Alaska Department of Transportation & Public Facilities
Alaska University Transportation Center

Soil Stabilization Manual
2014 Update

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Soil Stabilization is used for a variety of activities including temporary wearing curses, working platforms, improving poor subgrade materials, upgrading marginal materials, dust control, and recycling old roads containing marginal materials. There are a number methods of stabilizing soils including modifying the gradation, the use of asphalt or cement stabilizers, geofiber stabilization and chemical stabilization. Selection of the method depends on the soil type, environment and application. This manual provide tools and guidance in the selection of the proper stabilization method and information on how to apply the method.

A major portion of this manual is devoted to the use of stabilizing agents. The methods described here are considered best practices for Alaska.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic	m^3
		meters NOTE: volumes greater than 1000 L shall be		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inc h	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
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1.0 INTRODUCTION

1.1 Background

Soil stabilization is used for a number of activities including (1) temporary wearing surfaces, (2) working platforms for construction activities, (3) improving poor subgrade conditions, (4) upgrading marginal base materials, (5) dust control, and (6) recycling old roads containing marginal materials. A wide variety of materials have been used to stabilize soils or marginal aggregates (e.g., asphalt, Portland cement, lime and lime/fly ash, chemicals, salt, and other techniques). Selecting the stabilizer type depends on a number of factors, including:

1. gradation,
2. plasticity index (PI),
3. availability and cost of the stabilizer and appropriate construction equipment, and
4. climate.

When correctly designed, stabilization can produce numerous benefits for pavement construction and rehabilitation. However, inappropriate designs and/or construction can lead to premature failures.

Most stabilization of road materials carried out in Alaska has involved the use of asphalt emulsions, Portland cement, chemicals, and salt. The reasons for this are:

1. The stabilizing agents are available.
2. Equipment to incorporate the agents is available and proven.
3. The performance of these agents is well documented.
4. Their use is considered cost effective.

Therefore, the major part of this guide is devoted to the use of these stabilizing agents. However, other techniques are also addressed (for example, lime and lime-fly ash, geotextiles, and drainage).

We consider the contents of this guide to be best practice for Alaska conditions. However, with the advances that continue to take place with respect to materials, equipment, and techniques, we expect these practices to continually improve.

1.2 Scope

This guide provides practical information to people involved in the design, materials characterization, construction, and maintenance of pavements incorporating stabilized layers. It includes advice on how to select additive types, how to test materials, and construction procedures.

It also provides guidance on the best practices for selecting, designing, and constructing stabilized layers for new road pavements, as well as for maintaining and rehabilitating existing road pavements. This guide brings together stabilization technology research and experience from public agencies, contractors, research organizations, and material suppliers.

1.3 Limitations

The guide is based on information developed both within and outside the state of Alaska. It may not be applicable to all soils and/or environmental conditions found in the state. For more information, refer to the references at the end of each chapter.

2.0 TYPES OF STABILIZATION

2.1 Classification of Stabilization Types

In the past, stabilization has been classified primarily on the type of stabilizer used. Now that mechanistic design procedures are available to help evaluate the performance of the stabilized layer in a pavement structure, the type of stabilization can further be classified by structural performance.

The three main categories of materials in terms of performance criteria (Table 2.1) are

1. Unbound Materials—Materials that do not exhibit significant tensile strength and that do resist traffic through a combination of cohesion and interparticle friction, such as natural gravels and fine-grained soils.
2. Modified Materials—Unbound materials to which small amounts of stabilizing agents are added to
    - correct a material deficiency without causing a significant increase in stiffness,
    - increase the strength, or
    - reduce the moisture or frost susceptibility of fine-grained soils.
3. Bound Materials—These are produced by adding stabilizing agents to produce a bound material with significant tensile strength. The bound material acts like a beam in the pavement to resist traffic loading. Compared with unbound and modified materials, it has increased structural capacity.

2.2 Types of Stabilizing Agents

Engineers use stabilization to enhance materials properties for pavement design procedures or to overcome deficiencies in available materials. Stabilization agents fall into a number of categories:

1. Asphalt—emulsions, cutbacks, and other proprietary products
2. Portland cement—in accordance with AASHTO standards
3. Lime—includes hydrated lime [$Ca(OH)_2$] and quicklime [CaO]
4. Blends of the above
    - Asphalt/cement
    - Asphalt/lime
    - Lime/fly ash
    - Cement/lime/fly ash
5. Chemicals—generally proprietary chemicals
6. Salt—generally $CaCl_2$
7. Others—in Alaska, these include drainage, geotextiles, and mechanical stabilization

Table 2.2 gives a broad indication of the application and effects of the various stabilizing agents.

Table 2.1 Material Categories and Characteristics

Characteristics	Unbound Materials	Modified Materials	Bound Materials
Materials Types	Crushed rock, natural gravel, granular materials, and fine-grained soils.	Unbound materials with small amounts of stabilizing agents incorporated; Bitumen-stabilized materials; some bitumen/ cement-, lime-, and cement-stabilized materials and chemical stabilizers.	Unbound materials stabilized with cementitious or other binders (e.g., cement, lime, supplementary cementitious materials, bitumen/cement) and chemical stabilizers
Behavior Characteristics	Development of shear strength through cohesion and internal friction between particles.	Development of shear strength through cohesion and internal friction between particles.	Development of shear strength through particle interlock, chemical bonding, and cohesion. Significant tensile strength.
Distress Modes	Deformation through shear and densification. Disintegration through breakdown of particles and/or material structure.	Deformation through shear and densification. Disintegration through breakdown of particles and/or material structure.	Cracking developed through shrinkage, fatigue, and overstressing. Erosion and pumping in the presence of moisture.
Parameters Required for Structural Design	Modulus Poisson's Ratio Degree of anisotropy	Modulus Poisson's Ratio Degree of anisotropy	Modulus Poisson's Ratio Fatigue characterization
Performance Criteria	Current materials specifications (e.g., strength, grading, plasticity, density). Thickness governed by subgrade strain criteria.	Current materials specifications (e.g., strength, grading, plasticity, density). Thickness governed by subgrade strain criteria.	Fatigue and erosion

For definition of terms such as modulus and Poisson's Ratio, see Appendix A.

Table 2.2 Application of Stabilizing Agents

Stabilization Agent	Process	Effects	Applicable Soil Types
Cement	Cementitious interparticle bonds are developed.	• Low additive content (<2%): decreases susceptibility to moisture changes, resulting in modified or bound materials. • High additive content: increases modulus and tensile strength significantly, resulting in bound materials.	Not limited apart from deleterious components (organics, sulphates, etc., which retard cement reactions). Suitable for granular soils but inefficient in predominantly one-sized materials and heavy clays.
Lime (including hydrated lime and quicklime)	Cementitious interparticle bonds are developed but rate of development is slow compared to cement. Reactions are temperature dependent and require natural pozzolan to be present. If natural pozzolan is not present, a blended binder that includes pozzolan can be used.	Improves handling properties of cohesive materials. • Low additive content (<2%): decreases susceptibility to moisture changes, and improves strength, resulting in modified or bound materials. • High additive content: increases modulus and tensile strength, resulting in bound materials.	Suitable for cohesive soils. Requires clay components in the soil that will react with lime (i.e., contain natural pozzolan). Organic materials will retard reactions.
Blended slow-setting binders (for example, slag/lime, fly ash/lime, and slag/lime/fly ash blends)	Lime and pozzolan modifies particle-size distribution and develops cementitious bonds.	Generally similar to cement but rate of gain of strength similar to lime. Also improves workability. Generally reduces shrinkage cracking problems.	Same as for cement stabilization. Can be used where soils are not reactive to lime.
Bitumen (including foamed and high impact bitumen, cutback bitumen, and bitumen emulsion)	Agglomeration of fine particles.	Decreases permeability and improves cohesive strength. Decreases moisture sensitivity by coating fines.	Applicable to granular materials with low cohesion and low plasticity.
Bitumen/cement blends	Agglomeration of fine particles with some cementitious bonding.	Decreases permeability and improves strength. Cement aids in providing early strength.	Applicable to granular materials with low cohesion and plasticity.
Mechanical stabilization	Mixing two or more materials to achieve planned particle-size distribution.	Some changes to soil strength, permeability, volume stability, and compactibility. Materials remain granular.	Poorly graded soils, granular soils with a deficiency in some size(s) of the particle-size distribution.
Miscellaneous chemicals	Agglomeration of fine particles and/or chemical bonding (see trade literature).	Typically increased dry strength, changes in permeability and volume stability.	Typically poorly graded soils.

2.3 Selecting the Correct Stabilizing Agent

2.3.1 Soil Type

Particle-size distribution and Atterberg limits are commonly used to gain a preliminary assessment of the type of stabilization required for a particular material. The usual range of suitability of various types is based on the 75 μm sieve (#200) and the plasticity index of the soil. Figure 2.1 provides initial guidance for selecting a stabilizer type.

2.3.2 Climate and Drainage

Climate can have a significant effect on your choice of stabilizer. In wetter areas, where the moisture content of the pavement materials is high, it is important to ensure that the wet strength of the stabilized material is adequate. In these conditions, cementitious binders are usually preferred, although asphalt and asphalt/cement blends would also work. Lime is suitable for cohesive soils, particularly when used as the initial agent to dry out the material. Lime can also work with silty soils if a pozzolan is added to promote the cementing reaction.

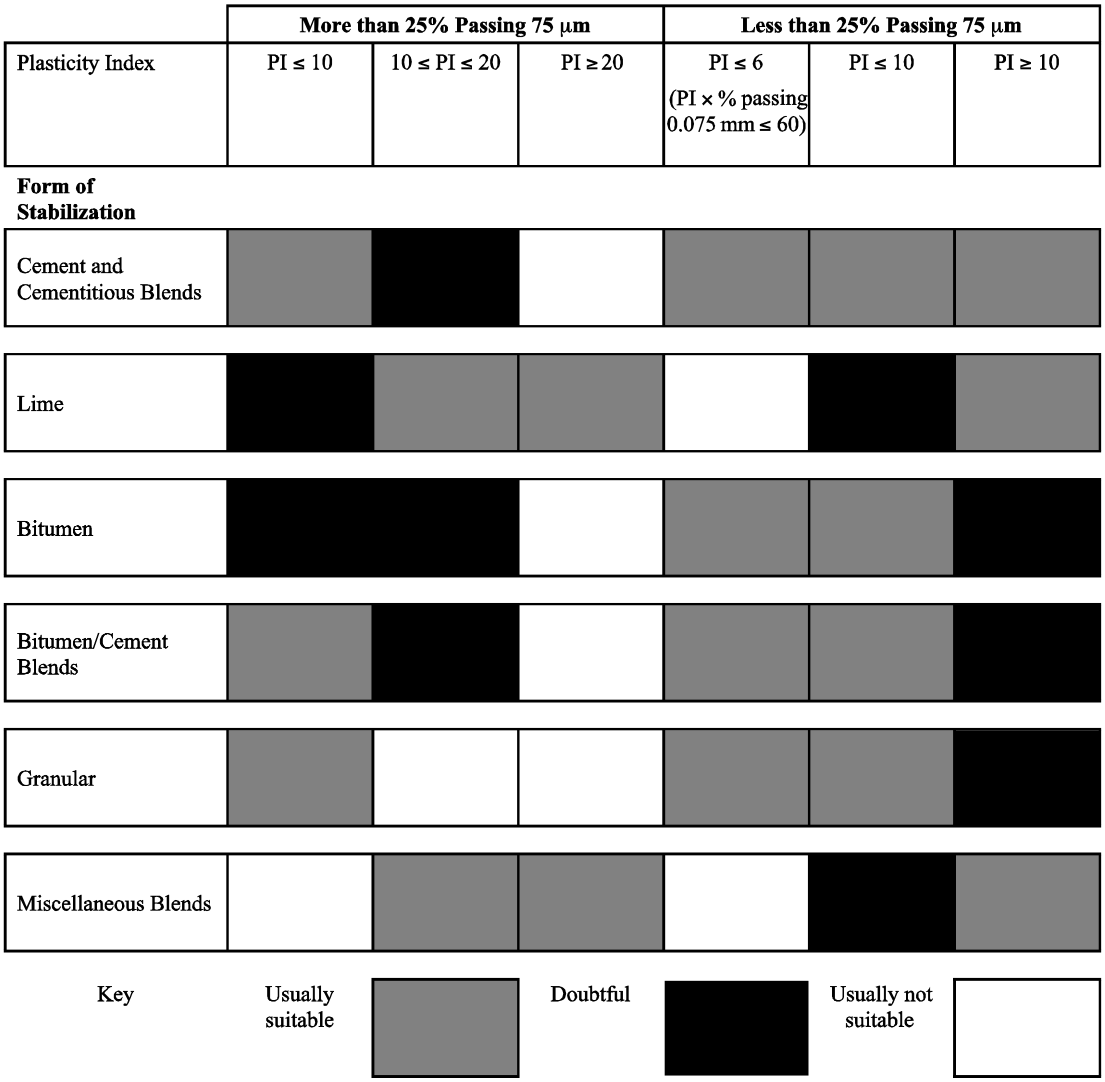
Using emulsions in cold dry climates requires using cement or lime to facilitate moisture removal from the emulsion during the stabilization process. It also promotes strength.

2.3.3 Sampling and Testing

It is essential with all stabilization work that you thoroughly assess all materials and properly evaluate their reactions with a specific admixture in the laboratory before any fieldwork begins. Stabilized materials should be tested to determine their quality and uniformity. Testing requirements are described in the sections of the guide dealing with the relevant methods of stabilization.

2.3.4 Final Selection

After analyzing all available data, you may find there are a number of feasible solutions. The decision is usually based on costs and/or expected performance. You also need to consider the skills, resources, and equipment available in the area, past performance of similar work, and availability of materials and construction equipment.



Should be taken as a broad guideline only.

Note: The above forms of stabilization may be used in combination, for example, using lime stabilization to dry out materials and reduce their plasticity, making them suitable for other methods of stabilization.

Figure 2.1 Guide to Selecting a Method of Stabilization

2.4 References

Alaska DOT&PF, Standard Specification for Highway Construction, Metric Edition, 1998.

Austroads, Guides to Stabilization in Roadworks, Austroads, Inc., Sydney, 1998.

Epps, J.A., W.A. Dunlap, and B.M. Galloway, “Basis for Development of a Soil Stabilization Index System,” USAF Contract #F29601-70-C-0008, Texas A & M University, November 1970.

ERES Consultants, “Soil and Base Stabilization and Associated Drainage Considerations,” Vol. I and II, FHWA-SA-93-004, Federal Highway Administration, December 1992.

Transportation Research Board, “Soil Stabilization,” Transportation Research Record 1295, 1990.

3.0 DESIGN PHILOSOPHY

3.1 Design Considerations

The three main aspects influencing the successful design and use of stabilized pavement materials include:

- the mix design for the stabilized materials,
- the structural design of the pavements into which the stabilized materials will be incorporated, and
- construction of the stabilized layer.

These aspects are interrelated, since stabilized material performance depends on the thickness and composition of the pavement in which it is used, while the structural design process depends on the characteristics of the stabilized pavement material. Both are also affected by the quality of the construction process.

3.2 Materials Mix Design

The materials design aspects of stabilized materials require the engineer to investigate and know about both the pavement material to be stabilized and the stabilizing agents available. Sections 4–10 of this guide describe the desirable materials characteristics and assessment requirements. Important characteristics include:

- strength (compressive and shear),
- durability and erodability,
- shrinkage characteristics,
- setting and curing characteristics,
- moisture susceptibility,
- stiffness and variability, and
- fatigue performance (where applicable).

3.3 Structural Design

You cannot design stabilized materials without considering the composition and structural design of the pavement into which they will be incorporated. The performance of pavements that include stabilized layers will depend on many factors, including

- subgrade strength,
- thickness and stiffness of the stabilized and other pavement layers (including the wearing surface),
- design traffic, and
- environmental conditions—temperature and moisture conditions and provision for pavement drainage.

Use the flexible pavement design procedures recommended by Alaska DOT&PF or the material supplier to design pavements incorporating stabilized layers.

In Alaska DOT&PF's mechanistic design procedure, stabilized materials are characterized by their

- stiffness/modulus,
- Poisson's ratio, and
- fatigue and functional (deformation) performance criteria.

