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16. ABSTRACT

This document provides guidelines on subgrade soil stabilization and covers mechanical, cementitious, and asphalt methods. The following aspects are discussed:

- An overview of subgrade stabilization methods
- Project investigation
- Mechanical stabilization
- Cementitious stabilization
- Asphalt stabilization
- Construction considerations

A list of further reading is provided.

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Guidelines for the Stabilization of Subgrade Soils In California

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Partnered Pavement Research Center (PPRC) Contract Strategic Plan Element 3.14:
Subgrade Soil Stabilization Guide

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California Department of Transportation
Division of Research and Innovation
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This guideline has been adopted by the California Department of Transportation (Caltrans) for determining and developing soil stabilization practices on Caltrans owned roadways. Use by other agencies is at their discretion and should only be done after reviewing the facts and issues of their roadways. This guideline constitutes a summary of best practices and guidance on soil stabilization and supplement to Caltrans Design and Construction Manuals. This guide does not supercede Caltrans or Federal standards, specifications, or regulations.

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The assistance and interest of Mr. Rich Haro and Mr. Donatas Greb of HSI Engineering with regard to strength testing of cement stabilized materials is acknowledged.

PROJECT OBJECTIVES

The objective of this project (PPRC Contract SPE 3.14) is to prepare a guideline document on pavement subgrade stabilization.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transport Officials
ASTM	ASTM International (American Society for Testing and Materials)
CBR	California Bearing Ratio
Caltrans	California Department of Transportation
DCP	Dynamic Cone Penetrometer
ESAL	Equivalent single axle load
FA	Fly ash
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
GGBS	Ground granulated blast furnace slag
HMA	Hot-mix asphalt
ICL	Initial Consumption of Lime
ICS	Initial Consumption of Stabilizer
ITS	Indirect tensile strength
LL	Liquid limit
MDD	Maximum dry density
MPD	Mean profile depth
OFC	Optimum fluid content
OMC	Optimum moisture content
PI	Plasticity index
PPM	Parts per million
PPRC	Partnered Pavement Research Center
PTR	Pneumatic-tired roller
SPE	Strategic Plan Element
TSR	Tensile strength retained
UCPRC	University of California Pavement Research Center
UCS	Unconfined compressive strength

LIST OF REFERENCED TEST METHODS AND SPECIFICATIONS

CT 201	Method of soil and aggregate sample preparation
CT 202	Sieve analysis of fine and coarse aggregates
CT 204	Method of testing plasticity index of soils
CT 216	Method of test for relative compaction of untreated and treated soils and aggregates
CT 221	Unconfined Compression of Soil
CT 226	Determination of moisture content by oven drying
CT 231	Relative compaction of untreated/treated soils and aggregates (area concept utilizing nuclear gauges)
CT 301	Method for determining the Resistance "R" Value of treated and untreated bases, subbases and basement soils by the Stabilometer
CT 312	Designing and testing Classes "A" and "B" cement treated bases
CT 371	Method of testing resistance of compacted bituminous mixture to moisture induced damage
CT 373	Unconfined compressive strength of lime treated soils and aggregates
CT 417	Method of testing soils and waters for sulfate content
CT 422	Method of testing soils and waters for chloride content
AASHTO M 145	Standard specification for classification of soils and soil-aggregate mixtures for highway construction purposes
AASHTO T 180D	Standard method of test for moisture-density relations of soils
AASHTO T 193	Standard method of test for the California Bearing Ratio
AASHTO T 290	Standard method of test for determining water-soluble sulfate ion content in soil
AASHTO T 291	Standard method of test for determining water-soluble chloride ion content in soil
ASTM D559	Standard test methods for wetting and drying compacted soil-cement mixtures
ASTM D560	Standard test methods for freezing and thawing compacted soil-cement mixtures
ASTM C593	Standard specification for fly ash and other pozzolans for use with lime for soil stabilization
ASTM D1557	Standard test methods for laboratory compaction characteristics of soil using modified effort (56,000 ft-lbf/ft^3[2,700 kN-m/m^3])
ASTM C1580	Standard test method for water-soluble sulfate in soil
ASTM D1632	Standard practice for making and curing soil-cement compression and flexure test specimens in the laboratory
ASTM D1633	Standard test methods for compressive strength of molded soil-cement cylinders
ASTM D1883	Standard test method for CBR (California Bearing Ratio) of laboratory-compacted soils
ASTM D2487	Standard practice for classification of soils for engineering purposes (Unified Soil Classification System)
ASTM D2974	Standard test methods for moisture, ash, and organic matter of peat and other organic soils
ASTM D4972	Standard test method for pH of soils
ASTM D5102	Standard test method for unconfined compressive strength of compacted soil-lime mixtures
ASTM D6276	Standard test method for using pH to estimate the soil-lime proportion requirement for soil stabilization
ASTM D6931	Standard test method for indirect tensile (IDT) strength of bituminous mixtures

CONVERSION FACTORS

SI* (MODERN METRIC) CONVERSION FACTORS				
Symbol	**Convert From**	**Convert To**	**Symbol**	**Conversion**
LENGTH				
mm	millimeters	inches	in	mm x 0.039
m	meters	feet	ft	m x 3.28
km	kilometers	mile	mile	km x 1.609
AREA				
mm^2	square millimeters	square inches	in^2	mm^2 x 0.0016
m^2	square meters	square feet	ft^2	m^2 x 10.764
VOLUME				
m^3	cubic meters	cubic feet	ft^3	m^3 x 35.314
kg/m^3	kilograms/cubic meter	pounds/cubic feet	lb/ft^3	kg/m^3 x 0.062
L	liters	gallons	gal	L x 0.264
L/m^2	liters/square meter	gallons/square yard	gal/yd^2	L/m^2 x 0.221
MASS				
kg	kilograms	pounds	lb	kg x 2.202
TEMPERATURE (exact degrees)				
C	Celsius	Fahrenheit	F	°C x 1.8 + 32
FORCE and PRESSURE or STRESS				
N	newtons	poundforce	lbf	N x 0.225
kPa	kilopascals	poundforce/square inch	lbf/in^2	kPa x 0.145

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

1. INTRODUCTION

1.1 Background

California is geologically active and a wide variety of soil types occur across the state. A thorough understanding of these subgrade soils in any pavement project area is essential to appropriately engineer the construction, rehabilitation, or widening of a highway facility. *Subgrade* is defined in the California Department of Transportation (Caltrans) *Standard Specifications* as "Roadbed portion on which pavement, surfacing, base, subbase, or a layer of any other material is placed." It is the soil or rock material underlying the pavement structure, and unlike base and wearing course (surfacing) materials whose characteristics are relatively uniform, there is often substantial variability of engineering properties of subgrade soils over the length of a project. Since pavements are engineered to distribute stresses imposed by traffic to the subgrade, the subgrade conditions have a significant influence on the choice and thickness of pavement structure and the way it is designed. Depending on the existing soils and project design, the properties of the subgrade may need to be improved, either mechanically, chemically, or both, to provide a platform for the construction of subsequent layers and to provide adequate support for the pavement over its design life. This will limit shear that may lead to permanent deformation; limit deflection that may lead to fatigue cracking; and limit expansion and contraction that leads to roughness. Stabilization of subgrade materials (defined in Section 1.4) has a number of benefits. Firstly, it eliminates the need to excavate substandard materials, transport them to a suitable site where they can be disposed of, and then excavate and import more suitable materials. Secondly, it improves the properties of existing materials, thereby providing a good platform for the overlying pavement layers.

1.2 Purpose of the Guideline

The concept of subgrade stabilization is discussed in the Caltrans *Highway Design Manual* (HDM, Topic 614: Soil Characteristics); however, no details are provided in the HDM or other Caltrans design documentation regarding selection of an appropriate stabilization method, design of stabilized subgrades, or construction considerations.

Subgrade stabilization is considered as an alternative to thicker pavement structures to address one or more of the following three situations:

- The existing roadway grade must be matched when lanes are added.
- The construction time required for excavating problem soils and/or hauling in additional materials must be reduced.
- It is the most economical pavement strategy.

This document provides guidance on these topics and should be used in conjunction with the Caltrans *Highway Design Manual* and *Standard Specifications* (stabilized soils are covered in Section 24 of the *Standard Specifications*).

1.3 Scope of the Guideline

Lime or cement stabilization are the primary methods of subgrade stabilization used by Caltrans, and are the only subgrade approaches covered in the *Standard Specifications.* However, this guideline also includes brief coverage of other cementitious additives (fly ash and kiln dust), as well as mechanical (compaction and material blending, but excluding geogrids and geofabrics) and asphalt stabilization methods. Alternative methods to lime and cement may be considered in specific situations, or may be proposed as an alternative in project documentation. The decision on which method of subgrade stabilization to use will depend on a range of factors including past experience and available information from similar projects and/or soil types in the district, the pavement design/project requirements, material type and properties, environmental factors, cost, and construction methods. The use of methods other than cement or lime are not covered in Section 24 of the *Standard Specifications* and will require preparation and approval of a nonstandard design pursuant to HDM Topic 606 and a nonstandard special provision.

A range of proprietary products with various chemical formulations are promoted as soil stabilizers, many of which were originally developed as dust palliatives or compaction aids. Few of these have been adequately researched, most do not have any formal specification, and insufficient information is currently available on their long-term stabilization performance to warrant inclusion in this guideline. They are, however, briefly discussed in Section 2.5.

1.4 Definitions

The terms "modification" and "stabilization" are used throughout this guideline. Definitions for each, as they appear in the *Standard Specifications*, are provided below:

- **Modification:** Soil improvement during or shortly after mixing to improve engineering properties including plasticity and moisture sensitivity to facilitate or expedite construction operations.
- **Stabilization:** Chemical or mechanical treatment of a mass of soil to improve its shear strength and durability for inclusion in a pavement structure.

1.5 Overview of California Geology and the Need for Subgrade Stabilization

California has diverse geology and consequently a wide variety of soil types throughout the state, ranging from coarse, stony materials in the mountain ranges; alluvial, silty materials in the river valleys; silts,

clays, and sometimes peat in the floodplains; and sandy materials in the basins and some of the coastal areas. Stabilization needs therefore also vary considerably throughout the state, and local knowledge of the soil types is important in selecting an appropriate subgrade stabilization approach.

1.6 Guideline Layout and Content

This guideline has been structured to lead the practitioner through the design and construction of stabilized subgrade layers, summarizing key issues through the following chapters:

- Chapter 2: Introduction to subgrade stabilization methods
- Chapter 3: Project investigation
- Chapter 4: Mechanical stabilization
- Chapter 5: Cementitious stabilization
- Chapter 6: Asphalt stabilization
- Chapter 7: Construction considerations
- Chapter 8: References and further reading

Figure 1.1 summarizes the subgrade stabilization process followed in this guideline.

1.7 Further Reading

The information provided in this guideline is based on best practice from across the United States and from other countries. No specific research studies on subgrade stabilization were undertaken to prepare these guidelines. Specific sources of information are not cited in the text. Instead, they are included by topic area in the list of further reading provided in Chapter 8.

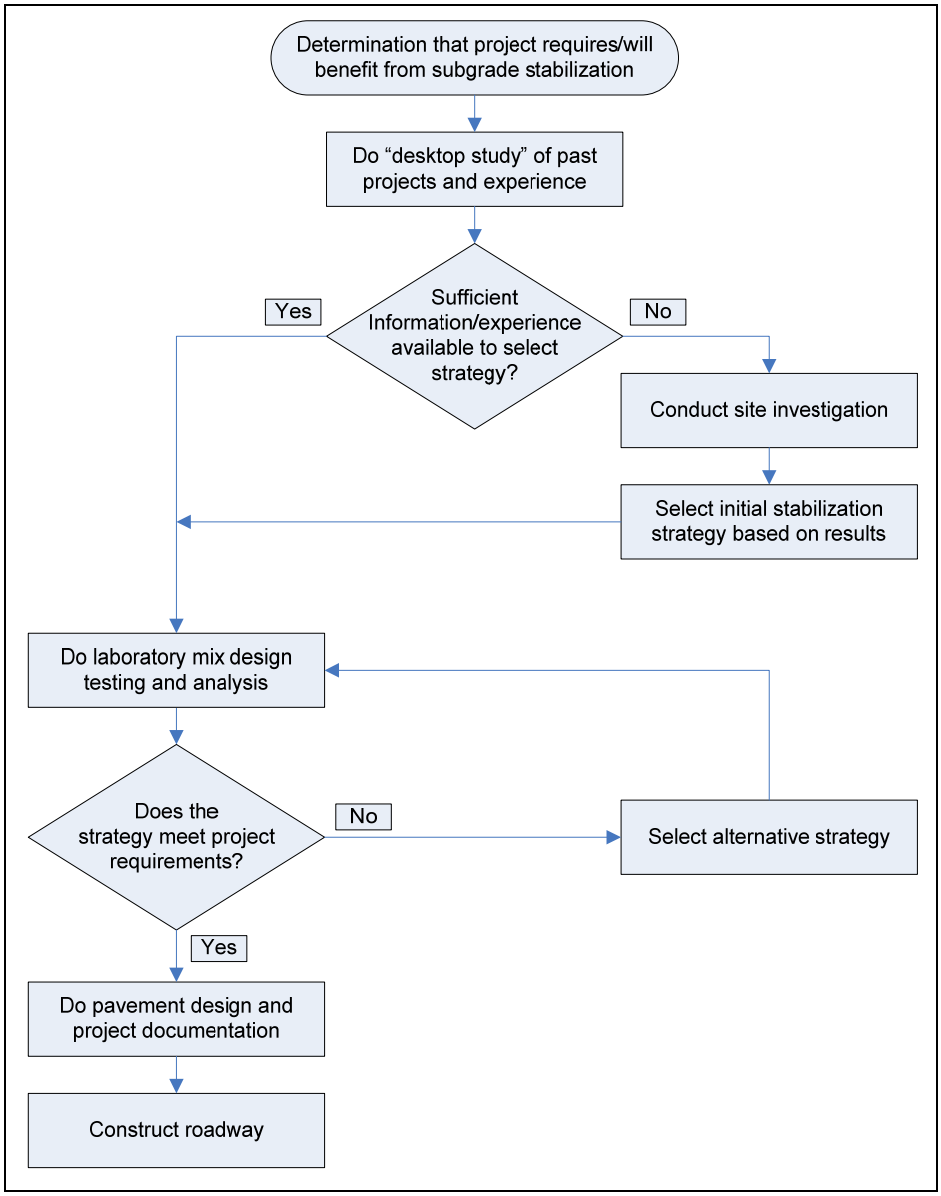

Figure 1.1: Summary of subgrade stabilization process.

2. INTRODUCTION TO SUBGRADE STABILIZATION METHODS

2.1 Introduction

The subgrade stabilization approaches discussed in this guide include mechanical, cementitious and asphalt methods. A brief introduction to each method is provided in this chapter with specific details on each provided in Chapters 4, 5, and 6, respectively. A brief introduction to nontraditional additives is also provided in this chapter.

2.2 Mechanical Stabilization

Mechanical stabilization is used to enhance soil–particle interlock through compaction, blending, and/or the use of geosynthetics (geogrids/geotextiles). Geosynthetics can also be used to manage drainage issues.

2.2.1 Compaction

Sufficient strength can often be achieved on certain subgrade materials that do not quite meet the design requirements by additional compaction, usually with a heavier or different type of roller than is normally used. Compaction improves aggregate interlock, and reduces air-void content, pore connectivity, and consequent susceptibility to moisture ingress.

2.2.2 Blending

Blending involves the mixing of materials that have different properties (typically particle size distribution and/or plasticity) to form a material with characteristics that improve upon the limitations of the source materials. Improving strength or plasticity is usually the primary reason for implementing mechanical stabilization. In most instances, blending will involve adding coarse aggregates to the finer in situ material. Less common in California is the addition of fine material to in situ sandy or coarse aggregates to fill voids and obtain a denser gradation.

2.2.3 Geosynthetics

Subgrade enhancement geosynthetics (SEG) are fabrics or grid interlayers placed between the pavement structure and the subgrade (the subgrade is usually untreated). They can be used for temporary improvement in terms of providing a platform for equipment during construction, and/or long-term improvement in terms of improving the structural integrity of the pavement structure. Mechanical stabilization with geosynthetics is covered in separate Caltrans guidelines (*Guide for Subgrade Enhancement Geotextiles*) and is therefore not covered in this document.

2.3 Cementitious Stabilization

2.3.1 Types of Cementitious Stabilizer Covered in Caltrans Standard Specifications

Cement

Portland cement is manufactured to meet various physical and chemical requirements for a range of applications. Eight types are generally available (I, IA, II, IIA, III, IIIA, IV, and V). Section 24 of the Caltrans *Standard Specifications* only allows the use of Type II and Type V portland cements for soil stabilization. These two cements have a moderate resistance to sulfate attack.

Lime

Both hydrated lime (or "slaked lime," $Ca(OH_2)$) and unhydrated lime (or "unslaked lime" or "quicklime," CaO) can be used for subgrade stabilization. Section 24 of the *Standard Specifications* only permits the use of high calcium quicklime (CaO >90) or dolomitic quicklime (CaO >55 and CaO + MgO >90) for subgrade stabilization. Unhydrated lime is considered more hazardous than hydrated lime and safety requirements for handling and working with this material should be strictly adhered to.

2.3.2 Other Cementitious Stabilizers

Fly ash, cement kiln dust, lime kiln dust, and ground-granulated blast furnace slag have been used as alternative cementitious stabilizers for subgrade soil modification or stabilization in various states. They are not commonly used in California and are not covered in Section 24 of the *Standard Specifications*, but they are included in this guideline for informational purposes in case they are considered for use in specific projects or proposed as an alternative stabilizer. In these instances, a nonstandard design pursuant to HDM Topic 606 will need to be prepared and a nonstandard special provision written and approved.

Fly Ash

Fly ash is available in a series of classes. Class-C (cementitious) has widest use in road stabilization, while Class-F fly ash (non-cementitious) can be used in combination with lime and cement as a stabilizer. Fly ash is not readily available in California and is usually imported from the midwest states where it is a by-product from coal burning electricity generation plants.

Cement Kiln Dust and Lime Kiln Dust

Cement kiln dust (CKD) and lime kiln dust (LKD) are by-products of portland cement and lime production, respectively. Their properties depend on the properties of the source materials and the production processes followed. Cement kiln dust and lime kiln dust are divided into three categories, namely precalciner, dry kiln, and wet kiln. Precalciner kiln dust is most suited to stabilization due to its

higher lime content, while dry- and wet-process kiln dust are more suited to drying of materials or for improving workability. Marginal improvements in plasticity reduction and strength may also be achieved.

Ground-Granulated Blast Furnace Slag

Ground-granulated blast furnace slag (GGBFS) is a by-product of iron and steel production and has limited use as a stabilizer. It is not produced in California and is not covered in this guideline.

2.3.3 Mechanism of Cementitious Stabilization and Effect on Soil Properties

The strength of cementitious stabilized soil and the rate of strength development depend on the following:

- Soil type and properties
- Quantity of stabilizer added (for permanent cementation to occur, the quantity of stabilizer must exceed the initial stabilizer demand of the material)
- Type and fineness of stabilizer
- Uniformity of mixing
- Density to which the stabilized soil or gravel is compacted, and time of compaction
- Temperature during compaction
- Curing period and conditions

Cement Stabilization

Cement stabilization primarily results in cementation, with a secondary reaction related to the calcium hydroxide generated during hydration. The end product is a cemented material consisting of the original soil, in which any clay minerals are partially or completely destroyed or altered, resulting in reduced plasticity. Various chemical reactions take place during cement stabilization, but in essence, crystals of hydrated calcium and alumina silicates generated during these reactions join together and bind the individual soil particles, usually providing significantly increased compressive and tensile strength.

Lime Stabilization

The development of strength in lime-treated soils is usually slower than that of cement treatment and is influenced by similar factors. Lime rapidly modifies the clay fraction of the material (involving ion exchange and flocculation of the clay mineral fraction of the material), and where sufficient stabilizer is available, continues with the development of hydrated calcium and alumina silicates and eventual cementation. Cementation usually takes longer than modification and will continue for as long as there is available clay, moisture, and a pH in excess of about 12.0 (i.e., the cementation reaction follows on from modification provided sufficient stabilizer remains). During this process, the clay mineral structure is broken down and forms colloidal gels of calcium aluminate and silicate hydrates, which have cementing properties similar to those of portland cement. High temperatures accelerate the process. Although lime is

mostly used to correct clay-related problems, the absence of clay does not necessarily mean that a material will not react with lime, and consequently comparative tests to determine the best stabilizer for a particular application is encouraged.

Fly Ash and Kiln Dusts

Fly ash and cement kiln dust and lime kiln dust contain varying but often substantial amounts of lime. In the presence of water, hydration reactions similar to portland cement take place leading to pozzolanic reactions in the soil. The level of strength development is, however, usually significantly less than that obtained from cement or lime. Pozzalanic properties of fly ash and kiln dust are assessed according to ASTM C593.

Reaction of Sulfates with Lime

Soluble sulfates present in the soil can react with hydrated lime and alumina to form the mineral ettringite. This is accompanied by severe expansion, causing a reduction in the strength of the stabilized material. Severe damage in the form of heave can result when clayey soils containing excess sulfates are stabilized with lime. Sulfate-rich soils occur in parts of California.

2.4 Asphalt Stabilization

Asphalt is not commonly used for subgrade stabilization in California and its use is not covered in Section 24 of the *Standard Specifications*. It is included in this guideline for informational purposes because of the growing interest in its use worldwide. Although asphalt (or bituminous) stabilization has been used on soils with plasticity indices of up to 15, it is more commonly used on sandy, well-graded soils with low fines contents and provides a relatively flexible platform for the pavement structure. It typically also provides some moisture resistance, but is not appropriate for use in poorly drained areas. If asphalt stabilization is considered, a nonstandard design pursuant to HDM Topic 606 will need to be prepared and a nonstandard special provision written and approved.

2.4.1 Types of Asphalt/Bituminous Stabilization

Asphalt Emulsion

Asphalt emulsion is a mix of asphalt binder, water, and emulsifying agent. It is liquid at ambient temperature to facilitate handling at lower application temperatures. Formulation of the emulsion (i.e., charge [anionic or cationic], type of emulsifier, and use of other additives) depends on the type of soil being treated and should be determined by the emulsion supplier. Anionic emulsions hold a negative charge and are compatible with positively charged aggregates (e.g., limestone), while cationic emulsions

hold a positive charge and are compatible with negatively charged aggregates (e.g., sandstone and quartz). Residual asphalt content in emulsions used for stabilization typically range between 50 and 60 percent. Higher residual asphalt contents usually result in poor aggregate coating and are not workable at ambient temperatures. Lower residual asphalt contents often lead to soil moisture contents significantly higher than the optimum compaction moisture content, resulting in construction delays while the soil dries back. Small amounts of active filler (typically cement at a concentration of between 1.0 and 2.0 percent by mass of dry aggregate) may be added to the soil during construction at the same time as the asphalt emulsion to accelerate breaking (separation of the asphalt from the water) of the emulsion and for additional early strength to accommodate traffic during curing of the layer.

