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Recommended Practice for Stabilization of Subgrade Soils and Base Materials

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Recommended Practice for
Stabilization of Subgrade Soils
and Base Materials

Dallas N. Little
Syam Nair
Texas Transportation Institute
Texas A&M University
College Station, Texas

Contractor's Final Task Report for NCHRP Project 20-07
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National Cooperative Highway Research Program

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ABSTRACT

Long-term performance of pavement structures is significantly impacted by the stability of the underlying soils. In situ subgrades often do not provide the support required to achieve acceptable performance under traffic loading and environmental demands. Although stabilization is an effective alternative for improving soil properties, the engineering properties derived from stabilization vary widely due to heterogeneity in soil composition, differences in micro and macro structure among soils, heterogeneity of geologic deposits, and due to differences in physical and chemical interactions between the soil and candidate stabilizers. These variations necessitate the consideration of site-specific treatment options which must be validated through testing of soil-stabilizer mixtures. This report addresses soil treatment with the traditional calcium-based stabilizers: Portland cement, lime, and fly ash. The report describes and compares the basic reactions that occur between these stabilizers and soil and the mechanisms that result in stabilization. The report presents a straightforward methodology to determine which stabilizers should be considered as candidates for stabilization for a specific soil, pavement, and environment. The report then presents a protocol for each stabilizer through which the selection of the stabilizer is validated based on mixture testing and mixture design. The mixture design process defines an acceptable amount of stabilizer for the soil in question based on consistency testing, strength testing, and in some cases (resilient) modulus testing. Within each additive validation and mixture design protocol, an assessment of the potential for deleterious soil-additive reactions is made.

EXECUTIVE SUMMARY

Long-term performance of pavement structures is significantly impacted by the stability of the underlying soils. In situ subgrades often do not provide the support required to achieve acceptable performance under traffic loading and environmental demands. Although stabilization is an effective alternative for improving soil properties, the engineering properties derived from stabilization vary widely due to heterogeneity in soil composition, difference in micro and macro structure of soils, heterogeneity of geologic deposits, and due to differences in physical and chemical interactions between the soil and candidate stabilizers. These variations necessitate the consideration of site-specific treatment options validated through testing of soil-stabilizer mixtures under simulated field conditions. This report addresses soil treatment with the traditional calcium-based stabilizers: Portland cement, lime, and fly ash. The report describes and compares the basic reactions that occur between these stabilizers and soil and the mechanisms that result in stabilization. The report presents a straightforward methodology to determine which stabilizers should be considered as candidates for stabilization for a specific soil, pavement, and environment. The report then presents a protocol for each stabilizer through which the selection of the stabilizer is validated through mixture testing and mixture design. The mixture design process defines an acceptable amount of stabilizer for the soil in question based on consistency testing, strength testing, and in some cases (resilient) modulus testing. Within each additive validation and mixture design protocol, an assessment of the potential for deleterious soil-additive reactions is made.

For successful soil stabilizer applications it is imperative to understand the mechanism of stabilization of each additive. A basic understanding of stabilization mechanisms assists the user agency in selecting the stabilizer or additive best suited for a specific soil not only from the standpoint of developing the engineering properties desired for the pavement sublayers but also to minimize the risk of long-term deleterious reactions that might compromise pavement structural capacity or even induce disruptive volumetric changes such as sulfate-induced heave. In order to determine an appropriate soil-additive combination and to reduce the risk of deleterious reactions for a specific soil-stabilizer combination field exploration is required. For soil stabilization operations, the exploration process is less complex than for structural foundations as the depth of the influence zone is less. Therefore, although geological data are valuable, the most important data come from pedological profiles that are available, for example, in the National Resources Conservation Service (NRCS) County Soil Surveys. This report describes how the NCRS surveys and geological data sources should be used to plan an effective exploration plan to more clearly define the extent and boundaries of soil series and the depth of soil horizons that may affect chemical stabilization.

The report provides a protocol for mixture design for each additive type. This protocol begins with stabilizer selection and then proceeds to the verification step in which the selected stabilizer is evaluated based on consistency and strength testing. An indispensable part of the verification protocol is mixture design in which the amount stabilizer required to provide long-term, durable performance is determined. A separate protocol is presented for the most widely used traditional, calcium-based stabilizers: Portland cement, lime, and fly ash.

INTRODUCTION

The purpose of this document is to support and serve as background for the draft Standard Recommended Practice for Modification and Stabilization of Subgrade and Base Soils in Pavement Structures. The Recommended Practice and this background report address soil exploration, modifier selection, verification of stabilizer selection, and mix design. This document addresses use of the traditional calcium-based stabilizer: lime (CaO or Ca(OH)$_2$), Portland cement, and fly ash.

Long term performance of pavement structures often depends on the stability of the underlying soils. Engineering design of these constructed facilities relies on the assumption that each layer in the pavement has the minimum specified structural quality to support and distribute the super imposed loads. These layers must resist excessive permanent deformation, resist shear and avoid excessive deflection that may result in fatigue cracking in overlying layers. Available earth materials do not always meet these requirements and may require improvements to their engineering properties in order to transform these inexpensive earth materials into effective construction materials. This is often accomplished by physical or chemical stabilization or modification of these problematic soils. Although the solution appears simple and straight forward, engineering properties of individual soils may vary widely due to heterogeneity in soil composition, difference in micro and macro structure among soils, variability and heterogeneity of geologic deposits and due to differences in physical and chemical interactions of air/water with soil particles. These differences necessitate the use of site-specific treatment options for stabilization.

Over the years engineers have tried different methods to stabilize soils that are subject to fluctuations in strength and stiffness properties as a function of fluctuation in moisture content. Stabilization can be derived from thermal, electrical, mechanical or chemical means. The first two options are rarely used. Mechanical stabilization, or compaction, is the densification of soil by application of mechanical energy. Densification occurs as air is expelled from soil voids without much change in water content. This method is particularly effective for cohesion less soils where compaction energy can cause particle rearrangement and particle interlocking. But, the technique may not be effective if these soils are subjected to significant moisture fluctuations. The efficacy of compaction may also diminish with an increase of the fine content, fraction smaller than about 75 μm, of the soil. This is because cohesion and inter particle bonding interferes with particle rearrangement during compaction. Altering the physio-chemical properties of fine-grained soils by means of chemical stabilizers/modifiers is a more effective form of durable stabilization than densification in these fine-grained soils. Chemical stabilization of non-cohesive, coarse grained soils, soils with greater than 50 percent by weight coarser than 75 μm is also beneficial if a substantial stabilization reaction can be achieved in these soils. In this case the strength improvement can be much higher, greater than ten fold, when compared to the strength of the untreated material. This report discusses key factors associated with stabilizing soils using chemical modifiers including:

Mechanisms of Stabilization

Soil Classification

Soil Exploration

Guidelines for Stabilization and

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Validation of Stabilizer Selection

The soil must first be classified as either a subgrade category or base category material. In order to be classified as a base material the following criteria must be met: (1) a maximum of 25 percent of the soil mass passes the No. 200 sieve (0.074 mm or 0.003 in.), (2) not more than 40 percent of the soil mass passes the No. 40 sieve (0.42 mm or 0.0165 in.), (3) a maximum plasticity index of 12 percent, and (4) a maximum liquid limit of 40 percent. Otherwise, it is classified as a subgrade material for stabilization purposes.

The definition of modification and stabilization can be ambiguous. In this document *modification* refers soil improvement that occurs in the short term, during or shortly after mixing (within hours). This modification reduces the plasticity of the soil (improves the consistency) to the desired level and improves short-term strength to the desired level (short-term is defined as strength derived immediately within about 7-days of after compaction). Even if no significant pozzolanic or cementitious reaction occurs, the textural changes that accompany consistency improvements normally result in measurable strength improvement. *Stabilization* occurs when a significant, longer-term reaction takes place. This longer-term reaction can be due to hydration of calcium-silicates and/or calcium aluminates in Portland cement or class C fly ash or due to pozzolanic reactivity between free lime and soil pozzolans or added pozzolans. A strength increase of 50 psi (350 kPa) or greater (of the stabilized soil strength compared to the untreated soil strength under the same conditions of compaction and cure) is a reasonable criterion for stabilization. Construction steps in the stabilization process are not addressed in this document or in the Standard Practice associated with this document.