These can be obtained either by testing or by estimation and guidance.

Figure 3.1 illustrates the mechanistic design system used in Alaska.

Stabilized materials for pavements fall into two broad categories:

- unbound materials (includes modified materials), and
- bound materials.

3.3.1 Unbound Materials

Granular stabilized materials, modified materials, and some asphalt-bound materials are considered as unbound granular materials for structural design purposes. There are no performance criteria available for these types of materials that can be checked during the structural design process. Unbound materials gain their load-spreading ability from a combination of internal friction and cohesion and are assumed to perform satisfactorily if they meet their respective materials specification requirements.

Subgrade materials are also considered, for design purposes, as unbound granular materials. If subgrades are stabilized to form a working platform or to increase their bearing capacity, then they are treated, for design purposes, as subbase layers. Unbound materials are considered to be anisotropic and their stiffness is stress dependent (that is, it varies depending on where it is located within the pavement structure).

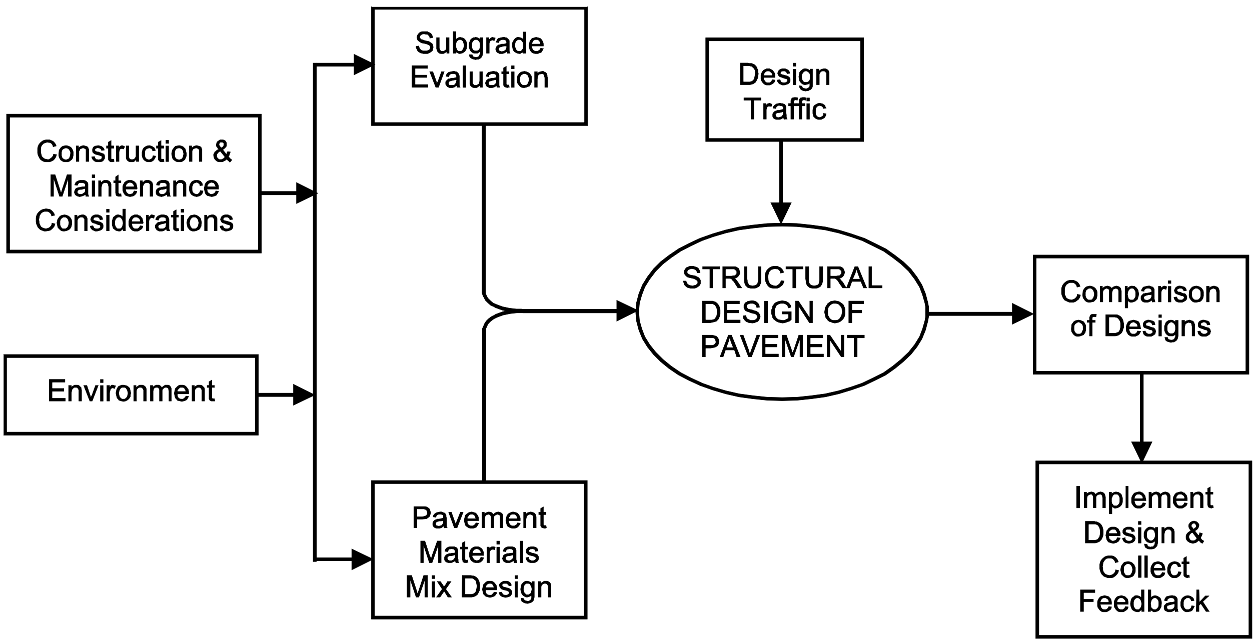


Figure 3.1 Structural Design System for Pavements with Stabilized Layer

3.3.2 Bound Materials

Bound materials can be classified as cementitiously bound materials or asphalt-bound materials as described in the following sections.

3.3.2.1 Cementitiously Bound Material

Materials bound with sufficient amounts of cementitious binders to achieve significant tensile strength are considered cemented materials and should meet a fatigue performance criterion for structural design. For design purposes, cementitiously bound materials are considered isotropic, and their stiffness is not stress-dependent.

The fatigue behavior of cemented materials can be described by a relationship that takes the form

$$N = (K/\sigma)^{\alpha} \qquad \text{Eq. 3.1}$$

where *N* is the number of stress repetitions to failure, *K* and *a* are constants dependent on binder content, and σ is the tensile stress in the cemented material. For cemented materials, the value of the exponent *a* in the above equation is usually about 12, which means that a small change in the tensile stress in the cemented material will result in a large change in the fatigue performance of the material. In other words, small changes in layer thickness (as a result of poor construction control or application of construction tolerances), density, or uniformity can lead to major deficiencies in long-term performance. This is why you should use conservative values of the characteristics of cemented materials for structural design.

Bonding between layers is another critical aspect of the design of pavements incorporating cemented layers. If cemented pavement courses are used the Alaska DOT&PF mechanistic design procedure assumes there is full bonding between all layers. If the section is constructed in more than one layer, it is vital that these layers bond together to act as a single structural layer. The possible effect of not achieving bonding between stabilized layers is illustrated in Figure 3.2.

The structural design process results in design layer thicknesses for each pavement layer. For stabilized layers, you should consider the design thickness as a minimum construction thickness because of their sensitivity to curing, density, and uniformity. Apply appropriate construction tolerances to ensure this design thickness is achieved in the field.

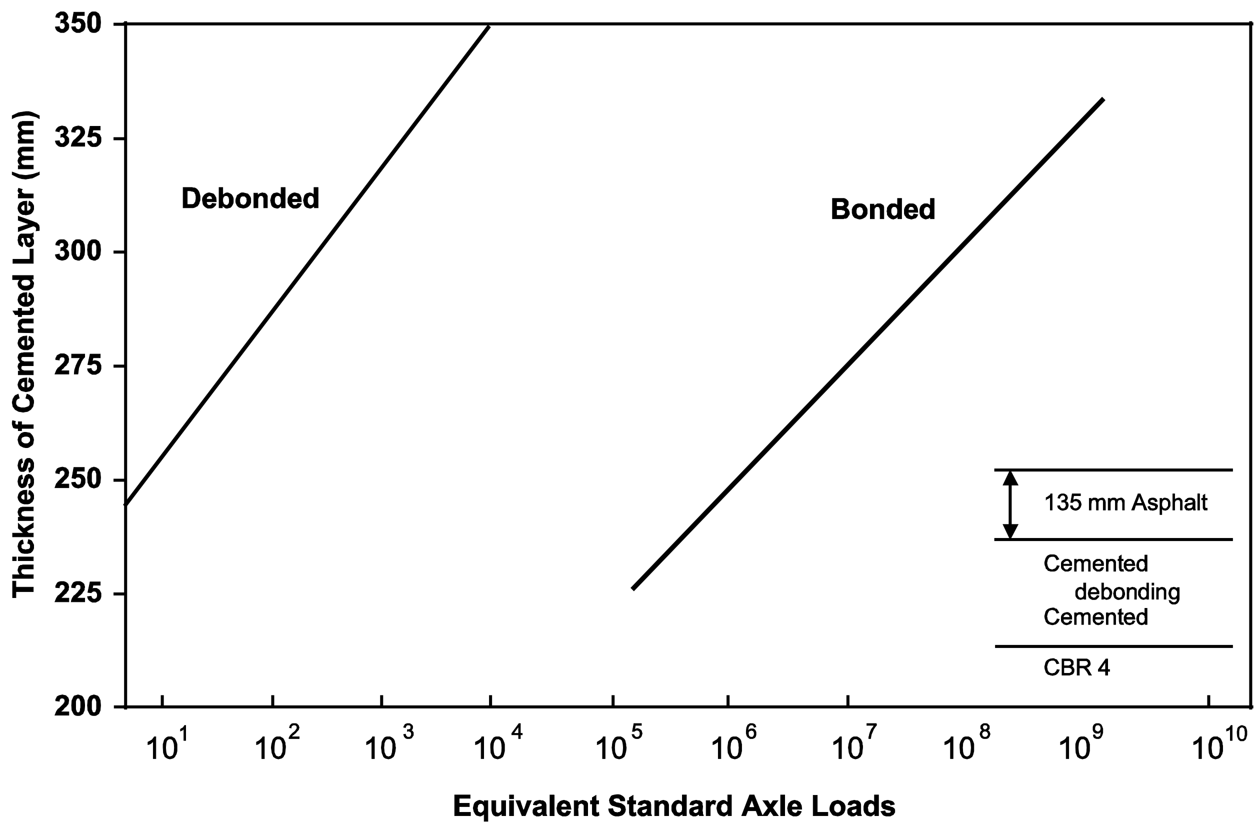


Figure 3.2 An Illustration of the Loss of Performance that Results from Debonding of Cemented Layers (Based on Mechanistic Modeling)

3.3.2.2 Asphalt-Bound Material

These are a combination of asphalt and aggregates that are mixed together, spread, and compacted to form a pavement layer. Some materials that are stabilized with asphalt, usually also with supplementary additives, behave structurally in a similar way to asphalt. This means that they can be classified for structural design purposes in the same way as asphalt. This characterization requires knowledge of stiffness, which may be dependent on temperature and rate of loading, Poisson's ratio, and a fatigue performance relationship in the same form as previously described for cemented materials.

3.3.3 Assessment of Existing Pavements or Recycling

Pavement recycling can also be considered a form of stabilization. Asphalt emulsions are generally used in this type of stabilization. You must investigate existing pavements that are being considered as candidates for recycling to determine their current structural capacity, composition, and variability. There are a number of ways to carry out this investigation work. Investigation may include one or more of the following techniques:

- measure pavement deflections and curvatures using falling weight deflectometer (FWD),
- assess bearing capacity using a dynamic cone penetrometer (DCP),
- excavate test pits to measure material properties and sample materials, and
- perform materials mix design in the laboratory.

Apart from the properties of the layer to be recycled, the structural properties of the subgrade and all the other layers are required as input into the structural design process that again is carried out in accordance with the mechanistic design system depicted in Figure 3.1.

3.3.4 Design Traffic

Design traffic is the total traffic loading over the design period of the pavement. Keep in mind that cementitious-bound materials and asphalt-bound materials have different fatigue performance relationships. This means you will have to determine separately for each material the number of equivalent single axle loads (ESALs) that will cause the same level of accumulated damage as the actual traffic spectrum.

3.4 Construction Considerations

3.4.1 Stabilization Equipment

Stabilization may be carried out using one of two methods: (1) mix-in-place or in situ stabilization, and (2) stationary or pugmill type stabilization. For in situ stabilization, two types of equipment are generally used: an additive spreader and a mixer or reclaimer. The additive spreader is used to distribute the additive uniformly on the soil to be treated. These include spreaders that are capable of applying either dry or liquid additives. The mixer is used to thoroughly mix the additive with the soil. These machines come in various sizes and include

- motor patrols,
- rotary type mixers, and

- reclaimers/stabilizers.

For mixing in place with cement and lime, it is essential to have a water truck connected to the mixer to introduce water into the mixing unit. More information on the types of equipment that are used can be obtained from the manufacturers.

When stationary plants are employed, the materials and additives are blended through a pugmill, which then discharges the material into a truck. The blended materials are normally laid with a paver.

3.4.2 Compaction Equipment

Compaction is generally achieved with conventional compaction equipment. Static steel- wheeled compactors of at least 14 tons will be required when the pavement layers approach 200 mm (8 inches). Use care when using heavy vibratory rollers since they can damage underground structures or the stabilized layer.

3.4.3 Procedures and Operations

While the detailed procedures vary depending on the type of additive used, the following procedures are common in most circumstances:

- First, prepare the materials to be stabilized so that the final grade is as planned.
- Then spread the additive to the recommended quantity using an appropriate spreader.
- Next, mix the additive into the host materials and add water as needed. Different types of additives have different requirements for degree of mixing.
- After mixing, begin compaction. In deep stabilization techniques, padfoot rollers are used until walk-out occurs. Smooth steel vibratory rollers are then used to complete the compaction process.
- While most pavements stabilized with a binder having a cementing component may be trafficked immediately after compaction, it is very important that the pavements be cured to ensure they have adequate strength to carry the traffic.

3.5 References

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4.0 ASPHALT STABILIZATION

4.1 General

Asphalt stabilization of pavement material is usually intended either to introduce some cohesion into nonplastic materials or to make a cohesive material less sensitive to loss of stability with increased moisture. Various bituminous materials can be used for this purpose. The process is more successful with granular material than with cohesive material. Asphalt stabilization is therefore primarily used on base and, to a lesser extent, subbase materials.

Using a mixture of bituminous and cementitious binders together has the advantage of improving strength as well as increasing cohesion and reducing moisture susceptibility. Using these types of stabilizing agents, even with poor quality pavement materials, improves performance.

4.2 Materials

4.2.1 Suitability of Materials

A relatively wide range of materials is suitable for asphalt stabilization, including materials that have been pretreated with lime. Figure 4.1 summarizes the broad selection process to determine an appropriate asphalt stabilizing agent, while Figure 4.2 indicates material types suitable for stabilization with asphalt and asphalt/cement blends.

4.2.2 Asphalt Materials

Bituminous stabilization may be carried out with any of the following materials:

- hot asphalt cement,
- cutback asphalt,
- asphalt emulsion, either as cationic or anionic emulsion, and
- the above with cementitious binders used in conjunction.

Typical binder contents range from 4% to 8%.

4.2.3 Stabilization with Hot Asphalt

Stabilization with hot asphalt involves a temporary change of state of the stabilizing agent by significantly increasing its surface area at the point of mixing. The two main methods are:

- foamed asphalt process, and
- high-impact process (HIP).

These processes require using specialized equipment to distribute the binder. They also eliminate the extra manufacturing process required in the use of cutback asphalt or asphalt emulsion. Stabilization can be carried out in place or in a central plant.

Several types of asphalt can be used in the hot process. Asphalt produced by the propane precipitation process includes an antifoaming agent. This ingredient must be neutralized before the asphalt can be used for foamed asphalt. Both foamed asphalt and the high impact process allow only a very short mixing time while the asphalt is in a finely dispersed condition. Mixing must be completed and the particles coated soon after the application of the binder.

Both methods require mixing to be carried out at or near optimum fluid (asphalt plus moisture) content. The fine aggregate particles are preferentially coated, leaving the coarse particles relatively uncoated with asphalt. You can place it immediately, but you must be careful with initial compaction to prevent instability. The foamed asphalt process results in materials with the desirable properties of asphalt mixtures: durability, flexibility (crack resistance, stabilization of fines against weakening by moisture), better cohesion, and decreased permeability.

The performance of hot asphalt stabilized materials varies with the quality of the material stabilized. A maximum plasticity index of 6–15 for the material to be stabilized is recommended for this type of treatment (Figure 4.1). Secondary additives such as fly ash, cement works flue dust, or lime may be added to alter the characteristics of the finished product or to make it more amenable to treatment with the asphalt binders.

Cementitious additives should not exceed 2% by mass, to avoid possible shrinkage cracking.

