is improtant when designing a lime-fly ash mixture that the ultimate mixture have sufficient fines to provide the amount of bonding surface area needed per unit volume to produce a sound mixture. As a general guide, the final mix should contain a minimum of 50% passing the No. 4 sieve; the minus 4 material can be a combination of fly ash, as filler, plus aggregate fines (19).

Corrections to Mixed-in-Place Design Mixtures

Mixes prepared in the laboratory are always better controlled than field mixes. In-place mixing is less uniform and application rates are not always exact. In-place mixing efficiency, as measured by the strength of the treated soil, may be only 60% to 80% of that obtained in the laboratory. This reduced efficiency is often accounted for by an adjustment increasing laboratory determined stabilizer content by one or two percent. When using a lime-fly ash combination the lime plus fly ash content should be increased by about 2% and the lime content by about 1/2 percent; (e.g. the additional 2% increase is made at a 1:4 lime to fly ash ratio).

In spite of the most careful selection and evaluation of both stabilizer type and application rate, proper construction technique and control are still essential to the success of the end product. Construction follows logical procedures but is certainly not a high technolgy exercise. However, neglecting to complete each step in its proper sequence and in the time alloted can cause irreparable damage to some soil stabilizing exercises. Any engineer who accepts responsibility for designing a stabilized low-volume road and does not offer guidance and some quality control methodology for its construction is risking a failure that need not occur.

CONSTRUCTION

The key to successful soil stabilization is to achieve a thorough mixture of a pulverized soil or aggregate with the correct amount of stabilizer and enough moisture to obtain maximum compaction. There must be favorable temperature and moisture conditions for strength development during the curing period and the stabilized material must be protected from traffic, both to prevent abrasion and to ensure adequate time for strength development.

Either mixed-in-place or central plant mixing are viable operations. The method selected will depend on local job conditions and equipment availability. Whatever type of equipment is available, the general construction principles and procedures are the same. Figure 15: <u>Soil Stabilization Construction Equipment</u>, on page 60, defines construction methods by type of stabilized soil mixing. Table 13: <u>Equipment Typically Associated with Mixed-In-Place Subgrade Stabilization Operations</u>, on page 75, outlines in greater detail the construction operation for stabilizing existing soil, whether it be natural soil; existing granular or other surface; or imported material to be mixed-in-place by itself or in combination with in-situ material. Mixed-in-place construction typically follows the following stabilization operations (1):

1) Soil Preparation - The soil is brought to the proper line and grade. The material is scarified to the specified width and depth of the stabilized layer and partially pulverized. A grader-scarfier, bull- dozer-scarfier and/or disc harrow can be used for scarification. A disc harrow or rotary mixer can be used for pulverization. If the soil is too dry, water should be added to aid pulverization. If it is extremely wet, the harrow or mixer can be used to aerate and dry the soil. Plows, various types of cultivators, and other agricultural equipment can be substituted for the normal highway construction equipment for the soil preparation operation.

2) Stabilizer Application - Asphalt is spread or distributed from an asphalt distributor or directly through a travelling mixing machine during the mixing process. The soil must be at the proper moisture content (not more than 3%) prior to asphalt application to achieve uniform mixing. Incremental asphalt applications and mixer passes are often necessary to achieve the specified mixture.

Cement and lime can be distributed dry by spotting bags on the roadway, by spreading from suitably equipped self-unloading bulk transport trucks, or by mechanical spreaders loaded from bulk hauling units. Lime can also be spread as a slurry through tank truck spray bars. The slurry usually consists of about one ton of hydrated lime to 500 gallons of water.

A double application of lime may be required when stabilizing extremely plastic clays (P.I.>50). Lime is added in two increments to permit adquate pulverization and uniform mixing. Normally 2 to 3% lime is added, partially mixed, and lightly rolled to seal the surface. After one to two days the mix is repulverized, the second lime increment is added, and the mixing is completed(20).

Lime and fly ash are spread separately in most lime-fly ash stabilization projects. Lime is spread as described above. Fly ash is normally spread in the conditioned state (i.e. with a moisture content of 15 to 25%). It is delivered in open dump trucks, dumped, and spread by a grader or spreader box.

The primary objective of stabilizer application is to obtain uniform distribution of the proper proportion of the stabilizer. The mixing operation will NOT improve distribution uniformity.

3) Pulverization and Mixing - Single-and multiple-shaft rotary (flat type) mixers are commonly used to pulverize and mix asphalt, cement, lime, and lime-fly ash with the prepared soil. Motor graders and agricultural equipment can be used, but uniform mixing is sometimes difficult to achieve with such equipment.

Asphalt stabilization requires several repetitions of asphalt distribution and mixing when flat-type rotary mixers and motor graders are used. Mixing the asphalt with soil and water should continue until a uniform mixture (all the same color) is obtained.

Cement stabilization mixing must continue until the fine grained soils are pulverized enough so that at the time of compaction 100% of the soil-cement mixture will pass the one-inch sieve and a minimum of 80% will pass the No. 4 sieve, exclusive of any gravel or stone.

Lime stabilization pulverizing and mixing should continue until 100% of the solid binder passes a one-inch sieve and at least 60% passes the No. 4 sieve. The proper water content for compaction must be incorporated during the pulverizing and mixing of the lime and soil. The pulverization and mixing requirements for lime-fly ash is the same as for lime stabilization, but uniform mixing is even more important because the stabilizers must be uniformly blended.

4) Compaction - Compaction should always be sufficient to reach the required density if the stabilized soil is to perform as expected.

Emulsified asphalt mixtures should be compacted as soon as the emulsion begins to break (this is indicated by a marked color change from brown to black)(21). Also at this time the mixture should be able to support the roller without undue displacement. Cut back asphalt mixtures should be properly aerated before compaction. Correct aeration is achieved when the volatile content is reduced to about 50% of the original content and the moisture content does not exceed 2% by weight of total mixture. This may occur almost immediately in open graded mixes and may take as long as a day in dense graded mixtures and cool temperatures. Trial rolling can be used to determine the proper moisture and volatile contents when test data are not available. Undue lateral movement (shoving) should not take place under the roller. Since asphalt stabilized materials are granular, pneumatic, steel wheel, and vibratory rollers can be used.

Cement stabilized mixtures should be compacted as soon as possible after the water is applied and thoroughly mixed with the previously pulverized and mixed soil-cement material. Most specifications require that the materials be compacted within four hours of mixing but less compactive effort is necessary for the same amount of compaction and there is less water evaporation if the material is compacted within an hour of adding and mixing water. The same type of roller should be used on cement stabilized soil as would be used on the soil alone(22). Cement stabilization of fine-grained soils sometimes uses sheepsfoot rollers while granular cement stabilized materials are rolled with pneumatic, steel wheel, and vibratory rollers.

Lime stabilized soil should be compacted shortly after uniform mixing is achieved. However since the pozzolanic action is long term, additional time is available for mixing and pulverizing lime stabilized soil. If proper pulverizing is extremely difficult, the mixture can be lightly rolled and allowed to mellow for one or two days, after which it can be repulverized and remixed without harm. Sheepsfoot rollers are commonly used for compacting fine-grained lime stabilized soils. After the sheepsfoot roller has walked out, pneumatic rollers are used for surface compaction, sometimes followed by steel wheel rollers for final finishing. More granular soils may be initially compacted with vibrating impact rollers or heavy pneumatic rollers.

Lime-fly ash mixtures should be compacted as soon as possible with compaction completed within four hours. Since lime-fly ash materials are basically granular in nature with little or no cohesion at the time of compaction, pneumatic and vibratory rollers are used for initial compaction(19). As with other stabilized soil mixtures, the final surface is usually brought to grade with a motor grader prior to final rolling with a steel wheeled roller.

5) Curing - Proper curing of asphalt stablized soils involves the further loss of volatile material. If traffic must travel over these stabilized materials during the curing period, a sand or aggregatre seal should be placed over the stabilized

mixture. The final asphalt seal or wearing surface should not be placed for at least seven days.

Proper curing of cement, lime, and lime-fly ash stabilized soils involves a gain of strength that is dependent on time, temperature, and the presence of moisture. The stabilized layer may be sprinkled with water at frequent intervals to prevent moisture loss. The perferred method however is to seal the damp surface with single application of cutback asphalt (at 0.10 to 0.25 gal/sq. yd.) within a day of final rolling. Emulsified asphalt sealing must be done incrementally during the curing period. If traffic is allowed on a curing membrane, a sand coat must be applied and the traffic limited in weight and speed.