MECHANISMS OF STABILIZATION

The stabilization mechanism may vary widely from the formation of new compounds binding the finer soil particles to coating particle surfaces by the additive to limit the moisture sensitivity. Therefore, a basic understanding of the stabilization mechanisms involved with each additive is required before selecting an effective stabilizer suited for a specific application.

Chemical stabilization involves mixing or injecting the soil with chemically active compounds such as Portland cement, lime, fly ash, calcium or sodium chloride or with viscoelastic materials such as bitumen. Chemical stabilizers can be broadly divided in to three groups: Traditional stabilizers such as hydrated lime, Portland cement and Fly ash; Non-traditional stabilizers comprised of sulfonated oils, ammonium chloride, enzymes, polymers, and potassium compounds; and By-product stabilizers which include cement kiln dust, lime kiln dust etc. Among these, the most widely used chemical additives are lime, Portland cement and fly ash (*1*). Although stabilization with fly ash may be more economical when compared to the other two, the composition of fly ash can be highly variable. The mechanisms of stabilization of the traditional stabilizers are detailed below.

Traditional Stabilizers

Traditional stabilizers generally rely on pozzolanic reactions and cation exchange to modify and/or stabilize. Among all traditional stabilizers, lime probably is the most routinely used. Lime is prepared by decomposing limestone at elevated temperatures. Lime-soil reactions are complex and primarily involve a two step process. The primary reaction involves cation exchange and flocculation/agglomeration that bring about rapid textural and plasticity changes (*2*). The altered clay structure, as a result of flocculation of clay particles due to cation exchange and short-term

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pozzolanic reactions, results in larger particle agglomerates and more friable and workable soils. Although pozzolanic reaction processes are slow, some amount of pozzolanic strength gain may occur during the primary reactions, cation exchange and flocculation/agglomeration. Extent of this strength gain may vary with soils depending on differences in their mineralogical composition. Therefore, mellowing periods, normally about one-day in length but ranging up to about 4-days, can be prescribed to maximize the effect of short term reactions in reducing plasticity, increasing workability, and providing some initial strength improvement prior to compaction. The second step, a longer-term pozzolanic based cementing process among flocculates and agglomerates of particles, results in strength increase which can be considerable depending on the amount of pozzolanic product that develops, and this, in turn depends on the reactivity of the soil minerals with the lime or other additives used in stabilization.

The pozzolanic reaction process, which can either be modest or quite substantial depending on the mineralogy of the soil, is a long term process. This is because the process can continue as long as a sufficiently high pH is maintained to solubilize silicates and aluminates from the clay matrix, and in some cases from the fine silt soil. These solubilized silicates and aluminates then react with calcium from the free lime and water to form calcium-silicate-hydrates and calcium-aluminate-hydrates, which are the same type of compounds that produce strength development in the hydration of Portland cement. However, the pozzolanic reaction process is not limited to long term effects. The pozzolanic reaction progresses relatively quickly in some soils depending on the rate of dissolution from the soil matrix. In fact, physio-chemical changes at the surface of soil particles due to pozzolanic reactions result in changes in plasticity, which are reflected in textural changes that may be observed relatively rapidly just as cation exchange reactions are.

Portland cement is comprised of calcium-silicates and calcium-aluminates that hydrate to form cementitious products. Cement hydration is relatively fast and causes immediate strength gain in stabilized layers (*3*). Therefore, a mellowing period is not typically allowed between mixing of the components (soil, cement, and water) and compaction. In fact it is general practice to compact soil cement before or shortly after initial set, usually within about 2 hours. Unless compaction is achieved within this period traditional compaction energy may not be capable of developing target density. However, Portland cement has been successfully used in certain situations with extended mellowing periods, well beyond 2 to 4 hours. Generally, the soil is remixed after the mellowing periods to achieve a homogeneous mixture before compaction. Although the ultimate strength of a soil cement product with an extended mellowing period may be lower than one in which compaction is achieved before initial set, the strength achieved over time in the soil with the extended mellowing period may be acceptable and the extended mellowing may enhance the ultimate product by producing improved uniformity. Nevertheless, the conventional practice is to compact soil cement within 2 hours of initial mixing (*4*). During the hydration process, free lime, $Ca(OH)_2$ is produced. In fact up to about 25 percent of the cement paste (cement and water mix) on a weight basis is lime. This free lime in the high pH environment has the ability to react pozzolanically with soil, just as lime does and this reaction continues as long as the pH is high enough, generally above about 10.5.

Fly ash is also generally considered as a traditional stabilizer. While lime and Portland cement are manufactured materials, fly ash is a by-product from burning coal during power generation. As with other by-products, the properties of fly ash can vary significantly depending on the source of the coal and the steps followed in the coal burning process. These by-products can broadly be classified into class C (self-cementing) and class F (non-self cementing) fly ash based

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on AASHTO M 295 (ASTM C 618). Class C fly ash contains a substantial amount of lime, CaO, but almost all of it is combined with glassy silicates and aluminates. Therefore upon mixing with water, a hydration reaction similar to that which occurs in the hydration of Portland cement occurs. As with Portland cement, this hydration reaction produces free lime. This free lime can react with other unreacted pozzolans, silicates and aluminates, available within the fly ash to produce a pozzolanic reaction, or the free lime may react pozzolanically with soil silica and/or alumina. Class F ash, on the other hand, contains very little lime and the glassy silica and/or alumina exists almost exclusively as pozzolans. Therefore, activation of these pozzolans requires additives such as Portland cement or lime, which provide a ready source of free lime. The hydration or "cementitious" reactions and the pozzolanic reactions that occur when fly ash is blended with water form the products that bond soil grains or agglomerates together to develop strength within the soil matrix. As discussed previously, maintenance of a high system pH is required for long term strength gain in fly ash-soil mixtures.

The kinetics of the cementitious reactions and pozzolanic reactions that occur in fly ash stabilized soils vary widely depending on the type of ash and its composition. Normally, class C ashes react rapidly upon hydration. However, class F ashes activated with lime or even Portland cement produce substantially slower reactions than Portland cement – soil blends. Generally compaction practice of fly ash - soil blends varies depending on the type of ash used or whether or not an activator is used, but the standard practice is to compact within 6 hours of initial mixing (*5*).

By-product Stabilizers

Like traditional stabilizers, pozzolanic reactions and cation exchange are the primary stabilization mechanisms for many of the by-product stabilizers. Lime kiln dust (LKD) and cement kiln dust (CKD) are by-products of the production of lime and Portland cement, respectively.

Lime kiln dust (LKD) normally contains between about 30 to 40 percent lime. The lime may be free lime or combined with pozzolans in the kiln. The source of these pozzolans is most likely the fuel used to provide the energy source. LKDs may be somewhat pozzolanically reactive because of the presence of pozzolans or they may be altogether non reactive due to the absence of pozzolans or the low quality of the pozzolans contained in the LKD. Cement kiln dust (CKD) is the by product of the production of Portland cement. The fines captured in the exhaust gases of the production of Portland cement are more likely (than LKD) to contain reactive pozzolans and therefore, to support some level of pozzolanic reactivity. CKD generally contains between about 30 and 40 percent CaO and about 20 to 25 percent pozzolanic material.

The purpose of this document is not to establish specific guidelines regarding composition of by-product LKD or by-product CKD as the oxide composition of each can vary widely depending on the composition of the feed stock, the nature of the fuel, the burning efficiency, and the mechanism and efficiency of flue dust capture. For example if coal is used, then ash produced as a by-product of burning coal could be captured in the bag house or other mechanism used to capture exhaust fines with the by-product lime. If the source of the LKD is from the production of dolomitic lime, then magnesium oxide may form a significant part of the LKD. Magnesium oxide, MgO, takes longer and is more difficult to fully hydrate than CaO, and upon hydration it expands. If the LKD contains more than about 5 percent MgO then care should be taken to insure full hydration of the MgO if this LKD is used for modification or stabilization.