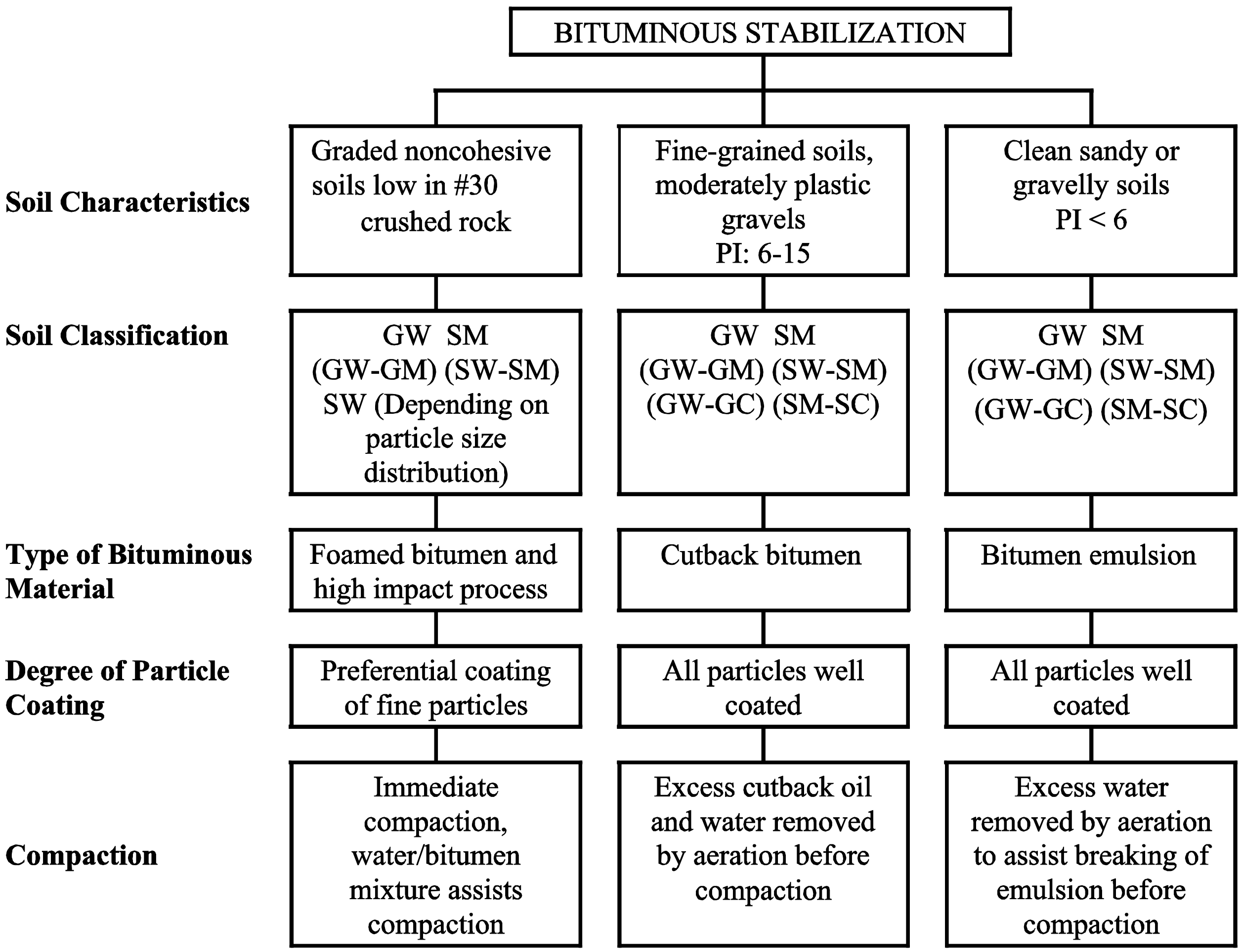

Figure 4.1 Guide to the Selection of Asphalt Materials for Stabilization

4.2.3.1 Foamed Asphalt

Foamed asphalt is gaining popularity in the U.S. in recent years as a base course stabilizer. In most cases foamed asphalt is associated with full depth reclamation. It has proven particularly effective in reducing frost heave and thaw weakening in high fines content base courses. Resilient moduli of between 75 and 120 ksi can be expected from full depth reclamation using foamed asphalt.

Foamed asphalt is produced by injecting 1 – 1.5% water and compressed air into hot asphalt cement. During this process the asphalt expands approximately 10 times its original volume. The expanded asphalt in then mixed into the base course. The purpose of foaming is to allow mixing and coating of cold aggregates reducing the cost of transporting material to a central plant. Cement, lime or fly ash are commonly added to improve coating of the aggregate.

Experience suggests that foamed asphalt not be used when the air temperature is below 50^{o}F or when the aggregate temperature is less than 60^{o}F.

It is important to perform a mix design for foamed asphalt mixes to determine the asphalt content, Portland cement, and water contents. Asphalt content is typically around 3% by weight. Mix design procedures and construction specifications are available from the Alaska DOT&PF Statewide Materials Section.

Construction requires a specially designed reclaimer capable of injecting water into hot asphalt cement while pulverizing and mixing the existing pavement with the underlying base course. Once the mixing and shaping is complete, a pads foot roller is commonly used to achieve compaction. Additional water may be used as a compaction aid if required.

A good source of information related to foamed asphalt stabilization can be found at http://www.wirtgen.de/en/ and the references at the end of this chapter.

4.2.4 Stabilization with Cutbacks

Cutbacks are asphalt mixed with light cutter oil, producing binders that are fluid at ambient temperatures. The cutback asphalt can be sprayed cold or with slight heating and mixed with premoistened soil. This method of stabilization results in a material that gains strength very slowly and as a result is not used very often. Environmental constraints often limit the use of cutbacks in urban areas.

4.2.5 Stabilization with Asphalt Emulsion

Asphalt emulsions may be readily mixed with damp soil to produce a good dispersion of asphalt throughout the soil. Asphalt emulsions are most widely used for soil stabilization.

4.2.5.1 Classes and Types of Emulsion

Asphalt emulsions are manufactured to comply with Alaska Standard Specifications 702, which allows for two classes, depending on the charge of the suspended particles:

- anionic asphalt emulsion, where the particles of asphalt are negatively charged, or

- cationic asphalt emulsion, where the particles of asphalt are positively charged.

Both classes of asphalt emulsion are prepared in three grades, rapid setting (RS), medium setting (MS), and slow setting (SS). Only the medium- and slow-setting grades are suitable for use in stabilization.

Most manufacturers make all classes and grades. However, emulsions of the same class made by different manufacturers may react differently with the same soil. Therefore, it is important to test the emulsion first in the laboratory.

Asphalt emulsion is normally manufactured with 120–180 pen asphalt, which is usually satisfactory for soils with lower fines content (0–10% passing the 75 μm sieve). For soils with higher fines content (15–25% passing the 75 μm sieve), it may be necessary to use a softer asphalt if you have difficulty in distribution. Under extreme conditions, emulsion-containing oil may have to be used (for example, CMS-2S).

Application rates of 2 to 3% of residual binder (the rate often limited by the natural moisture content of the aggregate) are commonly used. Lower rates of about 0.5% to 1% may be satisfactory for well-graded materials in dry climates. Lower application rates, when added to granular base materials, can be useful as a construction expedient to reduce raveling and potholing under traffic. In any case, determine the application rate by laboratory testing.

If no other data are available, a guide to the amount of emulsion to form a heavily bound material may be obtained from the following equation (Asphalt Institute, 1989):

$$\% \text{ (by mass)} = 0.75\,(0.05\,A + 0.10\,B + 0.50\,C) \qquad \text{Eq. 4.1}$$

where
A = % retained on 2.36 mm (#8) sieve,
B = % passing the 2.36 mm (#8) sieve, but retained on the 75 μm sieve (#200), and
C = % passing the 75 μm (#200) sieve.

4.2.5.2 Conditions of Mixing

The soil moisture content influences how efficiently the emulsion is distributed throughout the soil. Dry soil causes the emulsion to break prematurely, resulting in the asphalt forming blobs and not spreading evenly. As the moisture content of the soil increases, the tendency of the emulsion to break prematurely is reduced and the time of mixing can be extended to enable improved distribution. If the moisture content becomes too high, you may have to aerate the soil to remove excess moisture before compaction begins. Once the soil has been compacted, it is difficult to get any more water to penetrate.

4.2.6 Additives

You can improve the stabilization of gap-graded granular materials and/or materials with smooth rounded grains by adding mineral filler, rock dust, fly ash, etc. Hydrated lime or portland cement (1 to 2%) may also be added as a secondary additive to improve particle coating. You can also use lime as a preliminary modifying treatment to render particular soils more amenable to receiving an asphalt stabilizing agent (for example, asphalt emulsions).

You can improve the bond between soil particles with asphalt binders by using surface active agents or antistripping additives. These agents usually improve the wet strength and water absorption resistance, and you can mix them with the soil before adding the asphalt binder or combine them with the asphalt binder before use. The proportion of such additives is usually only about 0.3 to 1.0% by mass of the stabilizing agent.

You can successfully use a mixture of portland cement and asphalt to improve the properties of low-grade pavement materials by increasing stiffness and reducing permeability. Also, adding cement promotes the removal of excess water and helps the emulsion to break.

4.2.7 Water Quality

The quality of compaction water used in asphalt stabilization is not critical. Salty water has been used with no apparent harm. You may have to be careful to prevent damage by salt accumulation, which may lead to asphalt surfacing failures. You should avoid using salt water to dilute emulsions.

4.3 Mix Design

4.3.1 General

You can design asphalt-stabilized materials in a number of ways. Historically, such design has relied on strength and water absorption testing. However, a similar approach that is now being used for cementitious stabilized materials and asphalt is gaining in acceptance.

The mix design for asphalt-stabilized materials must ensure the best composition of mix components to meet the mix design criteria and to realize the structural parameters required. Your assessment of the materials should follow three basic steps for all types of asphalt stabilization:

1. Test and classify the material to be stabilized (grading and Atterberg limits).
2. Select the type of additive, depending on material to be stabilized, climate, and construction equipment availability, and determine the laboratory optimum-fluid content (OFC) and compacted density.
3. Determine target residual asphalt contents, optimum moisture content, and required density.

The Asphalt Institute design manuals (1989, 2000) give detailed guidance of the procedures involved. The various factors that affect the behavior and design of asphalt-stabilized materials are shown in Figure 4.2.

4.3.1.1 Classifying Material

Determine grading and Atterberg limits in accordance with AASHTO test methods. If you plan to use lime, cement, or other secondary additives in the stabilization process, you should incorporate them in the material to be tested.

4.3.1.2 Determining Laboratory Optimum Fluid Content and Compacted Density

Optimum fluid content (OFC) is defined as the fluid content at which maximum dry density is achieved where the total fluid content consists of the asphalt-stabilizing agent plus compaction moisture. You should do this testing on the likely blend to be used in the field.

4.3.1.3 Determining Target Residual Asphalt Content

The target residual asphalt content will depend on the performance required for the stabilized material in the field as well as the economics of the mix. The testing regime to determine the target residual asphalt content will depend on the performance criteria adopted. For heavily bound materials, you may need to perform stiffness and indirect tensile testing, as well as an assessment of fatigue performance, to optimize structural performance. For lightly bound materials, you should assess stiffness and deformation resistance or Marshall stability and flow.

You also need to assess water absorption, usually by capillary rise of water in compacted cylinders that have been oven dried at 60°C. Evaluate moisture sensitivity by either long-term soaking or vacuum soaking. In any case, try to make the testing regime reflect construction, curing, and performance conditions likely to occur in the field.

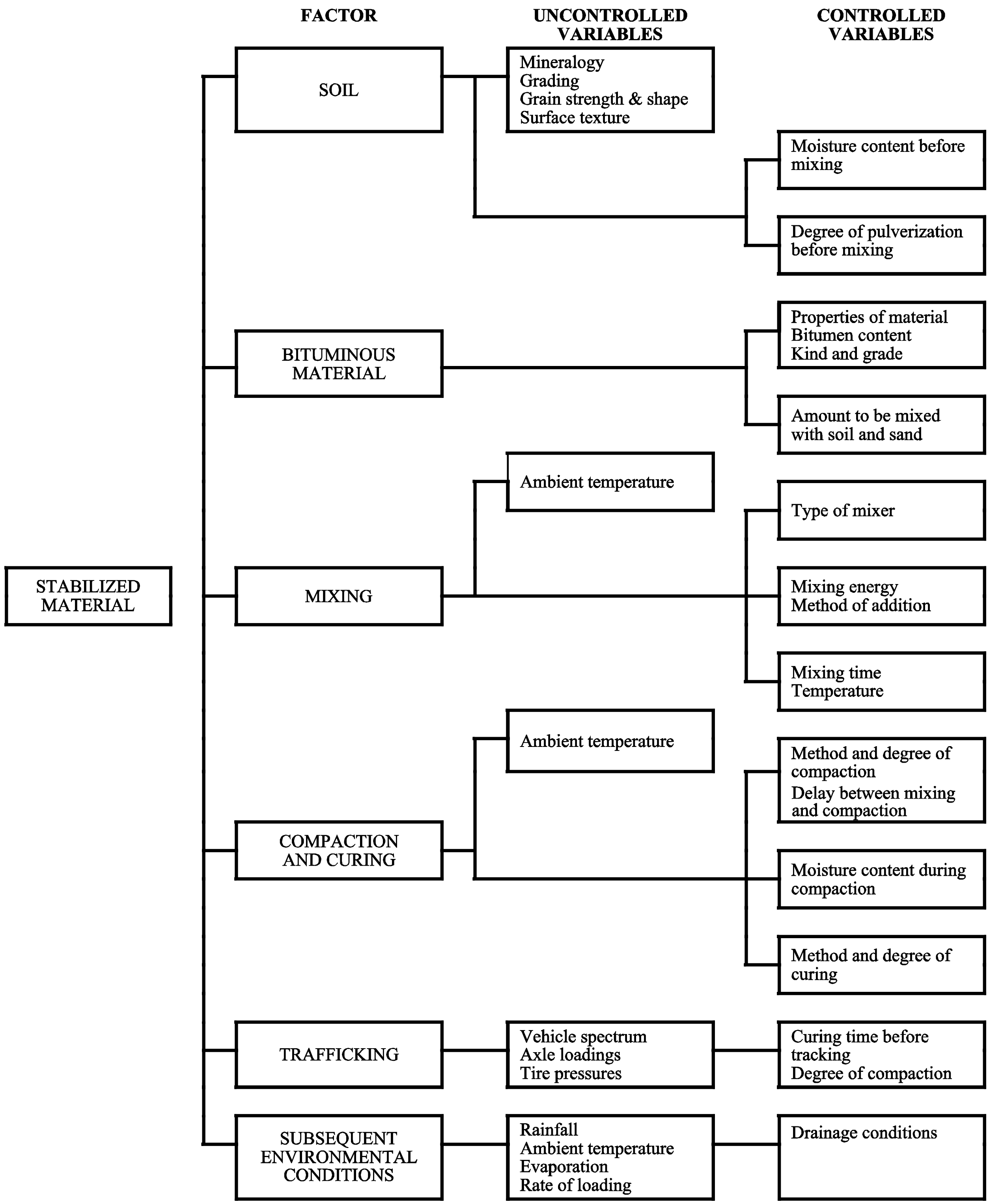

Figure 4.2 Factors Affecting the Design and Behavior of Asphalt-Stabilized Materials

4.3.2 Testing and Design Criteria

For the purpose of pavement design, asphalt-stabilized materials are either unbound, modified, or bound materials, depending on the type and quantity of binder. Poisson's ratio would vary between 0.20 and 0.45 (0.20 for bound, 0.45 for unbound).

As a guide to selecting suitable binder contents for preparing test specimens, soils and granular materials normally require adding 2 to 5% by mass of residual asphalt. You can use the diametral modulus test to test for resilient modulus (Figure 4.3), deformation, and fatigue characteristics of asphalt stabilized materials.

If the diametral test is not available, use the indirect tensile strength (ITS) test to obtain an indication of resilient modulus.

Alternatively, the resilient modulus (E) may then be estimated using the equation:

$$E\,(\text{MPa}) = 2.2 \times \text{ITS}\,(\text{kPa}) + 168 \qquad \text{Eq. 4.2}$$

A minimum value ITS of 100 kPa (14.5 psi) is recommended.

4.4 Construction Factors Effecting Design Considerations

You should stabilize with emulsion and cutback asphalts when conditions are dry and warm. In hot, dry areas, medium- to slow-setting cutback asphalts can be used, depending on the soil type, but in cooler areas, medium- to rapid-setting cutbacks would be required.

Foam asphalt stabilization is not subject to climatic restrictions for mixing and compaction. In all cases, pre-wet the soil with water to achieve better dispersion of the asphalt binder. It is quite difficult to add water after the mixing has been carried out.

When preparing test specimens of mixes incorporating asphalt binders other than hot asphalt, it is essential that you aerate the mixture before compacting. The time of aeration should be enough for the excess water to escape from emulsions and volatiles from cutback asphalt. Do not stabilize with asphalt binders, particularly cutbacks and emulsions, if rain is likely before the process is completed.