Soil moisture plays an important role in dispersing the asphalt emulsion and preventing a premature break during mixing. Once mixed, the asphalt emulsion must break to allow the asphalt to adhere to the soil particles. Since the asphalt emulsion acts as a lubricant, the break should occur only after the material has been fully compacted. Strength is achieved once the excess water has evaporated from the material. The treated material will have a "speckled" appearance after dry-back.

Emulsions are also categorized based on the time that they take to break. Medium-setting (MS) emulsions and slow-setting (SS) emulsions are generally used for stabilization; however, due to environmental constraints, only slow-setting emulsions are mostly used in California. Emulsified asphalt with more than three percent petroleum solvent is prohibited in most Air Quality Control/Air Quality Management Districts in California.

Foamed Asphalt

Foamed asphalt is commonly used in partial- and full-depth reclamation of existing pavements, but is not normally used for subgrade stabilization since it is generally more expensive than other methods and is not effective on materials with high fines contents (typically more than 12 percent passing the #200 [0.075 mm] sieve). It requires specialized equipment that injects a mixture of air and water into hot asphalt binder, producing foam that adheres to fine particles, which in turn forms a mastic that binds larger particles together. Foamed asphalt is not covered in this guideline.

Cutback or Liquid Asphalt Stabilization

Asphalt binder with different types of solvent, referred to as *cutback* or *liquid asphalt,* and made liquid with petroleum-based solvents such as diesel, kerosene, or gasoline have been used for subgrade stabilization in the past, but have generally been replaced by emulsions in California because of air emission restrictions and worker safety concerns. In California, 21 out of 36 Air Quality Control/Air

Quality Management Districts have restrictions on the use of cutback asphalt emulsions (California Air Resources Board [*www.arb.ca.gov/ei/areasrc/ccosmeth/att_c_asphalt.doc*]). The list of districts is provided in Appendix A. The volatility of the solvent dictates how fast the mixture of aggregate and cutback asphalt will set. Three classes of cutback are available, namely rapid-curing (RC, 95 percent by weight of the diluent evaporates), medium-curing (MC, 70 percent by weight of the diluent evaporates), and slow-curing (SC, 25 percent by weight of the diluent evaporates) formulations. Only slow-curing formulations (typically < 0.5 percent petroleum solvent with boiling point < 500°F [260°C]) are permitted in California. Stabilization with cutback asphalt is not discussed in any detail in this guideline.

Tar

Tar is not recommended for subgrade stabilization in California due to health and environmental concerns, and is not covered in this guideline.

2.5 Nontraditional Additive Stabilization

Numerous so-called "nontraditional stabilizers" have been introduced over the last 30 years, with various claims made regarding performance. However, minimal specification of their properties or records of their performance have been made available and very few properly controlled independent comparative tests on the effectiveness of the products from different producers and suppliers have been carried out in full-scale field trials. There use is not covered in Section 24 of the *Standard Specifications*.

These stabilizers can be divided into seven main categories:

- Chlorides. Calcium chloride and magnesium chloride are commonly used for dust control in the United States. Marginal increases in strength are possible, mostly due to improved compaction. They are water soluble and do not provide sufficient strength improvement to warrant consideration as subgrade soil stabilizers.
- Organic Non-Petroleum/Natural Polymers. Organic non-petroleum or natural polymers are by-products from a sulfite process commonly used in the pulp and paper industries, from tannin extraction, sugar refining, and other plant-processing industries. Their composition is variable and depends on the vegetable matter and chemicals used during processing. They are effective dust palliatives but do not provide sufficient strength improvement for consideration as subgrade soil stabilizers.
- Petroleum Resins. Petroleum resins are usually a blend of natural polymers and petroleum-based additives. They have a similar binding action to natural polymers but are more resistant to leaching by water. They are effective dust palliatives but do not provide sufficient strength improvement for consideration as subgrade soil stabilizers.
- Synthetic Polymer Emulsions. Synthetic polymer emulsions, or polymer dispersions, are suspensions of synthetic polymers in which the monomers are polymerized in a dominantly aqueous

medium. Numerous formulations have been developed for various soil-conditioning applications, many of which are potentially suitable for improving the properties of subgrade materials.

- Sulfonated Oils. Sulfonated oils or sulfonated petroleum products (SPPs) rely on ionic exchange reactions to perform their expected functions satisfactorily. As they are all proprietary products, it is not possible to establish their exact compositions. It would appear, however, that their "active ingredients" are mostly mineral oils (hydrocarbon chains) modified with sulfuric acid to form a sulfonic acid. The mineral oil may be a natural petroleum derivative (oil or bitumen) or artificial oil. Sulfonated oils are all "surface active agents" (surfactants) and have the ability to fix, displace, or replace exchange cations in clays and to make the soil materials (particularly clay minerals but not necessarily only clays) hydrophobic by displacing adsorbed water and water of hydration, and preventing re-adsorption of this water. They are highly susceptible to ion exchange reactions in which appropriate inorganic ions present on mineral surfaces (particularly clays) and in clay interlayers are replaced by, or attached to, the organic molecules. This reduces the mobility of the ions and functionally reduces the plasticity of the material. Once an ion exchange reaction has occurred and the sulfonic acid is attached to a mineral particle, the so-called hydrophobic tails of the sulfonated oils are directed away from the particle and form an oily protective layer around it. In theory, this has the effect of reducing the thickness of the electrical double layer and of preventing water from gaining access to the clay mineral particle. With this reduced double-layer thickness, it now becomes theoretically possible to achieve a greater degree of compaction in the material and also to reduce the possible water absorption of the material in the long term.
- Synthetic Oils. Synthetic oils include base fluids, mineral oils, and unique formulations of synthetic isoalkanes. They are effective dust palliatives, but do not provide sufficient strength improvement for consideration as subgrade soil stabilizers.
- Enzymes. As with sulfonated oils, enzymes are proprietary products, the formulations of which are not made public. Very little useful documentation on the exact process of stabilization is available, although some form of microbial activity to neutralize the activity of the clay is a central theme of product brochures. However, material requirements (in terms of clay content and plasticity), application rates, and method and performance claims are similar to those advocated by the suppliers of sulfonated oils.

Limited research has shown that synthetic polymer emulsions can effectively bind low-plasticity sandy materials, while sulfonated oils and enzymes can influence the properties and moisture susceptibility of clayey materials. Given the limited research undertaken and absence of any formal specifications for these additives, no recommendations as to their use for subgrade stabilization in California are made in this guideline.

3. PROJECT INVESTIGATION

3.1 Introduction

The performance of a stabilized subgrade will be directly related to the materials in the layer, drainage, the choice of stabilizer, the quantity of stabilizer used, and the construction process followed. A four-phase project investigation is critical for understanding these issues and making informed decisions. The level of detail of the investigation will depend on the project scope (e.g., a new road or new alignment will require a more detailed investigation than a lane or shoulder addition on a widening project on an existing highway for which historical test data should already be available). Early pavement failures are often caused by insufficient data gathering during the project investigation phase. Subgrade stabilization project investigation includes these four phases:

1. Desktop study;
2. Site investigation;
3. Preliminary laboratory testing, analysis, and strategy selection; and
4. Mix design testing and analysis.

The first three phases are discussed in this chapter. Output from these three phases will be a proposed subgrade stabilization strategy (i.e., using mechanical techniques, or cementitious or asphalt additives). The fourth phase is discussed in Chapter 4 through Chapter 6 depending on the strategy selected.

3.2 Desktop Study

The desktop study is the first stage in the project investigation and involves collecting all relevant information pertaining to the road and the project including, but not limited to the following:

- **Project information,** usually obtained from the District Materials Engineer (DME) is used to understand the project scope and size, and to determine any specific required subgrade conditions for the proposed pavement. Provisional test results, including Dynamic Cone Penetrometer (DCP) survey, Atterberg Limits, R-value, and presence of deleterious materials (e.g., sulfates, chlorides, and organic matter) may be available from earlier investigations or from investigations for similar projects on similar materials in the district.
- **Geotechnical report,** if available, is used to understand site conditions on the proposed road alignment, including soil type, soil depth, groundwater conditions, water table depth range, and variability of all of these parameters along the proposed alignment. Geotechnical investigations are typically only undertaken on major projects or projects in areas with known problems.
- **Climate data** relevant to pavement design in California can be obtained from the Caltrans Office of Pavement Management or *dot.ca.gov/hq/esc/Translab/ope/Climate.html*, and are used to identify precipitation type and intensity, typical minimum and maximum temperatures, and issues such as

freeze-thaw. All of these need to be understood in order to ensure that the subgrade preparation strategy and drainage system are designed and constructed appropriately.

- **General maps** are available to Caltrans staff on the Caltrans Intranet.
- **Detailed geology and soil maps** can be obtained from the California Geological Survey and/or U.S. Geological Survey websites, (*conservation.ca.gov/cgs/maps/ and usgs.gov/pubprod/*) and the U.S. Department of Agriculture website (*soils.usda.gov/survey/online_surveys/california/*), respectively, and are used to identify geology, soil type, and associated potential problem areas such as expansive clay and sulfate-bearing soils.

3.3 Site Investigation

A site investigation should be carried out to supplement information that was not obtained during the desktop study. The site investigation, if required, has two primary purposes:

- To determine the in-situ strength of the subgrade and its variability along the proposed road alignment using a Dynamic Cone Penetrometer (DCP) survey, and
- To collect soil samples for tests to determine whether stabilization is required and, if it is, which stabilization method is most appropriate. Mix designs will also be determined using these samples.

3.3.1 Dynamic Cone Penetrometer (DCP) Testing

Method

A standard DCP with 60° cone is used for this assessment. The test interval for DCP measurements will depend on variability of the subgrade conditions, with measurement frequency increasing with increasing variability:

DCP Test

- Take one measurement at least every 1,500 ft (500 m) in flat, featureless terrain.
- Increase the frequency to every 300 ft (100 m) in areas with variability.
- Take additional measurements in potential problem areas (e.g., failed areas on existing pavements, changes in moisture condition, change between cut and fill, soil type, vegetation type, etc.).

Measure the penetration after every five blows up to a depth of 800 mm (31.5 in.). An example DCP data collection form is provided in Appendix B (Form 1). On uncompacted soils (e.g., investigations for new alignments, lane additions, or lane widening), do not do DCP measurements immediately after rainfall or if the ground surface appears wet as this will influence the rate of penetration and may result in weaker than actual soil strengths.

Analysis of DCP Results

DCP results for subgrade soil investigations are typically analyzed in terms of the DCP Number (DN). The DCP Layer Structure Number (DSN) and the DCP Pavement Structure Number (DSN_{800}) are used for assessing pavement structures and are not discussed in this guideline.

- The DCP Number (DN) is the DCP rate of penetration in millimeters (mm) per hammer blow (mm/blow). This provides an indication of the relative shear strength of the material at the depth where it was calculated. On most soils this shear strength will typically reduce with increasing depth (unless bedrock is present or surface soil is wet), and if the DN is plotted against depth distinct jumps are often apparent. The points of each jump can be used to identify changes in material type, material properties, or moisture conditions. Empirical relationships have been developed in a number of countries to relate the penetration rate to the effective layer stiffness and to the California Bearing Ratio (CBR). No documented comprehensive studies have been undertaken to relate DN to R-value. Although these relationships provide useful indications to identify and evaluate potential problem areas, the stiffness and CBR values obtained should be regarded as **approximations** only. An example of a relationship between stiffness and penetration rate, developed in South Africa, is defined below (Equation 3.1). A summary of DN ranges and corresponding calculated CBR values and stiffnesses is provided in Table 3.1, along with an elementary relationship to R-value based on laboratory work published by Huang (1993) (note that R-values below 50 appear to be too high for rate of DCP penetration).

$$E_{eff} = 10^{3.05-1.066(Log(DN))} \quad \textbf{(3.1)}$$

where: E_{eff} is the effective elastic modulus

Table 3.1: Example Relationship between DN, CBR and Stiffness

DN Range (mm/blow)	Approx CBR Range[1] (%)	Stiffness[1] (MPa)*	R-value[1,2]
< 4	>70	> 258	>80
4 – 5	50 – 70	204 – 258	75 – 80
5 – 8	30 – 50	124 – 204	65 – 75
8 – 14	15 – 30	68 – 124	50 – 65
14 – 19	10 – 15	49 – 68	42 – 50
19 – 25	7 – 10	37 – 49	35 – 42
25 – 30	3 – 7	30 – 37	18 – 35
30 – 35	1 – 3	26 – 30	1 – 18
> 35	< 1	< 26	< 1

[1] Values are approximate only and should be used with caution and only as a guide. There is no published correlation between DN and R-value.

[2] From Huang (1993) based on CBR and R-value laboratory testing comparison. Not developed from DCP and laboratory R-value test result analysis and not verified for use in California. R-values below 50 appear to be too high for rate of DCP penetration

Enter the DCP Numbers into a spreadsheet and calculate an average and standard deviation. Plot the results to delineate uniform sections and identify potential problem areas (example in Figure 3.1). In the example, eight uniform sections can be identified, which can be further divided into three different zones. Zone A has DNs between 15 and 25 and can be considered reasonably strong for subgrade materials. Zone B has DNs between 30 and 40 and can be considered to have marginal strength, while Zone C has

DNs between 45 and 55 and is clearly very weak indicating potentially wet, clay soils. Zone C may require additional attention (e.g., excavation and replacement, or additional drainage prior to stabilization).

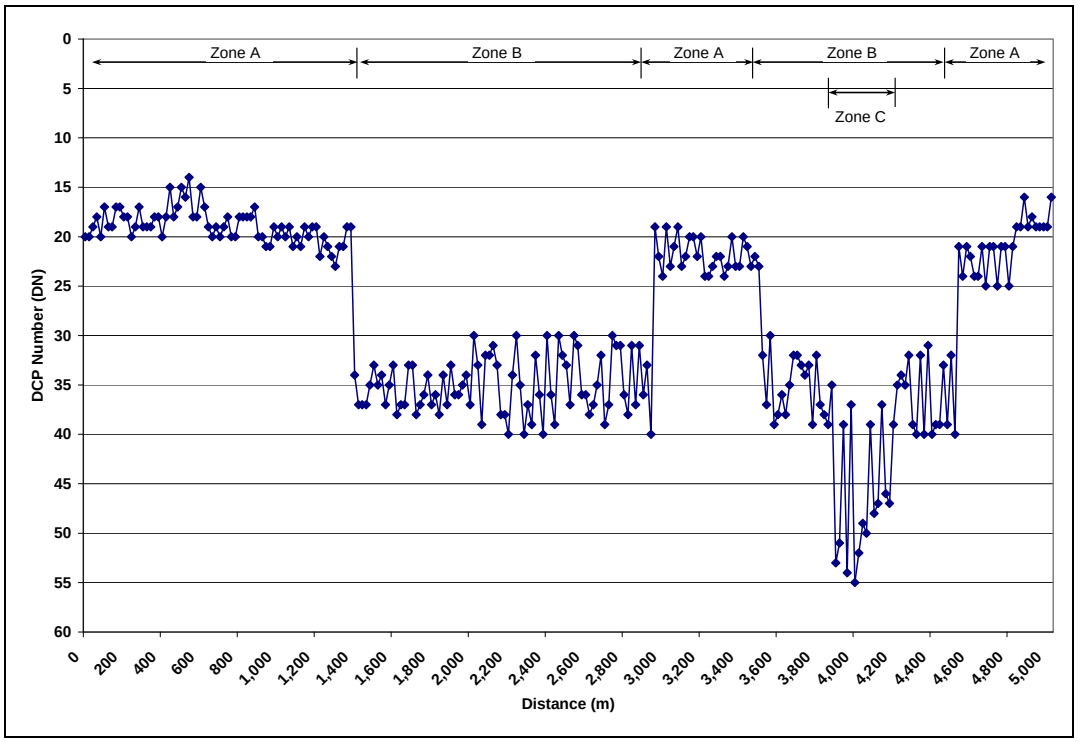

Figure 3.1: Example DCP Number (DN) analysis.

3.3.2 Material Sampling

Material sampling intervals will also depend on the type of project and problem being addressed (i.e., more intensive sampling will be undertaken for a new alignment than for drying out wet soils or improving the workability on a short problem section on a specific project), testing already undertaken, existing knowledge of the soils in the vicinity of the project, experience from previous projects in the area, as well as variability along the project. If there are no results available, or if the available results are too limited to make an informed decision, take at least one sample from each uniform section identified during the DCP investigation. Increase the sampling frequency if the soils or soil properties appear to be variable. If the presence of sulfates is suspected, increase the sampling frequency to about every 500 ft (150 m). Samples should be representative of all the material that is likely to be stabilized (i.e., from surface to below anticipated stabilization depth) in that uniform section. Record the location, size, and purpose of the sample on an appropriate form (example Form 2 in Appendix B). Sample quantities are summarized in Table 3.2 and distinguish between preliminary testing to classify the soil and identify an appropriate stabilization strategy (discussed in this chapter), and mix design testing that may be required after strategy

selection to determine stabilizer application rates and the properties that will be used for designing the pavement structure (discussed in Chapter 4 through Chapter 6).

Table 3.2: Sample Quantities and Test Methods for Subgrade Soil Assessment

Testing Phase	Test	Sample Size		Test Method
		(lbs)	(kg)	
Preliminary Testing	Grading Atterberg Limits Moisture content Optimum moisture content R-value (or California Bearing Ratio) Sulfate content Organic matter content Chloride content pH	200	90	CT 202 CT 204 CT 226 CT 216 CT 301 CT 417[1] ASTM D2974 CT 422[2] ASTM D4972
Design Testing	Grading Density and optimum moisture content Initial consumption of stabilizer[1] Unconfined compressive strength Indirect tensile strength	500	250	Refer to Chapters 4, 5, and 6

[1] Only for cementitious stabilization
[2] Alternatively, chlorides can be easily detected using an electrical conductivity test (AASHTO T 291).

3.4 Preliminary Laboratory Investigation

The test methods discussed in this section are used to characterize the soil, and the results are used to select an appropriate stabilization strategy if the soil does not meet the *Highway Design Manual* and/or project requirements. Additional testing may be required to refine the strategy (e.g., determine stabilizer application rates, compare different stabilizers, determine the level of improvement obtained, etc.).

3.4.1 Test Methods

Grading Analysis, (CT 202) Atterberg Limits (CT 204), moisture content (CT 226), optimum moisture content (CT 216), and R-value (CT 301) tests should be carried out (or data used from previous/similar investigations on or near the project) to characterize the sampled subgrade materials and to determine whether the material meets basic design requirements. Sulfate content (CT 417, AASHTO T 290, or ASTM C1580), organic matter content (ASTM D2974), chloride content (CT 422), and/or soil pH (ASTM D4972) tests should also be carried out if problems associated with these parameters are expected or have occurred in the project vicinity in the past. If the materials do not meet project design requirements the test results are used to select the most appropriate method of stabilization to achieve the required design objectives. Design test methods are discussed in Chapters 4 through 6.

3.4.2 Analysis

Grading (CT 202)

The two key sieve sizes used in the analysis are the #4 (4.75 mm) and #200 (0.075 mm). The percentage material passing and retained on these two sieves is used together with the Atterberg Limits to classify the soils according to the Unified Soil Classification System (USCS), discussed below.

Atterberg Limits (CT 204)

The liquid limit and plasticity index of the soil are both used in determining the need for and type of subgrade stabilization. The liquid limit is used to classify the soil and the plasticity index is used as an indicator for the degree of stabilization that will be required and the most likely stabilization method that will be used. **Soils with a plasticity index higher than 12 will typically require some form of modification or stabilization.**

Soil Classification (HDM Index 614.2)

The Unified Soil Classification System (USCS) (ASTM D2487, HDM Topic 6.14) (Table 3.3) is used to classify soils according to grain size distribution and plasticity. The AASHTO M 145 Soil Classification System is also commonly used to describe soils in pavement engineering publications and is provided in Appendix C for informational purposes.

In the USCS, soils are classified as coarse-grained or fine-grained as follows:

- **Coarse-grained** if more than 50 percent of the soil sample is **retained** on the #200 (0.075 mm) sieve. Coarse-grained soils are further classified as:
  - **Gravels** if 50 percent or more of the coarse fraction is **retained** on the #4 (4.75mm) sieve, or
  - **Sands** if 50 percent or more of the coarse fraction **passes** the #4 (4.75 mm) sieve.
- **Fine-grained** if 50 percent or more of the sample **passes** the #200 (0.075 mm) sieve. Fine-grained soils are further classified according to whether their liquid limit is less than or greater than 50 percent.

The USCS also includes peat and other highly organic soils, which are compressible and not recommended for roadway construction. Peat and other highly organic soils should be removed wherever possible prior to placing the pavement structure.