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Again, it is incumbent upon the agency involved to determine acceptable levels of oxides and trace elements that comprise the by-product.

As a general guide on the level of risk associated with the presence of oxides and trace elements in these by product stabilizers, the development of expansive mineral products may become intolerable when the SO_3 content exceeds about 3 percent or when the MgO content exceeds about 3 to 5 percent. The impact of organics can also be a problem as their presence can interfere with the availability of calcium to the soil or aggregate being treated. Several tests can be used to screen for the presence of organics. One quick test if loss on ignition (LOI). Although it does not identify the type of organic, which is definitely important, an LOI of greater than about 8 to 10 percent flags a potentially problematic quantity of organics.

Non Traditional Stabilizers

This standard practice is limited to traditional, chemical stabilizers like: Portland cement, lime and fly ash. However, it is important when considering treatment with these traditional products to broach the subject of non-traditional or alternative stabilizers.

The mechanism of stabilization for non-traditional stabilizers varies greatly among the stabilizers. Asphalt may or may not be grouped as a traditional stabilizer depending on perspective. Asphalt is not a "chemical" stabilizer in the sense that it does not react chemically with the soil to produce a product that alters surface chemistry of the soil particles or that binds particles together. Instead asphalt waterproofs aggregate and soil particles by coating them and developing an adhesive bond among the particles and the asphalt binder (*6*). The process is dependent on the surface energies of the aggregate or soil and the asphalt binder. Consequently, since this mechanism is more physical than chemical, soils with very high surface areas are not amenable to asphalt stabilization and such stabilization is normally limited to granular materials such as gravels or sands, and perhaps some silty sands. As a visco-elastic, visco-plastic material, temperature and/or dilution methods are required to make asphalt stabilization effective in soils. Either lower viscosity liquid asphalts (normally developed by mixing bitumen with diluents) or emulsified asphalts are used in soil stabilization. Because the nature of asphalt stabilization is so mechanistically different from chemical stabilization, asphalt stabilization is not considered as a candidate in this standard practice.

The mechanisms of stabilization of other non-traditional stabilizers including sulfonated oils, enzymes, ionic stabilizers, etc. are discussed in detail by Petry and Little (*1*). Such stabilizers may have a role in modification and/or stabilization, especially when high soluble sulfate contents in the soil limits the applicability of calcium-based, traditional stabilizers.

SOIL CLASSIFICATION

Soil is a broad term used in engineering applications which includes all deposits of loose material on the earth's crust that are created by weathering and erosion of underlying rocks. Although weathering occurs on a geologic scale, the process is continuous and keeps the soil in constant transition. The physical, chemical, and biological processes that form soils vary widely with time, location and environmental conditions and result in a wide range of soil properties (*7*). Physical weathering occurs due to temperature changes, erosion, alternate freezing and thawing and due to plant and animal activities causing disintegration of underlying rock strata whereas chemical weathering decomposes rock minerals by oxidation, reduction, hydrolysis, chelation, and carbonation. These weathering processes, individually or in combination, can create residual

in-place soils or facilitate the transport of soil fractions away from the parent rocks by geologic agents like wind, water, ice or gravity. These transport processes often result in mixing of soil minerals or introducing salts or organic material of a variety of species and concentrations. Soil impacted by the presence of organics and salts, such as sulfates, can exist as remote outcrops or over large areas and often do not have clearly defined boundaries. The soil pedological profile also varies considerably with location and even within a specific soil series or association. The complexity of soils requires a disciplined yet efficient method to identify and classify them for their use as a construction material.

Soil texture is defined, at least initially, by its appearance and is dependent on the size, shape and distribution of particles in the soil matrix. Soil particle sizes may vary from boulders or cobbles, roughly a meter in diameter, to very fine clay particles, roughly a few microns in diameter. Engineering properties of coarse fractions are dependent on physical interlocking of grains and vary with the size and shape of individual particles. Finer fractions in soil have a significantly higher specific surface area and their behavior is influenced more by electro-chemical and physio-chemical aspects than particle interaction. Among finer particles, clays exhibit varying levels of consistency and engineering behavior and demonstrate various levels of plasticity and cohesiveness in the presence of water. Silt fractions are also classified as fine-grained soils because more than 50 percent of the soil mass is smaller than 75 μm, which fits in the designation of fine-grained material according to the Unified Classification System (AASHTO M 145). However, the specific surface area of silt fines is several orders of magnitude larger than that of clay soil particles. This difference is part of the reason that clay particles are more reactive than silt particles. In addition, clay minerals have a unique sheet particle structure and a crystalline layer structure that is amenable to significant isomorphous substitution. As a result of the isomorphous substitution of lower valence cations for higher valence cations within the layer structure, clay mineral surfaces carry a significant negative surface charge that can attract positively charged ions and dipolar water molecules. The cumulative effect of high surface area and surface charge makes clay particles particularly reactive, especially with water, and is the root cause of the propensity of clay particles to shrink and swell depending on the availability of water.

The AASHTO (M 145) soil classification system differentiates soils, first based on particle size and secondly based on Atterburg limits. If 35 percent or more of the mass of the soil is smaller than 75 μm in diameter, then the soil is considered either a silt or clay and if less than 35 percent of particles are smaller than 75 micron sieve, then the soil is considered to be coarse-grained, either a sand or gravel. For stabilization purposes, soils can be classified into subgrade and base materials based on fractions passing No. 200 sieve. If 25 percent or more passes through the no. 200 sieve the soil can be considered as a subgrade, and if not, they may be classified as a base material. However, more than simple gradation impacts the definition of a subgrade or base. In order to be termed a base material, the material in question must also be targeted for use as a base layer from a structural perspective. On the other hand, an in situ coarse-grained soil with less than 25 percent fines, may be, by definition a native subgrade even though it may achieve the required classification of a base. For stabilization purposes, the soils may be differentiated into subgrade (soil) stabilization and base stabilization (coarse-grained) on the basis on the fine content index.

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SOIL EXPLORATION

Soil exploration is a vital part of the preliminary engineering survey for location, design, and construction of highways. Soil exploration provides information on conditions of the underlying strata that can affect the performance of pavement structures (*8*). The process also involves recovery of representative soil samples for classification and testing purposes. The general purpose of subsurface exploration is to:

- Identify and locate soil and rock strata
- Identify ground water table conditions
- Establish subsoil (moisture and density) conditions
- Define characteristics and relevant engineering properties of subsurface materials by sampling and in situ testing
- Provide preliminary assessment of the need for stabilization of sub-grade, sub-base and base materials
- Locate suitable materials for fills, subgrade treatment, materials for base and aggregate
- Identify local conditions requiring special considerations

Preliminary Data Collection

A close and interactive relationship exists among geology, pedology and engineering. Pedology is the study of the soil profile based on the soil forming processes and factors such as climate, age, vegetation, and drainage that have altered the parent material to form the soil. The soil profile is layered into horizons which can be used to identify the reactivity of the soil with stabilizers as a function of depth and to identify the presence of harmful minerals or compounds that may react negatively with the selected stabilizer. In addition, the mineralogical composition and chemical composition of the horizons within the profile can be used to assess whether or not the soil within the horizon will be reactive with the selected stabilizer. A pedological system of classification can be used as a basic approach for soil classification (*9*). The National Resources Conservation Service (NRCS) County Soil Survey is an excellent source of data that must be considered in any sampling effort in order to identify the required depth and frequency of sampling and to establish the expected results of the sampling process.

Geological data can be used to interpret the impact of land forms, processes that lead to their development, their history and also to identify the sub-surface terrain features that might affect the behavior of these layers. For example sulfate bearing seams below the layer to be stabilized may provide a source for sulfate diffusion into the stabilized layer via capillary rise. Geological and pedological knowledge at the location provides the ground work by which to differentiate earth materials and identify problem zones. This relationship is especially valid for highway construction as pavements are built on and, in some cases, of earth materials. In addition to the National Resource Conservation Service (NRCS) County Soil Survey Reports, geological data and information can be obtained from the United States Department of Agriculture, and the U. S. Geological Survey Reports (*2*). State Geologic Survey Reports, if available, can also be used as a source for geologic information for the location.