The final asphalt surface treatment, which is required to prevent ravelling and to provide waterproofing, can be applied instead of the curing membrane described above. However, during the first week of the stabilizer's curing period, no traffic heavier than a pneumatic roller should be allowed on the surface treatment.

Once the process of curing is well underway and traffic is using the roadway with no apparent detrimental effects, the engineer's job is considered a success requiring no further input. However, stabilized low-volume roads are no more immune to the need of proper timely periodic maintenance than are any other roads. An appropriate time to stress this fact is during the final inspection of the completed project while the concerned officials are viewing the road in the condition they hope will be typical for years to come.

Low-Volume Roads Definition

Low-Volume Roads are Service Roads in a Particular Area

Designed and Constructed with Minimum Serviceability Requirements

As Necessary and Sufficient to Enable All Vehicles Common to the Area

To Travel Unassisted and Safely with Reduced Priority for Speed and Comfort

Figure 2

UNIFIED SOIL CLASSIFICATION SYSTEM (Including field identification procedures)
UNITED STATES CORPS OF ENGINEERS AND BUREAU OF RECLAMATION

Major Divisions			Group Symbols	Typical Names	Field Identification Procedures (Excluding particles, larger than 3 inches and basing fractions on estimated weights)
1	2		3	4	5
Coarse-Grained Soils More than half of material is larger than No. 200 sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels More than half of coarse fraction is larger than No. 4 sieve size.	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Wide range in grain sizes and substantial amounts of all intermediate particle sizes
			GP	Poorly-graded gravels gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing
		Gravels with Fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures, see ML below).
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see CL below).
	Sands More than half of coarse fraction is smaller than No. 4 sieve size.	Clean Sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain size and substantial amounts of all intermediate particles sizes.
			SP	Poorly-graded sands, gravelly sands little or no fines.	Predominantly one size or a range of a range of sizes with some intermediate sizes missing.
		Sands with Fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).
			SC	Clayey sands, sand-clay mixtures	Plastic fines (for identification procedures see CL below).

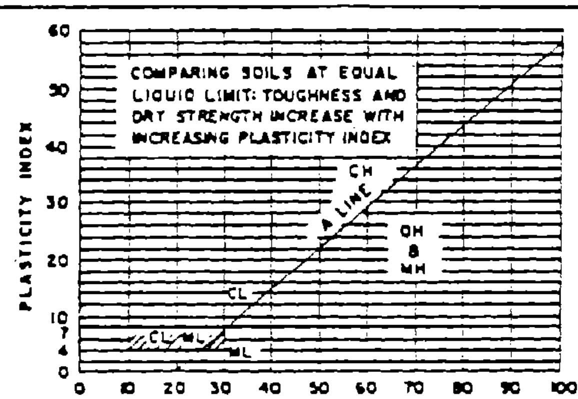
Major Divisions		Group Symbols	Typical Names	Identification Procedures on Fraction Smaller than No. 40 Sieve Size: Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near PL)
Fine-Grained Soils More than half of material is smaller than No. 200 sieve size.	Silts and Clays Liquid Limit less than 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	None to Slight	Quick to slow	None
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Medium to high	None to very slow	Medium
		OL	Organic silts and organic silty clays of low plasticity	Slight to medium	Slow	Slight
	Silts and Clays Liquid Limit greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Slight to medium	Slow to none	Slight to medium
		CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High
		OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium
Highly Organic Soils		Pt	Peat and other highly organic soils.	Readily identified by colour, odour, spongy feel and frequently by fibrous texture		

(1) Boundary Classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

Figure 2 (Contd.)

UNIFIED SOIL CLASSIFICATION SYSTEM (Including field identification procedures)
UNITED STATES CORPS OF ENGINEERS AND BUREAU OF RECLAMATION

Information Required for Describing Soils		Laboratory Classification Criteria	
6	7		
For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics. Give typical name; indicate approximate percentages of sand and gravel, max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses. Example: Silty sand, gravelly; about 20% hard, angular gravel particles 1/2 in. maximum size; rounded and subangular sand grains coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM).	Use grain-size curve in identifying the fractions as given under field identification.	Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5% GW, GP, SW, SP, More than 12% GM, GC, SM, SC 5% to 12% Borderline cases requiring use of dual symbols	$C_u = \frac{D_{60}}{D_{10}}$ greater than 6 $C_c = \frac{(D_{60})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Not meeting all grading requirements for GW
			Atterberg limits below "A" line or PI less than 4 / Atterberg limits below "A" line with PI greater than 7 — Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			$C_u = \frac{D_{60}}{D_{10}}$ greater than 4 $C_c = \frac{(D_{60})^2}{D_{10} \times D_{60}}$ Between 4 and 3
			Not meeting all grading requirements for SW
			Atterberg limits below "A" line or PI less than 4 / Atterberg limits above "A" line with PI greater than 7 — Limits plotting in hatched zone with PI between 4 and 7 are borderline cases requiring use of dual symbols.
Give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains, colour in wet condition, odour, if any, local or geologic name, and other pertinent descriptive information; and symbol in parentheses. For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions. Example: Clayey silt, brown, slightly plastic, small percentage of fine sand, numerous vertical root holes, firm and dry in place, loess (ML).			



LIQUID LIMIT
PLASTICITY CHART
For laboratory classification of fine grained soils

(2) Procedures for determination of Dry Strength, Dilatancy and Toughness are set out on next page.
(3) Sieve Sizes shown are U.S. Standard.

Figure 2 (Contd.): Field Identification Procedures for Fine-Grained Soils or Fractions.

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to Shaking)

After removing particles larger than No. 40 sieve size, prepare a patty of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the patty in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the patty which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the patty stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typcial rock flour, show a moderately quick reaction.

Dry Strength (Crushing characteristics)

After removing particles larger than No. 40 sieve size, mould a patty of soil to the consistency of putty, adding water if necessary. Allow the patty to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbing between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritting whereas a typical silt has the smooth feel of flour.

Toughness (Consistency near plastic limit)

After removing particles larger than No. 40 sieve size, a specimen of soil about one-half inch cube in size is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin type clays and organic clays which occurs below the A-line.

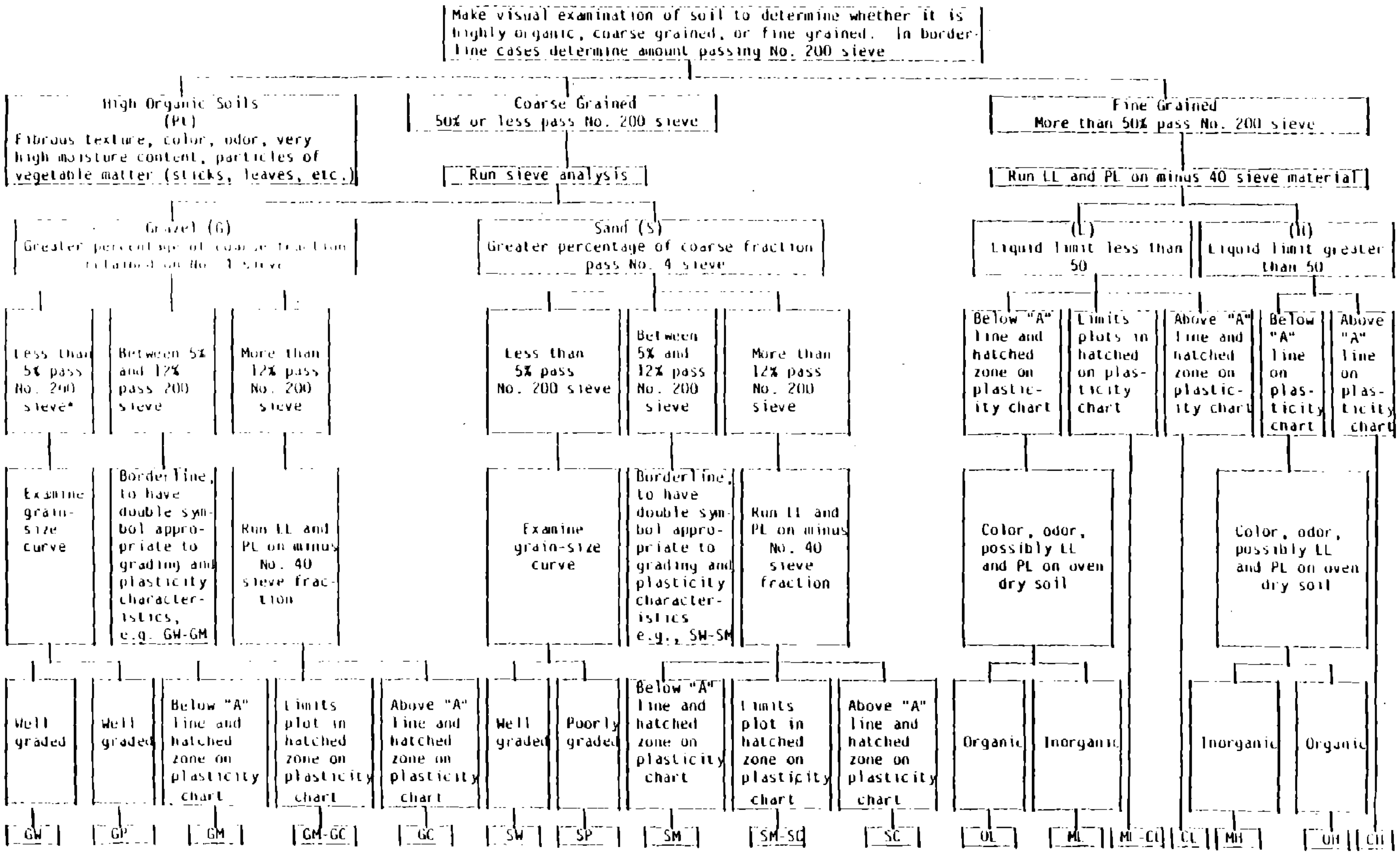
Highly organic clays have a very weak and spongy feel at the plastic limit.