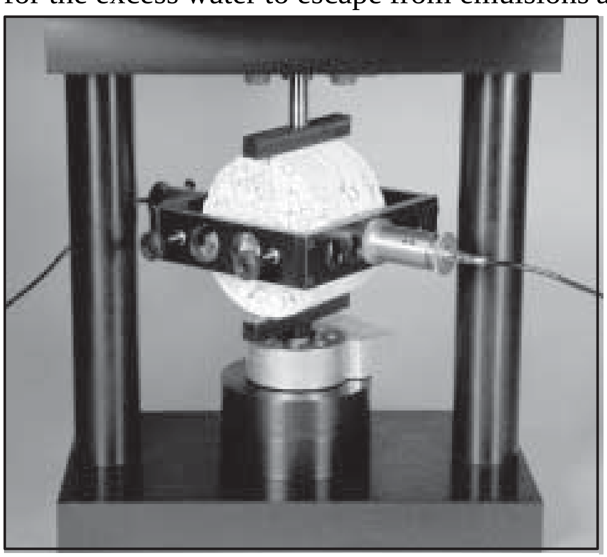

Figure 4.3 Diametral Modulus Testing (courtesy of OEM, Corvallis, OR)

To achieve good results with asphalt-stabilized materials in the field, you must

- thoroughly mix the stabilizing agent throughout the soil,
- ensure that the soil is compacted at a uniform moisture condition, and
- ensure adequate aeration of emulsion and cutback-stabilized materials to allow the excess moisture and/or volatiles to escape.

Asphalt stabilized materials are much slower setting than cementitiously treated materials, and you must not allow traffic on these materials until they have gained adequate strength. A limited amount of controlled traffic after setting is advantageous before sealing.

4.5 Expected Performance and Costs

Asphalt stabilized materials (using emulsions) have been widely used in Alaska with varying degrees of success. Local materials have been stabilized or pavements recycled using asphalt emulsions. Typical costs of using emulsions in highway construction in Alaska are not available.

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5.0 CEMENT STABILIZATION

5.1 General

Cement stabilization refers to stabilizing soils with portland cement. The primary reaction is with the water in the soil that leads to the formation of a cementitious material. These reactions occur almost independently of the nature of the soil, and for this reason, portland cement can be used to stabilize a wide range of materials.

Although there are several types of cement-stabilized soils, there are two types associated with highway construction:

1. Soil-cement—it contains enough cement (usually > 3%) to pass standard durability tests and achieves significant strength increase.
2. Cement-modified soil—an unhardened or semi hardened mixture of soil, water, and small quantities of cement.

In Alaska, soil-cement is the primary product used and it is the only one discussed in this chapter.

5.2 Materials

5.2.1 Types of Cement

Portland cement is a finely ground inorganic material that possesses strong hydraulic binding action when mixed with water to produce a stable, durable product. Several different cement types have been successfully used for cement stabilization of soils:

1. normal portland cement (Type I),
2. sulfate resistant (Type II), and
3. high early strength (Type III).

The most common cement used in Alaska is Type I. The portland cement used for stabilization should conform to Alaska Specifications, Section 701.

5.2.2 Soil-Cement Reactions

Regardless of the type used, the portland cement acts both as a cementing agent and a modifier. In primarily coarse-graded soils, the cement paste bonds the soil particles together by surface adhesion forces between the cement gel and the particle surfaces. In fine-grained soils, the clay phase may also contribute to the stabilization process through reaction of the free lime from the cement. In this manner, the cement acts as a modifier by reducing the plasticity and expansion properties of the soil.

5.2.3 Soils Suitable for Cement Stabilization

A wide range of soil types may be stabilized using portland cement (see Table 5.1 and Figure 5.1). It is generally more effective and economical to use it with granular soils due to the ease of pulverization and mixing and the smaller quantities of cement required. Fine-grained soils of low to medium plasticity can also be stabilized, but not as effectively as coarse-grained soils. If the PI exceeds about 30, cement becomes difficult to mix with the soil. In these cases, lime can be added first to reduce the PI and improve workability before adding the cement.

5.3 Design Considerations

5.3.1 Mix Design

Table 5.1 identifies the usual cement requirements for soil-cement for various soil types. You can select an approximate cement content from this table. However, note that the cement content ranges are for soil-cement, a hardened material that passes rather severe durability tests.

For major projects, you should use a more detailed testing program, as shown in Figure 5.1. Detailed test procedures are given in the Soil-Cement Laboratory Handbook (PCA, 1971). Criteria for satisfactory performance of soil cement in the durability tests are listed in Table 5.2.

5.3.2 Structural Design

High-strength stabilized materials have been used in many asphalt pavements throughout the world. The typical pavement section includes a minimum thickness asphalt concrete surface course over the stabilized base. In many applications, only an asphalt surface treatment is used. You can use the AASHTO design procedure to establish thickness requirements; however, the mechanistic-empirical design method is preferred.

Cementitious stabilizers typically increase the strength properties and modulus of elasticity. Also, stabilization enhances freeze-thaw and moisture resistance. Flexural strength, modulus, and thickness of the stabilized layer, as well as the subgrade modulus and strength, influence the structural response and performance of cement-stabilized layers. Some agencies limit the compressive strength to 700 psi maximum to minimize the potential for shrinkage cracks.

5.3.2.1 Strength Values

The strength of the stabilized material is a fundamental property of design and is often specified and used for construction control. The types of tests normally used are the flexural beam test, the split tensile test, and the unconfined compression test. The latter is normally used because of its simplicity.

5.3.2.2 Stress-Strain Relationship

This behavior is normally expressed in terms of an elastic or resilient modulus. For cement-stabilized soils, selecting an appropriate modulus value is complicated because:

- different test methods give different values,
- the relationship can be nonlinear at higher stress levels, and
- the modulus is lower in tension than in compression.

Table 5.1 Cement Requirements for Various Soils

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

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

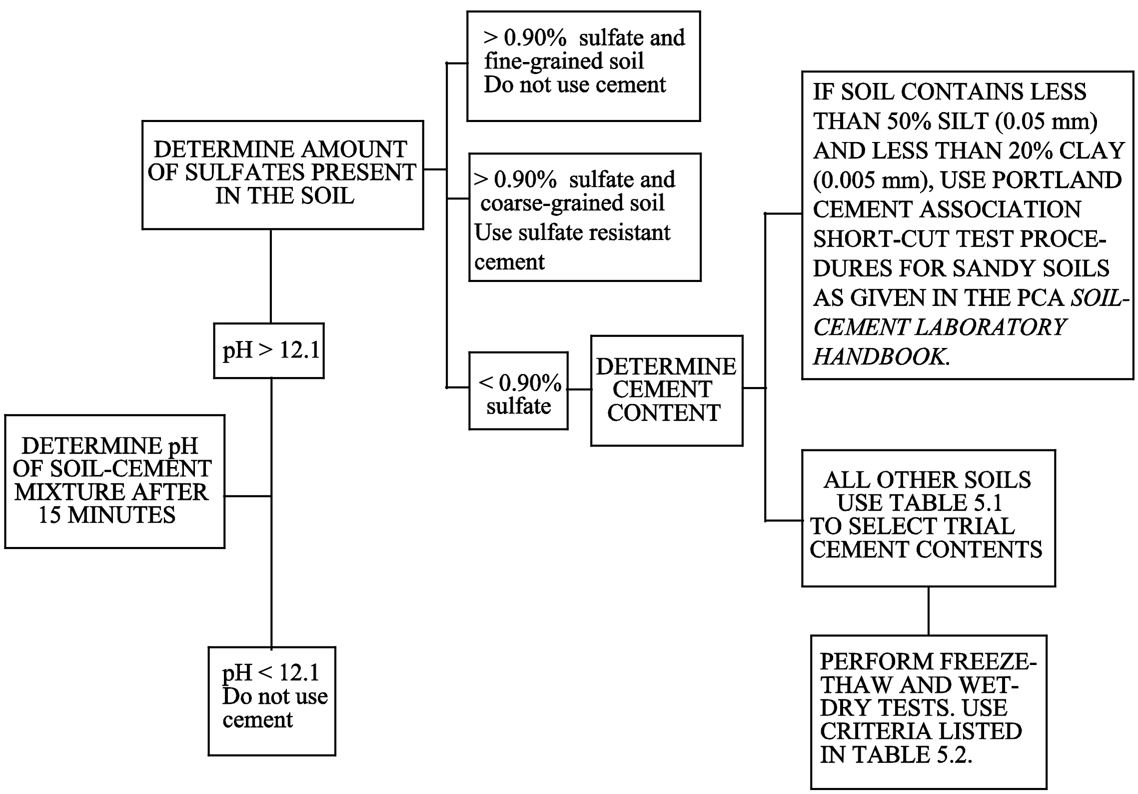

Figure 5.1 Subsystem for Base Course Stabilization with Cement

Because of this, one can use a relationship between compressive strength and modulus of elasticity in lieu of testing.

$$E = 1000 (f_c) \qquad \text{Eq. 5.1}$$

where

E = modulus of elasticity, psi
f_c = unconfined compressive strength, psi

AASHTO Soil Classification	Unified Soil Classification*	Usual Range in Cement Requirement		Estimated Cement Content and That Used in Moisture-Density Test	Cement Contents for Wet-Dry and Freeze-Thaw Tests
		Percent by Volume	Percent by Weight	Percent by Weight	Percent by Weight
A-1-a	GW, GP, GM, SW, SP, SM	5–7	3–5	5	3–5–7
A-1-b	GM, GP, SM, SP	7–9	5–8	6	4–6–8
A-2	GM, GC, SM, SC	7–10	5–9	7	5–7–9
A-3	SP	8–12	7–11	9	7–9–11
A-4	CL, ML	8–12	7–12	10	8–10–12
A-5	ML, MH, CH	8–12	8–13	10	8–10–12
A-6	CL, CH	10–14	9–15	12	10–12–14
A-7	OH, MH, CH	10–14	10–16	13	11–13–15

*Based on correlation presented by Air Force.

Table 5.2 Criteria for Soil-Cement as Indicated by Wet-Dry and Freeze-Thaw Durability Tests

5.3.2.3 Fatigue Characteristics

The fatigue characteristics of cement-stabilized materials are normally reported in terms of stress ratio vs. number of repetitions as shown below:

$$\text{Log } N = (0.9722 - S)/0.0825 \qquad \text{Eq. 5.2}$$

where
N = allowable number of repetitions
S = flexural stress/flexural strength

This relationship, developed by the PCA, is also illustrated in Figure 5.2. You can also use this relationship to establish the thickness requirement when employing a mechanistic design approach.

5.4 Construction

When working with cement, always follow the manufacturer's instructions and seek advice about worker safety. The Portland Cement Association is a good source of information for construction of soil-cement types of pavements (see website www.portcement.org).

5.5 Expected Performance and Costs

Cement stabilized materials have been widely used in parts of Alaska with varying degrees of success. Local materials have been stabilized for use in highways and airfields. Typical costs of using portland cement are not available.

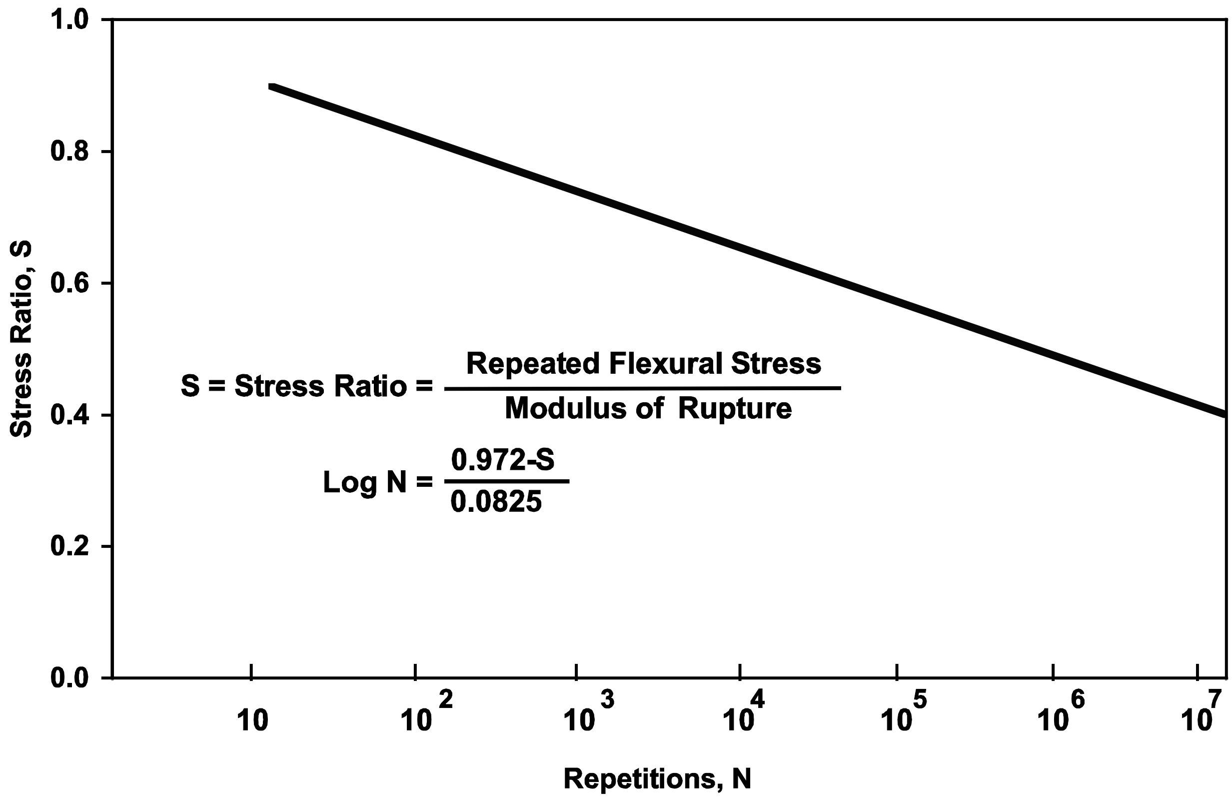

Figure 5.2 Recommended Stress Ratio-Fatigue Relation for Cement-Stabilized Materials

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6.0 LIME AND LIME/FLY ASH STABILIZATION

6.1 General

Lime is an effective additive for plastic soils, improving both workability and strength. It is not effective in cohesionless or low cohesion materials without the addition of pozzolanic additives. There are many similarities between materials stabilized with cementitious stabilizing agents and lime. They have similar composition, resulting in comparable behavior, and require similar materials characterization, structural design procedures, and construction considerations.

However, there are significant differences in the nature and rate of the cementitious reactions, and these differences often provide a basis for choice between cementitious stabilizing agents and lime.

6.2 Materials

6.2.1 Types of Lime

Lime comes in a number of forms:

- hydrated (or slaked) lime (calcium hydroxide),
- quicklime (calcium oxide);
- dolomite lime (calcium/magnesium oxide), and
- agricultural lime (calcium carbonate).

Agricultural lime is not suitable for stabilization, and dolomitic lime is not usually as effective as hydrated lime or quicklime.

All commercial lime products have impurities such as carbonates, silica, alumina, etc., which dilute the active additive but are not harmful to the stabilization reaction.

Hydrated lime comes as a dry, very fine powder or as slurry. The water contents of common lime slurries can range from 80 to 200%. Quicklime and dolomitic limes are commonly much more granular than the hydrated products and are available only as a dry product. These forms of lime react rapidly with available water, producing hydrated lime and releasing considerable amounts of heat. Table 6.1 summarizes the properties of lime. Lime contents are expressed as the equivalent of 100% pure hydrated lime.

Table 6.1 Properties of Different Types of Lime

	Hydrated Lime	Quicklime	Slurry Lime
Composition	$Ca(OH)_2$	CaO*	$Ca(OH)_2$
Form	fine powder	granular	slurry
Equivalent $Ca(OH)_2$ /Unit Mass (Available Lime)	1.00	1.32	0.56 to 0.33**
Bulk Density (tons/yd^3)***	0.32 to 0.39	0.73	0.875

* $CaO + H_2O \rightarrow Ca(OH)_2$ + Heat
** Moisture contents of slurries can vary from 80% to over 200%
*** tons/yd^3 = 0.695 tonnes/m^3

6.2.2 Reactions

Because the oxide reacts immediately with any available water to form hydroxides, the main reactions between all common lime types and materials are alike. Adding lime results in the following:

- It has an immediate effect on clay, improving its grading and handling properties by promoting flocculation of the clay particles. The effect varies with the actual clay minerals present (that is, it depends on the degree of pozzolanic material in the soil). The effect is large with montmorillonite group clays and low with the kaolinite clay groups.
- It has long-term strength gains.
- It allows reduced pavement thickness since the stabilized material can be treated as a base/subbase layer.