Table 3.3: Unified Soil Classification System (ASTM D2487)

Major Classification Group	Sub-Groups		Classification Symbol[1]	Description
Coarse-Grained Soils (>50% retained on the #200 sieve)	**Gravels** (>50% of the coarse fraction is retained on the #4 sieve)	Clean gravels	GW	Well-graded gravels and gravel-sand mixtures (little or no fines)
			GP	Poorly graded gravels and gravel-sand mixtures (little or no fines)
		Gravels with fines	GM	Silty gravels (gravel-sand-silt mixtures)
			GC	Clayey gravels (gravel-sand-clay mixtures)
	Sands (≥50% of the coarse fraction passes the #4 sieve)	Clean sands	SW	Well-graded sands and gravelly sands (little or no fines)
			SP	Poorly graded sands and gravelly sands (little or no fines)
		Sands with fines	SM	Silty sands (sand-silt mixtures)
			SC	Clayey sands (sand-clay mixtures)
Fine-Grained Soils (≥50% passes the #200 sieve)	**Silts and clays with liquid limit ≤50%**		ML	Inorganic silts (very fine sands, rock flour, silty or clayey fine sands)
			CL	Inorganic clays of low-to-medium plasticity (gravelly/sandy/silty/lean clays)
			OL	Organic silts and organic silty clays of low plasticity
	Silts and clays with liquid limit >50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity (fat clays)
			OH	Organic clays of medium-to-high plasticity
			PT	Peat muck and other highly organic soils

[1] Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay with LL ≤50%, H = Clay with LL >50%

Moisture Content and Optimum Moisture Content (CT 226 and CT 216)

The actual and optimum compaction moisture contents are used to determine whether some form of chemical modification is required to dry the material back to a suitable level for compaction as an alternative to ripping and allowing to dry back through evaporation. The choice of modification or allowing the soil to dry back naturally will depend on the project requirements and climatic conditions. **Non-plastic soils with moisture contents higher than the optimum compaction moisture content typically only require ripping, blading, and recompaction after excess water has evaporated, without the need for additional modification. Clay soils can also be dried back in this way if time and weather conditions permit, but will typically require some form of modification to dry them back to an appropriate level within a reasonable time period.**

R-value (CT 301)

The R-value is used to determine whether the soil meets the minimum strength requirements required for the project. **Soils with an R-value lower than 20 are considered to be weak subgrades in the HDM (Index 6.14.3) and will typically require some form of stabilization for conventional pavement designs.**

Sulfate Content (CT 417, AASHTO T 290, ASTM C1580)

The sulfate content test is used to determine the presence of sulfates. High soil sulfate contents can result in swell and heave problems and can have a deleterious influence on cementitious and asphalt stabilization mechanisms. **Soils with a sulfate content higher than 3,000 parts per million (ppm) will require specific stabilization procedures.** If sulfate contents exceed this value, a more detailed sampling and testing program should be carried out to accurately determine sulfate contents along the project.

Organic Matter Content (ASTM D2974)

The organic matter content is used to determine whether the soil can be stabilized, or whether it should be excavated and replaced with other material. High organic matter contents also retard hydration processes in cementitious stabilization projects and consequently lower than required strengths may result. **Soils with organic matter contents higher than 1.0 percent of the mass of the dry soil will typically require higher-than-economical quantities of stabilizer.** The costs of using higher quantities of stabilizer will need to be compared with the costs of other strategies (e.g., removing the organic soil and replacing it with different material, thicker pavement designs, etc.).

Chloride Content (CT 422)

Soils with high potassium and sodium contents, often in the form of potassium chloride or sodium chloride, can negatively impact stabilization by competing with calcium cations in calcium-based (i.e., cementitious) stabilizers. **No limits for chloride content are proposed in this guide; however, international literature recommends maximum electrical conductivities of 0.4 S/m (at 77°F [25°C]) for subgrade soils and 0.15 S/m (at 77°F [25°C]) for subbase and base soils. Engineers should be aware of the potential problem and that higher stabilization rates are usually required to achieve the required results.** The costs of using higher quantities of stabilizer will need to be compared with the costs of other strategies (e.g., removing the chloride soil and replacing it with different material, thicker pavement designs, etc.).

Soil pH (ASTM D4972)

Soil pH is used to distinguish between acidic and alkaline soils. **Soils with a pH less than 5.3 will typically require higher-than-economical quantities of cement if cement stabilization is being considered.** The costs of using higher quantities for stabilizer will need to be compared with the costs of other strategies. Pretreatment with lime may be required.

3.4.3 Reporting

Summarize the laboratory test results on an appropriate form (example Form 3 in Appendix B).

3.5 Selecting a Project Stabilization Strategy

3.5.1 Decision Guide

A basic decision flowchart for selecting a first-level project stabilization strategy, using past district experience and the information collected in the desktop study, preliminary site investigation, and preliminary laboratory testing is shown in Figure 3.2. The process is summarized as follows:

1. If the plasticity index of the sampled materials is less than 12, the R-value is greater than 20, and DCP number is less than 20 (typically GW, GP, and SW soils, and potentially GM, GC, and SP soils in Table 3.3), then the subgrade in these zones will most likely not need any additional stabilization beyond standard subgrade preparation requirements (i.e., rip and recompact). If the materials do not meet these requirements, or if they are to be stabilized for use as a subbase, then some form of improvement will be required.
2. If the problem is related only to high soil moisture content, check that the drainage design is satisfactory and consider mechanical (ripping and mixing) or cementitious (typically lime, cement, or kiln dust) modification to dry out the soil.
3. If the laboratory test results are just below the design plasticity or R-value requirements for the project (e.g., within 10 percent of the plasticity index or R-value and typically GW, GM, SW, SM

soils), consider mechanical stabilization (e.g., additional compaction or blending [Chapter 4]) or cementitious modification (Chapter 5) depending on project requirements, soil type, site conditions, and past district experience. Asphalt stabilization (Chapter 6) can also be considered if satisfactory results are not obtained using mechanical or cementitious stabilization, or it is shown to be more cost-effective.

4. If the problem is primarily related to soil gradation and not plasticity (typically GP, GC, and SP soils), consider mechanical stabilization (blending) (Chapter 4).
5. If the problem is primarily related to low-strength coarse-grained soils (typically G- and S- soils), consider cement (Chapter 5) stabilization. Asphalt stabilization (Chapter 6) can also be considered if satisfactory results are not obtained using mechanical or cementitious stabilization, or it is shown to be more cost-effective.
6. If the problem is primarily related to high plasticity soils and reducing the plasticity alone will achieve the desired results (typically CL, MH, CH, and OH soils), consider cementitious modification or stabilization (typically lime or cement) (Chapter 5).
7. If the problem is primarily related to low-strength fine-grained soils (typically M-, C-, and O-soils), consider cement stabilization (Chapter 5).
8. If the problem is primarily related to high organic content in the soil (PT soils), consider cementitious stabilization, but compare costs of higher stabilizer application rates with an alternative strategy (e.g., removing and replacing the soil).

3.5.2 Reporting

Summarize the stabilization strategy selection on an appropriate form (example Form 4 in Appendix B). Highlight the strategy selected with a justification for that choice.

3.6 Investigation Report

Prepare the first part of an investigation report by attaching a cover sheet with project details to the forms completed in this part of the study. If a stabilization strategy has been selected, results and analysis from additional strategy-specific testing discussed in Chapters 4 though 6 will be used to refine the stabilization strategy, determine stabilizer application rates, prepare design information for the pavement design, and complete the investigation report.

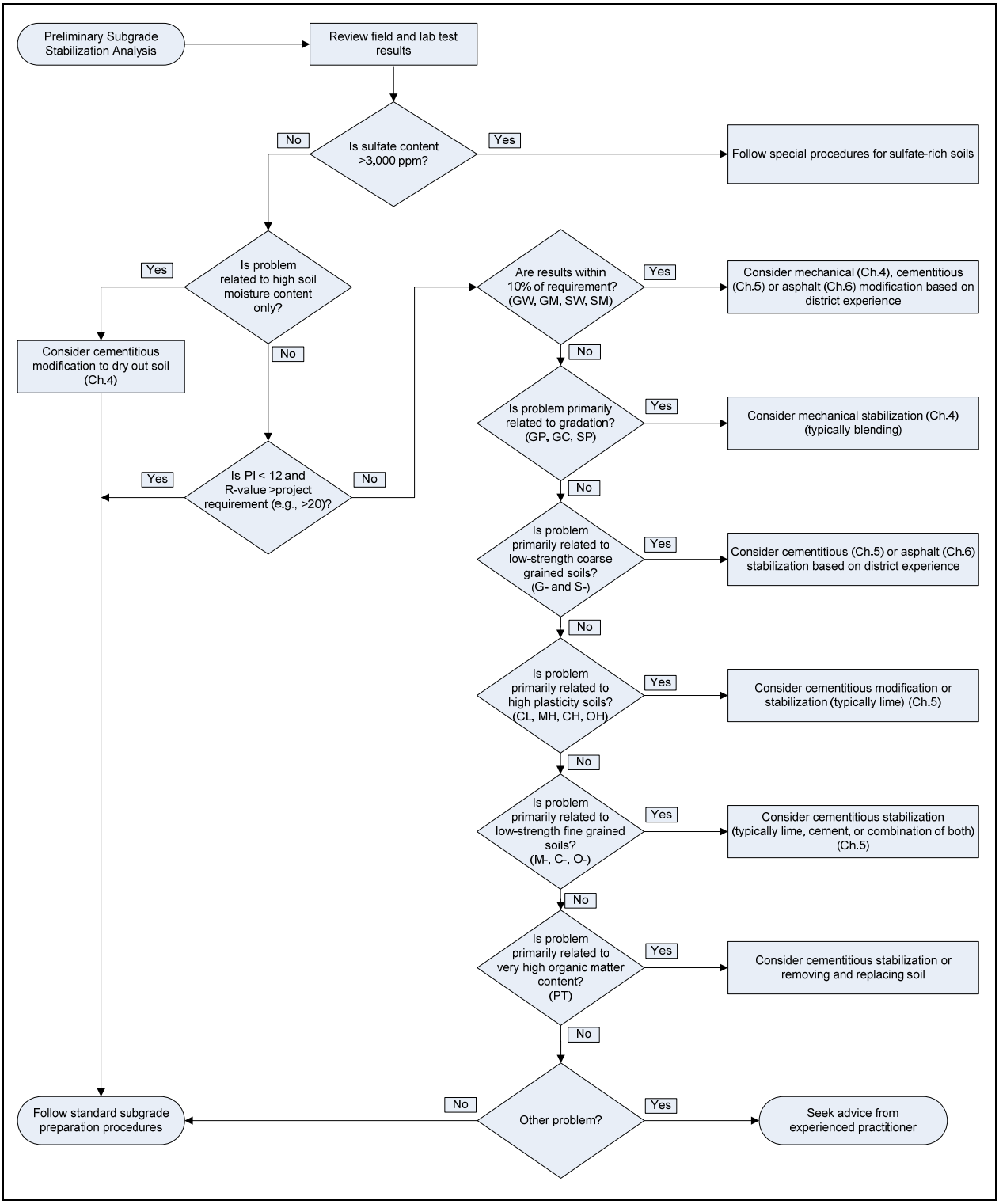

Figure 3.2: Selecting a first-level project stabilization strategy.

(Note: Geosynthetics are covered in a separate Caltrans guideline and are not included in this figure.)

4. MECHANICAL STABILIZATION

4.1 Introduction

This chapter covers subgrade stabilization using the mechanical methods of compaction and/or blending. Geosynthetics (i.e., geogrids and geofabrics) are covered in a separate guideline (*Guide for Subgrade Enhancement Geotextiles*) and are not discussed in this guide.

4.2 Compaction

4.2.1 Introduction

Caltrans earthwork specifications require that subgrade soils be compacted to at least 95 percent of the laboratory-determined density to a minimum depth of 2.5 ft (0.75 m) below finished grade over the full width of the pavement (traveled way and shoulders) plus 3.0 ft (0.9 m) beyond the edge of pavement. However, small increases in bearing capacity can often be achieved with additional compaction (e.g., 98 to 100 percent of the laboratory-determined density instead of 95 percent), which can usually be accomplished with either a heavier roller and/or additional roller passes. On projects where the strength/bearing capacity is marginally below the design requirements (i.e., within 10 percent of the required R-value or California Bearing Ratio [CBR]), additional compaction can be considered to increase it to the required levels as an alternative to more costly and time-intensive chemical stabilization, which may not be cost-effective for the small increase in strength that is required. The use of additional compaction to meet strength requirements should not be considered on expansive soils (i.e., plasticity index greater than >12 [MH, CH and OH soils]). Care must also be taken not to overcompact the soil, which can lead to crushing of aggregate particles.

4.2.2 Site Investigation

No additional site investigation information to that discussed in Chapter 3 is required.

4.2.3 Testing, Analysis, and Reporting

Laboratory testing to assess improvements achieved from additional compaction is limited to density tests (Table 4.1). The Unconfined Compressive Strength test (UCS) or California Bearing Ratio (CBR, if equipment is available) test can be considered as quick and simple tests for comparing shear strengths at different densities/levels of compaction. The UCS or CBR methods are suggested because of the simplicity of both tests, especially with regard to compacting specimens to different densities. Note that the CT 221 UCS test is primarily designed for testing undisturbed samples collected in the field, but is suitable as an indicator test to compare strengths of specimens compacted to different densities.

Table 4.1: Tests for Assessing Compaction as a Potential Stabilization Method

Parameter	Test Method	Reason
Relative Compaction	CT 216 or AASHTO T 180D	Determine densities for different compaction efforts.
UCS	CT 221	Determine shear strength at different compaction efforts
CBR	ASTM D1883 or AASHTO T 193	Alternative strength test to assess improvement with additional compaction.

Use the following procedure to determine whether sufficient strength/bearing capacity improvement can be achieved from additional compaction using the UCS or CBR test:

1. Determine the maximum dry density and optimum moisture content of the material using CT 216 or AASHTO T 180D. Note that AASHTO T 180 is preferred over Caltrans CT 216, which is a relative compaction test that primarily assesses wet density and is designed for quality control during construction. It may not be sufficiently accurate for assessing the effects of different compaction levels on shear strength or for determining maximum dry density and optimum moisture content for mix design laboratory tests.
2. Prepare three sets of specimens at 95 percent, 98 percent, and 100 percent of the maximum dry density (i.e., reduce the number of blows during compaction to achieve lower densities). Replicates in each set are recommended to assess variability.
3. Determine the average UCS or CBR for each set of specimens. Test the specimens within 60 minutes of compaction, or take appropriate steps to ensure that the material does not dry back.
4. Determine whether the UCS or CBR is sufficiently increased (e.g., depending on design requirements, UCS [unsoaked] above 115 kPa or CBR above 10 are usually satisfactory minimums for subgrade soil; increasing the UCS or CBR will provide increased bearing capacity [i.e., increased shear strength]).
5. If satisfactory results cannot be achieved, an alternative stabilization strategy will need to be considered.

Reporting

If satisfactory results are achieved, a recommendation on the use of this additional compaction strategy can be made and the project specifications will need to be revised accordingly. If not, then an alternative stabilization strategy will need to be considered (see Section 3.5).

Summarize the recommended compaction requirements on the stabilization strategy form (example Form 5 in Appendix B).

4.2.4 Construction Considerations when Using Compaction for Strength Improvement

The contractor will need to use a suitable roller to ensure that the specified additional compaction can be achieved. Material type and required compaction depth will dictate what type of roller is used, with increasing static weight required for increasing compacted lift thickness (recommendations are provided in Table 4.2). Padfoot rollers are typically used on silt and clay subgrade materials, while smooth-drum

and/or rubber-tired rollers are used on sandy subgrade materials. Moisture contents need to be carefully controlled, ensuring that in situ moisture is factored into the calculation of optimum moisture content. Silt and clay soils are usually compacted at or slightly above optimum moisture content (i.e., typically one or at most two percent above optimum to reduce permeability). Silt and clay soils are, however, unworkable at higher moisture contents. Cohesionless soils are typically compacted at moisture contents close to or slightly above optimum moisture contents.

Table 4.2: Recommended Primary Roller Weights for Different Layer Thicknesses

Compacted Layer Thickness		Static Weight
(in.)	(mm)	(tons)
< 6	< 150	12
6 – 8	150 – 200	15
8 – 10	200 – 250	18
10 – 12	250 – 300	20
> 12	> 300	25

Achieving optimal density during the compaction of subgrade layers is critical to ensure that the required structural capacity of the pavement structure is obtained, thereby limiting the likelihood of permanent deformation and/or fatigue of the asphalt surfacing under traffic. Density requirements are usually specified in terms of a percentage of a reference density, typically the maximum wet or dry density determined in the laboratory during the project investigation. Caltrans specifications require a density of at least 95 percent (determined according to CT 231) of the laboratory-determined density (CT 216) for subgrade compaction. However, the material properties on most projects vary along the length of the project. Consequently, the density of the material will also change and although the CT 231 test accounts for this to some extent, differences of a few percent can be the difference between accepting and rejecting a section of the road, and/or between over- and undercompaction. Representative reference densities will therefore need to be determined by carrying out a moisture-density relationship test on a sample collected from most locations where the field density is measured to ensure that correct values are being used. This can increase the number of laboratory tests required on projects with considerable variability and could result in reworking completed layers if there is any delay in obtaining test results.

A *refusal density* can be considered as a supplement to using a reference density. Refusal density can be defined as the maximum density that can be achieved on a material under the prevailing field conditions (i.e., material properties, moisture content, and support conditions). This can be assessed using two methods, or a combination of the two methods. The quality control procedures prescribed in the *Standard Specifications* must still be followed for both methods. The two methods are:

1. Complete a short test strip at regular intervals along the project, during which a nuclear density gauge reading is taken after each roller pass. Stop rolling if the same density is recorded for two

consecutive passes. The number of passes required to achieve this density is then used for that particular production run. The minimum required density must still be achieved.

2. Use an "intelligent compaction" system fitted to the roller. These systems incorporate a "compactometer" on the vibrating drum to measure "rebound acceleration," which is a measure of the response of the material to an impulse (vibration). This provides an indication of the density of the material. These measurements in combination with a GPS locator allow multiple measurements taken at the same location to be compared. Results are displayed in the cab, allowing the roller operator to continuously monitor density development at every location. When successive passes of the roller show no further increase in density, the refusal density has been achieved and the roller can move forward to compact the next section of work.

Satisfactory results can be achieved using a refusal density compaction specification provided that the correct equipment is used, appropriate procedures are followed, changes in material properties are monitored, and the moisture content of the material being compacted is in the required range relative to the optimum moisture content of the material.

4.3 Blending

4.3.1 Introduction

Caltrans specifications do not specify a grading for subgrade soils. On projects where the compacted in-situ soil does not quite meet the required strength/bearing ratio and appears to be poorly graded (i.e., too coarse, too fine, or gap-graded), mixing with a material with a different gradation may increase the bearing capacity sufficiently. The process is ideal if suitable materials (e.g., cut materials) are available on the project. Care needs to be taken to ensure that there are no new performance consequences after blending (e.g., different freeze-thaw behavior after adding fine material). On projects where the blend materials need to be imported, costs will need to be compared with other methods of stabilization. In these instances, chemical stabilization of fine materials (i.e., silts and clays) will usually be cheaper than importing coarse materials, while sourcing and transporting nearby fine material ("fill dirt") to improve the properties of coarse- or gap-graded materials (e.g., single-sized sands) may be less expensive than chemical stabilization.

4.3.2 Site Investigation

In addition to the site investigation requirements discussed in Chapter 3, the following actions will need to be taken if blending is being considered:

- Identify potential sources of blend materials on the project site. Coarse material from cuts can often be used to improve the properties of finer materials in lower-lying areas, while the same fine materials can be used to improve the grading of coarse and sandy materials. Document this information in the site investigation report.

- Identify potential sources of blend materials in the area if suitable materials are not available on the project, or if the quantities of suitable materials are insufficient. Sources could include fill dirt from other construction projects or farming activities, construction waste (e.g., crushed concrete), or tailings from crushing plants. Document this information in the site investigation report.
- Collect additional samples of materials to determine appropriate blends. Identify the location and purpose of these materials on the site investigation report.

4.3.3 Testing, Analysis, and Reporting

Optimal blends can be determined using either arithmetical or graphical methods. The arithmetical method is less tedious, but does not provide a range of potential blends. Both methods require material grading data. Atterberg Limit and R-value (or UCS or CBR) tests are used to check if the properties of the blended material meet the project requirements and specifications. Test methods are listed in Table 4.3.

Table 4.3: Mix Design Test Methods for Determining Optimal Soil Blending

Parameter	Test Method	Reason
Grading Analysis	CT 202	Determine before blend and after blend grading.
Atterberg Limits	CT 204	Check that plasticity requirements are met after blending.
R-value	CT 301	Check that R-value requirements are met after blending.
UCS CBR	CT 221 ASTM D1883 or AASHTO T 193	Alternative strength tests to R-value.

<u>Arithmetical Method</u>

Use the following procedure to determine an optimal blend of two materials using the arithmetical method. The procedure is suited to blending more than two materials; however, it is very difficult to achieve a quality blend of more than two subgrade materials and consequently this should not be considered.

1. Prepare a calculation table (example template in Table D.1 in Appendix D) and complete Columns 1 through 5 with available information from the grading analyses and target grading envelope or specifications. Example gradings for two materials and a target grading envelope are provided in Table 4.4, which were used to complete the example calculation table provided in Table 4.5.

Table 4.4: Example Gradings for Potential Blend Materials

	Percent Passing		
Sieve Size	Soil A	Soil B	Target or Specification Limits
1”	100	100	100
3/4”	92	72	70 - 100
3/8”	83	45	50 - 80
#4	75	27	35 - 65
#10	67	15	20 - 50
#40	52	5	15 - 30
#200	33	1	5 - 15

Table 4.5: Table for Determining Optimal Blend Using Arithmetical Method

Column										
1	2	3	4	5	6	7	8	9	10	11
Sieve Size	Soil A Grading	Soil B Grading	Target Limits		Target Mid Point (TMP)	|TMP - A|	|TMP - B|	0.43 A	0.57 B	Blend
			Low	High						
1”	100	100	100	100	100	0	0	43.0	57.0	100.0
3/4”	92	72	70	100	85	7	13	39.6	41.0	80.6
3/8”	83	45	50	80	65	18	20	35.7	25.7	61.3
#4	75	27	35	65	50	25	23	32.3	15.4	47.6
#10	67	15	20	50	35	32	20	28.8	8.6	37.4
#40	52	5	15	30	22.5	29.5	17.5	22.4	2.9	25.2
#200	33	1	5	15	10	23	9	14.2	0.6	14.8
						Σ|TMP - A| = 134.5	Σ|TMP - B| = 102.5			

2. Complete Column 6 by determining the midpoint of the target or specification range for each sieve size.
3. Complete Column 7 by subtracting the value in Column 2 from the value in Column 6 for each sieve size.
4. Complete Column 8 by subtracting the value in Column 3 from the value in Column 6 for each sieve size.
5. Sum the values in Columns 7 and in Column 8.
6. Calculate multipliers for each soil using the following formulas. These multipliers will be the approximate percentages of the two soils used in the blend.

   Soil A = Sum of Column 8 / (Sum of Column 7 + Sum of Column 8)
   Soil B = Sum of Column 7 / (Sum of Column 7 + Sum of Column 8)
7. Complete Columns 9 and 10 by determining the optimal percentages of each sieve size using the following formulas. These values will be used to check whether the target for each sieve size is met.

   Percentage Soil A = Value in Column 2 x (Soil A multiplier from Step 6)
   Percentage Soil B = Value in Column 3 x (Soil B multiplier from Step 6)
8. Complete Column 11 by adding the values in Columns 9 and 10 for each sieve size, to check whether the blend will fall within the target range.
9. Prepare a sample blend (e.g., in the example, a blend of 4.3 lbs of Soil A and 5.7 lbs of Soil B will provide 10 lbs of soil for testing) and check the grading to determine whether it meets the target. Adjust the percentages of each soil if necessary.
10. Test the Atterberg Limits and R-value of the proposed blend to check that they fall within the required design and/or specification. Adjust the percentages and retest if necessary.