NOTES: (For Laboratory Classification

Cu = uniformity coefficient
Cc = coefficient of curvature
D_{60} = grain diameter at 60% passing
D_{30} = grain diameter at 30% passing
D_{10} = grain diameter at 10% passing

The grain-size distribution of well-graded materials generally plot as smooth and regular concave curves with no sizes lacking or no excess of material in any size range. The uniformity coefficient (C_u) of well-graded gravels is greater than 4, and of well-graded sands is greater than 6.

Figure 3: Chart for Laboratory Identification procedure. (From Corps of Engineers). Source (4)



Note: Sieve sizes are U.S. Standard.

* If fines interfere with free-draining properties use double symbol such as GW-GM, etc.

SOIL CHARACTERISTICS

Major Divisions (1)	(2)	Ltrs. (3)	Name (4)	Foundation Value When Not Subject to Frost Action (5)	Value As Base Direct Under Bituminous Pavement (6)	Potential Frost Action (7)	Compressibility and Expansion (8)	Drainage Characteristics (9)	Compaction Equipment (10)	Unit Weight lb/ft (11)	Field CBR (12)
Coarse Grained Soils	Gravel and Gravelly Soils	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125-140	60-80
		GP	Poorly graded gravels or gravel sand mixtures, little or no fines	Good to Excellent	Poor to fair	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel wheeled roller	110-130	25-60
		GM	Silty gravels, gravel-sand-silt mixtures*	Good to Excellent	Fair to good	Slight to medium	Very Slight	Fair to Poor	Rubber-tired equipment, sheepsfoot roller; close control of moisture	130-145	40-80
				Good	Poor	Slight to Medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120-140	20-40
		GC	Clayey gravels, gravel-sand-clay mixtures	Good	Poor	Slight to Medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120-140	20-40
	Sand and Sandy Soils	SW	Well-graded sands or gravely sands, little or no fines	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110-130	20-40
		SP	Poorly graded sands or gravelly sands, little or no fines	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100-120	10-25
		SM	Silty sands, sand-silt mixtures*	Good	Poor	Slight to high	Very slight	Fair to Poor	Rubber-tired equipment, sheepsfoot roller; close control of moisture	120-135	20-40
				Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105-130	10-20
		SC	Clayey sands, sand-clay mixtures	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105-130	10-20

Figure 4: Page 2 of 2

SOIL CHARACTERISTICS

Major Divisions (1)	(2)	Ltrs. (3)	Name (4)	Foundation Value When Not Subject to Frost Action (5)	Value As Base Direct Under Bituminous Pavement (6)	Potential Frost Action (7)	Compressibility and Expansion (8)	Drainage Characteristics (9)	Compaction Equipment (10)	Unit Weight lb/ft (11)	Field CBR (12)
Fine grained Soils	Silts and Clays LL<50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Fair to poor	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller; close control of moisture	100-125	5 15
		CL	Inorganic clays of low medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Fair to poor	Not suitable	Medium to High	Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100-125	5-15
		OL	Organic silts and organic clays of low plasticity	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90-105	4-6
	Silts and Clays LL>50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils elastic silts	Poor	Not suitable	Medium to very high	High	Fair to poor	Sheepsfoot roller	80-100	4-8
		CH	Inorganic clays of high plasticity, fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller	90-110	3-5
		OH	Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller	80-105	3-5
High Organic Soils		Pt	Peat and other highly organic soils	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical		

Source: (6)

*Material above dashed line
LL > 25, PI > 5

Figure 5

SIX CLIMATIC REGIONS IN THE UNITED STATES FOR USE IN HIGHWAY TECHNOLOGY

As defined by University of Illinois

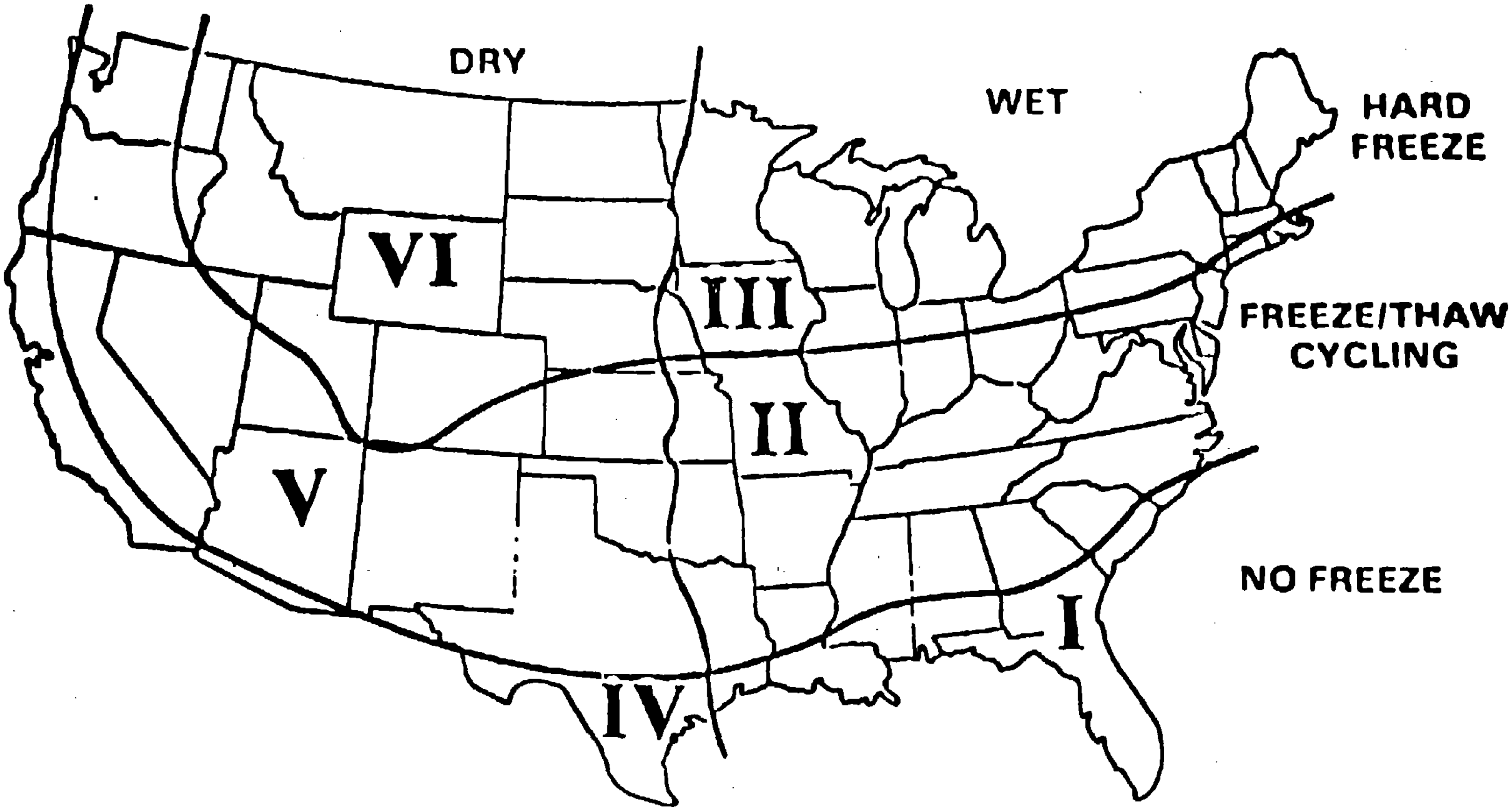


Figure 5 (Contd.)