Long-term strengthening (pozzolanic reactions) occurs in the highly alkaline environment (pH >12.3) that promotes the dissolution of the clay, particularly at the edges of the clay plates, and permits the formation of calcium silicates and aluminates at these sites. These cementitious products are similar in composition to those of portland cement.

This process is relatively slow because the available lime has to diffuse through both the matrix of the material and the initial cementitious products. The stabilization reactions cannot proceed unless there are clays or some other pozzolanic material within the pavement that will react with the lime. Lower temperatures (below 15°C) slow the lime-pavement material reactions, and high organic contents similarly impede those reactions.

6.2.3 Properties of Clays Stabilized with Lime

For clays, the effect of lime on volume and moisture stability, strength, and elastic behavior is similar to the effects of cement. The following sections deal only with those aspects of materials stabilized with lime that are significantly different from materials stabilized with portland cement.

6.2.3.1 Rate of Strength Gain

For materials stabilized with lime, the rate of strength gain (tensile strength or UCS) is considerably less than with cementitiously bound materials. Materials stabilized with lime and supplementary cementitious materials will continue to gain strength with time, provided curing is sustained. The rate of strength gain is temperature-sensitive and depends on the lime content. Therefore, exercise caution in accepting results of high-temperature accelerated testing without validation at field temperatures. High temperatures can cause other types of bonds to form that would not normally occur in the field. Accelerated curing temperatures should not exceed 40°C.

Lime-stabilized materials are usually evaluated at both 7 and 28 days. High lime contents will not necessarily produce high early strengths. Figure 6.1 illustrates the variations in strength with time and lime content for lime-stabilized materials.

6.2.3.2 Moisture-Density Relationships

Increasing lime content increases the optimum moisture content of the material being stabilized, due to the fine-grained nature of hydrated lime. This effect is further increased by delaying compaction once the lime is added (see Figure 6.2).

6.3 Design Considerations

6.3.1 Appropriate Conditions for Stabilization with Lime

As with all forms of stabilization, there are two areas of consideration:

- material factors, dealing with the composition of the material to be stabilized and its response to lime, and
- production factors.

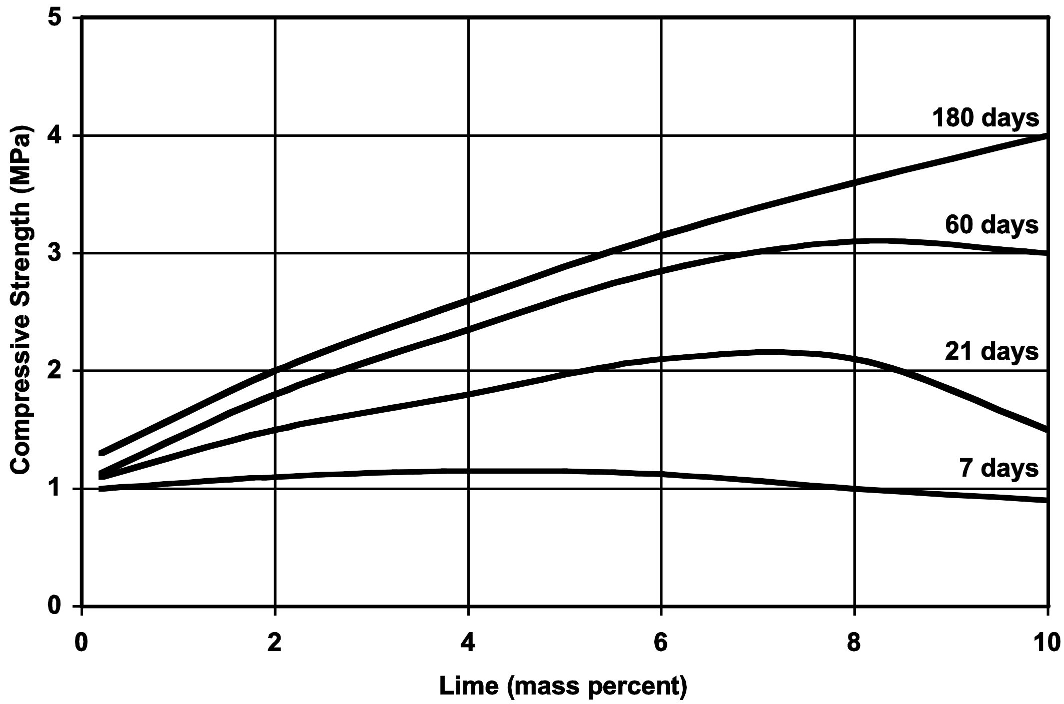

Figure 6.1 Variation in Compressive Strength as a Function of Lime Content and Time

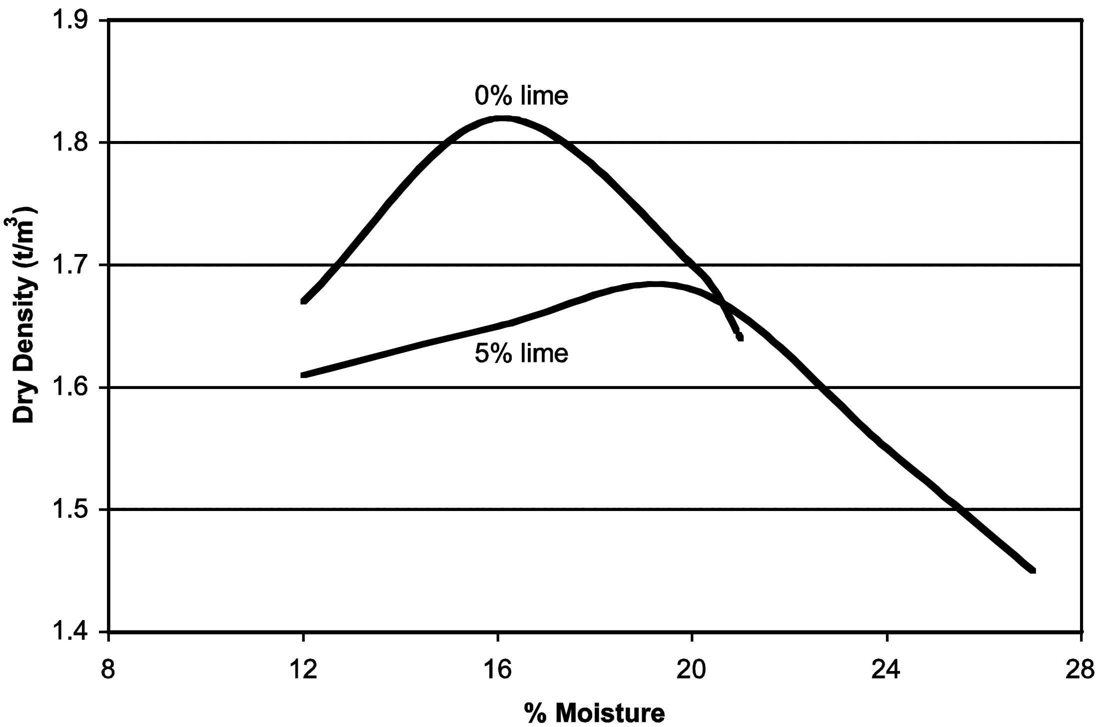

Figure 6.2 Effect of Lime on Optimum Moisture Content and Density

6.3.1.1 Material Factors

For lime to be effective, the material being treated must contain clay particles or pozzolanic materials that are reactive to lime. In general, the more plastic the clay fines and the higher the clay content, the larger the lime content required to produce a specific strength gain or other effect. However, the amount of bonding achievable with lime is limited by the amount of reactive material. Assess the initial lime demand for the soil to be stabilized. Then increase the lime slightly. This ensures that you achieve the stabilized long-term properties after the initial reaction of the lime with the soil.

The advantage of using lime instead of cementitious stabilizing agents increases with increasing plasticity and fines content. Generally, soils with a PI < 10 will respond better to cementitious binders. For plasticity index (PI) reduction and workability improvement using lime modification, add enough lime so that additional quantities do not result in further changes in PI.

For lime stabilization, use pH testing to determine whether a soil is reactive to lime and to estimate an approximate lime content, augmented by 28-day unconfined compressive strength (UCS) testing to establish the optimum lime content. The optimum lime content occurs when the plot of UCS vs. lime content peaks. An additional 1% is usually used to allow for losses and mixing variations. Sugars and reactive organic materials can retard the development of cementitious bonds with both cementitious binders and lime.

6.3.1.2 Production Factors

The following factors significantly affect the quality of lime-stabilized materials:

- quality of water,
- quality of lime,
- uniformity of mixing and curing,
- compaction, and
- clay content.

Adding lime normally promotes granulation of the material being stabilized. In materials that are difficult to break down, the lime-material mix is sometimes moist cured, from a few hours to a day, after light rolling to reduce contact with air, and then remixed. The initial lime addition may be a portion or the whole of the design lime content. This process is sometimes called mellowing.

Lime will diffuse slowly throughout clays and stabilize the lumps. Unless high early strength is particularly important, it is unnecessary to seek fine granulation. About 80 to 90% of the soil's clods should pass the 26.5 mm sieve. If temperatures are low at the time (< 15˚C), then more attention should be given to breakdown.

Using quicklime to establish a working platform on a wet clay is a useful construction expedient and uses the exothermic reaction of the lime as it hydrates to reduce the moisture content of the soil.

6.3.1.3 Compaction Process

The initial rate of reaction with lime allows time to achieve adequate compaction and riding qualities of lime-stabilized materials. If you are seeking high strengths, you need to perform early compaction to achieve as high a density as possible. Delayed compaction lowers the density but this is not as severe as cementitious binders.

6.3.2 Evaluation and Use of Materials Stabilized with Lime

The evaluation techniques and methods for materials stabilized with lime are similar to those used for cementitious binders. Lime is often used to modify materials, particularly those with high plasticity. If modification without achieving high strengths is the aim, the stabilized material can be reworked one or two days after initial compaction. If high strengths are required, you need to exercise careful control over field procedures, particularly moisture control, early rolling, and effective curing. Hydration cracking of lime-stabilized materials is not usually a major problem.

The similar range of materials for subgrade, subbase, and base can be treated with lime or cement. Certain conditions will favor the use of lime. Quicklime and, to a lesser extent, hydrated lime are particularly suitable for treating wet plastic clay subgrades. They provide effective working platforms from otherwise untraffickable situations. Lime slurry is not suitable for this application.

Lime is effective in modifying excessive plastic properties of subbase and base materials. Such modification of base materials is a widely accepted and successful practice. At lime contents of less than about 3%, the risk of undesirable shrinkage cracking is low, and it would rarely be necessary to take special measures to combat reflective cracking.

The use of lime slurry may have advantages in urban areas since it reduces environmental issues such as skin irritation to workers and passersby during hot and/or windy conditions.

For lime pozzolan stabilization and other supplementary cementitious materials, the lime and the pozzolan (or other component additives) are dependent variables. This requires a comprehensive testing program to determine the optimum lime-to-pozzolan ratio (or ratio of other components). In this regard, take into account the following:

- the costs involved for each of the additive components, and
- the need for filler to correct a particle-size distribution deficiency.

6.3.3 Choice of Lime Type

While the type of lime does not appear to be significant for determining the long-term structural properties of the stabilized materials, it has considerable influence on the construction processes. In selecting the type of lime for any particular job, take into account the following:

- Nuisance—hydrated lime can cause a dust problem even with very light winds. Minimize its use in urban or windy areas. Dust is not a serious problem with quicklime. Lime slurry is dust free.
- Soil moisture—quicklime and hydrated lime are effective in drying out wet soils, but slurry limes cannot be used for this purpose. Slurry limes are very suited to dry soil conditions where water may be required to achieve effective compaction.
- Lime content—if the content of the additive is to be kept low, quicklime is particularly effective. The total amount of lime slurry will usually be limited by the soil moisture

content and, in general, lime slurry is limited to low additive contents (< 3%) and dry construction conditions.
- Available equipment and expertise—these factors are always important. Automated spreaders, adequate mixing, and compaction equipment are essential to achieve good results.

While quicklime can be immediately mixed into wet subgrades without problems, you should allow the lime to hydrate before it is mixed into subbase and base materials because unhydrated particles of quicklime may cause expansion with possible blowouts in the compacted materials.

6.4 Construction Considerations

The National Lime Association recommends that lime stabilization be conducted only when the temperature is above 40°F (and rising). Lime modification can be used in colder temperatures. Hydrated lime should not be applied on frozen ground. Lime-stabilized bases should be completed one month before the first hard freeze.

Basic instructions on first-aid procedures and appropriate facilities—such as protective creams, burn creams and ointment, fresh water, and eyewash glasses—should be available for the safe handling of these materials, particularly in hot weather.

6.5 Expected Performance and Cost

Lime has not been widely used in Alaska because of the lack of appropriate soils. Lime could be used with silty sands or sandy gravels if fly ash is added to facilitate the pozzolanic reaction. There is no cost data on lime-stabilized soils for the state of Alaska.

6.6 References

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7.0 MECHANICAL STABILIZATION

7.1 Mechanical Stabilization with Granular Material

7.1.1 General

Improving one material by blending it with one or more other granular materials is referred to as mechanical stabilization. This type of stabilization provides a direct means of altering the particle-size distribution. Plasticity changes may also result.

Mechanical stabilization may involve the following:

- mixing materials from various parts of a deposit at the source of supply,
- mixing selected imported material with in situ materials, and
- mixing two or more selected, imported natural gravels, soils, and/or quarry products on site or in a mixing plant.

Materials produced by mechanical stabilization have properties similar to conventional unbound materials and can be evaluated by conventional methods for unbound granular materials. Alaska DOT&PF has added silt and/or clay to gravel containing little or no fines.

7.1.2 Materials

Materials requiring mechanical stabilization have properties that make them deficient to be used as base or subbase materials. Typically, such materials are:

- poor-graded products;
- dune- or river-deposited sands;
- silty sands, sandy clays, silty clays;
- crusher run products;
- waste quarry products;
- industrial byproducts; and
- high-plasticity pavement materials.

7.1.3 Design Criteria for Granular Stabilization

The principal properties affecting the stability of base and subbase materials are internal friction and cohesion.

Internal friction is generated primarily as a result of characteristics of the coarser soil particles and the particle-size distribution (PSD), or grading. Cohesion (and shrinkage, swelling, and compressibility) results primarily from the quantity and nature of the clay fraction as indicated by the plastic properties, sand equivalent, and maximum dry compressive strength (MDCS).

7.1.3.1 Particle-Size Distribution

While maximum frictional strength does not necessarily coincide with maximum density, achieving high density will generally provide high frictional strength. Maximum density grading is obtained with the closest packing and minimum voids when:

$$p = \left[\frac{d}{D}\right]^n \qquad \text{Eq. 7.1}$$

where
p = % passing sieve size,
d = particle size
D = maximum particle size, and
n = 0.45 to 0.50 for most materials.

For materials with a maximum size of 19 mm, the amount of fines passing the 75 μm (#200) sieve will be 6 to 8% for n values of 0.50 and 0.45, respectively. When relatively low permeability is required, materials should be of uniform particle-size distribution within the limits of 0.50 and 0.33 for n.

Where n is less than 0.33, the fines content may be excessive. A high fines content will result in reduced permeability and may lead to the development of positive pore pressures and instability during compaction and in service. However, these materials will perform well if moisture conditions are controlled. When n is greater than 0.50, the material tends to be harsh and may be prone to segregation and raveling.

7.1.3.2 Plastic Properties

The limits for liquid limit and plasticity index (PI) given in Table 7.1 are generally accepted as satisfactory design criteria for granular stabilized bases. Linear shrinkage (on material passing the 425 μm [#40] sieve) and the PI of a material are usually related. Linear shrinkage limits may be determined by test or estimated from PI values. Typical limiting values for linear shrinkage are 2% for sealed pavements and 3% for unsealed pavements.

Table 7.1 Desirable Limits for Plastic Properties of Granular Stabilized Base Materials

Pavement Type	Liquid Limit (%)	Plasticity Index (%)
Sealed		
Chip Seal	≤ 25	2 to 6
HMA	≤ 25	≤ 6
Unsealed	≤ 35	4 to 9

7.1.3.3 Strength Tests

For strength tests, such as the California R-value and the California bearing ratio (CBR), use the criteria normally specified for base and subbase materials. You may also use repeated load triaxial testing to characterize the elastic and plastic deformation characteristics of granular stabilized materials.