Graphical Method

Use the following procedure to determine an optimal blend of two materials using the graphical method:

1. Plot the gradings of the two soils on the x_1 and x_2 axes on a comparative chart. (A template is provided in Appendix D, Figure D.1.) An example grading and plot are provided in Table 4.4 and Figure 4.1, respectively.
2. Connect the corresponding points on the x_1 and x_2 axes (Figure 4.2).

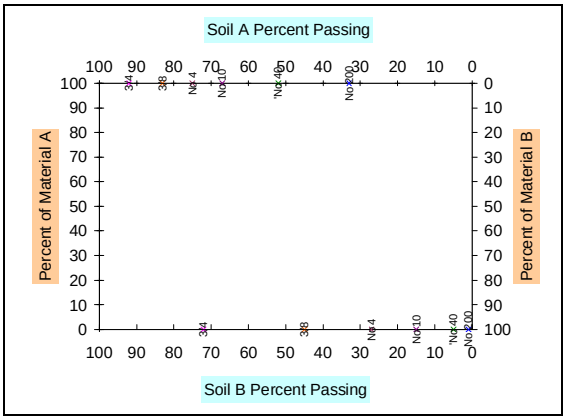

Figure 4.1: Example plot of potential blend materials (Step 1).

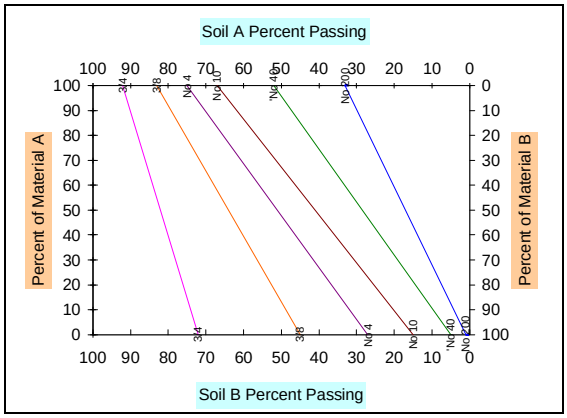

Figure 4.2: Example plot of potential blend materials (Step 2).

3. Plot the minimum and maximum target or specification limits for each relevant sieve size on these connected lines (using the x_1 and x_2 axes) (Figure 4.3).
4. Draw perpendicular lines from each target or specification limit point to the x_1 axis. These lines represent the allowable range for each sieve size in the blend (Figure 4.4, Step 4).

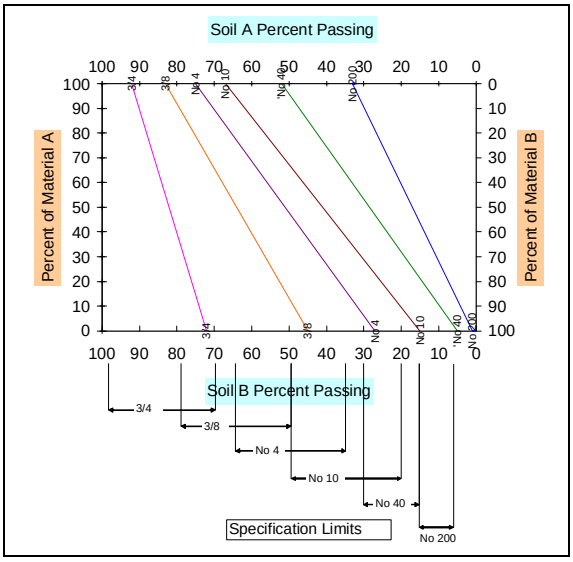

Figure 4.3: Example plot of potential blend materials (Step 3).

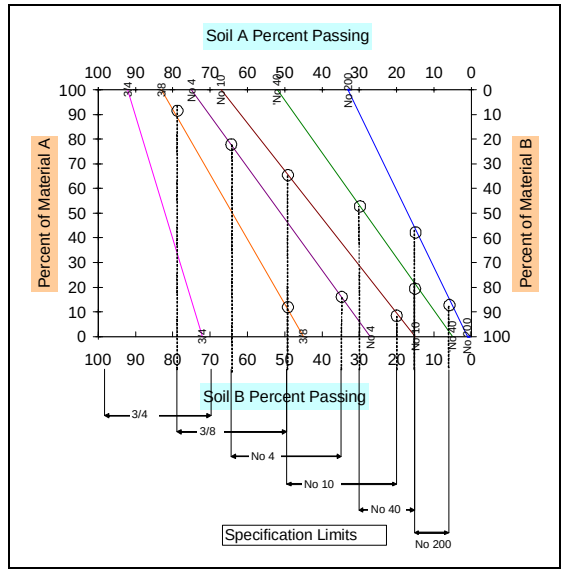

Figure 4.4: Example plot of potential blend materials (Step 4).

5. Identify the perpendicular line with the two closest points. Draw horizontal lines through these two points to connect to the y axes. These lines represent the minimum and maximum percentages of each material that can be used in the blend (Figure 4.5).
6. Select a realistic blend within these ranges (e.g., 20 to 43 percent of Material A and 57 to 80 percent of Material B in Figure 4.5) based on material availability, but avoiding the extremes of each range where possible (e.g., a blend of 25 percent of Material A and 75 percent of Material B would probably be appropriate, but note that the arithmetical method proposed a blend of 43 percent of Material A and 57 percent of Material B, which are the extremes of the range determined using the graphical method.).
7. Test the Atterberg Limits and R-value of the proposed blend to check that they fall within the required design and/or specification. Adjust the percentages and retest if necessary.

Reporting

If satisfactory results are achieved, a recommendation on the use of this strategy can be made. If not, then an alternative stabilization strategy will need to be considered (see Section 3.5).

Summarize the recommended blend and source of blend materials on the stabilization strategy form (example Form 5 in Appendix B).

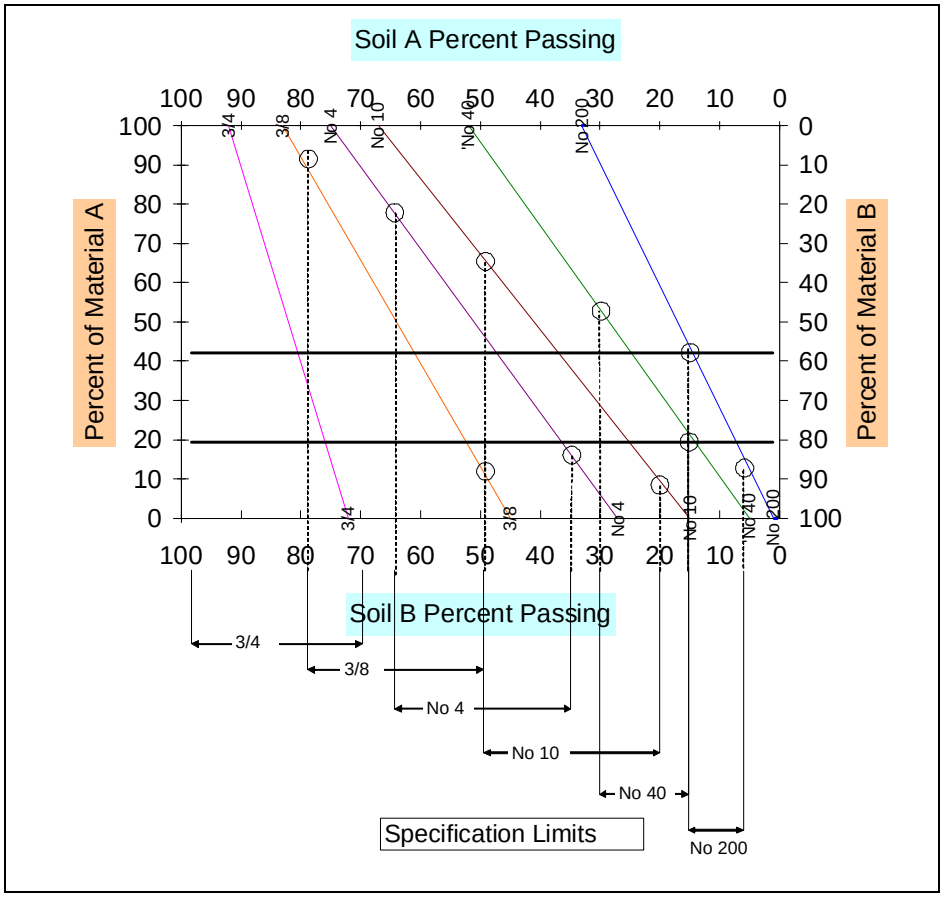

Figure 4.5: Example plot of potential blend materials (Step 5).

4.3.4 Construction Considerations

Key issues that need to be considered during construction are discussed in Chapter 7. Use the same procedure for mixing-in and compacting blend materials as that used for cement or lime.

4.4 Cost Assessment Considerations

The following factors need to be considered for completing a cost assessment to compare mechanical stabilization with other stabilization strategies:

- The costs of additional compaction (e.g., heavier compaction equipment and/or additional roller passes).
- The costs of moving material from one part of a project to another or the purchase and transport of material from another source. The potential savings from being able to use materials elsewhere on a project instead of transporting them off site and disposing of them should also be factored into the analysis.
- The costs of equipment to spread and blend the materials.

4.5 Pavement Design and Project Documentation

Design the pavement according to the *Highway Design Manual* using the laboratory-determined plasticity index and R-values after compaction and/or blending. Detail the additional compaction requirements and/or sources and mix ratios of blend materials in the project documentation.

5. CEMENTITIOUS STABILIZATION

5.1 Introduction

This chapter covers subgrade stabilization using primarily lime and cement. Common problems covered include those related to high soil moisture content, high plasticity, and low strength. Also covered are issues related to soils with high sulfate content, high organic matter content, low pH, and high chloride content, as well as potential problems associated with incorrect curing/incorrect stabilizer contents.

Fly ash and kiln dust, although rarely used for subgrade stabilization in California (they are not covered in the *Standard Specifications*) are also included for informational purposes as they may be considered for specific projects. In these instances, a nonstandard design pursuant to HDM Topic 606 will need to be prepared and a nonstandard special provision written and approved.

5.2 Problems Related to High Soil Moisture Content

5.2.1 Introduction

High soil moisture contents are common in many areas in California, especially in Central Valley silt and silty-clay soils. This can lead to problems with workability and in achieving desired compaction levels. Ripping the soil and allowing dry back through evaporation is the simplest method, but time and weather constraints often limit this approach. Instead, small quantities of lime, cement, kiln dust, or fly ash can be added to the soil to dry it back to the desired moisture content. Lime is the most commonly used modifier; however, the final choice will depend on district experience and/or on performance and cost.

<u>Alternative Methods</u>

- Mechanical (ripping and then allowing moisture to evaporate, with periodic mixing) can be considered as an alternative if time and weather conditions permit.
- Asphalt stabilization is not effective in reducing subgrade soil moisture content.

5.2.2 Site Investigation

No additional site investigation information to that discussed in Chapter 3 is required for correcting high soil moisture content problems. Note that subgrade moisture conditions change throughout the year and results obtained during the site investigation may be different than the conditions during construction.

5.2.3 Testing, Analysis, and Reporting

Laboratory testing to assess dry back of soil using chemical modification is limited to moisture content and density tests (Table 5.1).

Table 5.1: Tests for Assessing Cementitious Modification to Dry Soil and Improve Workability

Parameter	Test Method	Reason
Moisture content	CT 226	Determine optimum stabilizer content.
Relative compaction	CT 216	Check compaction.

Use the following procedure to determine the optimum stabilizer content to dry the soil back to the optimum moisture content:

1. Prepare five samples at the actual moisture content of the subgrade soil.
2. Add lime to each sample at rates of 0.5, 1.0, 1.5, 2.0, and 2.5 percent respectively, mix thoroughly, and allow to equilibrate for 60 minutes.
3. Determine the moisture content of each of the samples.
4. Select the lime content that reduces the moisture content closest to the optimum moisture content. If more than 2.5 percent lime is required, a combination of mechanical and lime treatment may be required. Very high moisture contents may be an indication of a water problem, which should be investigated in more detail.
5. Check the relative compaction (CT 216) by preparing samples at the actual moisture content and selected stabilizer content. The wet density should be equal to or higher than the relative compaction determined earlier (see Section 3.4).
6. Repeat the test if required to compare different stabilizers (e.g., portland cement, kiln dust, or fly ash).

<u>Reporting</u>

Summarize the recommended soil modification approach for drying back the soil on the stabilization strategy form (example Form 5 in Appendix B).

5.3 Problems Related to High Plasticity

5.3.1 Introduction

High plasticity is common in many areas of the state, especially in the Central Valley silty-clay and clay soils. This affects workability, compactability, and layer strengths. Plasticity can be reduced by modifying or stabilizing the soil with lime or cement. Lime is the most commonly used modifier; however, final choice will depend on district experience and/or on performance and cost. Kiln dust or fly ash can also be considered as alternatives provided that satisfactory reductions in plasticity index can be achieved in laboratory tests, and a cost assessment shows that they are more cost-effective.

Alternative Methods

- Mechanical stabilization (blending) can be considered if only marginal plasticity reduction is required.
- Asphalt stabilization is not suitable for reducing the plasticity of subgrade soils.

5.3.2 Site Investigation

No additional site investigation information to that discussed in Chapter 3 is required for assessing soil plasticity reduction.

5.3.3 Testing, Analysis, and Reporting

Laboratory testing to assess plasticity reduction using cementitious modification is limited to Atterberg Limits tests (Table 5.2).

Table 5.2: Tests for Assessing Cementitious Modification to Reduce Soil Plasticity

Parameter	Test Method	Reason
Atterberg Limits	CT 204	Determine if satisfactory plasticity reduction can be achieved.

Use the following procedure to determine the optimum lime content to reduce the soil plasticity to an acceptable level:

1. Prepare five soil samples according to Part IV of CT 204. Use modifier rates of 0, 1.0, 2.0, 3.0, and 4.0 percent of the dry soil weight. Note that soils with high salt contents and/or low pH will typically require higher than expected stabilizer contents and may require pretreatment with lime.
2. Determine the Atterberg Limits of the samples according to CT 204.
3. Select the lime content that reduces the plasticity index in the defined depth range to below the design requirements (i.e., usually below 12).
4. If the plasticity index is not reduced to a satisfactory level, repeat the tests using cement or other modifier, or increase the lime content in 1.0 percent intervals. Note that lime or cement contents above 4.0 percent may be uneconomical and can lead to problems related to shrinkage.

Reporting

Summarize the recommended soil modification approach for reducing the soil plasticity on the stabilization strategy form (example Form 5 in Appendix B).

5.4 Problems Related to Low Strength

5.4.1 Introduction

Economical, constructible pavement structures, or pavements that can match surface elevations of adjacent lanes, often cannot be designed for existing subgrade soil conditions. Consequently, the soil needs to be stabilized with an appropriate stabilizer that will increase its strength to a satisfactory level.

Cement and lime are the most commonly used cementitious stabilizers to improve subgrade soil strengths in California. However, fly ash and kiln dust can also be considered provided that satisfactory strengths can be achieved at lower cost than cement or lime.

Alternative Methods

- Mechanical stabilization (compaction or blending) can be considered if only marginal strength improvements are required.
- Asphalt stabilization can be considered as an alternative to cementitious stabilization on sandy and coarser-grained materials.

5.4.2 Site Investigation

No additional site investigation information to that discussed in Chapter 3 is required for assessing soil strength improvement.

5.4.3 Testing, Analysis, and Reporting

Laboratory testing to assess strength improvement using cement or lime stabilization requires a number of tests, which are listed in Table 5.3.

Table 5.3: Tests for Assessing Cement Stabilization to Increase Strength

Parameter	Test Method	Reason
ICL/ICS	ASTM D6276	Determine starting stabilizer content.
UCS (cement)	ASTM D1633[1]	Check whether required strength can be obtained using cement.
UCS (lime)	ASTM D5102[1]	Check whether required strength can be obtained using lime.
Durability	ASTM D559+D560	Check wet/dry and freeze/thaw durability.
ITS	CT 371 or ASTM D6931	Additional strength test.
CBR	ASTM D1883 or AASHTO T 193	Alternative strength test for screening different stabilizers.

[1] ASTM D1633 (cement) or ASTM D5102 (lime) with specimens compacted according to ASTM D1557 is preferred over CT 312/CT 373 as they typically result in lower stabilizer contents associated with higher compaction densities, which is preferred to higher stabilizer contents/lower compaction densities, both from an engineering and economic standpoint. CT 312 requires a 6-hour soak compared to a national standard of 4 hours. CT 373 does not include soaking prior to testing, which is considered important for assessing treated subgrade soils.

Use the following procedure to determine the optimum stabilizer content for strength improvement:

1. Confirm that soil sulfate content is less than 3,000 parts per million and organic matter content is less than 1.0 percent. If the values are higher, then refer to Section 5.5 (stabilizing sulfate-rich soils) or Section 5.6 (stabilizing soils with high organic matter content), respectively.
2. Choose a stabilizer based on past experience for best practice in the district (typically lime or cement). If the soil has a high plasticity, consider a combination of lime and cement to achieve the desired result if a single treatment does not meet the requirements (e.g., first treat with lime to correct the plasticity and then treat with cement to improve the strength). If there is no standard/preference in the district or if alternatives are being investigated, compare the performance of different stabilizers. A guide to stabilizer selection, based on percent passing the #200 (0.075 mm) sieve and plasticity index, is provided in Figure 5.1.

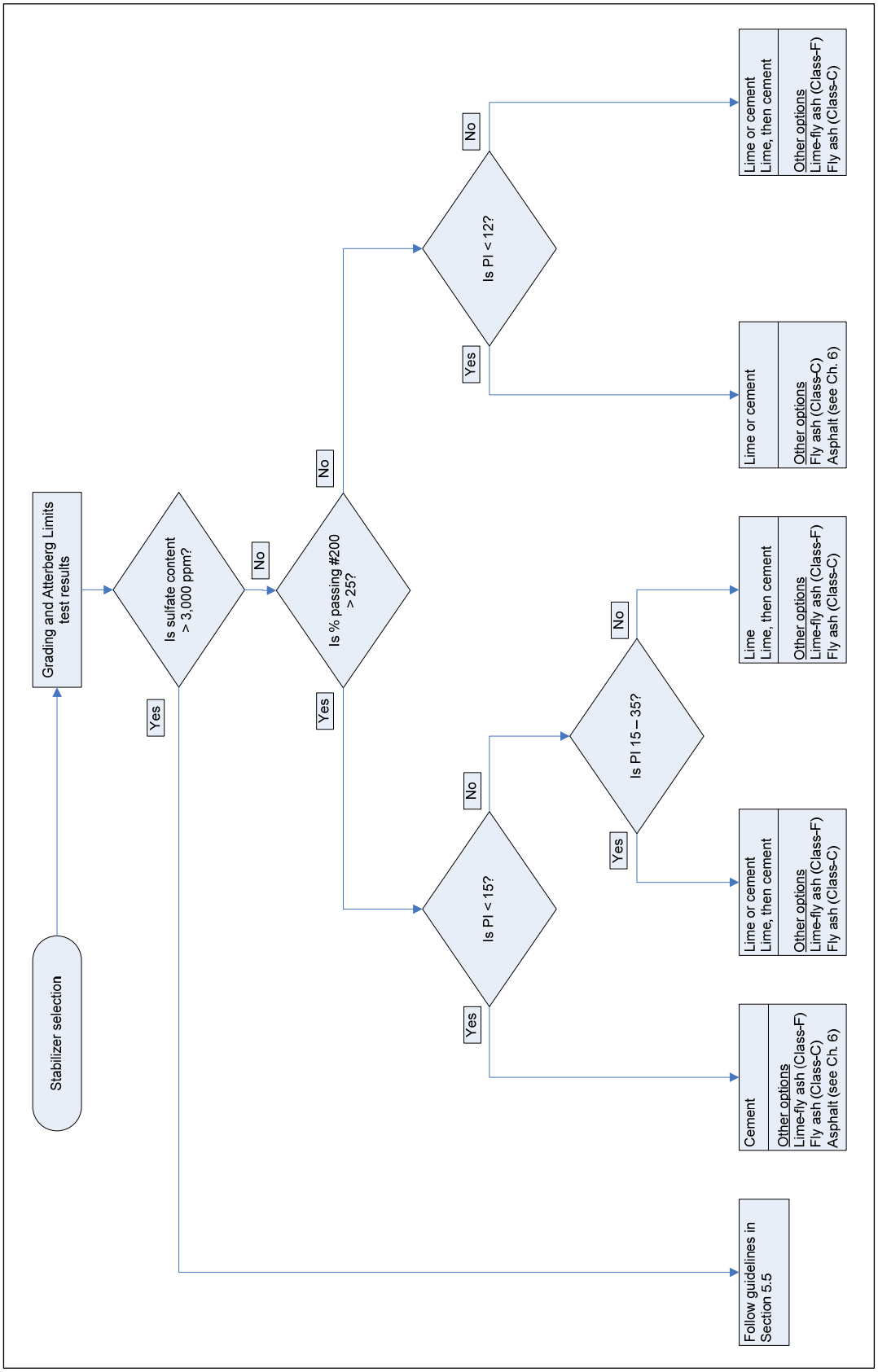

Figure 5.1: Decision tree for selecting cementitious stabilizer type.