SIX CLIMATIC REGIONS IN THE UNITED STATES FOR USE IN HIGHWAY TECHNOLOGY

As defined by University of Illinois

	NO FREEZE	FREEZE/THAW CYCLING	HARD FREEZE
WET	I	II	III
DRY	IV	V	VI

Thornthwaite Index = Zero

Freezing penetrates 5 inches

Freeze Index endures 60 days per year

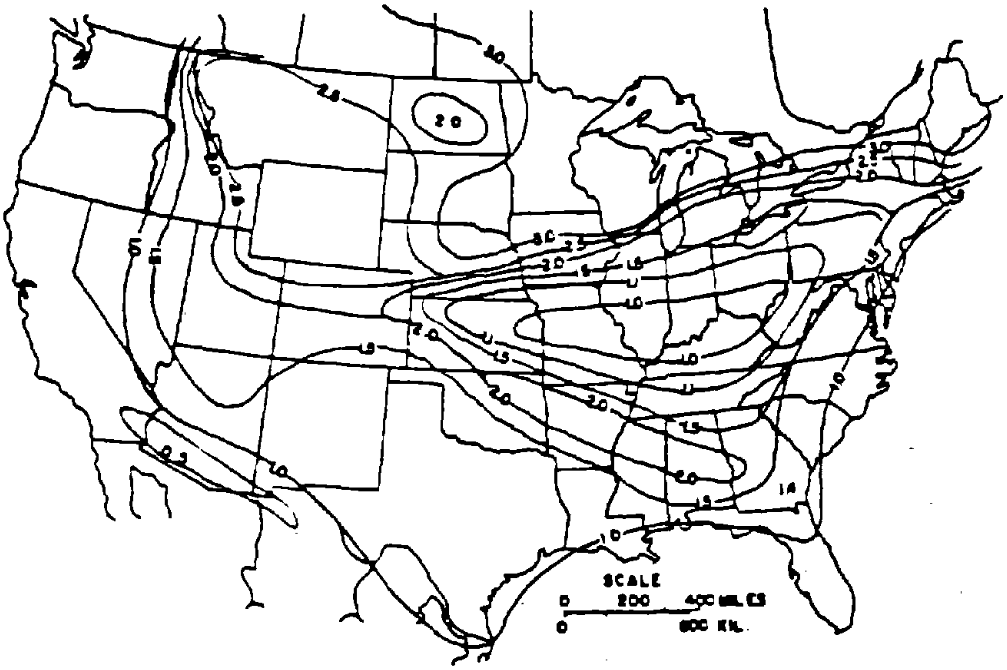


FIGURE 6: CONTOURS OF EQUAL REGIONAL FACTORS

SOURCE: (4)

Figure 7

Regional Factor "R"

The Forest Service suggests the following method for determing the regional factor. It is included because it shows the variables involved in determining the regional factor and their solution for specific sites.

Step 1. Use base "R" of 1

Step 2. Add to Base "R" the appropriate value for each column

Added Value	Annual Percipitation (Inches)	Average % Grade
+0.1	50-60	7-8
0.2	60-70	8-9
0.3	70-80	9-10
0.4	80-90	10-11
0.5	90-100	11-12
0.6	>100	>12

Step 3. For any soil which swells over 3% add 0.5 to the Base "R".

Step 4. In frost where frost penetration does not exceed 10 inches in frost susceptible soils, where the drainage is adequate to keep the water Table 3 feet below the top of the subgrade, where the subgrade is covered with a layer of stabilized soil, and where snow is removed from the road surface:

Add 0.5 to the Base R if subgrade is CL or CH soil.

Add 1.0 to the Base R if subgrade is SMu, ML, or MH.

However, if snow is not removed from surface the following should be used instead:

Add 0.4 to the Base R if subgrade is CL or CH soil.

Add 0.7 to the Base R if subgrade is SMu, ML, or MH.

Step 5. If the road surface does not include shoulders of two feet more more add 0.3 to the Base R.

The result of the above five steps is the Regional Factor for the site under consideration.

Source: (11)

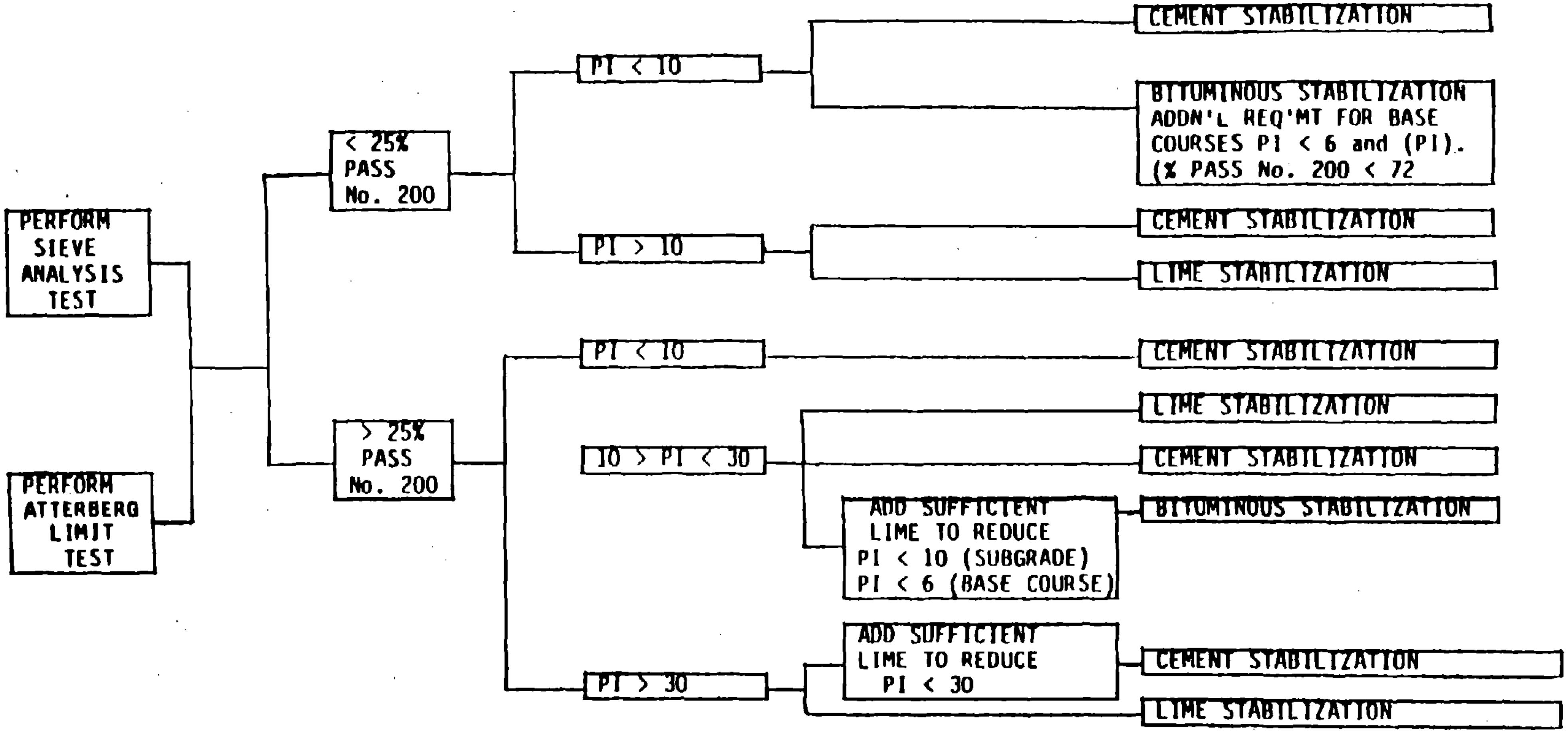


Figure 8. Selection of Stabilizers

Source: (1)