Subsurface Investigations

Subsurface investigations are most often site specific and should be guided by the purpose, requirements, and geographical settings of the project location (*10*). Available data regarding the

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project location should be reviewed prior to beginning the field investigation. Geologic maps, topographic maps, aerial and satellite photos, statewide or county soil surveys, and engineering maps are some of the useful sources of information regarding soil properties. Subsurface investigation reports of adjacent projects, if any, should also be studied. Field exploration methods, sampling requirements, and the type and frequency of field tests should be determined based on existing information, design requirements, availability of equipment and local practices. Subsurface exploration should be conducted in accordance with AASHTO R13. A comprehensive exploration plan should be developed to communicate the intent and level of testing required for the project (*8, 10*). The success of a subsurface investigation depends primarily on the effectiveness of the geotechnical engineers and technicians involved in field operations and therefore should only be performed by responsible, well-trained and experienced people.

Sampling Plan

A properly designed sampling plan should be developed to minimize sampling error and to optimize sampling efficiency. Samples should be taken in a manner that minimizes bias of the person taking the samples. This requires a plan to randomize sampling locations (*10*). However, boring and sampling programs must be planned and executed within budget constraints with appropriate consideration of other variables that can affect the site investigation. The development of a good sampling plan may include:

- Statement of the problem for which sampling is required
- Collection of available, relevant soil data
- Evaluation of different possible sampling plans, in terms of over-all cost, precision and difficulty

Sampling of Soils

Direct observation of subsurface conditions and retrieval of field samples can be achieved by examination of soil formations using accessible excavations, such as shafts, tunnels, test pits, or trenches, or by drilling and sampling to obtain cores or cuttings (*10*). Since stabilization operations involve mixing and compaction operations that destroy the original soil fabric, disturbance of samples during extraction does not normally compromise the quality of neither the sample nor its acceptability for testing. Hence undisturbed soil samples are not normally required for testing to evaluate the efficacy of soil stabilization. The testing involves evaluation of the soil properties including gradation, Atterberg limits, mineralogy, organic content and sulfate content.

Sample units of roadway materials should be selected randomly in accordance with ASTM D 3665. The number of field samples to be collected depends on the level of confidence required by project specifications. Guidance in determining the number of samples required to obtain the desired confidence levels are detailed in ASTM test methods E 105, E 122 and E 141.

Frequency and Depth of Sampling

Subsurface conditions can be identified at the individual test pits, boring holes or by examining open cut sections. Soil strata can show significant spatial variability and the soil conditions can vary significantly between test pits. Therefore the continuity of soil and rock formations should be considered during the investigation. Geophysical techniques may be used to obtain general information pertaining to location of boundaries between bedrock and overlying deposits.

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Spacing of test pits during soil investigations should be dependent on the geologic complexity of the project area. Frequency of sampling should be based on the uniformity of soil, intent and level of investigation required and the potential for detrimental reactions with the soil during chemical stabilization processes. A general recommendation on frequency of sampling based on varying soil conditions is given in Table 1.

Table1. Guideline regarding spacing between sampling locations (*11*).

Soil condition	Frequency of sampling
Uniform	0.5 to 1.0 mile
Non-Uniform	0.25 to 0.5 mile
Highly variable	1,000 ft to 0.25 mile
Sulfate bearing	500 ft

Exploration should be deep enough to identify all strata that can significantly influence the outcome of the stabilization project. The chemical stabilization operation seldom proceeds deeper than 12-inches. However, the material below this layer affects stabilization. The most important factor is the depth of the water table. This depth and its annual fluctuation will probably require a combination of soil borings and a study of pedological and geological data sources to establish. However, knowledge of the fluctuation of the water level with respect to the stabilized layer will help define the risk of and extent of intrusion of moisture into the stabilized layer through capillary rise. The potential for capillary rise into the stabilized layer will also help assess whether or not diffusion of deleterious salts into the stabilized layer are probable.

The Texas Department of Transportation (TxDOT) recommend continuous material sampling to a depth of at least 15 feet in locations where water fluctuations are high (*11*). For cuts exceeding these depths, sampling should be done to the road bed depth plus an additional 2 feet. Samples should be collected every time there is a change in observed physical characteristics of the material. AASHTO R-13 recommends that the depth of exploratory borings or test pits for road beds be at least 1.5 m (5 feet) below the proposed subgrade elevation. The boring depths and spacing requirements mentioned above should not be considered as either a minimum or a maximum, but instead should be used as a guide. In locations where project construction or performance may be affected by water or where impervious materials block internal drainage, borings should be extended to a sufficient depth to determine the engineering and hydro geologic properties relevant to the project design.

GUIDELINES FOR SOIL STABILIZATION

Stabilization projects are site specific and require integration of standard test methods, analysis procedures and design steps to develop acceptable solutions. Many variables should be considered in soil treatment, especially if the treatment is performed with the intent of providing a long-term effect on soil properties. Soil-stabilizer interactions vary with soil type and so does the extent of improvement in soil properties. Hence developing a common procedure applicable for all types of stabilizers is not practical. Instead, a generalized, flowchart-based approach, which provides the steps that should be followed in stabilizer selection, is presented in Figure 1.
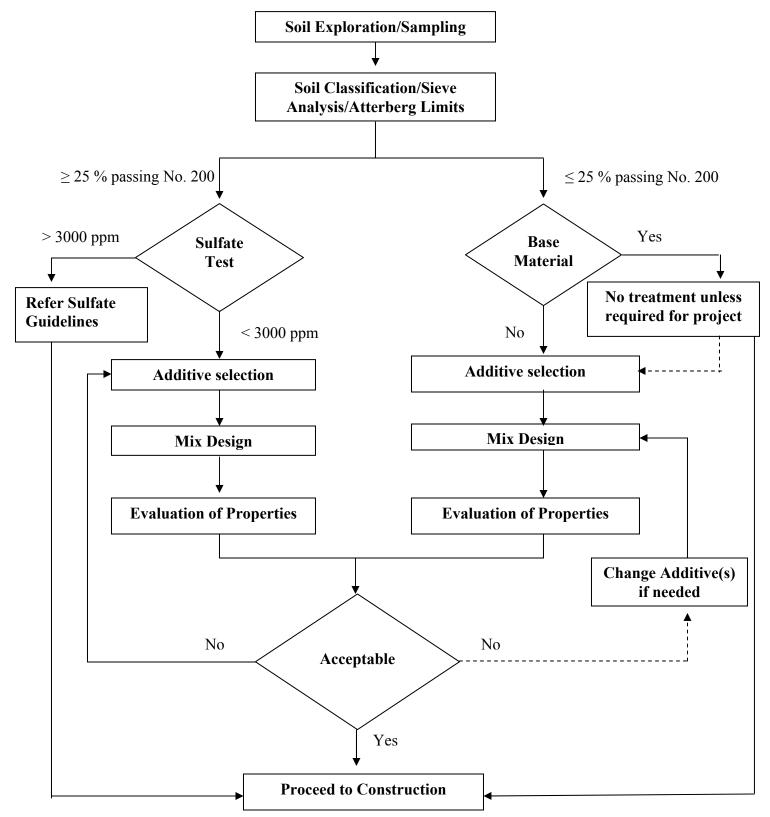
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Figure 1. Guideline for stabilization of soils & base materials for use in pavements (*12*).

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Soil exploration and sampling should be performed as described in the preceding sections. The soil can be classified as either a subgrade category or base category material on the basis of AASHTO M145. A key decision factor in selecting the appropriate subgrade additive is the concentration of water soluble sulfates in the soil. Sulfate testing should be done in accordance with the modified version of AASHTO T 290 or equivalent. Soils with sulfate levels above 3,000 ppm may be considered problematic and should be addressed separately from the standpoint of additive selection all the way through mix design and construction. Sampling, testing, stabilizer selection, and mix design for these soils should follow the draft recommended practice for stabilizing sulfate-bearing soils (*13*). A second key factor to be considered when deciding on the type of stabilizer to be used is the concentration of organic matter in the soil. Organic contents can interfere with strength gain mechanisms and should be determined prior to proceeding with mix design with any calcium-based stabilizer.