Satisfactory performance may be obtained from more than one type of stabilizer, in which case selection should be based on experience and/or cost. **Note that successful cementitious stabilization relies on physiochemical reactions between the stabilizer and the soil and is thus dependent on soil chemistry, which varies widely across California. Experience with a particular stabilizer and/or application rate in one area does not necessarily mean that the same success will be obtained in another, and consequently experimentation with both lime and cement and a range of application rates is encouraged to identify the most appropriate stabilization strategy.**

3. **Do not use "standard" application rates.** Determine the initial consumption of stabilizer (ICS) using ASTM D6276 or the revised version of the test provided in Appendix E. This test, originally developed for lime but also suitable for other cementitious stabilizers, is used to determine the minimum stabilizer content required to maintain the soil pH at 12.4 or higher and provides a **starting point** for selecting an appropriate stabilizer content. Stabilizer contents lower than the ICS value may result in temporary modification of the soil (i.e., plasticity reduction), and strength reduction over time and/or carbonation (see Section 5.11.4) is possible. Note that soils with high organic matter and salt contents and/or low pH will typically require higher-than-expected stabilizer contents. If the initial lime or cement demand cannot be satisfied (i.e., the pH of the soil cannot be raised to 12.4), or if an uneconomical amount of stabilizer is required to meet the lime or cement demand, an alternative stabilization method should be considered.
4. Prepare three sample batches for the UCS test according to ASTM D1633 if cement is the selected stabilizer, or according to ASTM D5102 if lime is the selected stabilizer. Use the percentage stabilizer required to satisfy the ICS, ICS + 1.0 percent, and ICS + 2.0 percent. Use ICS +1 percent to determine optimum moisture contents if required. Compact the specimens according to ASTM D1557. (With ASTM D1557, more energy is applied using this test method, and consequently the soil particles are more closely packed. The general overall result is a higher maximum dry unit weight, lower optimum moisture content, greater shear strength and stiffness, lower compressibility and air voids, and decreased permeability, all of which lead to lower, more realistic cement contents.)
5. Determine the UCS of the samples according ASTM D1633 (cement) or ASTM D5102 (lime).
6. Compare the results for the three batches. A soaked UCS of between 30 psi and 60 psi (200 kPa to 400 kPa) is considered sufficient for general subgrade improvement. Strive to achieve the target project strength requirements and not necessarily "maximum" strength as this could be a waste of stabilizer and may lead to shrinkage-related problems. If the required result has not been achieved, repeat the test with higher stabilizer contents and/or with an alternative stabilizer or stabilizer combination. Note that stabilizer contents above 4.0 percent may be uneconomical and, depending on the soil properties, can lead to problems related to shrinkage.
7. If the subgrade is susceptible to wetting and drying cycles (i.e., fluctuating high water table or periodic flooding) or freeze-thaw cycles, check the durability of the stabilized soil using ASTM D559 and/or ASTM D560. Caltrans does not have criteria for wet-dry or freeze-thaw durability in subgrade soils. Instead, the limits summarized in Table 5.4, which are based on national experience, can be used as a guide for assessing durability.

Table 5.4: Recommended Limits for Durability of Stabilized Subgrade Soils

Soil Type	Recommended Limit	
	Maximum Weight Loss (%)	Minimum Soaked UCS (psi [kPa])
Granular, PI <10	11	30 (200)
Granular, PI >10	8	
Silt	8	
Clay	6	

Using Stabilized Subgrade as a Subbase in Flexible Pavements

If the untreated subgrade soil being investigated has a relatively high R-value, or stabilization increases the soaked UCS significantly (i.e., to between 100 psi and 300 psi [~750 kPa and 1.5 MPa]), consideration may be given to using it as a subbase in order to reduce the overall pavement thickness on flexible pavements (note that the *Highway Design Manual* states that "treating the subgrade does not eliminate or reduce the required aggregate subbase for rigid or composite pavements in the rigid pavement catalogue" and that lime should not be used to reduce pavement structure thickness for design lives greater than 20 years). If this strategy is pursued, use the UCS to calculate the gravel factor, which will be used to determine layer thicknesses. Additional UCS testing with higher stabilizer contents may be required to achieve the desired gravel factor. Note that the formula provided in Section 614 of the *Highway Design Manual* (Equation 5.1) for calculating the gravel factor of lime-treated soils is only applicable to UCS determined according to CT 373, which does not require soaking of the specimens prior to testing.

$$G_f = 0.9 + \frac{UCS(psi)}{1{,}000} \qquad \textbf{(5.1)}$$

The formula is **not directly applicable** to unconfined compressive strengths determined according to ASTM D1633 and ASTM D5102, which include soaking periods. Instead, the formula can be used with results from these tests with modification, as detailed in Section 24 of the *Standard Specifications*. These modifications include (see details on test method modification in Appendix E):

- Sealing the specimen immediately after compaction to prevent moisture loss.
- Curing the specimen in an oven for seven days at 100°F ± 5°F (40°C).
- Air cooling the specimen after curing.
- Testing for UCS without soaking.

The modified method should only be used if a gravel factor needs to be calculated. It does not replace the standard method discussed above used for selecting an appropriate stabilizer and determining optimal stabilizer contents, where understanding moisture sensitivity is very important (i.e., testing after a four-hour soak).

If feasible, determine the indirect tensile strength (ITS) as well (according to CT 371 using specimens prepared according to ASTM D1557) to ensure that the treated material has both sufficient tensile and compressive strength. ITS values should typically be higher than 20 psi (140 kPa).

Reporting

Summarize the recommended soil stabilization approach for improving strength on the stabilization strategy form (example Form 5 in Appendix B).

Additional Considerations for Class-C Fly Ash Stabilization

Research has shown that Class-C fly ash can be used as a standalone subgrade soil stabilizer. However, there are no standard mix design procedures available for it and its use in subgrade soil stabilization is not covered in the *Standard Specifications*. Hydration rates will also differ depending on the source and properties of the fly-ash and consequently the strength achieved will depend on the compaction moisture content and the delay between the start of hydration and the start and completion of compaction. Optimal strengths are typically achieved at between 1.0 percent and 8.0 percent below optimum moisture content. Note that fly ash is not produced in California in significant quantities and will need to be imported from other states. Cost effectiveness will need to be determined before a decision is made on whether to use this option.

Consider the following additional procedures to those described above if using fly ash as a stabilizer:

1. Determine a starting stabilizer content using the initial consumption of stabilizer test using fly ash from the same source that would be used on the actual project. **Do not use a "standard" from previous projects.**
2. Determine a moisture-density relationship using soil sampled from the project at rates of ICS + 1.0 percent, ICS + 2.0 percent, and ICS + 3.0 percent.
3. Prepare replicate specimens at the determined optimum moisture content (OMC), and at moisture contents between OMC – 1.0 percent and OMC – 5.0 percent. Ensure that the same delay between addition of water and compaction of the specimens is applied to all mixes.
4. Cure the specimens for seven days at 100°F (38°C).
5. Determine the UCS of the specimens.
6. Compare the results with the project requirements to see if satisfactory improvement has been achieved. Select the stabilizer content/moisture content that gives the required result. If the required result has not been achieved, repeat the test with higher stabilizer contents and/or different moisture contents.
7. Determine the sensitivity of strength to moisture content. If there is a significant difference in strength between the different moisture contents, determine whether moisture contents can be adequately controlled to the extent required during construction. If not, consider an alternative stabilizer.

Additional Considerations for Lime/Fly Ash (Class-F) or Cement/Fly Ash (Class-F) Stabilization

There is some reference in the literature to the use of lime or cement and Class-F fly ash for subgrade soil stabilization, but there is no documented experience in California and their use is not covered in the *Standard Specifications*. The materials tend to have variable properties and strength gain tends to be much slower than lime, cement, and Class-C fly ash.

Consider the following additional procedures to those described above if using Class-F fly ash with either lime or cement as a stabilizer:

1. Determine a starting lime or cement content using the initial consumption of stabilizer test. Use this value plus 1.0 percent as a starting rate for the fly ash and half this value as a starting point for the lime or cement (i.e., a ratio of one part lime or cement to two parts fly ash).
2. Determine a moisture-density relationship for this blend using soil sampled from the project.
3. Determine the UCS of the blend using this moisture content and the same curing procedure used for lime or cement, depending on which stabilizer is used.
4. Compare the results with the project requirements to gauge the strength of the blend. Using this as a basis, experiment with a range of blends to determine an optimal blend that will result in the required UCS, maintaining a ratio of one part activator (lime or cement) to two parts fly ash. Determine an optimum moisture content for each blend.

5.5 Stabilizing Sulfate-Rich Subgrade Soils

5.5.1 Introduction

Sulfate-bearing soils generally occur in lower lying areas in central and southern California and are mostly associated with gypsum deposits. Stabilization of these soils with calcium-based stabilizers in the presence of excess moisture can lead to the formation of minerals such as ettringite and/or thaumasite, which expand rapidly potentially causing severe distress in pavements due to heaving. Consequently it is important to test for sulfates before deciding on a stabilization strategy, and if sulfates are present, to develop an appropriate strategy to deal with them.

5.5.2 Site Investigation

If past experience, the desktop study, and/or soil test results from the initial site investigation indicate a presence or potential presence of sulfate in the soils, check whether any other investigations on the issue have been undertaken. If not, a more thorough investigation to accurately map the sulfate content along the length and width of the alignment will need to be completed. Sulfate concentrations in soil exhibit high spatial heterogeneity and consequently the selection of locations to sample is critical. The minimum recommended frequency for drilling and logging test holes is every 500 ft (150 m) on alternating sides of the width of the road bed. Select additional test locations if required based on the geological and soil map

studies, and a visual assessment of the alignment and surrounding topography. Indicators for potential problems include but are not limited to:

- Occurrence of gypsum, pyrite, and other minerals containing sulfates. Gypsum typically occurs in seams and/or pockets, depending on deposition.
- Rolling terrain that could lead to concentrated accumulations of salts in lower-lying areas as a result of runoff.
- Dry streams and fluctuating water tables that could leave a concentration of salts near the surface.
- Presence of shale outcrops and clays with known low hydraulic conductivity, high capillarity, and/or high suction properties that could create sulfur reservoirs.

Drill test holes to a depth corresponding to the final grade in cut areas. Document the precise location of each sampling point on the sample.

5.5.3 Testing, Analysis, and Reporting

Laboratory testing to determine sulfate content is limited to a soil sulfate content test (Table 5.5) plus the mix design tests discussed in Sections 5.3 and 5.4.

Table 5.5: Tests for Assessing Sulfate Content

Parameter	Test Method	Reason
Sulfate content	CT 417 or AASHTO T 290[1]	Determine sulfate content.
[1] AASHTO T 290 with modifications recommended in NCHRP 145 ("Recommended practice for stabilization of sulfate-rich subgrade soils") is recommended instead of CT 417.		

Use the following procedure to assess the sulfate content of sampled soils:

1. Determine sulfate content according to CT 417 or preferably AASHTO T 290 with modifications recommended in NCHRP Report 145 (*Recommended practice for stabilization of sulfate-rich subgrade soils*). Modifications include a higher soil-water dilution ratio (1:10 instead of 1:3), testing on material passing the #4 (4.75 mm) sieve instead of the #10 (2.0 mm) sieve, and the use of water alone for extracting sulfates from the soil.
2. Determine the level of risk for each sample using the criteria in Table 5.6.

Table 5.6: Level of Risk Associated with Cementitious Stabilization in Sulfate Soils

Risk	Soluble Sulfate Concentrations	
	Parts per Million	Percent Dry Weight
1. Low	< 3,000	< 0.3
2. Moderate	3,000 to 5,000	0.3 to 0.5
3. Moderate to high	5,000 to 8,000	0.5 to 0.8
4. High to unacceptable	> 8,000	> 0.8
5. Unacceptable	> 11,000	> 1.0

<u>Analysis</u>

Analyze the results as follows. A flowchart summarizing the approach is provided in Figure 5.2:

1. If the level of risk is low for the entire project, follow the procedures discussed in Section 5.2 through Section 5.4.
2. If the level of risk varies along the project, plot the levels of risk on a map of the road alignment using the sample locations recorded on site. If the risk areas are limited in number and/or extent, consider removing the affected soil and replacing it with sulfate-free and low-sulfate content soils, or blending the affected soil with sulfate-free soil to dilute the sulfate content to an acceptable level (particular attention will need to be given to ensure uniform blending during construction). Retest the soil to check that a satisfactory level has been achieved.
3. If the risk areas are more extensive and the risk level is moderate or moderate-to-high, consider the use of lime and/or cement, but following a revised mix design testing approach and special construction procedures, discussed in the following sections. Alternatively, consider non-cementitious stabilization if the material properties are suitable.
4. If the risk levels are high or unacceptable, consider realignment of the road to avoid the problem areas, removal and replacement, dilution, or a non-calcium stabilizer. Other site-specific treatments such as fly ash/lime blends or ground-granulated blast furnace slag with lime, together with extended mellowing periods (i.e., time allowed for water and stabilizer to distribute evenly throughout the soil and begin reacting before mixing and compaction) can be experimented with, although there is no record of their successful use in California.

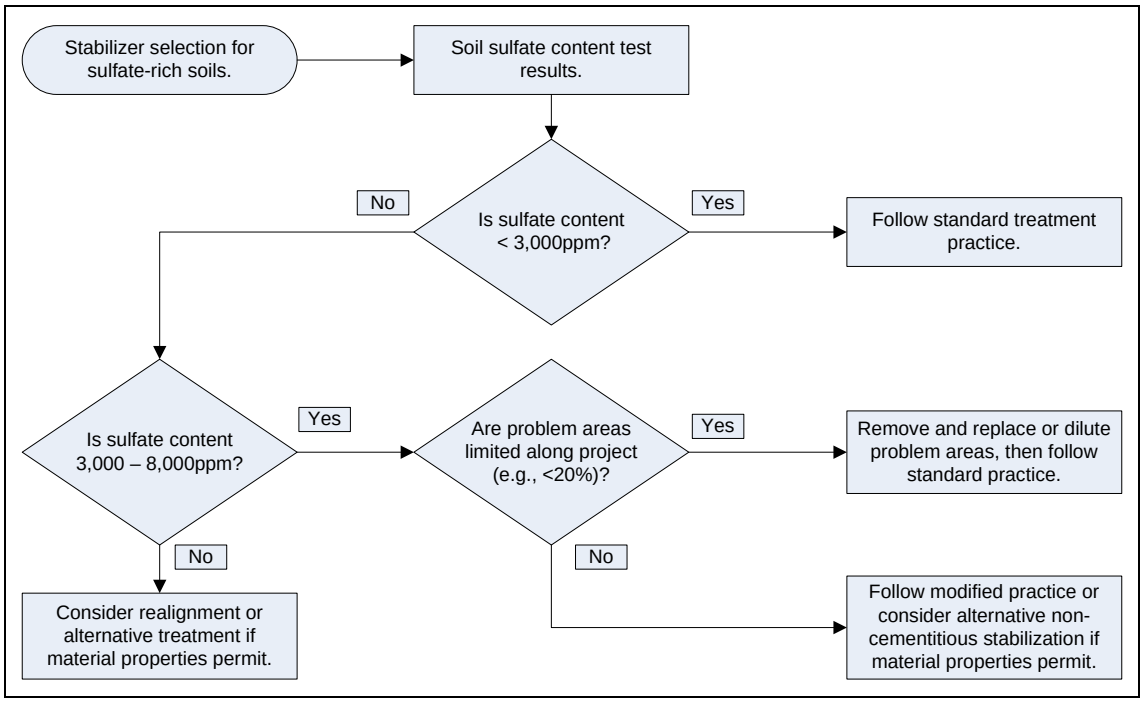

Figure 5.2: Guide for selecting a stabilization strategy for sulfate-rich soils.

Revised Mix Design Process for Stabilizing Sulfate-Rich Soils

Use the following procedure to determine stabilizer contents, moisture contents, mellowing periods, and strength on sulfate-rich soils:

1. Choose a stabilizer using the same criteria described above. Lime is the most commonly used stabilizer for sulfate-rich soils.
2. Determine the initial consumption of stabilizer (ICS) as described above on samples removed from the same position as those used to determine the sulfate content.
3. Determine the optimum moisture content according to CT 216.
4. Prepare three replicate samples at the design moisture content, and three replicate samples each at the design moisture content plus one, two, three and four percent.
5. Determine the sulfate content immediately after preparation and then at 24-hour intervals for seven days using CT 417 or AASHTO T 290.
6. Determine the point at which the sulfate content drops below 3,000 ppm. Select the best combination of moisture content and mellowing time that achieved the sulfate-content reduction.
7. Prepare additional samples at the design stabilizer content at the determined moisture content and allow to mellow.
8. Complete the mix design tests discussed in Section 5.4.3 to determine whether the project strength requirements can be met.
9. Check the results against the project requirements to see if satisfactory improvement has been achieved. If the required result has not been achieved, repeat the test with higher stabilizer contents, an alternative stabilizer, or a combination of lime and an additional stabilizer. If a combination is used, the sulfate treatment discussed in tasks 1 through 8 above should be completed first before the additional stabilizer is added.

The use of sulfate-rich materials in subbase or base layers is not recommended.

Construction Considerations for Stabilizing Sulfate-Rich Soils

Special construction procedures are required for stabilizing sulfate-rich soils. These are discussed in Section 7.12.

Reporting

Summarize the recommended soil stabilization approach for stabilizing sulfate-rich soils on the stabilization strategy form (example Form 5 in Appendix B).

5.6 Stabilizing Soils with High Organic Matter Content

5.6.1 Introduction

Soils with high organic matter contents are found in California, especially in the Central Valley and other valleys in the state. Soils with organic matter content in excess of one to two percent are usually difficult to stabilize due to inhibition of the normal hydration process and the consequent influence on strength gain. Uneconomical quantities of stabilizer are therefore usually required to achieve the required strengths. The presence of organic matter can also affect the compactability of the soil (i.e., organic matter may

compress during compaction and then decompress later resulting in a lower density than that recorded during construction and consequent potential for densification/rutting under traffic).

5.6.2 Site Investigation

If the desktop study (organic matter is usually adequately covered on soil maps) and soil test results from the initial site investigation indicate a presence of organic matter content in the soils, undertake a more thorough investigation to accurately map the problem along the length and width of the alignment. Organic matter is usually more evenly distributed and consequently easier to map than the sulfate-rich soils discussed above. A centerline study every 1,000 ft (300 m) will probably suffice, although more frequent sampling may be required to identify problem area boundaries. Drill test holes to a depth corresponding to the final grade in cut areas. Document the precise location of each sampling point on the sample.

5.6.3 Testing, Analysis, and Reporting

Laboratory testing to assess organic matter content is limited to an organic matter content test (Table 5.7), plus the mix design tests discussed in Sections 5.3 and 5.4. Compacted densities after treatment should also be checked to ensure that this parameter is not negatively affected. The wet density should be equal to or higher than the relative compaction determined earlier (see Section 3.4).

Table 5.7: Tests for Assessing Organic Matter Content

Parameter	Test Method	Reason
Organic matter content	ASTM D2974	Determine organic matter content.
Relative compaction	CT 216	Check effect of organic matter on compaction.

<u>Analysis</u>

Analyze the results as follows. A flowchart summarizing the approach is provided in Figure 5.3.

1. If the organic matter content is less than 1.0 percent for the entire project, follow the procedures discussed in Section 5.3 and Section 5.4.
2. If the organic matter content is less than 5.0 percent for the entire project, follow the procedures discussed in Section 5.3 and Section 5.4 to determine whether satisfactory results can be obtained using realistic quantities of stabilizer. The initial consumption of stabilizer test will give an indication of whether excessive stabilizer contents will be required. If they can, continue with the project following standard stabilization practices at the determined stabilizer content. If not, consider an alternative stabilization strategy (e.g., mechanical stabilization, asphalt stabilization, remove and replace, or dilution).
3. If satisfactory results cannot be obtained, or uneconomical quantities of stabilizer are required, and the organic matter content varies along the project, plot the contents on a map of the alignment using the sample locations recorded on site. If the problem areas are limited in number and/or

extent, consider removing the affected soil and replacing it with cleaner soils, or blending the affected soil with clean soil to dilute the organic matter content to an acceptable level.

4. If organic matter is more widespread, stabilization with asphalt emulsion can be considered if the material properties are suitable and satisfactory results can be achieved. Alternatively, a remove-and-replace strategy will need to be followed.

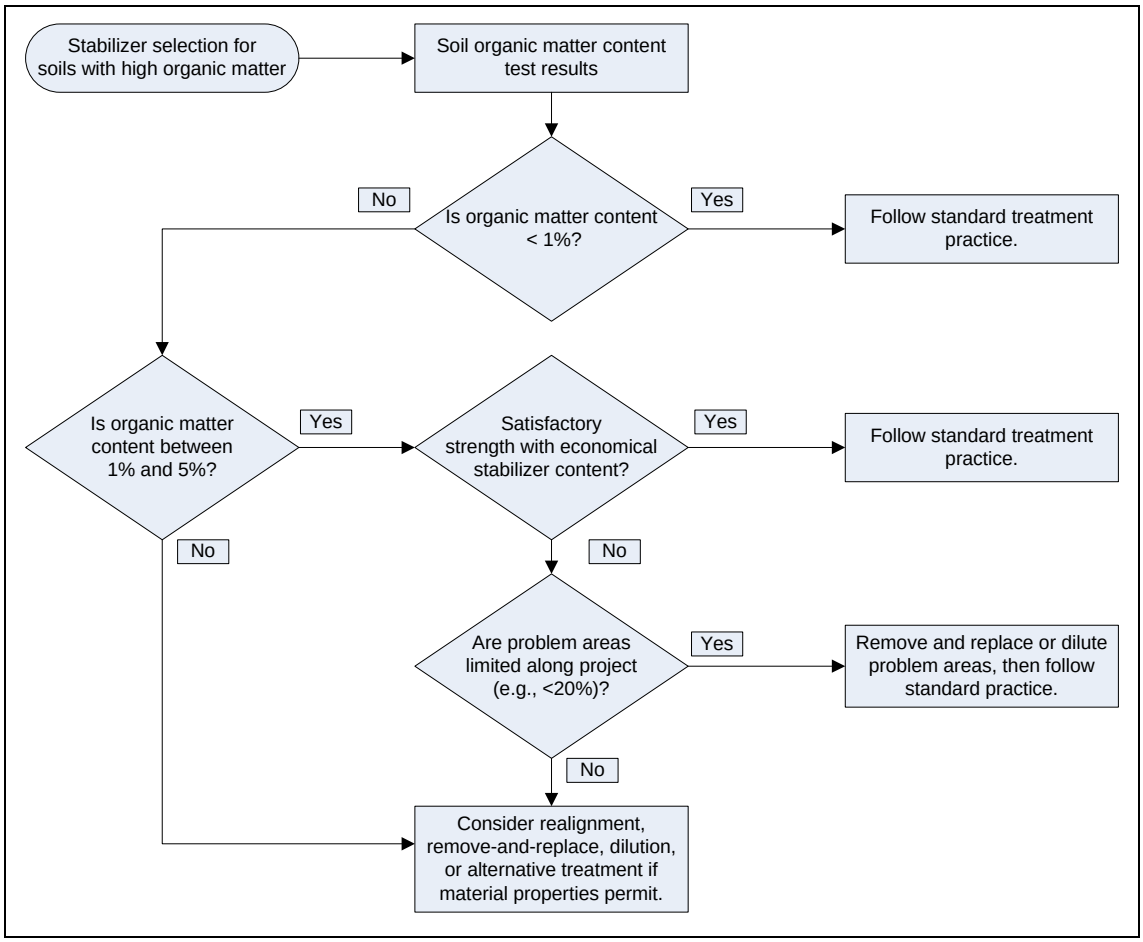

Figure 5.3: Guide for selecting a stabilization strategy for soils with high organic matter.