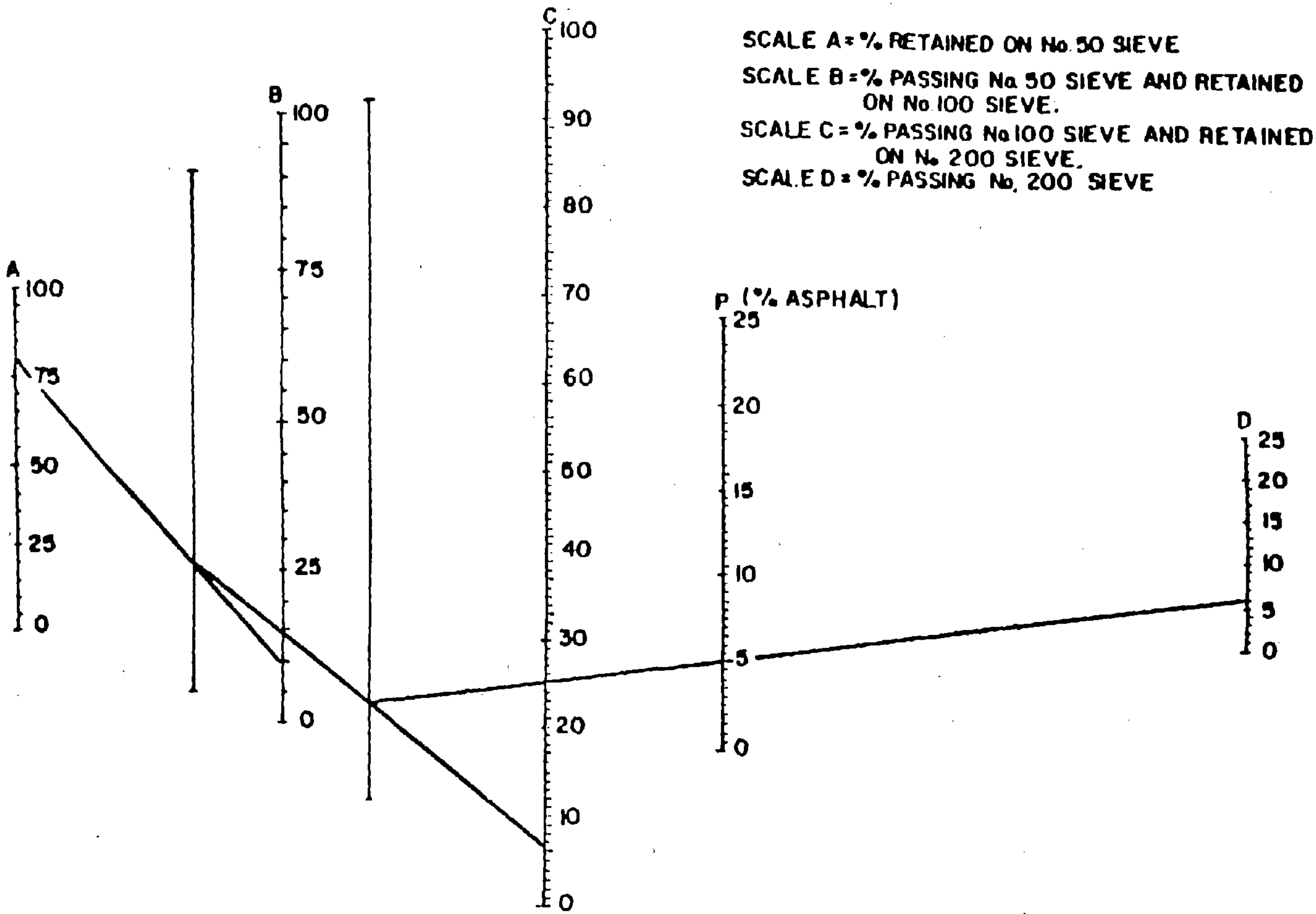


FIGURE 9 - CHART FOR DETERMINING CUTBACK ASPHALT REQUIREMENT

SOURCE (13)

Figure 10: Determination of Quantity of Cutback Asphalt

1. Formula for Determination of Estimated percent of Cutback Asphalt Requirement.

$$p = 0.02a + 0.07b + 0.15c + 0.20d$$

Where p = % of asphalt material by wt. of dry aggregate

a = % of mineral aggregate retained on No. 50 sieve

b = % of mineral aggregate passing No. 50 and retained on No. 100 sieve

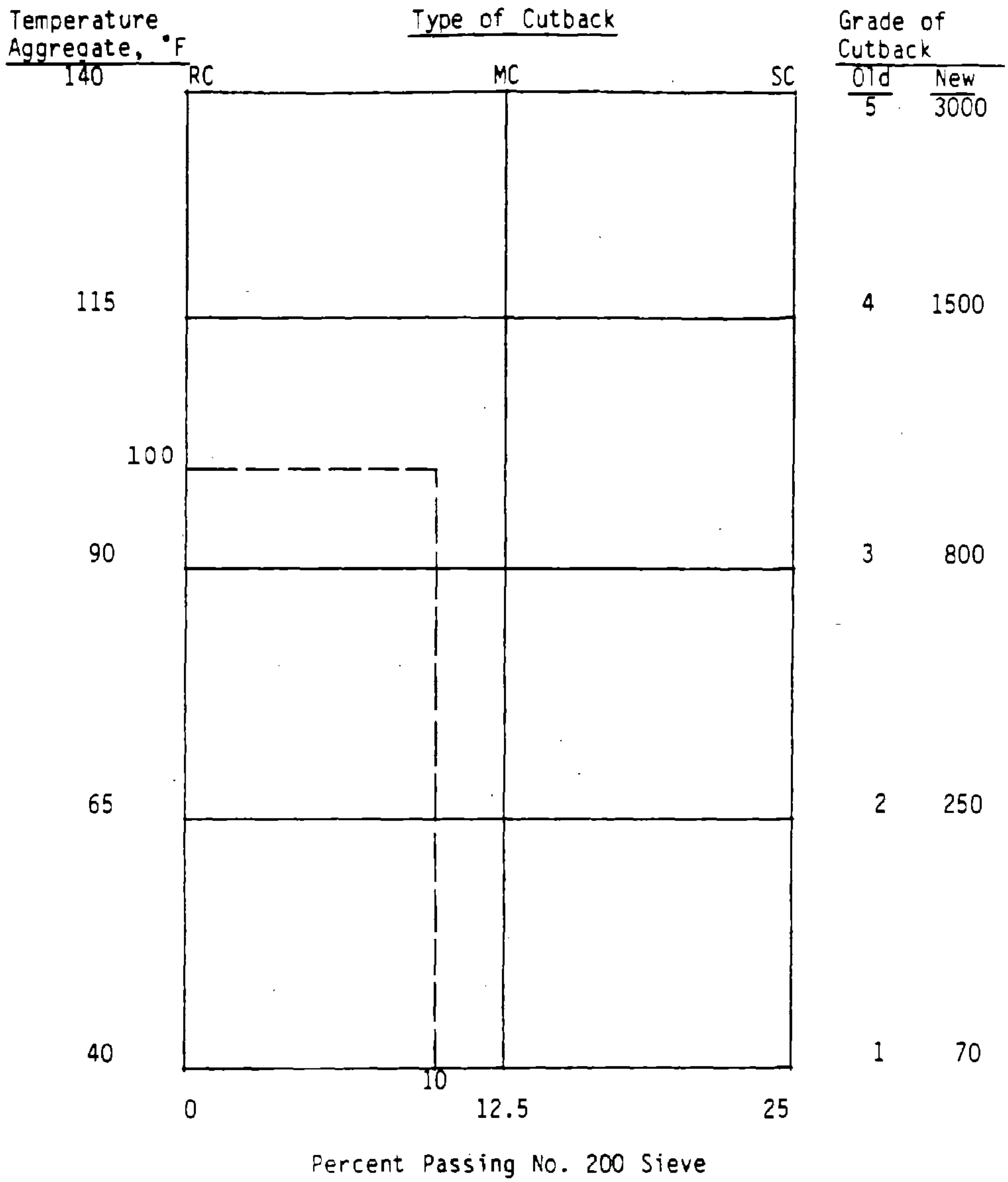
c = % off mineral aggregate passing No. 100 and retained on No. 200 sieve

d = % of mineral aggregate passing No. 200 sieve.