Selecting design criteria, particularly for lightly trafficked roads, should take into account local experience. Many materials that do not meet normal specifications perform well in lightly trafficked, well-drained situations.

7.1.4 Construction

In addition to adequate investigation and design, good construction practice and testing are essential to achieve a properly performing material. You must carefully proportion and thoroughly mix the constituent materials to produce a homogeneous material that can be compacted and finished as specified.

One of the major uses for granular stabilization is in constructing unsealed pavements to minimize dust. It involves mixing materials to ensure that the correct grading and plasticity requirements for a low-dust wearing surface are met. This is usually a well-graded mixture with a specifically designed amount of cohesive fines. This is not easy to find without using special additions such as commercial clay (for example, montmorillonite or stabilite).

A suitable grading for wearing surfaces for unsealed pavements is given in Table 7.2.

Table 7.2 Suggested Particle-Size Distribution for Unsealed Pavements

Sieve Size (mm)	% Passing Sieve	
	Wearing Course (Base)	Subbase
26.5	100	100
19.0	85–100	70–100
9.5	65–100	50–80
4.75	55–85	32–65
2.36[1]	20–60	25–50
0.425	25–45	15–30
0.075[2]	10–25	5–15

1. The maximum particle size for subbase is often increased to 40 mm.
2. The 0.075 mm fraction is the fraction containing the dust particles.

7.1.5 Expected Life and Performance

Alaska DOT&PF has not used this technique extensively; hence, there is little performance or cost data available.

7.1.6 References

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Skorseth, Ken, and Ali A. Selim, Gravel Roads – Maintenance and Design Manual, South Dakota LTAP Program, Federal Highway Administration, November 2000.

7.2 Mechanical Stabilization with Geofibers

7.2.1 General

This is a different form of mechanical stabilization rather than with the addition of granular material as covered in Section 7.1. Instead of improving one material by blending it with one or more granular materials, the added material addressed within this section is synthetic geofiber. The most common types of geofibers are made of polypropylene. The appearance of individual geofiber strands is similar to the fibrous strands that are the woven constituents of a slit-film woven geotextile.

Alaska research experience has indicated that fibrillated geofibers with lengths of about 51 mm may be the best performing of the available geofiber types. Individual geofibers are not solid strands of polypropylene. Instead each strand is fibrillated, i.e., composed of many smaller, individual interconnected filaments.

Stabilization is achieved by mechanically mixing a soil or aggregate with a prescribed quantity of geofibers. The resulting mixture is effectively stabilized by means of the intertwining root-like structure provided by the geofibers, which are (ideally) dispersed throughout the volume of the granular material. Results of such stabilization can transform relatively unstable materials, e.g., poorly graded sands, into load bearing fill materials that also tend to resist penetration because of the intertwining, root-like structure. Such materials actually become stronger when penetrated or rutted with use.

In certain materials, the use of geofibers can greatly increase both the bearing capacity (CBR) and the shear strength—thus transforming low-bearing strength materials into useful fill capable of handling heavy vehicles.

7.2.2 Materials

Materials most appropriate for stabilization with geofibers include inherently unstable granular types that must (by economic necessity in certain areas of Alaska) be employed as loadbearing layers within the pavement structure. Such stabilized materials would be used, for example, as subbase, or base course.

It is possible to achieve some degree of stabilization of silt using geofibers, but silt is generally not as responsive to geofiber treatment as coarser granular material. On the other hand, coarser, poorly graded sands, for example, usually benefit more from the addition of geofibers and fines (very fine sand or silt), than by the addition of geofibers alone.

7.2.3 Design Criteria for Geofiber Stabilization

Poorly graded sands can exhibit little stability and low CBR values unless confined. The addition of -#200 fines and geofibers may greatly influence the CBR value of unconfined sands.

Figure 7.1 indicates that poorly graded sands with little natural fines can benefit from adding a combination of geofibers and/or fines. Note that the CBR values for sands with high natural fines content may be decreased by adding geofibers.

Laboratory testing is required to quantify the benefit of adding geofibers and fines to any particular sand material. The broad, shaded lines in Figure 7.1 indicate the large amount of scatter in the actual test data.

Alaska research indicates success with sand-type materials with geofibers in amounts less than 1% (by total weight). In fact, a geofiber content of 0.5% appears to be about optimal for several Alaska sands studied. Silts might also benefit from small amounts of geofibers (<.2%).

Table 7.3 shows how four materials benefited from use of geofiber additives under laboratory conditions.

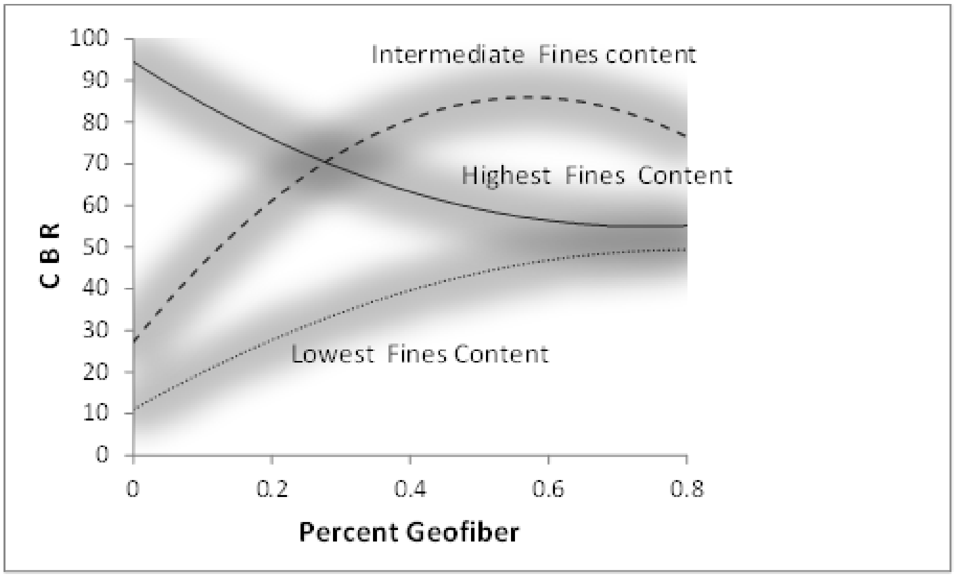

Figure 7.1 Influence of Fines and Geofiber Content on CBR Value (Generalized)

Table 7.3 CBR Improvements Measured in the Laboratory

% Fines Added	% Geofibers Added	CBR Improvement
Ottowa Sand 20	0.8	CBR 20 increased to CBR 80
Monterey Sand 20 to 30	0.2 to 0.5	CBR < 20 increased to CBR 85+
Horseshoe Lake Sand* (natural content 6 to 7% P200)	0.5	CBR 25 increased to CBR 45
Fairbanks Silt* n/a	0.2	CBR <35 increased to CBR 60+

*Natural Alaska Material

The example in Figure 7.2 (western Alaska fine sand) indicates at least a doubling of CBR for geofiber versus non-geofiber-treated material. The right-hand side of Figure 7.2 indicates much more than an additional doubling of CBR after geofibers in the sample have been stretched by further penetration of the CBR load device.

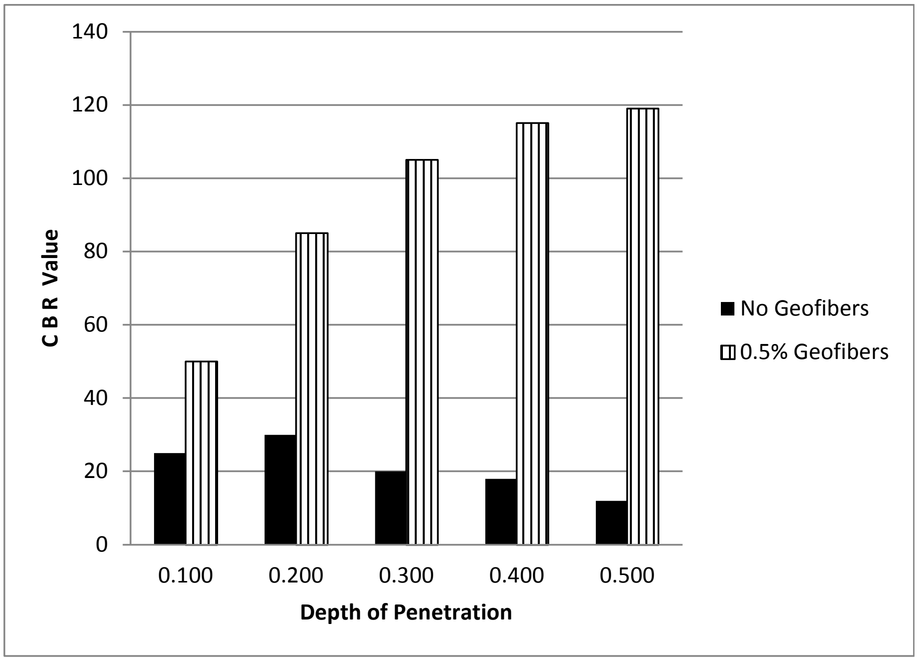

Figure 7.2 CBR Values Versus Increasing CBR Penetration for Sandy Material with and without Geofibers

Therefore, specifications written for projects with geofiber-stabilized sandy materials should account for:

1. an increase in material strength due to the simple presence of the geofibers, plus
2. an additional significant increase in strength after the geofibers have been stretched by construction equipment and traffic.

7.2.4 Laboratory Strength Tests

Research conducted at the University of Alaska has used the laboratory CBR test (ASTM D1883) and the direct shear test (ASTM D3080) to characterize the strength of materials stabilized solely with geofiber reinforcement and with a combination of geofibers and chemical additives.

7.2.5 Construction

The exact weight percent addition of geofibers can be critical, especially in silty materials. A form of construction project mix design for such material can be done as in the following example for determining optimum geofiber content:

Run a series of CBR tests using various percentages of geofibers (mixed at optimum moisture). Plot CBR values versus % fiber content for fiber contents between about 0 and 1%. Several CBR values will be plotted for each % fiber content—these represent the range of variation in CBR value with increasing CBR penetration. The optimum fiber content (often around 0.5% for sands and less for silt) is selected by subjective interpretation of the plotted data. Usually, the % fiber content exhibiting a set of obviously higher CBR values will be selected.

Treatment of soil or aggregate materials with geofibers requires that the design quantity of geofibers be distributed uniformly throughout the treated material. Too little or too much geofiber within a given volume of the treated material will completely change the expected properties within that volume.

Construction equipment must be employed that thoroughly mixes the geofibers throughout the intended thickness of material.

Considering the "strain hardening" property of geofiber-stabilized materials (see Figure 7.2), construction specification requirements should:

1. limit scarification and re-grading of geofiber-treated materials, and
2. require compaction of treated material using a rubber tire roller (maximizes stretching of the geofibers during compaction).

Random sampling/testing will be necessary to verify that the required uniform mixture has been obtained. Post-mixing field CBR tests (ASTM D4429 Standard Test Method for CBR—California Bearing Ratio—of Soils in Place) can be used to verify that required minimum bearing capacities are being achieved.

Geofibers are not susceptible to deterioration through contact with natural fluids. Geofibers are susceptible to oxidative destruction by ultra violet (UV) exposure. Geofibers contained within a compacted volume of soil or aggregate would normally be exposed only along the surface of the volume—so degradation should be insignificant.

Limit UV exposure! During construction, protect bulk quantities of geofibers from direct sunlight exposure prior to incorporating geofibers into soil or aggregate materials.

7.2.6 References

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8.0 SALT STABILIZATION

8.1 General

Salt stabilization has focused primarily on using calcium chloride as an additive to improve performance of granular materials. The stabilizing action of this additive is in its ability to attract and hold moisture and to reduce the void space of the compaction material. More information on this topic is available from the Salt Institute

(www.saltinstitute.org).

8.2 Materials

The aggregate gradations suitable for salt stabilization are those typically recommended for aggregates used as an unsealed surface. These gradations have an increased amount of fines, or the percent passing the No. 200 sieve. A minimum value would be 5% with typical values in the range of 6% for base course and 10% for a wearing surface.

8.3 Design Considerations

Quantities vary slightly with application purpose and gradation. Typical application rates are from 1.5 to 2.0 lbs. of calcium chloride/square-yard/year/inch of material for full stabilization efforts. The ability to hold moisture can improve the workability and maintainability of the aggregate surface, and the material may also be stronger. However, if the surface becomes wet, it may be less stable since the salt holds in the moisture for longer periods of time.

8.4 Construction Considerations

You should add aggregate material in layers to establish the specified grade and crown after compaction. Add the salt shortly afterward to take advantage of all subsequent mixing operations. Blend all materials thoroughly using a travel plant, a motor grader, or a pugmill. After the material is mixed, spread it out to provide a lift of at least 3 inches for compaction. Continue to compact until sufficient fines are brought to the surface to bind and seal the aggregate. You can leave the surface unsealed or seal it with a chip seal or a thin hot mix asphalt overlay.

8.5 Expected Performance and Costs

Most of the salt applications used in Alaska have been for dust control and not for full depth stabilization. As a result, there is little performance or cost data for these types of applications. Also, the salt can migrate over time, which causes a loss in the stabilizing effects.

8.6 References

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9.0 Non-Traditional Stabilization

9.1 General

The vast majority of pavement stabilization carried out in Alaska is conducted using cement or asphalt stabilizers or blends of these stabilizing agents.

However, other forms of stabilization have been used in trial sections and these are briefly described in this section. Many of these chemical binders have been used as dust suppressants but may also have the ability to alter other properties such as strength and permeability. These binders should be assessed for their ability to improve the structural performance of pavements in a similar way to the cementitious, lime, and asphalt binders.

One area where other methods of stabilization are used is on unsealed pavements or pavements under construction to reduce dust nuisance and improve safety. This is an area that is continuously developing, so individual stabilizing agents should be assessed on the merits of their performance for a particular application. Broad guidance for the application of other stabilization agents follows.

9.2 Dust Suppression

Short of sealing a road, there are no known ways to eliminate dust emissions on a long-term basis by using a single process or just one application of a chemical dust suppressant. Dust suppression techniques fall into three main categories:

- good construction and maintenance practices,
- mechanical stabilization, and
- chemical dust suppressants.

9.2.1 Good Construction and Maintenance Techniques

Good construction and maintenance techniques are fundamental for a longer life and high level of service for unsealed roads. Providing a 3 to 5% crown and adequate drainage are critical in retaining a hard road surface that minimizes dust.

9.2.2 Mechanical Stabilization

Section 7.1 describes mechanical stabilization. It involves mixing aggregate materials to ensure that the correct grading, plasticity, and strength requirements are met. Section 7.2 describes mechanical stabilization with mixtures of poorly graded aggregate materials and geofibers. Such mixtures can produce large stability increases as measured by bearing capacity.

9.2.3 Chemical Binders

If you continue to have a lot of dust and cannot achieve dust suppression through mechanical stabilization, consider using chemical dust suppressants as an adjunct to the other methods mentioned in Sections 4–6. Chemical dust suppressions generally have a limited life and require regular applications to maintain satisfactory control of dust on a long-term basis. Selecting a particular type of dust suppressant depends on material composition and climatic factors.

Chemical dust suppressants can be broadly classified as

- organic non-asphalt products,
- water attracting chemicals,
- waste oil (not recommended)
- petroleum-based products,
- electrochemical products,
- microbiological binders, and
- polymers.

9.2.3.1 Organic Non-asphalt Products

Organic non-asphalt products consist primarily of lignin sulphonates, a byproduct of the paper pulping industry. Their action in the soil is to adhere to and "glue" together soil particles. They also act as a clay dispersant, making clay more plastic and increasing its density after compaction. Lignosulphonates are effective dispersing agents for clays. During rain, the dispersed clays in the soil swell, filling the pore spaces, which tends to reduce water infiltration. During drying out, the lignosulphonate distributed throughout the soil reduces the rate of evaporation. These products are water soluble and have a limited lifespan if used in wet environments.