Reporting

Summarize the recommended soil stabilization approach for stabilizing soils with high organic matter content on the stabilization strategy form (example Form 5 in Appendix B).

5.7 Stabilizing Soils with Low pH or High Chloride Content

Soils with low pH or high chloride content can be effectively stabilized with lime or cement to reduce plasticity and/or increase strength. However, the lime or cement contents needed to achieve the design plasticity or strengths are typically higher than on soils with normal pH and chloride contents, and project

costs will consequently also be higher. Consideration should first be given to neutralizing the soil with lime or other stabilizer, before undertaking the mix designs discussed in Sections 5.3 and 5.4. If an uneconomical amount of stabilizer is required to meet the lime or cement demand, an alternative stabilization method should be considered.

5.8 Cost Assessment Considerations

The following factors need to be considered for completing a cost assessment to compare cementitious stabilization with other stabilization strategies:

- The costs of the stabilizer required to achieve the required unconfined compressive strength. Include the costs of transporting and storing the stabilizer.
- The costs of removing and replacing materials against the use of higher stabilizer contents (i.e., in soils with high organic matter contents and/or low pH).
- The costs of moving material from one part of a project to another or the purchase and transport of material from another source. The potential savings of being able to use materials from elsewhere on a project instead of transporting them off site and disposing of them should also be factored into the analysis.
- The unit costs of one stabilization method over another (include the costs of extended mellowing periods/water spraying when stabilizing sulfate-rich soils).

5.9 Pavement Design and Project Documentation

Design the pavement according to the *Highway Design Manual* using the laboratory-determined plasticity index and/or strength values after modification and/or stabilization. Detail the stabilizer type and application rate, along with any specific construction requirements (e.g., for dealing with sulfate-rich soils) in the project documentation.

5.10 Shrinkage Cracking Mitigation on Cementitious Stabilized Soils

Shrinkage cracking in stabilized subgrade or subbase soils is not as significant a problem as it is in base material, since cracks are unlikely to reflect through the overlying untreated base and into the surfacing layers. However, it is still important to minimize any shrinkage cracking to ensure that the treated layer acts as a uniform stable platform for the pavement structure. Shrinkage cracking mitigation is discussed in Section 7.13.

5.11 Postconstruction Durability Issues

Postconstruction durability problems in cementitious stabilized layers are normally manifested as one of the following:

- Disintegration or weakening of the upper portion of the stabilized layer, leading to a loose layer of material;
- Partial or even complete loss of cementation of the treated layer, leading to reduced strength, rutting, cracking, and/or shearing; or
- Erosion of the stabilized layer.

These problems are of less concern in subgrades compared to base and subbase layers, but knowledge of them can alleviate potential performance issues later in the life of the pavement. They can usually be attributed to various causes, including inappropriate construction and curing methods, prolonged periods between completion of construction of the stabilized subgrade and starting construction of the next layer, damage by soluble salts or acids, degradation of the aggregate, or carbonation of the stabilized layer.

5.11.1 Inappropriate Curing Methods

The common practice of curing stabilized layers by "keeping it continuously wet" frequently results in weakening or loosening of the top 0.25 in. to 1.0 in. (6 mm to 25 mm) as the constant moisture content requirement is seldom met (i.e., the layer is subjected to repeated wetting and drying cycles). This loose layer will typically have a low pH, an increased reactivity with hydrochloric acid, and will generally contain more calcium carbonate than material deeper in the layer that remains consistently moist and well-cemented. This weak, loose layer will have to be broomed or scraped off before a subsequent layer can be applied.

5.11.2 Soluble Salts and Acids

Loss of cementation attributed to soluble salts or acids in the soil and/or compaction water can be prevented by ensuring that the electrical conductivity and sulfate content of the soil being treated do not exceed design and specification limits.

5.11.3 Aggregate Degradation

Volume increases associated with degradation of marginal aggregates, particularly basic crystalline rocks (e.g., basalt), in stabilized layers could lead to problems within those layers. The retention of a high pH (i.e., > 12.4, ensuring that the initial cement or lime demand is satisfied) in these materials assists in the prevention of material breakdown.

5.11.4 Carbonation

Mechanism

Lime and, to a lesser extent, cement are unstable under normal environmental conditions and carbonate readily. Carbonation is the process by which lime and cementation products added to a material or developed during the hydration reactions of cementitious stabilization of a material react with carbon dioxide in the atmosphere or soil air. Calcium carbonate is produced and the pH of the stabilized material reduces to that of calcite (about 8.3), resulting in instability of the remaining cementitious products. Although most subgrades stabilized with lime, cement, or combinations of the two perform well, problems may occur with weak or marginal quality materials, especially residual basic crystalline (e.g., basalt) and calcareous (e.g., limestone) materials.

Disintegration of the stabilized layer during carbonation is caused by volume increase (about 10 percent) as lime ($Ca(OH)_2$) changes to calcium carbonate ($CaCO_3$) and to a lesser extent, volume decrease (about 2 percent) as the cementation products revert to silica and calcium carbonate. If the stresses generated by these volume changes exceed the tensile strength of the stabilized material, microcracking will occur. As these microcracks coalesce, the material loses strength and deteriorates. (*Note that in this document, microcracking associated with carbonation should not be confused with precracking, which is defined as intentionally inducing some microcracks in the stabilized layer shortly after final compaction to limit later, more severe shrinkage cracking [see Sections 7.8 and 7.13].*)

Consequences

Carbonation can have a number of negative effects on the stabilization process, including:

- Loss of cementation
- Reduction of lime
- Reduction of the pH from approximately 12.4 (lime) and sometimes higher (certain cements) to about 8.3 (pH of calcium carbonate).
- Decreased solubility
- Expansion, leading to densification of the material under traffic. Stabilized material generally also has a lower maximum dry density and higher optimum moisture content than the equivalent unstabilized material.

Effects typically seen in the field and laboratory include:

- Reduction of the pH to between 8.3 and 10.
- Increase in plasticity index (i.e., return of plasticity index to that of the original material).
- Microcracking of the cemented matrix.
- Loss of strength (average reduction of 40 percent of laboratory-determined UCS and ITS values).
- Formation of a loose layer of material between the subgrade and next layer.

- Deformation in the form of rutting.
- Decrease in electrical conductivity.

Rate of Carbonation

Typical rates of carbonation are as follows:

- Material exposed to the atmosphere: 0.02 in. to 0.08 in. (0.5 mm to 2.0 mm) per day on all exposed surfaces (most rapid penetration occurs during curing and before placing of the next layer).
- Material exposed to soil air (upwards and sideways): 0.08 in. to 2.0 in. (2.0 mm to 50.0 mm) per annum (notably on the bottom and sides of stabilized layers; also from the top to the bottom of stabilized subgrades beneath unstabilized upper layers).

The pH as well as the solubility of lime reduces with an increase in temperature. If a prime coat is applied as a temporary seal on top of the stabilized subgrade during hot summer months, carbonation could accelerate as a result of the drop in the solubility and pH of the stabilizer. The stability of the stabilizing action can also be negatively influenced. The influence of moisture condensation beneath a prime layer as a result of daily temperature fluctuations can also affect the pH, causing a reduction in the available lime in the upper portion of the layer as a result of dilution.

Tests to Confirm Carbonation

Phenolphthalein and hydrochloric acid are useful field indicators to determine whether carbonation has occurred. Solutions of these are sprayed onto the soil being tested, generally on the profile of a freshly exposed face in a trench. The phenolphthalein test indicates where the carbonation has occurred in the treated layer (e.g., where the top 0.75 in. [20 mm] of a layer does not change color relative to the remaining depth, carbonation of the top surface of the layer is likely). Interpretation of phenolphthalein test results is summarized in Table 5.8.

Safety
Phenolphthalein is a strong laxative and a suspected carcinogen and care should be taken when working with it. Hydrochloric acid is corrosive and should also be handled carefully. Suitable protective equipment should be worn when using these chemicals.

Table 5.8: Interpretation of Phenolphthalein Test Results

Reaction	pH	Interpretation
Sprayed area remains colorless	< 8.4	Carbonation is complete or no stabilizer was present
Sprayed area turns pink	8.4 – 10.0	Significant but incomplete carbonation
Sprayed area turns red	> 10.0	Little or no carbonation

If the phenolphthalein test shows no reaction, it must be established whether a stabilizer was originally added. This is achieved using dilute hydrochloric acid. Diluted hydrochloric acid reacts (in the form of effervescence) with the following materials commonly found in pavement layers:

- Carbonated stabilizer.
- Carbonates occurring naturally in the soil (e.g., limestone, dolomite).
- Old lime (fresh lime may show some effervescence).
- Old cement (fresh cement may show a weak effervescence, except for limestone cements, which contain between 6 and 20 percent limestone and effervesce strongly).

Where no effervescence is observed it can be concluded that no carbonation has taken place, or no stabilizer was added to the material. The hydrochloric acid test cannot generally be used on limestones and dolomites as the acid reacts with the carbonate soil, which predominates, masking any lesser reaction with the stabilizer. However, careful comparison between different portions of the stabilized layers, together with phenolphthalein testing, can provide an indication of whether carbonation has affected the material.

Construction Control to Limit Carbonation and Repair of Carbonated Layers

Construction procedures to limit carbonation and to repair it if it occurs are discussed in Section 7.14 and Section 7.15, respectively.

6. ASPHALT STABILIZATION

6.1 Introduction

This chapter covers subgrade stabilization with asphalt emulsion. Although subgrade stabilization with asphalt emulsion is not covered in the *Standard Specifications*, it is included in this guideline for informational purposes because of the growing interest in its use worldwide. If asphalt stabilization is considered, a nonstandard design pursuant to HDM Topic 606 will need to be prepared and a nonstandard special provision written and approved.

Asphalt stabilization is not commonly used in California since it is usually less effective than lime or cement stabilization on soils with high fines contents (i.e., more than 12 percent passing the #200 [0.075 mm] sieve) and higher plasticity indices (i.e., plasticity index higher than six), which are common throughout the state. However, asphalt stabilization can be used effectively on coarser-grained sandy soils, which do occur in some areas of the state. It can also be compared with cementitious stabilizers on soils with fines contents up to 20 percent and/or plasticity indices up to 10 to determine which stabilization method achieves the best results at lowest cost. Asphalt can also be considered for stabilizing soils with high sulfate contents if unsatisfactory results are obtained with cementitious stabilizers.

Asphalt emulsion is the most commonly used type of asphalt for subgrade stabilization. Foamed asphalt is rarely used in this application due to complexities in mix-design and construction, which render them less cost-effective. The use of cutback or liquid asphalt binder (asphalt mixed with petroleum solvent) is limited by air quality restrictions in California and therefore has limited application for subgrade stabilization. They are not discussed in this chapter.

6.1.1 Determining the Appropriateness of Asphalt Stabilization

Asphalt stabilization is generally limited to soils with low fines contents (< 20 percent passing #200 [0.075 mm] sieve), low plasticity (PI < 10, preferably ≤ 6), and relatively low moisture contents (well below optimum compaction moisture content). If these requirements are not met then consider cementitious stabilization (Chapter 5).

Factors to consider when deciding on the use of asphalt stabilization include the following:

- The formulation and grade of the asphalt emulsion will depend on soil characteristics and weather conditions.
- Cationic emulsions are usually more compatible with California soils than anionic emulsions; however, the supplier should make the recommendation on which to use.

- In terms of soil properties, medium-setting emulsions are generally used on soils with low fines contents and low plasticity, and on poorly graded soils if the gradation is coarse. Slow-setting emulsions are used on soils with higher fines contents and plasticity.
- In terms of weather conditions, medium-setting emulsions are used at colder temperatures, while slow-setting emulsions are used in warmer temperatures to provide sufficient time for mixing and compaction.
- Guidance on selecting the correct emulsion should be obtained from emulsion suppliers.

6.2 Problems Related to Moisture Content and Plasticity

Asphalt stabilization is not appropriate for correcting problems related to moisture content and plasticity. Cementitious additives or mechanical modification should be used in these applications.

6.3 Problems Related to Low Strength

6.3.1 Site Investigation

No additional site investigation information to that discussed in Chapter 3 is required for assessing soil strength improvement. However, additional representative soil samples as well as any other cementitious additives that may be used in conjunction with the emulsion must be collected and sent to the asphalt emulsion supplier to check the compatibility of the emulsion with the soil in terms of soil chemistry. If asphalt emulsion is being considered, soil moisture contents will also need to be closely monitored to determine whether the layer can still be effectively compacted after mixing in the emulsion, given that fluid content may be increased to a point well above the optimum compaction fluid content.

6.3.2 Testing, Analysis, and Reporting

Laboratory testing to assess strength improvement using asphalt stabilization requires a number of tests, which are listed in Table 6.1.

Table 6.1: Tests for Assessing Asphalt Stabilization to Increase Strength

Parameter	Test Method	Reason
ITS Durability	CT 371/ASTM D6931 ASTM D559+D560	Check whether required strength can be obtained. Check wet/dry and freeze/thaw durability.
UCS CBR	CT 221 ASTM D1883 or AASHTO T 193	Alternative strength tests for screening emulsion contents.

Use the following procedure to determine the optimum stabilizer content for strength improvement:

1. If there is a desire to use asphalt emulsion, but the plasticity criteria are not met, consider first modifying the material with lime or cement to correct the plasticity and then stabilizing with asphalt to improve the strength. If in doubt about the potential effectiveness of asphalt stabilization

or if alternatives are being investigated, compare the performance of both cementitious and asphalt stabilization and base the decision on which method provides the more appropriate performance and on cost. Satisfactory performance may be obtained from more than one type of stabilizer, in which case selection should be based on experience and/or cost. **Note that successful stabilization with asphalt will depend on the compatibility of the emulsion with any other additives used (i.e., cement or lime) and with the soil chemistry, which varies widely across California. Experience with a particular emulsion and/or emulsion application rate in one area does not necessarily mean that the same success will be obtained in another, and consequently emulsion formulation, choice of additional additives, and application rate need to be assessed for each project.**

2. Determine the starting point application rate using Table 6.2. Note that residual asphalt contents higher than 4.5 percent may not be economical and that asphalt emulsion contents higher than 7.7 percent may lead to very high soil fluid contents, causing workability and compaction problems.

Table 6.2: Start Point for Determining Asphalt Emulsion Contents

Percent Passing #200	Percent Asphalt Emulsion by Mass of Dry Aggregate at Percent Passing #10 (2.0 mm) Sieve (E = Emulsion, R = Residual Asphalt Content for 60:40 asphalt: water ratio)											
	≤ 50		51 - 60		61 - 70		71 - 80		81 - 90		91 - 100	
	E	R	E	R	E	R	E	R	E	R	E	R
2	6.3	3.8	6.5	3.9	6.7	4.0	7.0	4.2	7.2	4.3	7.5	4.5
4	6.5	3.9	6.7	4.0	7.0	4.2	7.2	4.3	7.5	4.5	7.7[1]	4.6[1]
6	6.7	4.0	7.0	4.2	7.2	4.3	7.5	4.5	7.7[1]	4.6[1]	7.9[1]	4.7[1]
8	7.0	4.2	7.2	4.3	7.5	4.5	7.7[1]	4.6[1]	7.9[1]	4.7[1]	8.2[1]	4.9[1]
10	7.2	4.3	7.5	4.5	7.7[1]	4.6[1]	7.9[1]	4.7[1]	8.2[1]	4.9[1]	8.4[1]	5.0[1]
12	7.5	4.5	7.7[1]	4.6[1]	7.9[1]	4.7[1]	8.2[1]	4.9[1]	8.4[1]	5.0[1]	8.6[1]	5.2[1]
14	7.2	4.3	7.5	4.5	7.7[1]	4.6[1]	7.9[1]	4.7[1]	8.2[1]	4.9[1]	8.6[1]	5.0[1]
16	7.0	4.2	7.2	4.3	7.5	4.5	7.7[1]	4.6[1]	7.9[1]	4.7[1]	8.2[1]	4.9[1]
18	6.7	4.0	7.0	4.2	7.2	4.3	7.5	4.5	7.7[1]	4.6[1]	7.9[1]	4.7[1]
20	6.5	3.9	6.7	4.0	7.0	4.2	7.2	4.3	7.5	4.5	7.7[1]	4.6[1]

[1] Residual asphalt contents higher than 4.5% may not be economical. Asphalt emulsion contents higher than 7.7% may lead to very high soil fluid contents, causing workability and compaction problems.

3. Determine the indirect tensile strength (ITS) using CT 371 (and/or UCS or CBR tests) using the selected emulsion content, and the selected content plus-and-minus 0.5 percent. Substitute water with asphalt emulsion during specimen preparation, adding water if required to meet the required compaction moisture content. Cure the specimens in a forced draft oven for 72 hours at 104°F (40°C) before testing. Prepare replicate specimens with 1.0 percent lime and/or cement to check if this enhances the breaking and curing of the asphalt emulsion, and counteracts problems associated with high soil-fluid contents. If the material is poorly graded (i.e., likely to have a high void content after compaction, typical of coarse, sandy subgrades), prepare replicate specimens with 1.0, 1.5, and 2.0 percent lime and/or cement to check if this improves the strength to a satisfactory level. The cement or lime content must not be higher than half the residual asphalt content.
4. Compare the results with the project requirements to see if satisfactory improvement has been achieved. Strive for achieving the target and not necessarily "maximum" strength as this could be a waste of stabilizer and may lead to problems related to soil workability and high soil fluid content.

5. If the required result has not been achieved, repeat the test with lower or higher asphalt emulsion contents and/or with an alternative stabilizer or stabilizer combination.
6. Repeat the above testing cycle if required to compare with cementitious stabilizers or stabilizer combinations (e.g., asphalt emulsion and lime or asphalt emulsion and cement).

Reporting

Summarize the recommended soil stabilization approach for improving strength on the stabilization strategy form (example Form 5 in Appendix B).

6.4 Cost Assessment Considerations

The following factors need to be considered for completing a cost assessment to compare asphalt stabilization with other stabilization strategies:

- The costs of the asphalt plus additives (i.e., lime or cement) needed to achieve the required indirect tensile strength (or unconfined compressive strength/California Bearing Ratio). Include the costs of transporting and storing the stabilizer and additives.
- The costs of delays if a dry-back period is required before compaction can commence because the soil fluid content is significantly higher than the optimum compaction moisture content after the addition of asphalt emulsion.
- The savings in moving material from one part of a project to another or the costs of the purchase and transport of material from another source. The potential savings from being able to use materials from elsewhere on a project instead of transporting them off site and disposing of them should also be factored into the analysis.
- The unit costs of one stabilization method over another.

6.5 Pavement Design and Project Documentation

Design the pavement according to the *Highway Design Manual* using the laboratory-determined strength values after stabilization. Detail the emulsion type and application rate, along with any specific construction requirements in the project documentation. Note that subgrade soil stabilization with asphalt emulsion is not covered in the *Standard Specifications* and project specific specifications will need to be prepared.

7. CONSTRUCTION CONSIDERATIONS

7.1 Introduction

Standard subgrade stabilization construction procedures are not discussed in this guide. Relevant Caltrans manuals should be followed, and standard and project specifications should be enforced at all times. However, the discussion below addresses key issues that might not be routinely considered in normal project specification requirements, but which are important to optimal performance and the extended life of pavement structures. These issues should be addressed in the just-in-time training or preconstruction meetings required in the *Standard Specifications*, and the way in which they are dealt with by the contractor should be assessed during construction of the test strip, which is also a requirement in the *Standard Specifications*.

Standard worksite safety procedures should be followed at all times. Also follow all safety standards specified for the stabilizer being used. Distribute copies of the stabilizer material safety data sheet (MSDS) to all workers.

7.2 Grade Correction

Correct the grade to the approximate final levels prior to beginning stabilization. Corrections after stabilization will be difficult and may affect the thickness of the layer.

7.3 Test Strip

Complete a test strip to ensure that the required results can be achieved with the stabilization strategy and available equipment according to the project specifications. Consider the following in conjunction with the *Standard Specifications*:

- Use the test strip to determine optimal lot sizes, mixing times and rates, compaction moisture contents, compactor speeds, rolling patterns, and approximate number of passes required to achieve the required (or refusal) density. Formulae for calculating stabilizer quantities are provided in Appendix F.
- Check the in situ moisture content, compare this to the content used in the design, and adjust compaction moisture contents accordingly. On lime stabilization projects, slaking water needs to be considered (typically three percent above optimum moisture content), and on cement stabilization projects, hydration water needs to considered. On asphalt emulsion projects, soil moisture and the water in the emulsion need to be considered when determining if additional water is required. On sulfate-rich soils, a mellowing period is required and compaction is usually carried out at above optimum moisture content. Use the test strip to refine these moisture contents.

An example checklist for test strips is provided in Appendix B, Form 6.

7.4 Mixing and Application Equipment

Depending on the method of stabilization used, best results will be achieved with the following equipment:

- Mechanical stabilization: A rotary mixer/recycler in combination with a water tanker to mix in-situ soils and to blend materials.
- Cementitious stabilization: A rotary mixer/recycler in combination with a water tanker (Figure 7.1a) if the stabilizer is spread onto the road in dry form. If the stabilizer is added in slurry form, a rotary mixer/recycler in combination with the slurry tanker, and water tanker if required (i.e., the slurry and additional compaction water, if required, are sprayed onto the soil in the mixing chamber of the rotary mixer/recycler) (Figure 7.1b). Note that hydration reactions will begin as soon as water and cement are mixed, and mixing and compaction times may need to be adjusted accordingly. Also note that adding the stabilizer as a slurry may increase the fluid content of the soil beyond optimum levels. Slurry applications should not be used if soil moisture contents are already high.

(a) Rotary mixer/recycler coupled to water tanker

(b) Rotary mixer/recycler coupled to slurry applicator

(c) Rotary mixer/recycler coupled to emulsion tanker and water tanker

Figure 7.1: Recommended mixing equipment.

- Asphalt stabilization: A rotary mixer/recycler in combination with the asphalt emulsion tanker, and water tanker if required (i.e., the asphalt stabilizer and compaction water, if required, is sprayed onto the soil in the mixing chamber of the rotary mixer/recycler) (Figure 7.1c).
- On projects where material blending is the selected strategy, spread the imported material to the thickness required to achieve the desired blend.
- If a combination of lime and cement or lime and asphalt emulsion are used to correct plasticity and improve strength, complete the lime modification before starting with the cement or asphalt emulsion stabilization (i.e., they should be completed as two separate activities).
- Where feasible, consider application of powdered stabilizers (e.g., lime, cement, fly ash) as a slurry instead of as a dry powder to prevent loss by wind (Figure 7.2) and to limit uneven distribution (Figure 7.3), which could lead to areas of over- and understabilization and consequent weak spots or spots with excessive shrinkage. Do not rely on mixing to achieve uniform distribution through the soil if stabilizer spreading is inconsistent. Note that difficulties in achieving uniform distribution increase with increasing stabilizer content.
- If powdered stabilizers are used, ensure that the stabilizer is as evenly distributed as possible before mixing. Level overlap windrows and other areas of excess or underapplication prior to mixing.