Note: All percentages are expressed as whole numbers, absorptive aggregates - such as slag, limerock, vesicular lava and coral - will require additional asphalt.

Source: (14)

Figure 11: Selection of Type of Cutback for Stabilization



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

Source: (13)

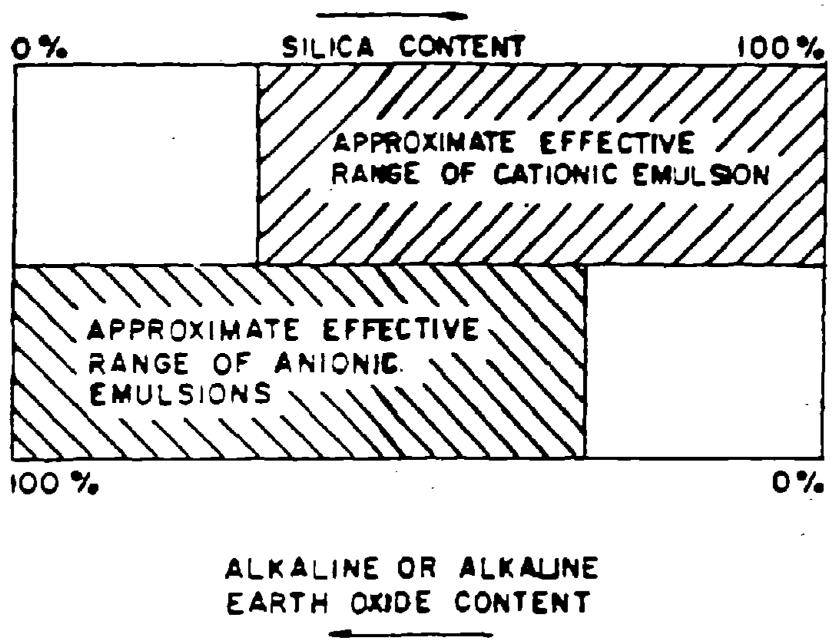
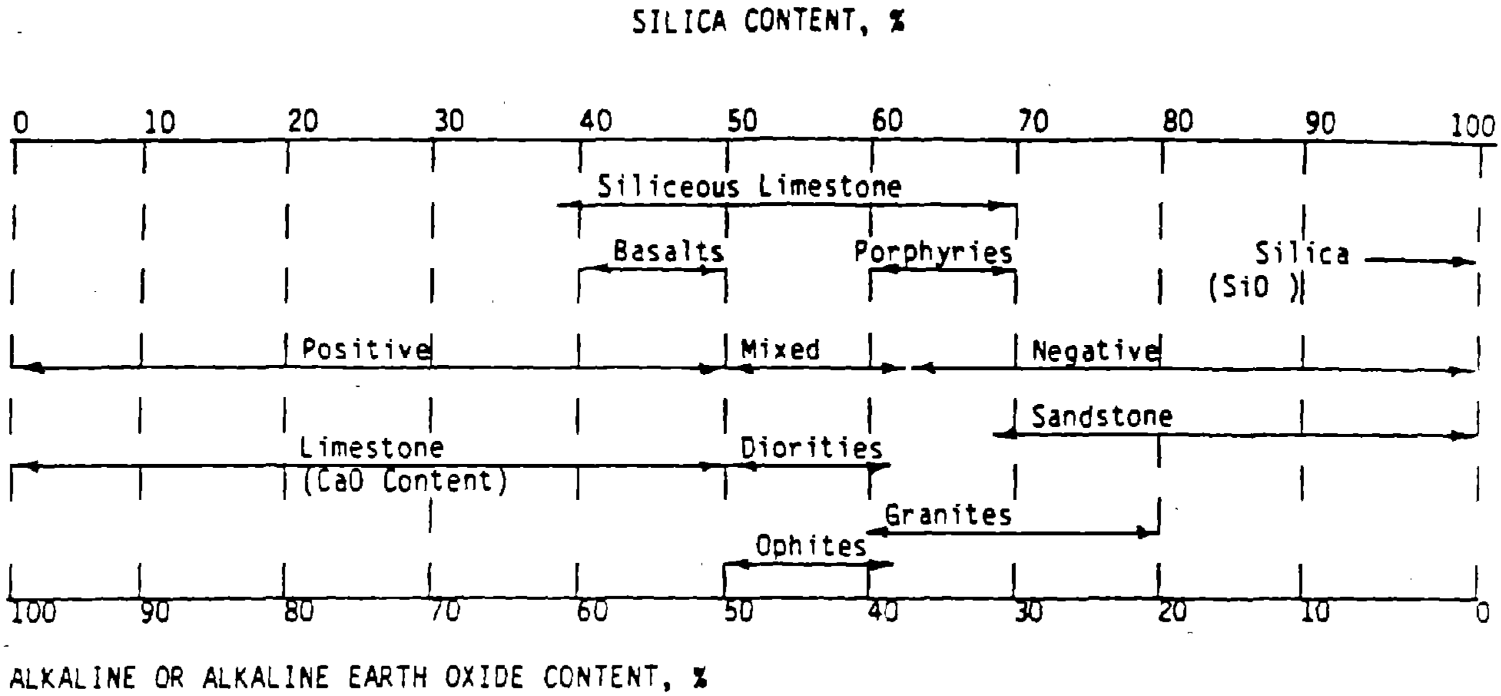


FIGURE 12 APPROXIMATE EFFECTIVE RANGE OF CATIONIC AND ANIONIC EMULSIONS ON VARIOUS TYPES OF AGGREGATES.

SOURCE: (2)

Figure 13: Classification of Aggregates



Source: (2)

Figure 14: Eades and Grim Procedure

1. Representative samples of air-dried, minus No. 40 sieve soil to equal 20 grams of oven-dried soil are weighed to the nearest 0.1 g and poured into 150 ml (or larger) plastic bottles with screw tops.

2. Since most soils will require between 2 and 5 percent lime, it is advisable to set up five bottles with lime percentages of 2, 3, 4, 5, and 6. This will insure, in most cases, that the percentage of lime required can be determined in one hour. Weigh the lime to the nearest 0.01 g and add it to the soil. Shake to mix the soil and dry lime.

3. Add 100 ml of CO_2-free distilled water to the bottles.

4. Shake the soil-lime and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.

5. Shake the bottles for 30 seconds every 10 minutes.

6. After one hour, transfer part of the slurry to a plastic beaker and measure the pH. The pH meter must be equipped with a Hyalk electrode and standardized with a buffer solution having a pH of 12.00.

7. Record the pH for each of the soil-lime mixtures. If the pH readings go to 12.40, the lowest percent lime that gives a pH of 12.40 is the percentage required to stabilize the soil. If the pH does not go beyond 12.30 but at least two consective percentages of lime give the same reading, the lowest percentage which gives a pH of 12.30 is that required to stabilized the soil. However, if only the highest percentage checked gives a

pH of 12.30, additional test bottles should be started with larger percentages of lime.

Source: Transportation Research Circular 180, September 1976, TRB as modified and published in Compendium 8, Chemical Soil Stabilization, Transportation Technology Support for Developing Countries, Transportation Research Board, 1979.