Base materials must satisfy plasticity and gradation requirements and restrictions that vary from state-to-state. As a typical example, the Texas Department of Transportation (TxDOT) specifies various classes of base materials in Item 247 of the Texas Standard Specifications (*14*). AASHTO M 147 also provides guidance in distinguishing among classes of base materials.

Guidelines for Stabilizer Selection

Soil characteristics including mineralogy, gradation and physio-chemical properties of fine-grained soils influence the soil-additive interaction. Hence stabilizer selection should be based on the effectiveness of a given stabilizer to improve the physio-chemical properties of the selected soil. The preliminary selection of the appropriate additive(s) for soil stabilization should consider:

- Soil consistency and gradation
- Soil mineralogy and composition
- Desired engineering properties
- Purpose of treatment
- Mechanisms of stabilization
- Environmental conditions and engineering economics

Soil index properties (i.e., sieve analysis, Atterberg limit testing, and moisture density testing) should be determined based on laboratory testing of field samples. Soil samples should be prepared following AASHTO T 87. The initial processing of most soils involves thorough air drying or assisted drying at a temperature not to exceed $60^{o}C$. Aggregations of soil particles should be broken down into individual grains to the extent possible. A representative soil fraction should be selected for testing following AASTHO T 248. The required quantity of soil smaller than 0.425 mm (No. 40 sieve) should be used to determine the soil index properties. Liquid limit testing should be performed following AASHTO T 89 and plastic limit and plasticity index testing should be measured following AASHTO T 90.

Lime Stabilization

Lime has been found to react successfully with medium, moderately fine and fine grained soils causing a decrease in plasticity and swell potential of expansive soils, and an increase in their workability and strength properties. Research has proven that lime may be an effective stabilizer in soils with clay content as low as 7 percent and in soils with plasticity indices below 10 (*15*).

The National Lime Association recommends a plasticity index of 10 or greater in order for lime to be considered as a potential stabilizer whereas the U.S Army Corps of Engineers recommends a plasticity Index of 12 or greater for successful lime stabilization (*6, 16*). Based on AASHTO classification, soil types A-4, A-5, A-6, A-7 and some of A-2-6 and A-2-7 are suitable for stabilization with lime (*17*).

Cement Stabilization

Cement stabilization is ideally suited for well graded aggregates with a sufficient amount of fines to effectively fill the available voids space and float the coarse aggregate particles. General guidelines for stabilization are that the plasticity index should be less than 30 for sandy materials. For fine-grained soils, soils with more than 50 percent by weight passing 75μm sieve, the general consistency guidelines are that the plasticity index should be less than 20 and the liquid limit (LL) should be less than 40 in order to ensure proper mixing (*6*). A more specific general guideline based on the fines content is given in the equation below which defines the upper limit of P.I. for selecting soil for cement stabilization (*17*).

$$P.I \leq 20 + \frac{50 - (\% \; smaller \; than \;\; 0.075\,mm)}{4}$$

Cement is appropriate to stabilize gravel soils with not more than 45 percent retained on the no. 4 sieve. The Federal Highway Administration recommends the use of cement in materials with less than 35 percent passing no. 200 sieve and a plasticity index (PI) less than 20 (*18*). Based on this system, soils with AASHTO classifications A-2 and A-3 are ideal for stabilization with cement, but certainly cement can be successfully used to stabilize A-4 through A-7 soils as well. The Portland cement Association (PCA) established guidelines to for stabilizing a wide range of soils from gravels to clays.

Fly Ash Stabilization

The literature lacks a clear direction in selection parameters for the use of fly ash in soil stabilization. However, the literature documents that a wide range of aggregates can be suitably stabilized with fly ash including sands, gravels, crushed stones and several types of slags. Fly ash can be used effectively to stabilize coarse grained particles with little or no fines. In coarser aggregates, fly ash generally acts as a pozzolan and/or filler to reduce the void spaces among larger size aggregate particles to float the coarse aggregate particles. After the appropriate amount of fly ash is added to coarse grained soils to fill the voids, optimize density, an activator is often used to maximize the pozzolanic reaction in the mixture. The activator content is generally in the range of 20 to 30 percent of the fly ash used to fill the voids. The activator is normally either lime or Portland cement, but lime kiln dust or cement kiln dust can also be used. Similarly, consider a clay soil that is stabilized with lime but the clay is not pozzolanically reactive. The addition of fly ash and lime can substantially increase strength in the blend due to the reactive pozzolans provided by the ash. In these fine-grained soils, fly ash is typically used in conjunction with lime or cement to enhance the reactivity of the fine-grained soil with lime or cement.

Class C fly ash has been used alone to stabilize moderately plastic soils. The basis for stabilization is free lime that becomes available upon hydration of the ash. The large majority of this lime is combined with the silica and alumina, but upon hydration, just as in the hydration of Portland cement, cementitious products are formed which stabilize the soil. However during this

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hydration process, just as in the hydration of cement, free lime is released, which can react pozzolanically with the clay. This reaction reduces clay particle plasticity and improves strength. Successful application is often achieved with fine grained, plastic soils, by first applying lime or cement to reduce plasticity and improve workability of the soil and then adding the fly ash to boost strength of the soil, lime blend. Again, the impact of a given class F (with activator) or a given class C fly ash without activator may be very different depending on the pozzolan content of each ash, the degree of self cementing property of the class C ash, etc. Hence, the superior filler cannot be determined before hand and without evaluation.

Techniques for Stabilizer Selection

A range of options are available for selecting soil stabilizers most of which are based on the soil classification following either the AASHTO or Unified classification system. A simple, but well accepted methodology by which to select the appropriate stabilizer is the Soil Stabilization Index System (SSIS). The methodology was developed by U.S Air Force, and is based on soil index properties: plasticity index and percent passing the no. 200 sieve (*19*). These laboratory tests are easy to perform and are necessary inputs for AASHTO and Unified systems. Both these characteristics can be effectively correlated to the engineering properties of the soil and therefore can be used to differentiate engineering applicability. Figures 2 (for soils) and 3 (for base materials) use these two index properties, PI and percent passing the no. 200 sieve (percent smaller than 75 μm), to identify the appropriate stabilizer (*12*). Once the stabilizer is selected, detailed laboratory tests to determine strength and performance characteristics of soils are required. Individual test methods required for mix design for three traditional stabilizers are discussed in the later sections of this report.
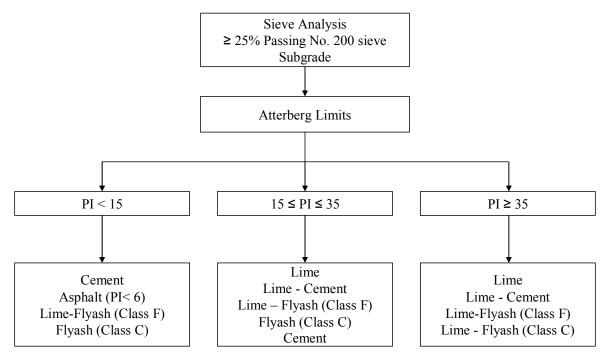
Figure 2. Decision tree for selecting stabilizers for use in subgrade soils (*12*).

Figures 2 and 3 present a set of general guidelines for selecting candidate stabilizers for soil and base materials. Agencies, however, should alter or adjust these guidelines based on their own

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unique experiences as tempered by local conditions. It is important to remember that Figures 2 and 3 are "guidelines" but the final selection should be based on a more specific analysis of the soils. These involve identifying the reactivity of the pozzolans in the clay with the selected stabilizers. For example, lime may be an ideal stabilizer for reactive plastic clay because the lime can immediately reduce plasticity due to cation exchange reactions. Pozzolanic reaction continues over time to further reduce plasticity and increase strength due to the formation of, primarily, calcium-silicate-hydrates. On the other hand, a different clay bearing soil may not be pozzolanically reactive, and, even though the application of lime initially reduces plasticity and improves workability, the desired strength gain does not develop. In this case the stabilizer of choice may have to be Portland cement or a combination of lime and fly ash or lime and cement.