Figure 7.2: Cement loss by wind.

Figure 7.3: Uneven distribution of stabilizer on road surface.

7.5 Mixing Train Crew Responsibilities

Check that the mixing train crew is appropriately trained. Crew duties include but are not limited to:

Pipeline Marker

- Checking for marked and unmarked services, power lines, culverts, overhanging branches, etc., that may influence the recycling train.
- Checking couplings between the tankers and recycler.
- Checking that stabilizer and water contents are correct at all times. Monitor dry-powder application rates as required in the *Standard Specifications* (Figure 7.4). Correct any problems immediately.

Figure 7.4: Monitoring stabilizer application rate.
(Place pan in middle of applicator path [a], apply the stabilizer [b], weigh the stabilizer, [c], and then check that correct unit weight has been applied.)

- Checking that powder stabilizers have been evenly spread at the required rate for a sufficient distance ahead of the mixing train (i.e., the train will never need to stop while powder stabilizer is being spread). Checks should also ensure that spread stabilizer is not being blown away by wind or passing traffic, or accumulating in windrows during mixing.
- Ensuring that an **experienced** technician is always present around the mixer to check:

+ Mixing depth on both sides of the mixer (Figure 7.5).
+ That a uniform blend is being achieved by digging holes across the roadway and observing the mixed materials (Figure 7.6), which should have a uniform consistency, color, and moisture content. On cement- and lime-treated materials, spray the sides of the hole with a phenolphthalein solution and check that the color of the soil changes to a uniform deep red (Figure 7.7). If it does not, there is insufficient stabilizer in the material. On projects requiring material blending, take samples periodically for grading tests to confirm the observations.
+ For oversize or organic material and other issues that may influence the quality of the finished pavement.
+ For leaks and spills while the mixing train is standing to ensure that no soft spots will be left in the pavement that may deform at a later date.
+ That appropriate safety procedures are followed during mixing, especially during the change-over of tankers, and working between the mixer, tankers, and rollers.

Safety
Phenolphthalein is a strong laxative and a suspected carcinogen and care should be taken when working with it.

Figure 7.5: Checking mixing depth.

Figure 7.6: Checking material consistency and color.

NOTE: Crew working on and around the mixing train should be cautious at all times. The monitoring of mixing depth and mixed material behind the mixer are particularly dangerous exercises.

Visual inspection

Inspection with phenolphthalein (note red color indicating high pH)

Figure 7.7: Monitoring stabilizer mixing.

7.6 Mixing Depth and Moisture Content

Consider the following with regard to mixing depth and moisture content:

- Mix the material to the specified depth and moisture content (determined from laboratory tests—note that the specified moisture contents may be above or below optimum depending on the stabilizer and the soil type) using appropriate equipment. Note also that mixing to a depth greater than the design will equate to a lower stabilizer application rate and potentially lower than required plasticity reduction and/or strength.
- On asphalt emulsion stabilization projects, monitor the in-situ moisture content continuously to ensure that excess water is not added. Note that total fluid content (i.e., in-situ moisture, emulsion, and additional compaction water) needs to be considered in moisture calculations.

7.7 Mellowing (Lime Stabilization)

If a mellowing period is required (e.g., for stabilizing sulfate rich soils):

- Lightly compact the layer prior to any mellowing to seal the surface and prevent oxidation and/or carbonation of the lime.
- Check that correct moisture contents are maintained throughout the mellowing period.
- Remix the material after the mellowing period is complete.
- When working with high plasticity clays, a 24- to 48-hour mellowing period between initial and subsequent mixing will improve the workability of the mix and facilitate even distribution of moisture.

7.8 Compaction

Consider the following with regard to compaction:

- On cementitious stabilization projects, start compaction as soon as possible after final mixing, and complete compaction in the time specified in the *Standard Specifications*. Typical compaction start times for various stabilizer types are listed in Table 7.1.

Table 7.1: Start Times for Compaction of Stabilized Layers

Stabilizer Type	Max. Time Before Starting Compaction (hours)
Lime modification	24 – 48
Lime stabilization	6 – 8
Cement	0.5
Cement kiln dust	1
Class-C fly ash	1
Class-F fly ash with lime	2
Class-F fly ash with cement	1
Asphalt emulsion	Complete compaction before emulsion "break"[1]

[1] Emulsion "break" is usually identified by color change as water evaporates and asphalt particles coalesce.

- On asphalt emulsion stabilization projects, wait until the fluid content in the soil has dried back to the optimum compaction moisture content before compacting. Remix the material after the dry-back period is complete prior to compaction. Do not add additional lime or cement (i.e., more cement than specified in the mix design) to dry out the material as this will change the design properties of the material.
- Compact the material to the specified density, or to a refusal density as discussed in Section 4.2.4. Compact at the speeds and using the rolling pattern determined during construction of the test strip.
- Consider the following if a rotary mixer/recycling train is used for application and/or mixing:
  + Initial compaction on subgrade soils prepared with a rotary mixer/recycler is different than that on conventional aggregate bases. Although the material exiting the rotary mixer/recycler is in a loose state, the rear wheels of the mixer and all the wheels of any following tankers run on this material, resulting in compaction of the loose material in the equipment wheelpaths (Figure 7.8). The in-place density of this compacted material is at least 10 percent higher than that of the adjacent uncompacted material. It is therefore imperative that the material between the wheelpaths first be compacted to at least the same density as that in the wheelpaths before any additional processing is initiated. **Grading prior to initial compaction or failure to compact this material in an appropriate sequence will result in a permanent density differential, which could lead to premature failure (rutting and/or longitudinal cracks) in the wheelpaths.**
  + Do not let the roller speed exceed 2.0 mph (3.0 km/h) or 165 ft/min (50 m/min).
  + The first pass of the roller (note that padfoot rollers are usually the most appropriate for deep mixes) should proceed down the center of the rotary mixer/recycling train wheelpaths, ensuring that the drum of the roller bridges the wheelpaths (Figure 7.9). At the end of the run, the blade should be lowered and the roller reversed back down the same path while dragging material into the padfoot impressions and wheelpaths of the rotary mixer/recycling train. The material between the wheelpaths should now be level with and of a similar density to that in the wheelpaths. A conventional rolling pattern can then be followed to obtain uniform compaction of the mixed material. The blade should be used on each reverse pass to fill in the padfoot impressions (Figure 7.10).

Figure 7.8: Differential compaction in rotary mixer/recycling train wheelpaths.

Figure 7.9: First padfoot roller pass.

Figure 7.10: Using the blade to redistribute material in padfoot impressions.

+ As more roller passes are applied, the increasing compactive effort results in increased density in the lower regions of the mixed layer, which increases the resistance to penetration of the individual pads on the roller, allowing the roller to "walk out" of the material. Only minor indentations, which can be skimmed by the grader during leveling, should be left on the surface after completion of initial compaction with the padfoot roller (Figure 7.11 and Figure 7.12).

Figure 7.11: First padfoot roller pass impressions.

Figure 7.12: Padfoot impressions after multiple passes.

- The use of low amplitude vibration or use of a roller with insufficient static mass during initial compaction will result in densification of the material in the upper horizon of the layer, forming a bridge of compacted material over a layer of relatively loose material. This loose material will eventually densify under traffic, resulting in settlement and consequent rutting of the pavement.
- Rollers must be correctly selected during test strip construction and quality control of density must be rigorously carried out to ensure that the specified density is obtained throughout the full depth of the stabilized layer.

- Final grading and compaction should follow initial compaction as quickly as possible.
- On cementitious stabilized materials, consider precracking the material after final compaction to limit shrinkage cracking (see Section 7.13).

7.9 Compaction Quality Control

Compaction must comply with *Standard Specifications*.

7.10 Curing

Cure the material as specified in the *Standard Specifications*. If water alone is used for curing cementitious stabilized materials, consider an irrigation system to maintain the treated material at constant moisture content. Spraying water with trucks invariably results in cyclical wetting and drying, which can lead to durability issues and varying strength along the alignment. Where feasible on cementitious stabilized materials, spray asphalt emulsion at a rate of 0.15 gal/yd^2 to 0.3 gal/yd^2 (0.7 L/m^2 to 1.4 L/m^2) within 24 hours after final compaction as a curing membrane, as an alternative to continuous water spraying. See discussion on postconstruction durability issues in Section 5.11 and Sections 7.14 and 7.15.

7.11 Trafficking

Keep all vehicles off the stabilized layer during the specified curing period and/or until the minimum specified strength for the project has been achieved.

7.12 Construction Considerations for Stabilizing Sulfate-Rich Soils

Incorporate the following modified construction practices when stabilizing sulfate-rich subgrade soils:

- Apply the stabilizer as a slurry in a single application to facilitate even distribution and mixing, and to prevent the formation of stabilizer pockets.
- Add sufficient water so that the compaction moisture content is the same as the laboratory-determined “best” optimum moisture content. Take the in situ moisture content and the slurry moisture content into consideration, but remember that higher than optimum moisture contents are required.
- Mix the material thoroughly to ensure that the stabilizer and moisture are uniformly distributed throughout the layer and to facilitate consumption of the sulfates in the soil.

- Lightly compact the layer after mixing to seal the surface and reduce oxidation and carbonation of the lime. This typically involves one or two roller passes. This is not final compaction.
- Extend the mellowing time based on the laboratory test results from the mix design. The extended mellowing time will promote ettringite formation prior to compaction. Use engineering judgment to adjust this mellowing time since field conditions will vary from laboratory conditions. Spray water at regular intervals to maintain the moisture content at the required level.
- Remix the material thoroughly after the mellowing period is complete.
- Compact the material to the specified density. The material may need to be dried back to achieve optimal density after the mellowing period has elapsed, but the layer should be compacted at as high a moisture content as possible to achieve the target density.
- If a combination of stabilizers is used, complete the pretreatment (usually with lime) described above first before mixing the second stabilizer.

7.13 Shrinkage Cracking Mitigation in Cementitious Stabilized Layers

Shrinkage cracking in stabilized subgrade and subbase soils is not as significant a problem as it is in base material, since cracks are unlikely to reflect through the overlying untreated base and into the surfacing layers. However, it is still important to minimize it to ensure that the treated layer acts as a uniform stable platform for the pavement structure. Limit shrinkage cracking using one or more of the following:

- Compact the layer at the lowest possible moisture content that achieves the required density and strength. Note, however, that certain situations require mixing and mellowing at above optimum moisture contents (e.g., lime-stabilized soils, all sulfate-rich soils) and consequently may require compaction at higher than optimal moisture contents (typically between one or three percent above optimum). Formulae for calculating saturation moisture contents are provided in Appendix F.
- Cure the layer correctly. If asphalt emulsion is used as a curing membrane, ensure that any damage or thin spots are repaired/resprayed immediately.
- Consider precracking the material 24 hours after final compaction with two to four passes of a vibrating smooth drum roller (frequency of 1,000 to 2,000 Hz and speed of 2 to 3 mph [3 to 5 km/h]). A slight reduction in stiffness may result in the short term; however, this usually has less impact on pavement performance than a layer with shrinkage cracks.

7.14 Construction Control to Limit Carbonation in Cementitious Stabilized Soils

The following construction procedures must be carefully controlled to prevent or at least minimize the carbonation of susceptible materials:

- Compact the stabilized layer to as high a density as possible (without disturbing the grading excessively) as soon as possible after mixing with the stabilizer, to expedite cementation and to prevent carbonation reactions. This action helps by sealing the layer and reducing the voids (not applicable to lime modification).
- If necessary, compact very thick stabilized layers (e.g., >12 in. [300 mm]) in multiple lifts to ensure that good compaction is obtained throughout the layer. Less than five percent air voids is desirable,

which may require additional compaction effort. Layers that are too thin (e.g., <2 in. [50 mm]) may result in "biscuits" and the related consequential problems.

- Where modification is required and the material is to be worked in two events, apply slightly higher stabilizer contents to compensate for carbonation. Take care to seal the surface after the initial application of stabilizer.
- Curing should be strictly controlled.
- Avoid unnecessary delays in the placing of the next layer on the stabilized subgrade.
- The stabilizer must be as fresh as possible and must not be stored for long periods before use.
- Where difficulties in achieving a well-compacted and cured layer are anticipated, increase the thickness of the layer by about 1.0 in. (25 mm) to allow skimming or brooming of the weak/loose upper layer prior to placing the next layer. The success of this will depend to a large degree on the grading of the material in the layer.

7.15 Repair of Carbonated Layers

Repair methods will depend on whether carbonation has occurred from the top, bottom, or on the sides of the layer.

- **Carbonation from the Top of the Treated Layer**
  Carbonation from above is usually caused by ineffective curing techniques or exposure of stabilized layers to the atmosphere for excessively long periods before being sealed or covered. If the carbonation has penetrated less than about 0.4 in. (10 mm) into the layer, sweep the loose material off with a mechanical broom or skim it with a grader before the next layer is applied. Take care not to disturb or weaken the remainder of the layer. Check the pavement design to ensure that the structural capacity of the pavement will not be affected by the reduced layer thickness.
- **Carbonation from the Bottom of the Treated Layer**
  Carbonation from the bottom of the treated layer usually starts immediately after construction as a result of very high carbon dioxide levels in soils (often about 13 percent or 400 times that of the atmosphere). It is difficult to predict how long it will take for the full layer thickness to carbonate in the absence of durability test results. However, it is likely to be at least several years. Where full layers have carbonated and the pavement has failed, it is recommended that the road or parts thereof be reconstructed.
- **Carbonation from the Sides of the Road or through Cracks**
  Carbonation from the sides usually takes the form of a horizontal wedge penetrating towards the center of the road. A sign of this is increased rutting in the outer wheelpath relative to the inner wheelpath. This effect can be limited by surfacing the road shoulders. Carbonation can also affect the material adjacent to any cracks in the road, especially the typical wide block cracks resulting from stabilization. It is thus important to seal cracks as soon as they become visible so that the penetration of carbon dioxide can be minimized.

8. REFERENCES AND FURTHER READING

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8.4 Geogrid/Geotextile Stabilization

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APPENDIX A: RESTRICTIONS ON CUTBACK ASPHALT

Air Pollution Control/Air Quality Management Districts with restrictions on cutback asphalt are listed in Table A.1. Further details on restrictions can be obtained from the California Air Resources Board (*www.arb.ca.gov/ei/areasrc/ccosmeth/att_c_asphalt.doc*).

Air Quality Districts	Restriction on cutback asphalt?	Rule No.
Amador County Air Pollution Control District	No	-
Antelope Valley Air Pollution Control District	Yes	1108
Bay Area Air Quality Management District	Yes	15-Aug
Butte County Air Quality Management District	Yes	241
Calaveras County Air Pollution Control District	No	-
Colusa County Air Pollution Control District	Yes	2.33
El Dorado County Air Pollution Control District	Yes	224
Feather River Air Quality Management District	No	-
Glenn County Air Pollution Control District	Yes	99.1
Great Basin Unified Air Pollution Control District	No	-
Imperial County Air Pollution Control District	Yes	426
Kern County Air Pollution Control District	Yes	410.5
Lake County Air Quality Management District	No	-
Lassen County Air Pollution Control District	No	-
Mariposa County Air Pollution Control District	No	-
Mendocino County Air Quality Management District	No	-
Modoc County Air Pollution Control District	No	-
Mojave Desert Air Quality Management District	Yes	1103
Monterey Bay Unified Air Pollution Control District	Yes	425
North Coast Unified Air Quality Management District	No	-
Northern Sierra Air Quality Management District	No	-
Northern Sonoma County Air Pollution Control District	No	-
Placer County Air Pollution Control District	Yes	217
Sacramento Metropolitan Air Quality Management District	Yes	453
San Diego County Air Pollution Control District	Yes	67.7
San Joaquin Valley Unified Air Pollution Control District	Yes	4641
San Luis Obispo County Air Pollution Control District	Yes	420
Santa Barbara County Air Pollution Control District	Yes	329
Shasta County Air Quality Management District	Yes	3.15
Siskiyou County Air Pollution Control District	No	-
South Coast Air Quality Management District	No	1108
Tehama County Air Pollution Control District	Yes	4.26
Tuolumne County Air Pollution Control District	No	-
Ventura County Air Pollution Control District	Yes	74.4
Yolo-Solano Air Quality Management District	Yes	2.28

APPENDIX B: FORMS AND CHECKLISTS

The following example forms and checklists are provided in this appendix:

1. Subgrade Stabilization Project Investigation: DCP Assessment
2. Subgrade Stabilization Project Investigation: Sample Record
3. Subgrade Stabilization Project Investigation: Preliminary Laboratory Test Results
4. Subgrade Stabilization Project Investigation: Stabilization Strategy Selection
5. Subgrade Stabilization Project Investigation: Stabilization Strategy Detail
6. Example Checklist for Construction of Test Strips

1	SG Stabilization Investigation: DCP Assessment							
Project Number and Description								
District			County			Date		
Road No.			Post Mile	to		Assessor		
Position			Position			Position		
0			0			0		
5	205	405	5	205	405	5	205	405
10	210	410	10	210	410	10	210	410
15	215	415	15	215	415	15	215	415
20	220	420	20	220	420	20	220	420
25	225	425	25	225	425	25	225	425
30	230	430	30	230	430	30	230	430
35	235	435	35	235	435	35	235	435
40	240	440	40	240	440	40	240	440
45	245	445	45	245	445	45	245	445
50	250	450	50	250	450	50	250	450
55	255	455	55	255	455	55	255	455
60	260	460	60	260	460	60	260	460
65	265	465	65	265	465	65	265	465
70	270	470	70	270	470	70	270	470
75	275	475	75	275	475	75	275	475
80	280	480	80	280	480	80	280	480
85	285	485	85	285	485	85	285	485
90	290	490	90	290	490	90	290	490
95	295	495	95	295	495	95	295	495
100	300	500	100	300	500	100	300	500
105	305	505	105	305	505	105	305	505
110	310	510	110	310	510	110	310	510
115	315	515	115	315	515	115	315	515
120	320	520	120	320	520	120	320	520
125	325	525	125	325	525	125	325	525
130	330	530	130	330	530	130	330	530
135	335	535	135	335	535	135	335	535
140	340	540	140	340	540	140	340	540
145	345	545	145	345	545	145	345	545
150	350	550	150	350	550	150	350	550
155	355	555	155	355	555	155	355	555
160	360	560	160	360	560	160	360	560
165	365	565	165	365	565	165	365	565
170	370	570	170	370	570	170	370	570
175	375	575	175	375	575	175	375	575
180	380	580	180	380	580	180	380	580
185	385	585	185	385	585	185	385	585
190	390	590	190	390	590	190	390	590
195	395	595	195	395	595	195	395	595
200	400	600	200	400	600	200	400	600

2	SG Stabilization Investigation: Sample Record

Project Number and Description:					
District:		**County:**		**Date:**	
Road No.:		**Post Mile:**	to	**Assessor:**	
Sample #					
Sample Location					
Sample Depth		**Sample Size**			
Sample Description					

Tests	Grading		Atterberg		R-value		Sulfate		Organic		pH	
	Moisture		Density		ICS		UCS		ITS		CBR	

Notes

Sample #					
Sample Location					
Sample Depth		**Sample Size**			
Sample Description					

Tests	Grading		Atterberg		R-value		Sulfate		Organic		pH	
	Moisture		Density		ICS		UCS		ITS		CBR	

Sample #					
Sample Location					
Sample Depth		**Sample Size**			
Sample Description					

Tests	Grading		Atterberg		R-value		Sulfate		Organic		pH	
	Moisture		Density		ICS		UCS		ITS		CBR	

Notes

3 SG Stabilization Investigation: Test Results

Project Number and Description:					
District:		**County:**		**Date:**	
Road No.:		**Post Mile:**	to	**Assessor:**	
Sample #					
Grading	% Passing #4		% Passing #200		
Atterberg Limits	Liquid Limit		Plasticity Index		
Classification					
R-value		**Moisture Content**		**Sulfate Content**	
Organic Content		**pH**		**CBR**	
Notes					

Sample #					
Grading	% Passing #4		% Passing #200		
Atterberg Limits	Liquid Limit		Plasticity Index		
Classification					
R-value		**Moisture Content**		**Sulfate Content**	
Organic Content		**pH**		**CBR**	
Notes					

Sample #					
Grading	% Passing #4		% Passing #200		
Atterberg Limits	Liquid Limit		Plasticity Index		
Classification					
R-value		**Moisture Content**		**Sulfate Content**	
Organic Content		**pH**		**CBR**	
Notes					

4 SG Stabilization Investigation: Strategy Selection

Project Number and Description:					
District:		**County:**		**Date:**	
Road No.:		**Post Mile:**	to	**Assessor:**	

1.	Is sulfate content > 3,000 ppm?	Yes	No	
	Follow special procedures for stabilizing sulfate-rich soils			*Go to 2*
2.	Is problem related to high moisture content only?	Yes	No	
	Consider cementitious modification to dry out soil			*Go to 3*
3.	Is PI < 12 and R-value >40?	Yes	No	
	Follow standard subgrade preparation procedures			*Go to 4*
4.	Are results within 10% of requirement (GW, GM, SW, SM)?	Yes	No	
	Consider mechanical, cementitious, or asphalt modification. Do LCCA			*Go to 5*
5.	Is problem primarily related to gradation (GP, GC, SP)?	Yes	No	
	Consider mechanical stabilization (blending)			*Go to 6*
6.	Is problem primarily related to low-strength coarse-grained soils (G-, S-)?	Yes	No	
	Consider asphalt, cementitious, or mechanical stabilization. Do LCCA			*Go to 7*
7.	Is problem primarily related to high plasticity soils (CL, MH, CH, OH)?	Yes	No	
	Consider cementitious modification or stabilization (typically lime)			*Go to 8*
8.	Is problem primarily related to low-strength fine-grained soils (M-, C-, O-)?	Yes	No	
	Consider cementitious stabilization (typically lime, cement, or combination)			*Go to 9*
9.	Is problem primarily related to very high organic matter content (PT)?	Yes	No	
	Consider removing and replacing soil. Compare with cementitious stabilization			*Go to 10*
10.	Are there any other problems?	Yes	No	
	Seek advice from experienced practitioner			*Follow standard procedures*