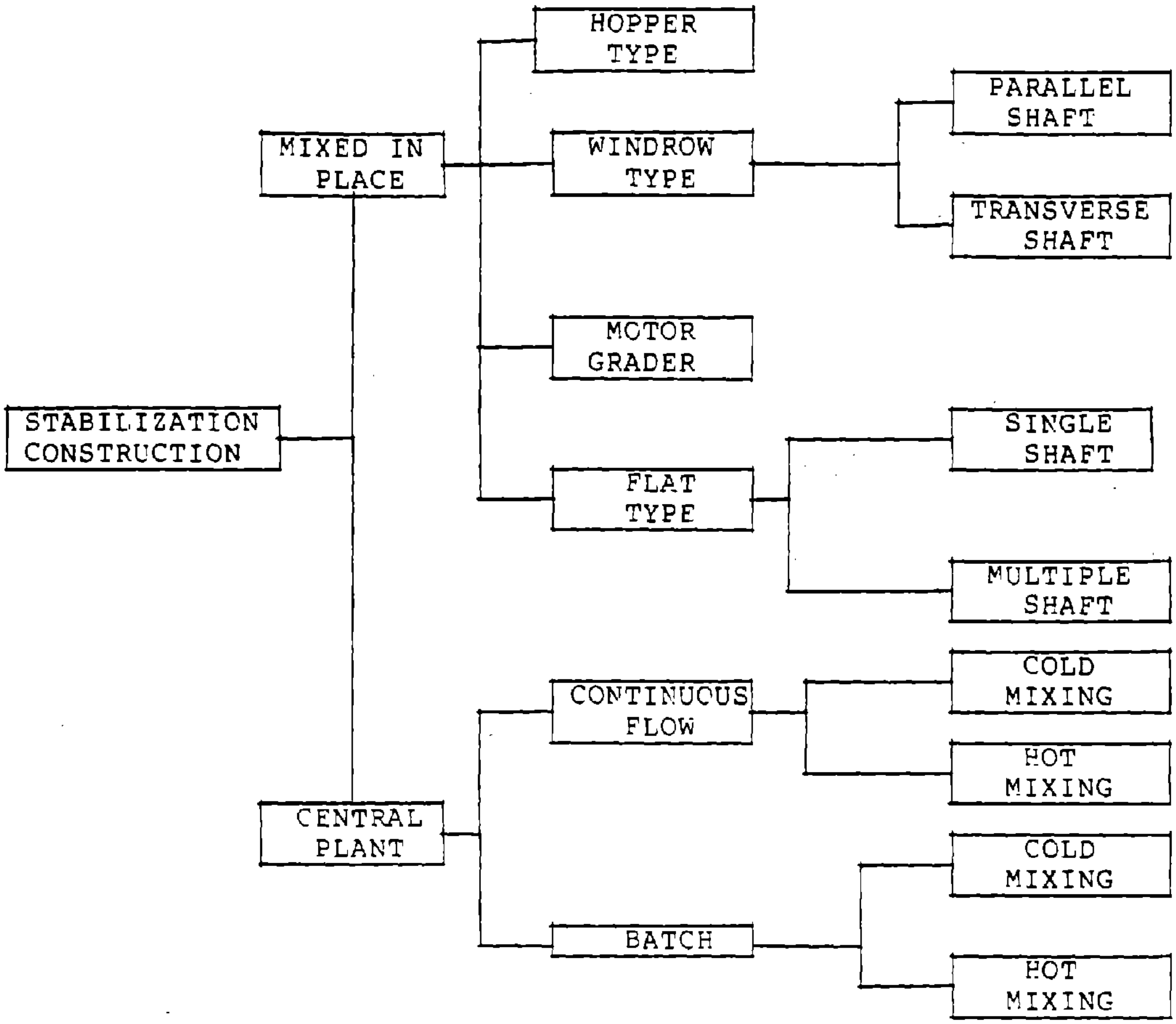


Figure 15: Soil Stabilization Construction Equipment.

Source: (1)

TABLE 1: Review of Axle Load Equivalences, Flexible Pavement

Axle Load	Equivalence Factor	Number Equal to A Standard Axle	Vehicles Included
2000 lbs	0.0002	5000 per lane	VW rabbit; Empty Ford F-100 F-150, F-250, F-350; Empty Ford Van E-100, E-150 Empty Dodge D-150, W-150, D-250; Empty Dodge Van B-150, B-250
4000 lbs	0.002	500 per lane	Loaded Ford F-100, F-150, E-100, E-150; Empty Ford E-350, Loaded Dodge D-150, W-150, B-150, B-250, Empty Dodge W-250, D-350, W-350, B-350
6000 lbs	0.01	100 per lane	Loaded Ford F-250; Dodge W-350, B-350
8000 lbs	0.04	25 per lane	Loaded Ford F-350, E-350; Load Dodge D-350. Includes wrecking trucks, stake body trucks, 1-1/2 yard dump trucks, small school buses, emergency rescue vehicles
10,000 lbs	0.09	11 per lane	
12,000 lbs	0.19	5 per lane	
18,000 lbs single axle	1.0	1 per lane	Previous national maximum load limit for single axle loading; 5 cy dump truck, GVW 27,500; 60 passenger school bus

TABLE 1: Review of Axle Load Equivalences, Flexible Pavement (Contd.)

Axle Load	Equivalence Factor	Number Equal to A Standard Axle	Vehicles Included
18,000 lbs tandem axle	0.08	12 per lane	
20,000 lbs single axle	1.56		New national maximum load limit for single axle loading
20,000 lbs tandem axle	0.12	8 per lane	
32,000 lbs. tandem axle	0.84		Old national maximum load limit for tandem axle loading
34,000 lbs tandem axle	1.08		Old national maximum load limit for tandem axle loading

TABLE 2

Frost Design Soil Classification

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

Source: (9)

Table 3: Total Equivalent 18-Kip Single Axle Load Applications Per Lane

Table 3a*, SN = 0.84: Total Equivalent 18-Kip Single Axle Load Applications Per Lane

Soil Structural	Regional Factor					
Value (SSV)	0.5	1.0	1.5	2.0	2.5	3.0
8	27,400	13,700	9,100	6,900	5,500	4,600
7.2	13,800	6,900	4,600	3,450	2,760	2,300
6.5	7,600	3,800	2,530	1,900	1,520	1,260
6.2	5,900	2,930	1,960	1,470	1,170	980
5.2	2,490	1,250	830	620	500	420
4.0	890	450	300	220	180	150
3.5	580	290	190	150	120	100
3.0	380	190	130	100	80	60

Table 3b*, SN = 1.20: Total Equivalent 18-Kip Single Axle Load Applications Per Lane

	Regional Factor					
SSV	0.5	1.0	1.5	2.0	2.5	3.0
8	145,000	72,500	48,300	36,200	29,000	24,200
7.2	73,100	36,500	24,400	18,300	14,600	12,200
6.5	40,000	20,100	13,400	10,000	8,000	6,700
6.2	31,100	15,500	10,300	7,800	6,200	5,200
5.2	13,200	6,600	4,400	3,290	2,630	2,200
4.0	4,700	2,360	1,570	1,180	940	790
3.5	3,070	1,540	1,020	770	610	510
3.0	2,000	1,000	670	500	400	330

Table 3c*, SN = 1.60: Total Equivalent 18-Kip Single Axle Load Applications Per Lane

	Regional Factor					
SSV	0.5	1.0	1.5	2.0	2.5	3.0
8	683,000	341,000	228,000	171,000	137,000	114,000
7.2	344,000	172,000	115,000	86,000	68,800	57,300
6.5	189,000	94,400	63,000	47,200	36,800	31,500
6.2	146,000	73,000	48,700	36,500	29,200	24,300
5.2	62,000	31,000	20,700	15,500	12,400	10,300
4.0	22,200	11,100	7,400	5,500	4,400	3,700
3.5	14,400	7,200	4,800	3,620	2,900	2,410
3.0	9,400	4,700	3,140	2,360	1,890	1,570

Table 3: Total Equivalent 18-Kip Single Axle Load Applications Per Lane (Cont'd)

Table 3d*, SN = 2.00: Total Equivalent 18-Kip Single Axle Load Applications Per Lane

	Regional Factor					
SSV	0.5	1.0	1.5	2.0	2.5	3.0
5.2	231,000	115,000	77,000	57,700	46,200	38,500
4.0	82,600	41,300	27,500	20,600	16,500	13,800
3.5	53,800	26,900	17,900	13,500	10,800	9,000
3.0	35,100	17,500	11,700	8,800	7,000	5,800

Table 3e*, SN = 2.40: Total Equivalent 18-Kip Single Axle Load Applications Per Lane

	Regional Factor					
SSV	0.5	1.0	1.5	2.0	2.5	3.0
4.0	257,000	128,000	85,000	64,100	51,300	42,800
3.5	167,600	83,600	55,000	41,800	33,400	27,900
3.0	109,000	54,500	36,300	27,200	21,800	18,200

*Tables give one-lane values. For two lane ADT the tabular results must be doubled and divided by (365 x number of years of assumed design life).