9.2.3.2 Water-Attracting Chemicals

Water-attracting chemicals consist of hygroscopic (water loving) materials, primarily chlorides and salts (see Section 8). Salts suppress dust by attracting and trapping moisture from the air, keeping the pavement wearing surface moist.

9.2.3.3 Petroleum-Based Products

Petroleum-based products are recycled waste oils, asphalt emulsions, and tars. Their effect is to produce heavy agglomerations of fine dust particles. Oils are often the longest lasting dust suppressants, but may carry the highest environmental consequences, particularly if waste oil is used.

9.2.3.4 Electrochemical Stabilizers

Electrochemical stabilizers consist of enzymes and sulphonated petroleum and are highly ionic. Electrochemical dust suppressants work by expelling adsorbed water from the soil, decreasing air voids and increasing compaction. Most of these products need some clay particles to work in the material. If traffic occurs during wet weather, you should apply a wearing surface to the electrolyte-treated material. The maximum strength of the material may not be attained for up to 20 days following application. This type of stabilization produces a more cohesive road surface, effectively increasing the energy required to dislodge particles.

9.2.3.5 Polymer Stabilizers

Soil stabilization polymers that have so far become familiar to Alaska researchers are of the polymer emulsion type. Most of the polymer emulsions intended for soil stabilization are copolymers of the vinyl acetate or acrylic-based types. Polymer emulsions can provide gains in strength under wet conditions, but, as with asphalt emulsions, they gain considerable additional strength after breaking and curing (aging). In addition to simple compression strength gains, polymer emulsions tend to improve the mechanical "toughness" of the stabilized materials. This improvement allows materials to undergo additional strain prior to failure—in effect reducing brittleness compared to cement stabilization. The addition of curing agents may not only expedite the strength gain of polymer stabilizers but actually increase strength as well.

9.3 Expected Performance

Chemicals have been used in Alaska with mixed success (Tables 9.1 and 9.2). They have also been used only in small test sections or in the laboratory. Hence, good performance and cost data are not available.

Table 9.1 Chemical Products Evaluated by Alaska DOT&PF Prior to 2002[1,2]

Class	Product
Clay Additives	Stabilite
Enzymes/Electrolytes	EMC[2] Perma-Zyme
Tree Resins	Road Oyl

[1] Stabilite, EMC[2], and Road Oyl are products from Soil Stabilization Products Co., Inc. (www.sspco.com)
[2] Perma-Zyme is from the Charbon Group Inc. (www.natural-industrial.com)

Since about 2002, newer chemical stabilizers have been laboratory- and field-tested by the University of Alaska. These stabilizers (polymer emulsion types) are identified in Table 9.2.

Table 9.2 Polymer Emulsions Evaluated by Alaska DOT&PF (after 2002)

Class	Product
Vinyl Copolymer Emulsion w/1 &w/2	SoilTac (a product of Soilworks LLC)
Aqueous Acrylate Polymer Emulsion w/1 & w/2	DirtGlue (a product of DirtGlue Enterprises)
Aqueous Acrylic Vinyl Acetate w/1 & w/2	SoilSement (a product of Midwest Industries

1 Polycure (Proprietary additive by DirtGlue Enterprises
2 Extended Use (proprietary curing additive by Midwest Industries)

Laboratory tests using the Table 9.2 polymers produced compression strength improvements for Alaska materials. Results from a number of tests on three Alaska materials are shown as generalized trends in Figure 9.1. The bottom curve of Figure 9.1 illustrates the case where a significant strength gain occurred over just a narrow range of curing agent percent. The broad, shaded lines in Figure 9.1 indicate the large amount of scatter in the actual test data.

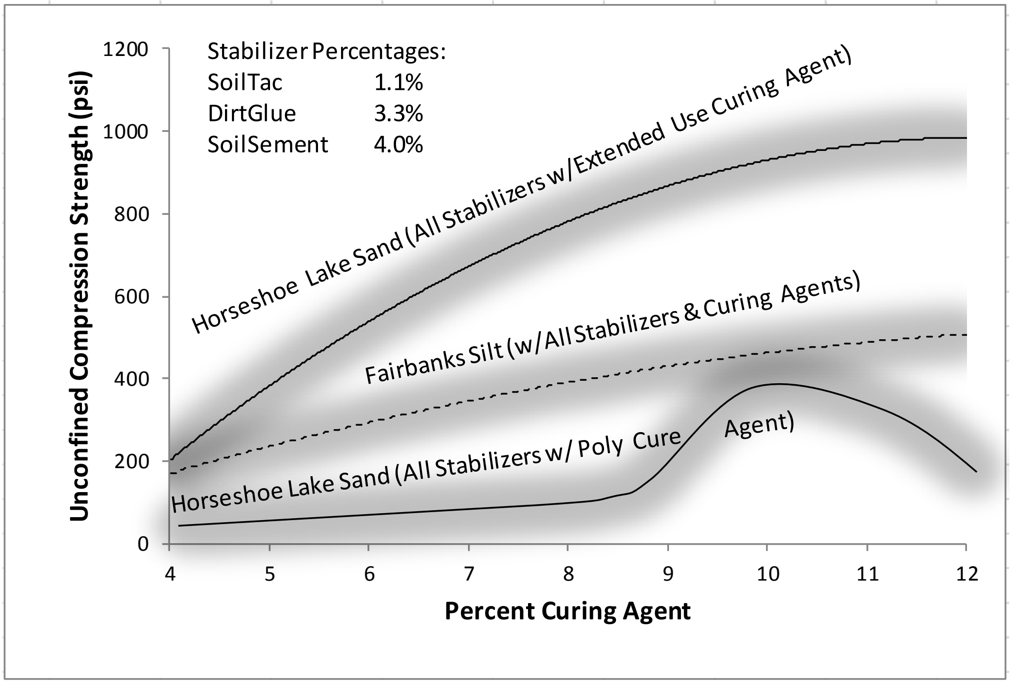

Figure 9.1 Strength Gains for Alaska Materials Using Polymer Stabilizers and Curing Agents

Figures 9.2 and 9.3 provide example moduli obtained for Midwest Extended Use with 4% Soil Sement additive. The moduli were obtained by measuring the slope of the strength curves from the unconfined compression strength tests. An estimate of resilient modulus can be obtained by taking 90% of the modulus obtained from the unconfined compression test. This value can then be used in the mechanistic pavement design procedures.

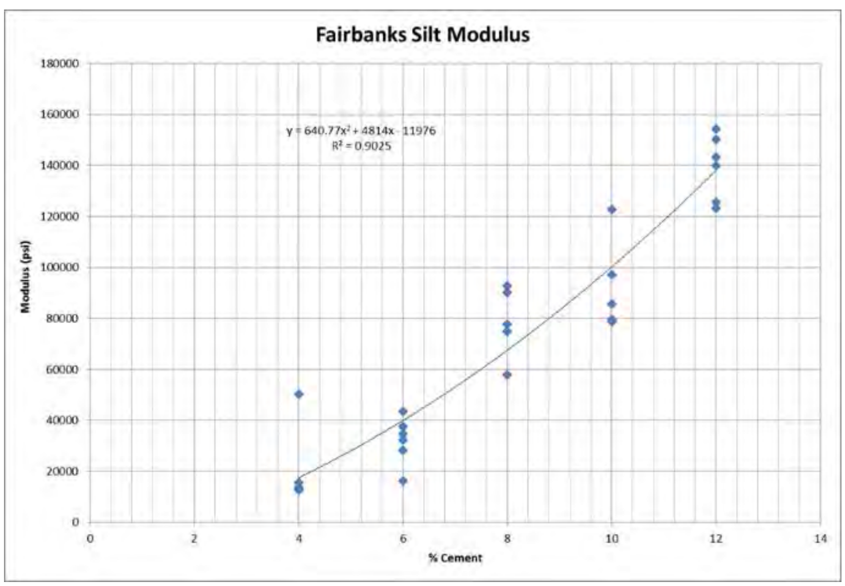

Figure 9.2 Example Modulus for Cement Modified Silts with Polymer Additives

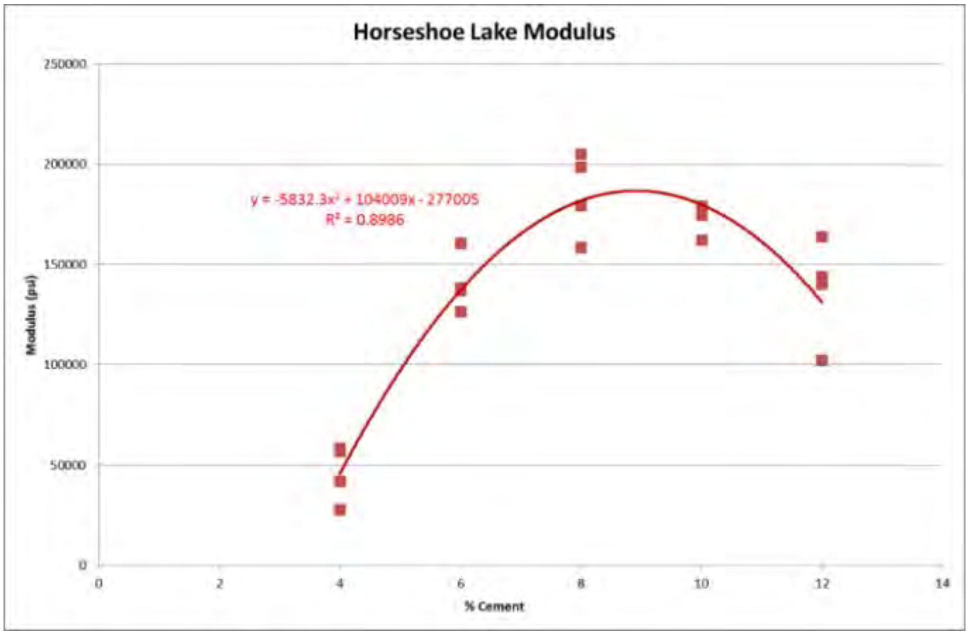

Figure 9.3 Example Modulus for Cement Modified Sands with Polymer Additive

Some additional laboratory testing was done on Fairbanks Silt using both polymer emulsions and geofibers. The results are shown in Table 9.3 and indicate CBR gains of 4 to 7 fold over the untreated silt.

Table 9.3 CBR Improvements for Fairbanks Silt Using Polymer Emulsions and Geofibers

% Stabilized by weight	% Curing Additive by weight	Approximate CBR Improvement
SoilTac @ 1.1%	No additive	CBR 15 increased to CBR 54
DirtGlue 3.3%	Polycure 10%	CBR 15 increased to CBR 95

9.3.1 Improvement of Fairbanks Silt with Polymer Emulsion Stabilizers and Geofibers

The CBR improvements gained using combinations of stabilizer and curing additive were significantly greater than those gained using only the stabilizer—even though two different stabilizers were used.

9.3.2 Improvement of Western Alaska and with Polymer Emulsion Stabilizers and Geofibers

The example in Figure 9.4 (western Alaska fine sand) indicates a doubling of CBR for geofiber versus non-geofiber-treated material. The right-hand side of Figure 9.4 indicates much more than an additional doubling of CBR after geofibers in the sample have been stretched by further penetration of the CBR load device.

Figure 9.4 also shows that there can be a very substantial added CBR benefit in using both geofibers and stabilizing fluid. Note however that the relative advantage of the stabilizing fluid additive decreases as geofibers are stretched by further penetration of the CBR load device.

Note in Figure 9.4 that CBR values for samples containing stabilizing fluid were aged before testing.

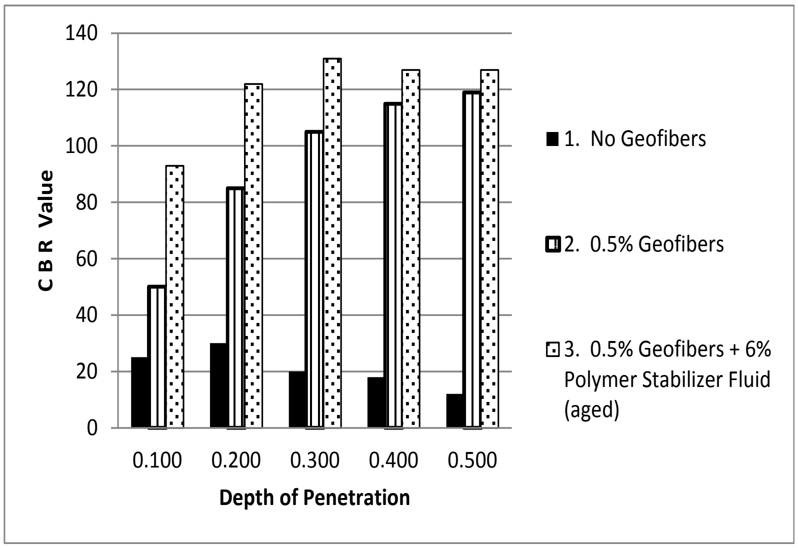

Figure 9.4 CBR Values Versus Increasing CBR Penetration for 1) Sandy Materials, and 2) With Geofibers, and 3) With Geofibers and Stabilizer Fluid

Figure 9.5 shows the typical failure plane in Fairbanks silt during an undrained-unconfined compression test. Fairbanks silt is again tested with the addition of silt. Figure 9.6 shows the failure is similar to that expected of granular material. This test also shows that the addition of fiber increases the compressive strength of the soil and causes the soil to be strain hardening; that is, strength increases with additional strain. This is consistent with the results of the CBR testing.

Therefore, specifications written for projects with sandy materials stabilized using both geofibers and polymer-stabilizing fluids should account for:

1. an increase in material strength due to the simple presence of the geofibers, plus
2. an additional significant increase in strength after the geofibers have been stretched by construction equipment and traffic, plus
3. an additional strength gain after the polymer stabilizer has aged (cured).

Figure 9.5 Undrained-Unconfined Compression Test of Silt without Fiber

Figure 9.6 Undrained-Unconfined Compression Testing of Fairbanks Silt with Geofibers

9.4 Construction Considerations

Work safely with stabilizing agents. Always follow the manufacturer's instructions and seek advice about working safely with the products from the manufacturer or contact the Research Office of Alaska DOT&PF.

9.4.1 Specific Concerns When Using Geofibers with Polymer Stabilizers

The exact combination and total weight percent of geofibers and stabilizer added to a particular material is critical for good results. A construction project mix design for material treated by using both geofibers and polymer stabilizer requires a two-step procedure similar to that indicated in the following example:

Step 1. Determine Optimum Fiber Content.

Run a series of CBR tests using various percentages of geofibers (mixed at optimum moisture). Plot CBR values versus % fiber content for fiber contents between about 0 and 1%. Several CBR values will be plotted for each % fiber content—these represent the range of variation in CBR value with increasing CBR penetration. The optimum fiber content (often around 0.5% for sands) is selected by subjective interpretation of the plotted data. Usually, the % fiber content exhibiting the set of obviously higher CBR values will be selected.

Step 2. Determine Optimum Stabilizer Content.

Run CBR tests using various percentages of stabilizer mixed with soil and the optimum content of geofibers (determined in Step 1). Evaluate CBR strengths of aged versus non-aged specimens for each stabilizer %.

Most polymer stabilizers will not mix well with really dry soil. Good mixing requires a significant amount of natural moisture in addition to the polymer. Also, the ability to obtain maximum density in the final product requires that the total moisture, i.e., the sum of natural moisture plus polymer, be close to optimum for the treated material.

Treatment of soil or aggregate materials with a combination of geofibers and polymer stabilizer requires that the design quantities of these additives (perhaps including curing agent) be distributed uniformly throughout the treated material. Too little or too much of any one component within a given volume of the treated material will completely change the expected properties within that volume.

Construction equipment must be employed that thoroughly mixes the geofibers and stabilizer throughout the intended thickness of material.

Considering the "strain hardening" property of materials treated with geofibers and polymer stabilizers (see Figure 9.2), construction specification requirements should:

1. limit scarification and re-grading of geofiber-treated materials, and
2. require compaction of treated material using a rubber tire roller (maximizes stretching of the geofibers during compaction).

Random sampling/testing will be necessary to verify that the required uniform mixture has been obtained. Post-mixing field CBR tests (ASTM D4429 Standard Test Method for CBR—California Bearing Ratio—of Soils in Place) can be used to verify that required minimum bearing capacities are being achieved.