First-Level Project Stabilization Strategy	

Notes

5 SG Stabilization Investigation: Strategy

Project Number and Description:					
District:		**County:**		**Date:**	
Road No.:		**Post Mile:**	**to**	**Assessor:**	

1.	Sulfate content > 3,000 ppm		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
2.	PI < 12 and R-value > 40		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
3.	High moisture content only		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
4.	Results within 10% of plasticity or R-value requirement		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
5.	Gradation		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
6.	Low-strength coarse-grained soils		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
7.	High plasticity soils		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
8.	Low-strength fine-grained soils		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
9.	High organic matter content		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						
10.	Other problem		Design requirement		Actual		
			After treatment		OK?	Yes	No
	Strategy						

6 SG Stabilization Test Strip Checklist

Project Number and Description:					
District:		**County:**		**Date:**	
Road No.:		**Post Mile:**	**to**	**Assessor:**	
Test Strip No.:		**Post Mile:**	**to**		

Check item				**Yes**	**No**	**Action required**
Equipment appropriate and calibrated?				Y	N	
Site preparation satisfactory?				Y	N	
In situ moisture content measured?				Y	N	
In situ moisture content						
Compaction moisture content						
Water application rate adjusted for in situ?				Y	N	
Ambient and surface temperatures measured?				Y	N	
Ambient		Surface				
Stabilizer spread rate satisfactory?				Y	N	
Spread rate						
Stabilizer spread uniformly?				Y	N	
Lot size appropriate?				Y	N	
Mix depth and uniformity satisfactory?				Y	N	
Mix start		Mix end				
Mix depth						
Mellowing period satisfactory? (if required)				Y	N	
Mellowing period						
Rolling pattern appropriate?				Y	N	
Compaction speed and time satisfactory?				Y	N	
Start		End				
Number of roller passes						
Satisfactory compaction achieved?				Y	N	
Density		Moisture				
Final grading satisfactory?				Y	N	
Quality control process satisfactory?				Y	N	
Curing process satisfactory?				Y	N	
Precracking?				Y	N	
Vehicle access control?				Y	N	
Specifications adhered to?				Y	N	

APPENDIX C: AASHTO SOIL CLASSIFICATION SYSTEM (M 145)

Properties	Group Classification										
	Granular Materials (35% or less passing the 0.075 mm sieve)							Silt-Clay Materials (>35% passing the 0.075 mm sieve)			
	A-1		A-2				A-3	A-4	A-5	A-6	A-7
	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7					A-7-5 A-7-6
Sieve Analysis, % passing 2.00 mm (#10) 0.425 mm (#40) 0.075 mm (#200)	 ≤ 50 ≤ 30 ≤ 15	 --- ≤ 50 ≤ 25	 --- --- ≤ 35	 --- --- ≤ 35	 --- --- ≤ 35	 --- --- ≤ 35	 --- ≤ 51 ≤ 10	 --- --- ≥ 36	 --- --- ≥ 36	 --- --- ≥ 36	 --- --- ≥ 36
Atterberg Limits (on 0.425 mm [#40]) Liquid limit (LL) Plasticity index (PI)	 --- ≤ 6		 ≤ 40 ≤ 10	 ≥ 41 ≤ 10	 ≤ 40 ≥ 11	 ≥ 41 ≥ 11	 --- N.P.[1]	 ≤ 40 ≤ 10	 ≥ 41 ≤ 10	 ≤ 40 ≥ 11	 ≥ 41 ≥ 11[2]
Usual types of significant constituent materials	Stone fragments, gravel and sand		Silty or clayey gravel and sand				Fine sand	Silty soils		Clayey soils	
General rating as a subgrade	Excellent to good							Fair to poor			

[1] N.P.: Non-plastic
[2] A-7-5: PI ≤ LL – 30. A-7-6: PI > LL – 30

APPENDIX D: MATERIAL BLENDING DESIGN TEMPLATES

D.1: Material Blending

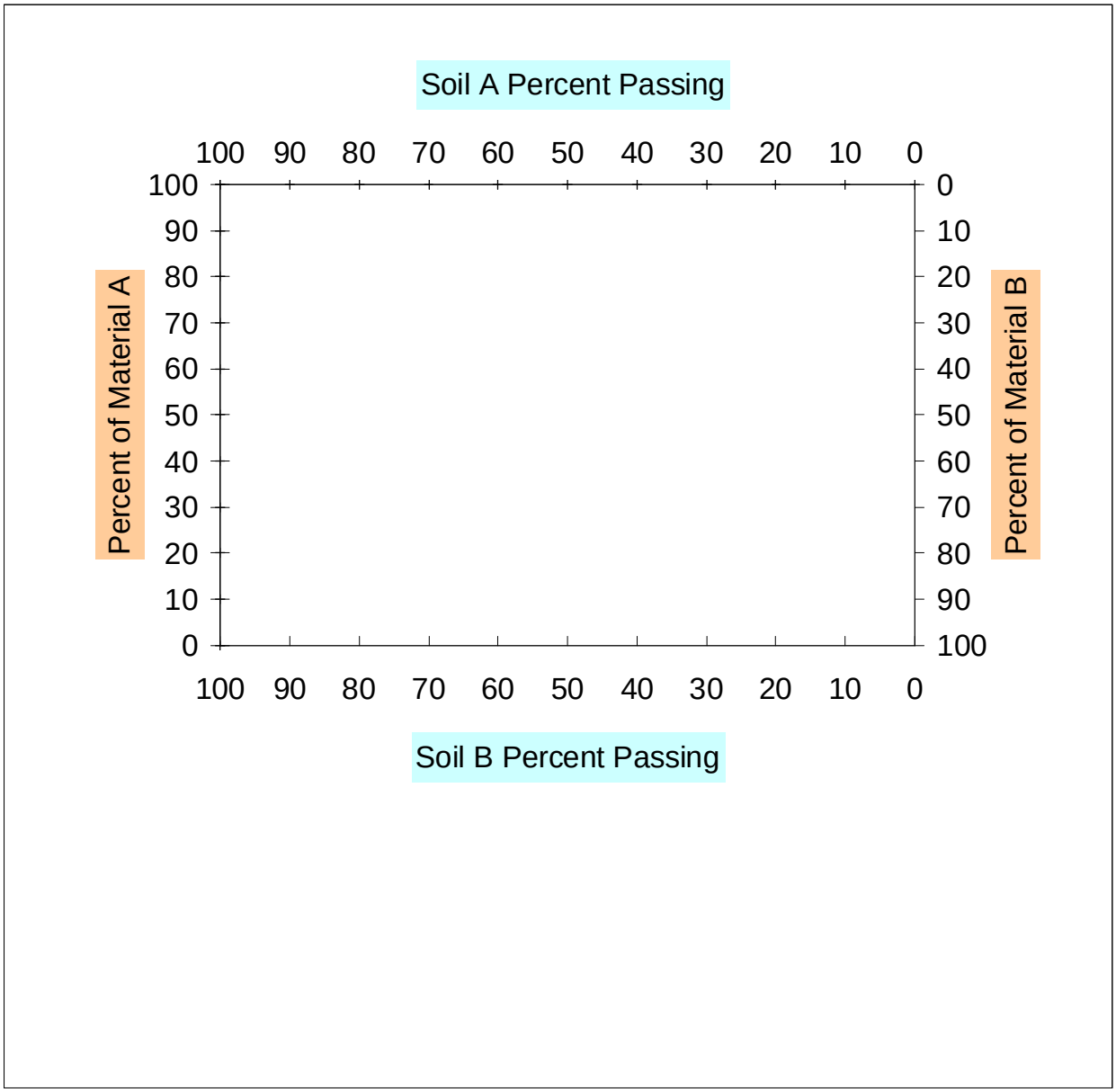

Figure D.1: Template for blending – graphical method.

Table D.1: Template for Arithmetical Method of Soil Blending

Column										
1	2	3	4	5	6	7	8	9	10	11
Sieve Size	Soil A	Soil B	Target Limits		Target Mid Point (TMP)	|TMP - A|	|TMP - B|	% Soil A	%Soil B	Blend
			Low	High						
1”										
3/4”										
3/8”										
#4										
#10										
#40										
#200										
						Σ|TMP - A| =	Σ|TMP - B| =			

APPENDIX E: NEW OR MODIFIED TEST METHODS

E.1 Initial Consumption of Stabilizer Test

E.1.1 Method

Determination of the initial consumption of stabilizer in soil (gravel ICS). (Note that this test is based on ASTM D6276 (Standard test method for using pH to estimate the soil-lime proportion requirement for soil stabilization), which was developed for determining lime content in lime treated soils. However, the method can also be used to provide an indication of appropriate starting cement contents as well. The results must always be verified with appropriate strength tests).

E.1.2 Scope

This method tests the construction material as a whole, and not only the -0.425 mm fraction. The objective of the test is to control the pH of chemically stabilized materials in order to allow the possible formation of cementitious materials and to ensure the stability of the reaction products. During the initial testing, lime would normally be used for the ICS test. Once the type of stabilizer to be used on the project has been decided, this stabilizer will be used for all further testing for that project.

E.1.3 Equipment

1. Balance (accuracy : 0.1 gram)
2. Calibrated pH meter (accuracy: 0.02 units)
3. 5 plastic beakers (150 mL, tall)
4. 200 ml glass beaker
5. Distilled water
6. Calcium hydroxide or cement
7. Water bottle
8. Spatula or pallet knife
9. Soft tissues
10. Sample splitter (25 mm)
11. Jet bottle (for spraying electrode)
12. Drying oven (105°C – 110°C)
13. Thermometer (accuracy 0.5°C)

E.1.4 Method

<u>Sample Preparation</u>

The test is carried out on material finer than 19 mm. Any oversize material is lightly crushed to pass the 19 mm sieve. The samples are dried overnight in the oven at 105°C to 110°C. The samples are then reduced to 200 g quantities using a sample splitter and placed into 150 mL or larger plastic containers.

Testing Procedure

Since most materials require between two and five percent stabilizer, it is advisable to set up six beakers with stabilizer percentages of 1, 2, 3, 4, 5, and 6 percent respectively of the dry soil mass. Weigh the lime to the nearest 0.1 g and add to the soil. Mix the soil and dry lime. Slightly oversaturate the samples with carbon dioxide (CO_2)-free distilled water. The material is considered to be oversaturated when its pores are water-filled and free water can be observed on the surface of the mix. The material shall be judged oversaturated when the pores of the material are water-filled and free water can be observed on the surface of the mix. The surface particles need not be submerged completely. Mix the soil-stabilizer and water until there is no evidence of dry material on the bottom of the beaker. Mix for a minimum time of 30 seconds. Continue mixing for 30 seconds every 10 minutes. After one hour, measure the pH of the paste by inserting the electrode gently into a hole, made in the material with a spatula to a depth of about 20 mm to 30 mm, and gently covering this part of the electrode with the displaced material. Tap the beaker gently to ensure contact between the electrode and material. Record the pH of each sample to the nearest 0.02 of a unit. The lowest percentage stabilizer at which the soil paste remains constant is the saturation stabilizer content of this particular material. The saturation pH of lime at 25°C is usually 12.4.

E.1.5 Recording of Results

Record the results to the nearest 0.5 percent of stabilizer required to produce the maximum pH in the paste.

E.1.6 Preparation of pH Meter

Adhere strictly to the calibration procedures provided by the manufacturer of the pH meter. The temperature-asymmetry and slope adjustment are of particular importance. The efficiency of the electrode must be assured.

E.2 Modified ASTM D1633 Method

Section 614 of the *Highway Design Manual* discusses the determination of gravel factors, used to determine layer thicknesses, using unconfined compressive strength (UCS) test results. The formula provided in the HDM for calculating the gravel factor of lime-treated soils is only applicable to unconfined compressive strengths determined according to CT 373, which does not require soaking of the specimens prior to testing. The formula is **not directly applicable** to unconfined compressive strengths determined according to ASTM D1633 and ASTM D5102, which include soaking periods. However, the formula can be used with results from these tests with modification, as detailed in Section 24 of the *Standard Specifications*. These modifications include:

- Sealing the specimen immediately after compaction to prevent moisture loss.
- Curing the specimen in an oven for seven days at 100±5°F (40°C).
- Air cooling the specimen after curing.
- Testing for unconfined compressive strength without soaking.

The sections of the test method with modifications, developed by HSI Engineering (www.hsi-engineering.com), are provided below.

E.2.1 Section 1: Scope

<u>Modification #1:</u> Two alternative procedures are provided in the test method, namely Method A or Method B. Only use Method A (provided below) for testing stabilized subgrade soils.

- *Method A*. This procedure uses a test specimen 4.0 in. (102 mm) in diameter and 4.58 in. (116 mm) in height. Height to diameter ratio equals 1.15. This test method may be used only on materials with 30 percent or less retained on the 3⁄4-in. (19.0-mm) sieve.

E.2.2 Section 2: Referenced Documents

No modifications to this section.

E.2.3 Section 3: Terminology

No modifications to this section.

E.2.4 Section 4: Significance and Use

No modifications to this section.

E.2.5 Section 5: Apparatus

No modifications to this section.

E.2.6 Section 6: Test Specimens

Modification #2: ASTM D1633 prescribes that specimens are molded in accordance with ASTM Test Methods D559 or D560, which in turn specify compaction requirements detailed in D558. In the modified test method, specimens are molded in accordance with D1557 (Modified Proctor), which uses a compacted energy more representative of current pavement compaction techniques/equipment.

Modification #3: ASTM D1633 prescribes a moist cure in accordance with D1632, which specifies 12 hours or longer in a moist room, followed by a four-hour soak and then testing. The modified method uses an accelerated cure, to yield an estimated 28-day compressive strength at seven days. Testing can also be carried out sooner (e.g., after three days) to give an early indication of the quality of the specimens.

Terminology used in ASTM D1633

6.1 Mold the test specimens as follows:

6.1.1 *Method A*—Specimens are 4.0 in. (101.6 mm) in diameter and 4.584 in. (116.4 mm) in height and are molded in accordance with Test Methods D559 or D560.

6.2 Moist cure the specimens in accordance with Practice D1632.

6.3 At the end of the moist-cure period, immerse the specimens in water for 4 hours.

6.4 Remove the specimens from the water and make compression tests as soon as practicable, keeping specimens moist by a wet burlap or blanket covering. (Note 4—Other conditioning procedures, such as air or oven drying, alternate wetting and drying, or alternate freezing and thawing may be specified after an initial moist curing period. Curing and conditioning procedures shall be given in detail in the report.)

6.5 Check the smoothness of the faces with a straightedge. If necessary, cap the faces to meet the requirements of the section on Capping Specimens in Practice D1632.

Modified Terminology

6.1 Mold the test specimens as follows:

6.1.1 *Method A*—Specimens are 4.0 in. (101.6 mm) in diameter and 4.584 in. (116.4 mm) in height and are molded in accordance with Test Method D1557.

6.2 Remove specimen from mold and seal with two (2) layers of a minimum 4-mil plastic, assuring that plastic is tight around specimens. Place duct-tape at all seams to seal from moisture loss.

6.3 Cure the sealed specimens in an oven at 43.3°C ± 2.7°C for seven days.

6.4 At the end of the seven-day curing period, remove the test specimens from the oven and allow to cool.

6.5 Remove the tape and plastic wrapping.

6.6 Cap both ends of the test specimen with capping plaster. Use lubricated glass plates to form a smooth surface on the caps, allowing the surface to extend slightly outside the width of the test specimen on all sides. If the capping plaster does not adhere firmly to the treated soil, allow a small excess of plaster to flow over the edge of the test specimen to grip the sides and hold the cap in place. The surface of the top and bottom caps should be as nearly as possible at right angles to the axis of the test specimen and parallel to each other.

E.2.7 Section 7: Procedure

No modifications to this section.

E.2.8 Section 8: Procedure

No modifications to this section.

E.2.9 Section 9: Procedure

No modifications to this section.

E.2.10 Section 10: Procedure

No modifications to this section.

E.2.11 Section 11: Procedure

No modifications to this section.

APPENDIX F: FORMULAE

F.1 Formula for Calculating the Quantity of Stabilizer

F.1.1 Spreading of Stabilizers Supplied in Pockets

The following formula (Equation F.1) should be used to calculate the spacing of pockets when spreading stabilizer supplied in pockets:

$$L = \frac{M \times 100{,}000}{T \times W \times P \times D} \qquad \textbf{(F.1)}$$

where:
L = Length in meters between pockets when such pockets are placed in a single row.
M = Mass of the pockets, in kg.
T = Thickness of the compacted layer, in mm.
W = Width in meters of the road to be stabilized.
P = Percentage stabilizer required, by mass of stabilizer to mass of raw soil.
D = Density of the soil after compaction, in kg/m^3.

When the pockets are laid out in more than one row the distance between the pockets can be obtained by multiplying the distance for a single row (L) by the number of rows.

Note: The density of the soil (D) is not the modified density of the soil but the specified minimum density that must be obtained after compaction. Supposing the soils MDD is 2,167 kg/m^3 and the soil must be compacted to 96 per cent, then:

D = 2,167 x 96/100 = 2,080 kg/m^3

<u>Example</u>

A layer 150 mm thick and 8.5 m wide is to be stabilized with 3 percent cement (50 kg pockets). The required density is 2,080 kg/m^3. Therefore the spacing for a single row is:

$$L = \frac{50 \times 100000}{150 \times 8.5 \times 3 \times 2080} = 0.63 \text{ m}$$

and for four rows: 4 x L = 2.52 m

F.1.2 Spreading of Cement in Bulk

<u>Control During Spreading</u>

A quick test can be carried out to determine whether the spreading at a specific point is correct. This test is carried out by placing a metal tray or canvas mat, 1.0 m^2 in the spreading path and measuring the mass of stabilizer deposited. If the distributor spread is equal to the width of the road to be stabilized, Equation F.2

can be used to determine what the mass of stabilizer on the tray/canvas should be. The mass (M) on the canvas in road must be:

$$M = \frac{L \times W \times T \times D \times P}{100000} \qquad \textbf{(F.2)}$$

where:
L = Length of tray/canvas, in m.
W = Width of canvas, in m.
T = Thickness of the compacted layer, in mm.
D = Density of soil after compaction, in kg/m^3.
P = Percentage stabilizer (e.g., for 3 percent, P = 3).

Example

Four percent cement must be mixed into a soil with a density of 2,080 kg/m^3. The compacted layer must be 125 mm thick. The tray is 1.0 m x 1.0 m. How much cement must be deposited on the canvas for the correct application? Mass on canvas, in kg:

$$M = \frac{1 \times 1 \times 125 \times 2{,}080 \times 4}{100{,}000} = 10.4 \text{ kg}$$

Note: Sometimes the spreading width is smaller than the total road width to be stabilized since the stabilizer is spread wider during mixing in. Equation F.2 must then be amended as shown in Equation F.3. Mass on canvas, in kg:

$$M = \frac{L \times H \times T \times D \times P \times R}{100{,}000 \times S} \qquad \textbf{(F.3)}$$

where:
R = Final road width that is to be stabilized, in m.
S = Width of spreader bar application, in m.

In the example above, if the spread width of the stabilizer is 2.5 m, and the road width to be stabilized must be 8.5 m, then the mass on the canvas (in kg) will change to:

$$M = \frac{1 \times 1 \times 125 \times 2{,}080 \times 4 \times 8.5}{100{,}000 \times 2.5} = 35.6 \text{ kg}$$

This would be too much for a single application run, so two runs may be required and hence M is adjusted to suit (Equation F.4):

$$M = \frac{L \times H \times T \times D \times P \times R}{100{,}000 \times S \times N} \quad \textbf{(F.4)}$$

where:
N = Number of spreading runs.

F.1.3 Determination of the Mass of Stabilizer Required

Equation F.5 can be used to calculate what mass of stabilization agent (in tonnes) is required for a given length of road.

$$M = \frac{L \times T \times W \times P \times D}{100{,}000{,}000} \quad \textbf{(F.5)}$$

where:
M = Mass of stabilizer in tonnes
L = Length of road, in m.
T = Thickness of the compacted layer, in mm.
W = Width of stabilized road, in m.
P = Percentage stabilizer.
D = Density of soil after compaction, in kg/m^3.

Example

A 300 m length of road is to be prepared. The length must be stabilized to a width of 8.5 m with four percent cement. The compacted layer must be 150 mm thick. The soil density is 2,080 kg/m^3. The mass required is:

$$M = \frac{300 \times 150 \times 8.5 \times 4 \times 2{,}080}{100{,}000{,}000} = 31.8 \text{ tonnes}$$

F.1.4 Determination of Length of Road Covered by a Consignment of Stabilizer

Equation F.6 can be used to determine how far spreading can take place along a length of road (in m) with a known mass of stabilizer.

$$L = \frac{M \times 100{,}000{,}000}{T \times W \times P \times D} \quad \textbf{(F.6)}$$

where:
L = Length of spreading, in m.
M = Mass of load, in tonnes.
T = Thickness of the compacted layer, in mm.
W = Width of stabilized road, in m.
P = Percentage stabilizer.
D = Density of soil after compaction, in kg/m^3.

Example

Over what distance must a load of 25 tonnes be spread if the stabilization width is 8.5 m, the layer thickness is 150 mm, the soil density is 2,080 kg/m^2, and the required application rate of stabilizer is four percent?

$$L = \frac{25 \times 100{,}000{,}000}{150 \times 8.5 \times 4 \times 2{,}080} = 236 \text{ m}$$

F.2 Calculation of Saturation Moisture Content

To prevent cracking of the stabilized layer it is necessary to limit the compaction moisture content to not more than 80 percent of saturation moisture content. Where this leads to a compaction moisture content below optimum moisture content (OMC), additional compactive effort must be allowed for on site. To calculate the maximum compaction moisture content (m80), the following formula is used (Equation F.7):

$$m_{80} = \left(\left(\frac{1 - A}{\frac{D}{1{,}000}} \right) - \frac{1}{\frac{G_b}{1{,}000}} \right) \times 80 \qquad \textbf{(F.7)}$$

where:
A = Air voids ratio (0.05).
D = Dry density of soil after compaction, in kg/m^3.
Gb = Bulk relative density of soil particles, in kg/m^3.

Example

Determine the maximum compaction moisture content (m80) for the construction of a cement-stabilized layer, if the bulk relative density of the soil particles is 2,600 kg/m^3, air-void ratio is 0.05, and the dry density of the compacted soil is 2,090 kg/m^3.

$$m_{80} = \left(\left(\frac{1 - 0.05}{\frac{2{,}090}{1{,}000}} \right) - \frac{1}{\frac{2{,}600}{1{,}000}} \right) \times 80 = 5.6\%$$