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┌─────────────────────────────┐
│       Sieve Analysis        │
│  < 25% Passing No. 200 sieve│
│        Base Material        │
└─────────────────────────────┘
               │
               ▼
┌─────────────────────────────┐
│      Atterberg Limits       │
└─────────────────────────────┘
        │              │
        ▼              ▼
┌──────────────┐  ┌──────────────┐
│   PI ≤ 12    │  │   PI ≥ 12    │
└──────────────┘  └──────────────┘
        │              │
        ▼              ▼
┌──────────────┐  ┌──────────────────────┐
│    Lime      │  │       Lime           │
│   Cement     │  │      Cement          │
│Asphalt (PI<6)│  │    Lime-Cement       │
│Flyash (Class C)│ │ Lime – Flyash (Class F)│
│              │  │   Flyash (Class C)   │
└──────────────┘  └──────────────────────┘
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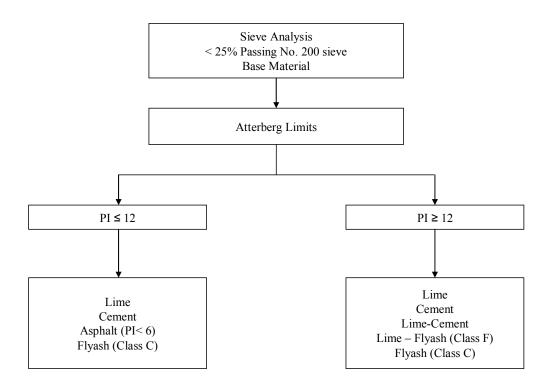
Figure 3. Decision tree for selecting stabilizers for use in Base materials (*12*).

The decision trees provide a first step toward stabilizer selection. Once a stabilizer is selected, detailed mixture design is recommended if stabilization is the objective. If modification is the objective, then verification tests are required to ensure that the objectives of reduction in plasticity and perhaps immediate strength gain requirements are met. As discussed earlier in this document, *modification* refers soil improvement that occurs in the short term, during or shortly after mixing (within hours) where as *Stabilization* is generally a longer term reaction and the degree of strength gain required to achieve stabilization varies based on the expectations of the user. Again, as discussed earlier a strength increase of at least 50 psi greater than that of the untreated soil fabricated and cured under the same conditions as the stabilized material is used in this document to define stabilization. This value was used by Thompson in the Illinois method of mix design for lime treated soils (*20*). The researchers on this project recommend that a method of moisture conditioning be included in all strength testing protocols. This research team recommends capillary soak as the form of moisture conditioning before strength testing. In the

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capillary soak protocol, the sample is placed on a porous stone and wrapped in an absorptive fabric and allowed to absorb water through capillary rise until the moisture front ceases to move or for at least 24 hours.

Additional Tests Involved in Stabilizer Selection

Once an additive has been selected based on the index properties of plasticity index and percent of the soil mass smaller than 75μm, the possible impact of deleterious components of the soil must be considered. Organic contents in excess of one percent on a mass basis have been proven to be potentially deleterious (*16*). However, some soils with organic contents well over one percent have been successfully treated and stabilized with lime and Portland cement. The second deleterious component is high salt content. A high potassium or sodium content may negatively impact stabilization by competing with calcium cations. However, this can normally be overcome simply by adding the additional calcium-based stabilizer. However, salts containing sulfates have the potential to react with calcium and aluminum released from soil in the high pH environment formed during stabilization to form expansive minerals that can disrupt the stabilized layer. The mechanisms of these mineral formations and the associated volume changes in pavement layers are detailed elsewhere (*13*).

Soil organic content should be measured following ASTM D 2974. Soils with an organic content of 1-2 percent as determined by ASTM D 2974 may be difficult to stabilize or may require uneconomical quantities of additives in order to stabilize. Stabilized soils, in some cases, may also not be able to meet the recommended strength criteria when excess amounts of organic matter are present. This is because the presence of organic materials in soils inhibits the normal hydration process and reduces the strength gain in stabilized soils.

Sulfate contents in soil should be determined following Modified AASHTO test method T 290 or equivalent. Generally, water soluble sulfate levels greater than 0.3 percent (3,000 ppm) suggest the potential for expansive reactions to occur that may result in disruptive volume change in the stabilized layer. Recommendations outlined in Guidelines for Stabilizing Sulfate-Bearing Soils should be followed in stabilizing these soils with lime (*13*).

VALIDATION OF STABILIZER SELECTION

The procedure outlined below provides a guideline for mixture design for lime, Portland cement and fly ash.

Lime Stabilization for Soils

Lime is an appropriate stabilizer for most cohesive soils but the level of reactivity depends on the type and amount of clay minerals in the soil. The steps described in the following paragraphs ensure that the appropriate amount of lime is used to meet design expectations. If design expectations cannot be met with lime, that will become clear by following this protocol described in this section.

Mix Design Considerations

The mix design protocol presented here follows the National Lime Association protocol (*21*). The mix design protocol is designed to optimize the potential for long-term strength gain and durability of lime stabilized soils.

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Soil Evaluation

The first step in the NLA protocol is similar to the approach described in Figure 2 and in fact either the criteria described in Figure 2 or the criteria described in this section can be used. In this step, the soil fraction passing the no. 200 sieve is determined following AASHTO T-27. Liquid limit and plastic limit should be determined following AASHTO T 89 and AASHTO T-90, respectively. Soils with a plasticity index of 10 or above and a minimum of 25 percent passing the no. 200 sieve are considered desirable for lime stabilization. The NLA protocol requires screening for organic contents above one percent following ASTM D 2974. The NLA protocol does not restrict or eliminate lime stabilization when the organic content of the soil is above one percent, but the protocol recommends that the designer maintain an awareness of this condition throughout the design process and also maintain an awareness of the fact that high organic contents may disrupt the pozzolanic reaction process and may require a greater lime content than normal for the soil in question to reach the desired strength. Water soluble sulfate should be evaluated following AASHTO T 290 (modified). The NLA protocol recommends that if the soluble sulfate content is greater than 3,000 ppm then the user should perform swell tests to verify the expected degree of expansion and take construction steps to mediate the potential expansive reactions. Additional steps to be followed in stabilizing soils with sulfate content above 3,000 ppm are detailed in the AASHTO draft recommended practice for stabilizing sulfate bearing soils (*13*).

Optimum Lime Content

The first step in assessing the optimum lime content to ensure optimal long term strength gain is to perform the Eades and Grim pH test. For reliable test results, the lime used in the pH test should be the same as that to be used in construction and this lime should be carefully stored to avoid carbonation. The lime used, whether it is in the form of CaO or $Ca(OH)_2$, must meet AASHTO M 216 (ASTM C 977) or equivalent for purity requirements. The standard test method, ASTM D 6276, is used to determine the amount of lime needed to achieve the design pH at 25^0C (77^0F), which is about 12.45, depending on specific soil characteristics. The goal of this test is to identify the amount of lime necessary to satisfy immediate lime-soil reactions and also provide a sufficient quantity of calcium to maintain a high residual pH and sustain significant long-term pozzolanic reactions. The pH test is only a first step. The optimum lime content must be validated based on strength testing.

Moisture Density Relationship

The addition of lime changes the optimum moisture content (OMC) and maximum dry density (MDD) of soils because the effects of cation exchange and short-term pozzolanic reactions between lime and the soil results in flocculation and agglomeration of clay particles leading to textural changes that are reflected in the moisture-density relationships. For this reason it is necessary to verify the moisture-density relationship of the lime-soil mixture when the amount of lime identified by the Eades and Grimm pH test has been added. The moisture-density relationship of lime-soil mixtures should be determined in accordance with AASHTO T 99.

Fabrication and Curing of Samples for Compression Testing

Lime-soil mixtures should be fabricated following ASTM D 3551 for compressive strength testing. The samples should be prepared at the moisture content and density expected in the field. Normally, for compressive strength testing, samples are not allowed to mellow before samples

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are fabricated. However, if it is difficult to achieve satisfactory homogeneity during laboratory mixing, it is reasonable to consider a mellowing period (between initial mixing and final mixing before compaction) of up to 24 hours to simulate field mellowing. However, with the high efficiency of lab mixing compared to field mixing, it is assumed that lab mellowing will not be necessary in most applications.