The above tables were developed from the following equation which can be used to determine the one lane total equivalent 18-kip single axle load applications for conditions not found in the above charts.

$$\log W_{t18} = 9.36 \log (SN + 1) - 0.20 + \frac{\log (2.2/2.8)}{0.40 + [1094/(SN + 1)\ 5.19]} + \log \frac{1}{R} + 0.372 (S_i - 3.0)$$

where

W_{t18} = total load applications for any subgrade condition i

SN = structural number of pavement

R = regional factor

S_i = soil support value for any condition i = SSV in table

Table 4: Stabilizing Agents/Soil Classifications

Bituminous Stabilizer's

Soil Classifications	Remarks
GW	
GW-GM,	PI not to exceed 10
GM or GC or GM-GC	Well graded material only, PI not to exceed 10, < 200 material not to exceed 30% by weight
SW or SP	
SW-SM or SP-SM	P.I. not to exceed 10
SM or SC or SM-SC	P.I. not to exceed 10, < 200 material not to exceed 30% by weight

Portland Cement Stabilizer

GW or GP	Material should contain at least 45% by weight of material passing No. 4 sieve
GW-GM or GP-GM	Material should contain at least 45% by weight of material passing No. 4 Sieve, P.I. not to exceed 30
GM or GC or GM-GC	Material should contain at least 45% by wt. of material passing No. 4 sieve. P.I. not to exceed No. indicated by equation: 20 + (50 - fines content)/4
CL or ML ML-CL or CL or MH or OH or CH	Liquid Limit less than 40 and P.I. less than 20. Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means.

Lime Stabilizer

GW-GM or GP-GM or GM or GC or GM-GC	P.I. not less than 12

Table 4: Stabilizing Agents/Soil Classifications (Cont'd)

SW-SM or SP-SM or SM or SC or SM-SC	P.I. not less than 12
OL or ML or ML-CL or CL or MH or OH or CH	Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means.

Table 5: Climatic Limitations and Construction Safety Precautions

Type of Stabilizer	Climatic Limitations	Construction Safety Precautions
Lime and Lime-Fly ash	Do no 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 T (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) hot mixed asphalt concrete Hot, dry weather is perferred 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}m

Source: (1)

Table 6: 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.

Source: (2)

Table 7: Emulsified Asphalt Requirements

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	7.0
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7 0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

*50 or less

Source: (13)

Table 8: Asphalt Cutback Composition

Type of Cutback	Solvent	Percent Solvent for Particular Grades				
		30	70	250	800	3000
RC	Gasoline or Naptha	--	35	25	17	13
MC	Kerosene	46	36	26	19	14
SC	Fuel Oil	--	50	40	30	20

Source: (12)

Table 9: Selection of Type of Emulsified Asphalt for Stabilization

Percent Passing No. 200 Sieve	Relative Water Content of Soil	
	Wet (5 percent +)	Dry (0-5 percent)
0-5	SS-1h (CSS-1h)	CMS-2h (or SS-1h*)
5-15	SS-1, SS-1h (CSS-1, CSS-1h)	CMS-2h (or SS-1h*, SS-1*)
15-25	SS-1 (CSS-1)	CMS-2h

*Soil should be pre-wetted with water before using these types of emulsified asphalts.

Source: (12)

Table 10: Cement Requirements for Various Soils

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.		
GW, GP, GW, SW SP, SM	5 - 7	3 - 5	5	3- 5- 7
GM, CP, SM, SP	7 - 9	5 - 8	6	4- 6- 8
GM, GC, SM, SC	7 - 10	5 - 9	7	5- 7- 9
SP	8 - 12	7 - 11	9	7- 9-11
CL, ML	8 - 12	7 - 12	10	8-10-12
ML, MH, CH	8 - 12	8 - 13	10	8-10-12
CL, CH	10 - 14	9 - 15	12	10-12-14
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.

Source: (12)

Table 11: Average Cement Requirements of Miscellaneous Materials

Type of miscellaneous material	Estimated cement content and that used in moisture-density test		Cement contents for wet-dry and freeze-thaw test percent by wt.
	percent by vol.	percent by wt.	
Shell soils	8	7	5- 7- 9
Limestone screenings	7	5	3- 4- 5- 7
Red dog	9	8	6- 8-10
Shale or disintegrated shale	11	10	8-10-12
Caliche	8	7	5- 7- 9
Cinders	8	8	6- 8-10
Chert	9	8	6- 8-10
Chat	8	7	5- 7- 9
Mart	11	11	9-11-13
Scoria containing material retained on the No. 4 sieve	12	12	9-11-13
Scoria not containing material retained on the No. 4 sieve	8	7	5- 7- 9
Air-cooled-slag	9	7	5- 7- 9
Water-cooled slag	10	12	10-12-14

Source: (15)

Table 12: Approximate Lime Contents

Soil Type	Approximate Treatment, Percent by Soil Weight	
	Hydrated Lime	Quicklime
GC, GM-GC	2-4	2-3
CL	5-10	3-8
CH	3-8	3-6

Source: (12)

Table 13: Equipment Typically Associated with Mixed-In-Place Subgrade Stabilization Operations

STABILIZER	CONTRUCTION OPERATION				
	SOIL PREPARATION	STABILIZER APPLICATION	PULVERIZATION AND MIXING	COMPACTION	CURING
Lime[1]	-Single-shaft rotary mixer (flat type) -Motor grader, -Disc Harrow -Other agricultural-type equipment	-Dry-bagged -Dry bulk -Slurry -Slurry thru mixer	-Single- and multi-shaft rotary mixers -Motor graders -Other agricultural-type equipment	-Sheep's foot -Pneumatic -Steel wheel	-Asphalt membrane -Water sprinkling
Lime or cement, Fly ash[2]	-Single-shaft rotary mixer (flat type) -Motor grader -Disc harrow -Other agricultural-type equipment	-Separate application -Lime--dry or slurry of fly ash—conditioned -Combined application -Dry-bagged -Dry bulk	-Same as lime	-Steel wheel -Pneumatic -Vibratory	-Asphalt membrane -Water sprinkling
Cement[3]	-Single-shaft rotary mixer (flat type) -Motor grader -Disc harrow -Other agricultural-type equipment	-Dry-bagged -Dry bulk	-Same as Time	-Sheep's foot -Pneumatic (clay soils) -Vibratory (granular soils)	-Asphalt membrane -Water sprinkling
Asphalt[4]	-Motor grader -Single-shaft rotary mixer (flat type)	-Asphalt spray distributor -During mixing process	-Single- and multi-shaft rotary mixer (flat type) -Motor grader	-Pneumatic -Steel wheel -Vibratory	-Volatiles should be allowed to escape and/or the pavement to cool

COMMENTS	SAFETY PROCEDURES
[1]Double application of lime may be required to facilitate mixing. The soil and air temperature should be greater than 40°-50°F to insure adequate strength gain. Construction should be completed early enough in summer or fall so that sufficient durability will be gained to resist freeze-thaw action.	Lime spreading should be avoided on windy days. Proper clothing should be worn so that Workmen can avoid skin contact with quicklime. Workmen should avoid prolonged contact with lime and breathing lime dust.
[2]Fly ash must be conditioned with moisture prior to distribution to prevent dusting. Mixing and compaction should be completed shortly after stabilizer application. The soils and air temperature should be greater than 40°-50°F to insure adequate strength gain. Construction should be completed early enough in summer or fall so that sufficient durability will be gained to resist thaw-freeze action.	Fly ash, lime and cement spreading should be avoided in windy days. Workmen should avoid prolonged contact with the stabilizers and breathing the the stabilizers.
[3]Mixing and compaction must be completed shortly after stabilizer application. The soil and air temperatures should be greater than 60°F to insure an adequate rate of strength gain. Construction should be completed early enough in summer or fall so that sufficient durability will be gained to resist freeze-thaw action.	Cement spreading should be avoided on windy days. Workmen should avoid prolonged contact with cement and breathing the cement dust.
[4]Proper soil moisture content must be achieved to aid distribution and mixing. Stabilized material should be properly aerated prior to compaction. The soil and air temperature should be above 40°F to allow for proper curing and sufficient time for compaction if hot mix processes are utilized. Thick lifts of hot, asphalt cement stabilized materials can be placed below 32°F.	Proper clothing should be worn so that workmen can avoid skin contact with quickline.

Source: (1)

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