Geofibers are not susceptible to deterioration through contact with natural fluids. Geofibers are susceptible to oxidative destruction by ultraviolet (UV) exposure. Geofibers contained within a compacted volume of soil or aggregate would normally be exposed only along the surface of the volume—so degradation should be insignificant.

Limit UV exposure! Protect bulk quantities of geofibers from direct sunlight exposure prior to incorporating geofibers into soil or aggregate materials.

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10.0 OTHER STABILIZING TECHNIQUES

10.1 General

Other techniques, including drainage and geotechnical fabrics, have also been used in Alaska to stabilize soils. They provide other options for stabilizing soft or wet soils.

10.2 Drainage

Removing excess water using drainage techniques can help stabilize soft wet soils. This technique has been used successfully in combination with fabrics throughout Alaska.

Wicking fabrics that move water from within the soil matric to the shoulder of the roadway have recently become available. These materials are comprised of reinforcing fabric with bundles of very fine fibers that create capillary channels through which the water moves. Capillary action will allow water to move uphill. The vertical height depends on the difference in relative humidity and the configuration of the fibers. However, around 2 feet is common.

Since the water will move from the wetter soil toward the drier soil or air, care must be taken to ensure that the water is not moved to an undesirable location. For example, wicking fabric terminated in a wet ditch may move water into the embankment.

Open-cell fabric allows water to travel to the fabric where it is removed by gravity. Care must be taken to ensure that the fabric has a downward slope in the direction of the desired water flow. Future deformation in the roadway may cause a concentration of water within the embankment.

10.3 Geotechnical Fabrics

Fabrics have been used to reinforce soft soils as well as to separate soils from aggregate layers.

Separation fabrics are commonly used to separate subgrade materials from structural materials. This technique eliminates the mixing of the fines into the structural layer. Care must be taken to ensure that the opening in the fabric called the Equivalent Opening Size (EOS) is appropriate for the fine-grained layer and that the fabric allows for water to freely move through it.

Geotextiles and geogrids are used to strengthen soil in single or multiple layers. It is important that a competent designer design the system when these materials are used. Manufactures provide a number of tools to assist the designer.

The Northern Region used Geoweb products by Presto (www.prestogeo.com) that can be filled with sand to construct the runway at Shishmaref. By using local materials in combination with the Geoweb, the cost of the runway construction was significantly reduced. The runway has been in service over 20 years and has performed well.

10.4 Expected Life and Costs

The limited use of these techniques has not resulted in good life or cost data. Better information on life and costs is required.

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APPENDIX A: Glossary of Terms

Absorption—The penetration of binder into an aggregate or base.

Acidic Soil—A soil having a pH value less than 7.0 (see also Alkaline Soil).

Additive—A substance added in small amounts to help in the manufacture or handling of a product or to modify the end properties.

Alkaline Soil—A soil having a pH value greater than 7.0 (see also Acidic Soil).

Anionic Asphalt Emulsion—A type of asphalt emulsion in which the suspended particles are negatively charged.

Binder—A material used for the purpose of binding particles together as a coherent mass.

Bound Material—Granular material to which cement, asphalt, or similar binders are added to produce structural stiffness.

California Bearing Ratio (CBR)—A measure of the bearing capacity of a soil or granular material obtained from a standard soil test.

Cationic Asphalt Emulsion—A type of asphalt emulsion in which the suspended particles are positively charged.

Cemented Material—Materials produced by the addition of cement, lime, or other hydraulically binding agent to granular materials in sufficient quantities to produce a bound layer with significant tensile strength.

Chip Seal—A thin layer of asphalt material sprayed onto a pavement surface and having a layer of aggregate rolled in.

Cohesive Soil—A soil whose relevant behavioral characteristics are derived largely or entirely from the cohesive bonds associated with the fine fraction.

Compaction Test (Field)—To compare field compaction with maximum dry density of the soil or pavement material.

Compaction Test (Laboratory)—A laboratory test to determine the maximum dry density of a soil or pavement material under specified test conditions.

Cutback Asphalt—A material made from asphalt by the addition of cutter oil for a temporary reduction in viscosity.

Deep Lift—A pavement construction technique whereby stabilization is carried out to depths in excess of 200 mm.

Design Life—Time period during which the quality of a pavement, for example, riding quality, is expected to remain acceptable.

Design Traffic—Cumulative traffic, expressed in terms of equivalent standard axle loads, predicted to use road over time.

Dynamic Cone Penetration (DCP) Test—A test in which the effort to push or drive a standard steel cone into soil at a controlled rate is used as a measure of certain soil properties, such as the field CBR.

Equilibrium Moisture Content—The moisture content that is reached in a soil in a particular environment after moisture movements have ceased.

Equivalent Standard Axle Loads (ESAL)—The number of standard axle loads that are equivalent in damaging effect on a pavement to a given vehicle or axle loading.

Field Density—The density of earthworks or pavement material measured in place.

Fly Ash (FA)—A fine powder of pozzolanic material extracted from the flue gases of a boiler fired with pulverized coal.

Foamed Asphalt—Hot asphalt greatly expanded in volume by the introduction of steam or water.

Gap-Graded Material—Material having a substantially continuous distribution of sizes from coarse to fine, the largest size being several times larger than the smallest size.

Indirect Tensile Strength—The tensile strength of a pavement material measured across the diameter of the sample.

Isotropic—Having properties that are equal in all directions.

Layer—A sequence of one thickness of pavement material placed during one construction operation.

Leaching—The removal of soluble material and colloids by percolating water.

Load Equivalency Factors—The ratio of the number of repetitions of the standard axle load that the pavement can sustain to the number of repetitions of another axle load that the same pavement can sustain for given damage criteria.

Maximum Dry Density—The greatest dry density of a soil obtained when a soil or pavement material is compacted in a specified manner over a range of moisture content. The moisture content at which this density is reached is called the optimum moisture content. Two amounts of compactive effort

are commonly specified, referred to as standard and modified.

Modification—Improving the properties of a material by adding small quantities of an additive.

Moisture Content—The quantity of water that can be removed from a material by heating at 105˚C until no further significant change in mass occurs, usually expressed as a percentage of the dry mass.

Noncohesive Soil—A soil that is lacking the fine fraction, resulting in a loss of the cohesive bonds associated with this fraction. Could also have fines, which are non-plastic.

Nuclear Density Meter (NDM)—An instrument for the nondestructive determination of the density and moisture content of material, using a radioactive source for its operation.

Optimum Moisture Content (OMC)—That moisture content of a material at which it will produce the maximum dry density under a standard test.

Plasticity Index (PI)—The numerical difference between the value of the liquid limit and the value of the plastic limit of a soil.

Poisson's Ratio—Ratio of radial to longitudinal strain.

Pozzolan—A siliceous or alumino siliceous material that in itself possesses little or no cementitious value, but which in finely divided form may be mixed with lime or portland cement to form a cementitious material.

Recycling—The reuse of pavement material by in situ or plant mixing, with or without the addition of new material components.

Reflection Cracking—A visible crack in the pavement surfacing resulting from the movement associated with cracks in the underlying pavement layer.

Rehabilitation—The restoration (that is, stabilization) of a distressed pavement so that it may be expected to function at a satisfactory level of serviceability for longer.

Relative Compaction—The percentage ratio of the field dry density to maximum dry density.

Resilient Modulus—The ratio of stress to recoverable strain under repeated loading conditions. Also referred to as elastic stiffness.

Rutting—The longitudinal vertical deformation of a pavement surface in a wheel path, measured relative to a straightedge placed at right angles to the traffic flow and across the wheel path.

Stabilized Soil—A material that has been modified to improve or maintain its load carrying capacity. Modification may be by the addition of other natural materials such as sand, loam, or clay, or of manufactured materials such as cement, lime, and asphalt.

Standard Axle Load (SAL)—A load of 80 kN (18,000 lb) applied over a single axle with a dual wheel at each end.

Stiffness—A measure of the elastic behavior of a pavement material (σ/ε) and can be determined either in compression or tension.

Subbase—The material layer on the subgrade below the base, for the purpose of making up additional pavement thickness required over the subgrade, or to prevent intrusion of the subgrade into the base, or to provide a working surface on which the remainder of the pavement can be constructed.

Subgrade—The trimmed or prepared portion of the formation on which the pavement is constructed.

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APPENDIX C: Soils of Alaska

C.1 General

The information presented in this appendix comes from the Exploratory Soil Survey of Alaska, which has been prepared for many different users, including farmers, foresters, and agronomists. Great differences in soil properties occur even within short distances. Soils may be organic, seasonably wet, shallow over bedrock or permafrost, gravely, and/or sandy. The type of soil in the region will have a profound effect on the type of additive that can be used to stabilize it.

C.2 Major Land Resource Areas

Alaska has 15 major land resource areas. Each is characterized by a unique pattern of topography, climate, vegetation, and soils. (For expected soil types in each area, contact the head Alaska DOT&PF geologist in each respective region.) Alaska's major land resource areas include the following:

- Southeast Alaska. This includes the mountains of the mainland east of the St. Elias Mountains and the islands of the Alexander Archipelago. It is dominated by rugged hills and mountains that rise from the sea. Strips of hilly moraines border most of the bays and valleys. The area has a cool climate characterized by high precipitation throughout the year. Frost-free seasons are long, but are offset by low summer temperatures and persistent cloud cover. The predominant soil types encountered include in road construction are sandy or silty soils and peat or muskeg.
- Southcentral Alaska Mountains. This area includes the St. Elias Mountains, Chugach, and the Kenai Mountains bordering the Gulf of Alaska and the Wrangell and Talkeetna Mountains farther inland. Moraines, outwash plains, and other glacier features are found throughout this area. The resource area has a variety of climates, ranging from high precipitation and moderated temperatures along the coastal regions to areas with low precipitation and marked seasonal temperature differences in the Interior. At higher elevations, the precipitation is mainly snow and summer temperatures are so low that ice persists throughout the year. The predominant soil types encountered in road construction include silt, sand, and gravel.
- Cook Inlet–Susitna Lowland. This area is a long narrow basin between the Kenai, Chugach, and Talkeetna Mountains to the east and the Aleutian and Alaska Ranges to the west. Most of the northern half of the lowland is drained by the Susitna River and its tributaries. The southern half borders Cook Inlet. The Matanuska Valley is an eastern extension of the lowland from the head of the Cook Inlet. The entire basin is underlain by sediments of the Tertiary age, but the surface consists mainly of glacial deposits, including low moraines interspersed with many lakes, bogs, and broad outwash plains. The climate of the lowland has both maritime and continental characteristics. The Alaska Range protects the area from the extreme temperatures of interior Alaska. Precipitation is moderate in the southern part and fairly low in the central part. The predominant soil types encountered in road construction include silts, silty clays, and muskeg.
- Alaska Peninsula and Southwestern Islands. This area includes the Aleutian Range, the Alaska Peninsula, the Kodiak Island group, other small islands south and east of the peninsula, and the Aleutian Islands. The mountains of the Aleutian Range are mostly

volcanic, whereas the Kodiak Mountains are a continuation of the non-volcanic mountains that border the Gulf of Alaska. The climate is generally maritime, but influenced by the mountainous terrain. Except in the high mountains, mean annual temperatures are above freezing. Summers are cool and winters relatively mild. Precipitation is heavy except in the northwest part of the area. The predominant soil types found in road construction are sand, silty sands, and muskeg.

- Copper River Plateau. This area is a broad basin of rolling to hilly moraines and glacial sediment interspersed with many lakes. It is surrounded by mountains: the Chugach to the south; the Alaska Range to the north; and the Talkeetna mountains to the west. The plateau is drained by three major rivers—the Copper, the Matanuska, and the Susitna. The climate is strongly continental—winters are long and cold. Summers are short and warm. Mean annual temperatures are below freezing and precipitation is low to moderate. The predominant soil types found in road construction are silt, sand, and gravels.
- Alaska Range. The long narrow mountain chain arcs around southcentral Alaska and separates it from the interior. It is very rugged and has many peaks above 10,000 feet. Many of the rivers in southcentral Alaska originate in the Alaska Range region. Mean annual temperatures are well below freeing even in the low passes. Precipitation is fairly heavy on the southern and southeastern slopes, but lighter on the north and western slopes. The predominant soil types are sands and gravels.
- Interior Alaska Lowlands. This area includes broad valleys and plains between the Alaska Range on the south and east, the Brooks Range on the north, and the Norton Sound Highlands on the west. The area is divided into parts—the Yukon Flats and the Kanuti Flats, which are large basins surrounded by hills; and the Koyukuk–Innoko and the Tanana–Kuskokwim Lowlands, which border major rivers in the region. Also included are natural levees, glacial outwash plains, piedmont slopes, and some rolling hills. The climate is continental, characterized by long cold winters and short warm summers. The predominant soil types for road construction include silt, sand, gravel, and permafrost.
- Kuskokwim Highlands. This area includes hills and low mountains between the central Yukon River and Bristol Bay. The northern part consists mostly of a series of rounded ridges 200 to 1,500 feet in elevation, separated by narrow valleys. The climate is strongly continental in most of the area but is modified by maritime influences near the Bering Sea. The mean annual temperature everywhere except the coast of Bristol Bay is below freezing. Precipitation is light in the north but increases southward toward the coast. The predominant soil types for road construction are coarse to fine sands.
- Interior Alaska Highlands. This area includes hills between the Tanana and Yukon Rivers and the Brooks Range and east of the Koyukuk and Selawik lowlands. The highlands consist mostly of rounded hills and ridges but include some mountains higher than 6,000 feet. Parts of the area adjacent to the major river valleys are as low as 300 feet. The higher parts have been affected by glaciers, but most of the area has never been ice covered. The climate is continental with long cold winters and short warm summers. The predominant soil types found in road construction are silt, sands, and gravels, and some low-PI clays.
- Norton Sound Highlands. This area consists of hills and low mountains on the Seward Peninsula and in the area east and south of Norton Sound. Elevations are generally less than 3,000 feet, though a few peaks are higher. Some of the mountainous areas were

glaciated, but most of the area has always been free of ice. There is a significant maritime influence on the climate in the area. Mean annual temperatures are below freezing, but winters are milder and summers cooler than in the inland areas. Precipitation is moderated in the regions bordering the sound, but low in the northern Seward Peninsula. The predominant soil types found in road construction are sand and silty gravels.

- Western Alaska Coastal Plains and Deltas. This area is made up of the Selawik–Kobuk Delta and the Yukon–Kuskokwim Delta and the Bristol Bay coastal plain. All are low and have very little relief. Permafrost underlies nearly all areas except the southern part of the Bristol Bay coastal plain. A cold maritime climate prevails. Mean air temperatures are generally below freezing except for Bethel, Kotzebue, and King Salmon. The predominant soil types found in road construction are silts, and sands.
- Bering Sea Islands. This area includes six islands of the Bering Sea—the Pribilofs, Nelson, St. Matthew, and St. Lawrence. The islands are volcanic rock and permafrost is virtually everywhere, except the Pribilof Islands. All of the islands have cool moist climates with mean annual temperatures increasing from the north to the south. The predominant soil types found in road construction are sands and gravels.
- Brooks Range. This area, the northern extension of the Rocky Mountains, extends across northern Alaska from Canada to about the Bering Strait and the Chukchi Sea. The southern slopes of the Brooks Range mark the northern limit of extensive forests in Alaska. In all but the southern slopes, the climate is arctic, with freezing temperatures occurring every month. Total precipitation is low. The predominant soil types found in road construction are sand and silty gravel.
- Arctic Foothills. This is an area of low ridges and intervening swales north and west of the Brooks Range. The elevation is generally less than 2,000 feet. The area has an arctic climate, modified slightly in the western part by a maritime influence. The predominant soil types found in road construction are silt, sands, and gravels.
- Arctic Coastal Plain. This is a gently rolling treeless area with many shallow elongated lakes and naturally drained lake basins. Rivers flowing from the mountains to the south meander across the plains to the Arctic Ocean. The climate is arctic with low mean annual temperatures and very low precipitation rates. The predominant soil types found in road construction are sand and silty sands.

C.3 Summary

This appendix briefly describes the different land resource areas in Alaska and the major soil types encountered in road construction. For more detail on soil types, the reader is referred to the USDA’s soil conservation publication titled Exploratory Soil Survey of Alaska, dated February 1979.