Triplicate samples are prepared for compressive strength testing following ASTM D 5102 procedure B with the lime content determined from the pH test. Samples are fabricated at between optimum moisture content (OMC) and OMC ± 1 percent. Additional mixtures with lime contents one and two percent higher than the optimal lime content identified by the Eades and Grim pH test as optimum should also be fabricated and tested following ASTM D 5102 to verify the optimum lime content, which may be greater than that identified by Eades and Grim pH test.

After compaction the test specimens should be wrapped in a plastic wrap and stored in an air tight moisture proof bag with about 10 ml of free water to ensure proper moisture for pozzolanic reactions. The specimens are then cured at 40°C (104^{0}F) for 7 days before compression testing. Since the accelerated cure is not always a good approximation of strength gain by long term normal cure, it is appropriate to subject one set of lime soil samples to normal cure for 28 days before compression testing.

After the curing period, the specimens are removed from the storage bags and plastic wraps are removed. The specimens are then wrapped with a wet absorptive fabric or geotextile and placed on a porous stone for capillary soak. Capillary soaking should continue for as long as it takes for the moisture front to move to the top of the sample or until the moisture front ceases to move. A soaking period of at least 24 hours is recommended. Research work by Thompson (*22*) and Little (*23*) demonstrated that the reduction in compressive strength due to soaking is not substantial (less than about 10 percent) for stabilized soil with a significant level of pozzolanic reaction. But the deleterious effects can be significant (up to 40 percent) if soaking occurs prior to significant pozzolanic strength gain. During capillary soak, the water used in soaking should never come in direct contact with the specimen (*24*). The water level should be maintained to the top of the porous stone and kept in contact with the fabric wrap.

Unconfined Compression Strength Testing

Following capillary soak, unconfined compression strength testing should be performed in accordance with ASTM D 5102 procedure B. The results of compression tests are compared with the suggested minimum requirements given in Table 2. If more than one lime contents are considered in compression testing, the lowest lime concentration that meets the compression strength requirement is considered as the required lime content for stabilization purposes. If the specimens do not meet the strength criteria, then the soils can be considered as modified soils and not stabilized soils. Higher lime content may be used in these soils and the mix design procedure, starting from moisture density relationship, should be repeated. It should be noted that the compressive strength values given in table below are suggested minimum values and field requirements may vary depending on purpose of stabilization, exposure conditions, expected freeze thaw cycles and cover material over stabilized soil.

Table 2. Compressive strength recommendations for lime stabilized sections (*22*).

Anticipated Use of Stabilized layer	Compressive strength recommendations for different anticipated conditions

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	Extended Soaking for 8 Days (psi)	Cyclic Freeze-Thaw		
		3 Cycles (psi)	7 Cycles (psi)	10 Cycles (psi)
Sub-base Material				
Rigid Pavements/Floor Slabs/Foundations	50	50	90	120
Flexible Pavement (>10 in.)	60	60	100	130
Flexible Pavement (8 in. - 10 in.)	70	70	100	140
Flexible Pavement (5 in. - 8 in.)	90	90	130	160
Base Material				
	130	130	170	200
For cyclic moisture conditioning the samples should be made to reach 80 percent saturation upon "wetting" followed by 50 percent saturation upon "drying". This is satisfactory to represent the damaging cyclic environment.				

Volume Change Measurements for Expansive Soils

Samples prepared for compression testing can be used to evaluate volume changes in lime stabilized expansive soils. Vertical and circumferential measurements of samples before and after soaking should be made to calculate volume changes between the dry and soaked conditions. A three dimensional volumetric expansion of up to 2 percent is typically regarded as acceptable (*24*). If the expansion in the treated soil is higher than the recommended value, then additional lime of 1 to 2 percent should be evaluated. This step is applicable only for expansive soils. Although this test can be used to validate swell in sulfate bearing soils, the period of exposure to moisture for sulfate bearing soils is considerable longer than 7-days. In the case of sulfate-bearing soils the period of swell should continue until swell ceases.

Lime Treatment of Base Courses

The protocol described above addresses lime-soil mixtures. In the event that lime is used as a stabilizer for base materials, it is important to understand that the purpose of lime is to interact with the fine material, normally finer than 75 μm, to form a matrix that will provide improved strength for the aggregate base. It is assumed that the candidate aggregate base material is of at least moderate quality, otherwise, the material should be treated as a soil. Moderate quality is defined as: (1) not more than 20 percent finer than the no. 40 sieve (0.425 mm or 0.0165 in.), (2) a maximum plasticity index of 12 percent, and (3) a maximum liquid limit of 40 percent.

Since in aggregate base courses, the fine material (smaller than about 75 μm) comprises no more than about 10 percent of the of the entire mixture by weight, the amount of lime used by weight of the total aggregate base will be considerably less than that used in soils. Normally, the amount of lime used in base stabilization is between about 1 percent and 3 percent by total weight of the aggregate base.

Adding lime to the fines matrix will decrease plasticity as well as increase strength, and it can generally be surmised that if an acceptable target strength is achieved, that the plasticity of the

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fines will be appropriately altered as well. However, it is prudent to test the plasticity of the minus no. 40 sieve fraction with the target lime content to verify the impact of lime on the plasticity of the fines.

The steps for stabilization of a base course are: (1) add the appropriate target percentages of lime (generally starting with 1 percent by weight of the entire mixture and increasing in 1 percent increments to 4 percent), (2) determine moisture density relationships for each aggregate-lime blend following either AASHTO T-99 or AASHTO T-180 based on agency requirements, and (3) determine unconfined compressive strengths of the lime-aggregate blends following curing for 7-days at 40^0C (104^0F) followed by capillary soak as described in the NLA protocol for lime-soil mixtures. The compressive strength testing procedure and target compressive strength requirements should be based on specifications defined by the user agency.

Cement Stabilization

The American Concrete Institute (ACI) defines soil cement as a mixture of soil and a measured amount of cement and water mixed to a high density (*25*). Soil cement has been classically defined as a stabilized soil in which the coarse aggregate, sand size and larger (coarser than 75 μm) is surrounded and bonded by a matrix of cement paste and fine soil particles. The goal of mix design for this type of soil is to float the coarse aggregate in the matrix. The durability of this matrix is determined by durability tests such as AASHTO T 135 and T 136 (or by their ASTM equivalents D 559 and D 560) or by compressive strength testing. However, Portland cement has also been successfully used to stabilize fine grained silt and clay soils. In fact cement stabilization of silty soils provides perhaps the most dramatic improvement of any soil type (when the properties of the cement treated silty soil are compared to the properties of untreated soil). However, the amount of cement required to stabilize fine grained soils can be substantially more than that required to stabilize coarse grained soils because of the higher surface area of fine grained soils. The transition from silt to clay means that the particle surface area increases by orders of magnitude. However, in actuality cement does not need to coat all particles for successful stabilization and substantial improvement of moderately plastic clay soils, plasticity indices of below 30, has been achieved with about the same amount of Portland cement as would be required of hydrated lime. This is primarily because the cement forms a stabilized matrix around agglomerates of clay particles. Obviously if the integrity of cement matrix surrounding the agglomerates is compromised, then the durability of the matrix will begin to degrade.

The ability to stabilize soils with plasticity indices above about 20 with cement is based on the ability to intimately mix cement with the soil to a degree that will produce a reasonably homogeneous and continuous, stabilized matrix of the agglomerates. This requires a certain efficacy of mixing, which is in turn associated with the energy imparted to the soil by the mixing equipment and by the time span over which mixing occurs. The limitation associated with mixing Portland cement with plastic clay soils is the short time of initial set of the cement, usually not more than 2 hours is provided for mixing before compaction. However, this mixing time has been extended under certain circumstances. During the extended mellowing period, the release of free lime during cement hydration alters plasticity and textural properties of the clay soil, which can improve workability. However, mixing following this extended mellowing must be performed with equipment that has the ability to impart sufficient energy to mix the soil and cement after the cement has reached a final set, which normally occurs within 8 hours. It must be

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understood, when extended mellowing is adopted, that all the strength lost during remixing may not be recovered with additional curing.

Hardened soil cement mixtures must withstand adverse environmental conditions. Other stabilization objectives include reducing plasticity index, increasing shrinkage limit, meeting strength thresholds, and improving resilient modulus. Soil cement can provide a strong and uniform support for pavement layers and provide a firm and stable working platform for construction.

In summary, most soil types, except those with high organic content, highly plastic clays and poorly reacting sandy soils, are amenable to stabilization with Portland cement. General gradation specifications limit the nominal maximum size at 2-inches with at least 55 percent passing the no. 4 sieve. For uniformly graded materials, the addition of non plastic fines like fly-ash, aggregate screenings, cement and lime kiln dust may help fill the voids in the soil structure and help reduce the required cement content

Mix Design Considerations

As with lime stabilization, soils must be screened for organic content and sulfate content prior to verifying whether Portland cement is an acceptable stabilizer. Soils with higher organic content may require a higher cement content as the organic matter can inhibit normal hardening processes. A pH test, as recommended by the U. S. Army Corps of Engineers, using a mixture of 10 parts soil to one part cement (by weight) is used to verify if organic matter might interfere with the hydration process (6). If the pH of the paste after 15 minutes of mixing is 12.0 or higher then it is probable that organics will not interfere with the normal hardening process. If not, then a higher cement content than that recommended based on AASHTO soil groups (Table 3) may be needed. Again the required cement content must be confirmed based on strength testing. The following procedure outlines the steps to be followed in developing an effective mix design for cement stabilized soils.

Preliminary Estimate of Cement Content

The first step in determining the required cement content is to classify the soil, AASHTO M 145. Table 3 defines a starting point to be considered in treatment. These cement contents are based on a data base of empirical evidence of soil cement mixtures that have proven to be able to meet the durability requirements established in AASHTO T 135 and T 136 or their respective ASTM equivalents D 559 and D 560. In Table 3, the cement quantities are proportioned on a weight basis in terms of the percent of oven dry soil.

Table 3. Cement requirement for AASHTO soil Groups (26).

AASHTO Soil Group	Usual Range in Cement Requirement		Estimated Cement Content, Percent by Weight
	Percent by Volume	Percent by Weight	
A-1-a	5-7	3-5	5
A-1-b	7-9	5-8	6
A-2	7-10	5-9	7
A-3	8-12	7-11	9
A-4	8-12	7-12	10
A-5	8-12	8-13	10
A-6	10-14	9-15	12
A-7	10-14	10-16	13

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These cement contents are only preliminary estimates and must be verified or modified based on additional test results. Additional cement requirement for soils with higher organic contents should be considered based on pH test of soil cement mixtures (6). It is important to understand that the requirements in Table 3 are based on durability tests, ASTM D 559 and D 560, and that many soils can be successfully stabilized with considerably lower cement contents.

Determine the Moisture Density Relationship

Changes in optimum moisture content and dry density with addition of cement are not always predictable (*4*). Flocculation of clay particles by cement can cause an increase in optimum moisture content and decrease in maximum dry density for cement-soil mixes whereas the higher density of cement relative to soil can result in a higher density for mixes. Therefore, it is appropriate to use the median cement content as estimated in Table 3 for determination of moisture density relationships as the maximum dry density varies only slightly with modest changes in percent cement content (*26*). However, as previously discussed, if it is expected that acceptable treatment can be achieved with considerably lower cement contents than those in Table 3, then that cement content should be used to determine the moisture-density relationship. After the required amount of cement is added to the soil, the blend should be mixed thoroughly until the color of the mixture is uniform. Fabrication and testing of samples for moisture density relationship should be done in accordance with AASHTO T 134 or its ASTM equivalent D 558.

Sample Preparation for Compressive Strength and Durability Testing

Two types of tests are typically used to evaluate the efficacy of a soil cement mixture: strength tests and durability tests. The Portland Cement Association (PCA) considers the ability to withstand adverse environmental conditions as the primary requirement for soil cements (*26*). The PCA manual recommends durability tests based on weight loss under wet-dry and freeze-thaw conditions for evaluating usability of soil cement mixtures. Both PCA and ACI determine the weight loss in samples subjected durability tests in accordance with ASTM D 559 or ASTM D 560 as appropriate. These methods are highly subjective and carry significant user variability. In addition, these test methods may not reflect field conditions that are applicable to all stabilized pavement layers. In flexible pavements the soil cement base is protected at the surface by a hot mix bituminous wearing surface and in rigid pavements by a concrete slab. Hence the extent of damage in pavement layers due to freeze-thaw activity will vary significantly depending not only on climate but also the pavement structure. Healing of micro cracks in the stabilized layers with time may also influence the extent of damage in field (*27*). This effect is not reflected in the recommended freeze thaw test criteria. It is most important to consider that the depth of penetration and the number of freeze-thaw cycles to which the pavement layer is exposed varies considerably from site to site.

Since the results of freeze thaw testing does not simulate field conditions, many state departments of transportation currently recommend minimum unconfined compressive strength testing based on ASTM 1633 in lieu of durability tests (*3*). The research work by Thomson and Dempsey in lime stabilized soils has shown that compressive strength of samples subjected to freeze thaw can be used as a criteria in deciding durability issues in soil cements (*28*). Thompson's data demonstrate that the compressive strength decreases by approximately 8-10 psi for every freeze thaw cycle endured. The U. S. Army Corps of Engineers recommends using 12 freeze-thaw cycles as described by ASTM D 560 (but omitting the wire brushing part) for

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cement modified soils. This method may also be considered an alternative method by which to assess the durability of cement stabilized soils.

Whether the cement requirements in Table 3 are used or alternative cement requirements are used, cement contents above and below the nominal value of cement should be considered. Therefore, the accepted approach is to prepare mixtures at the nominal stabilizer content and two percent above and below the nominal content. Again, the samples should be prepared following AASHTO T 134.

Unconfined Compressive Strength Testing

Compressive strength is indicative of the degree of reaction in the soil-cement-water mixture based on the rate of hardening of the mixture. Since the compressive strength is directly related to density, it is affected by the degree of compaction and water content in soil cement. Similar to lime stabilization, moisture conditioning of cement-soil mixtures is recommended prior to testing as most soil cement structures are either intermittently or permanently saturated during their service life. Preparation and curing of samples compressive strength testing should be performed in accordance with ASTM D 1632 which recommends moist cure for soil cement samples. Testing of cured samples should be done following ASTM D 1633 that requires the cured samples to be immersed in water for 4 hours prior to testing (*6*). Typical ranges of unconfined compressive strength criteria of moisture conditioned soil cement specimens for varying soil classifications are given in Table 4.

Table 4. Range of compressive strength in soil cements (*29*).

Soil Type	AASHTO Classification	Soaked Compressive Strength (psi)	
		7 Days	28 Days
Sand and gravelly	A-1, A-2, A-3	300-600	400-1,000
Silty	A-4, A-5	250-500	300-900
Clayey	A-6, A-7	200-400	250-600

Strength requirement for stabilized layers may vary considerably from agency to agency. The required compressive strengths for soil cements shown in the Table 4 are based on ACI and the U. S. Army Corps of Engineers recommendations (*4, 6*).

Strength criteria are presented in Table 5 are based on the experience of the U. S. Army Corps of Engineers and the ACI. The lowest cement content in the mixture design that meets the requirements in Table 5 should be used as the design content. If the selected samples does not confirm to the recommendations, then higher cement contents may be added to soil and strength and durability tests may be repeated till the strength values confirm to the requirements.

Table 5. U.S Army Corps of Engineers unconfined compressive strength criteria (*6*).

Purpose of Stabilized Layer	Minimum 7 day Unconfined Compressive Strength (psi)	
	Flexible Pavement	Rigid Pavement
Base Course	750	500
Sub base, select material or subgrade	250	200

The typical minimum requirement varies from around 200 psi for sub base layers to around 750 psi for base layers (*6*).

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Cement Treatment of Base Courses

The protocol described above addresses cement-soil and cement-base mixtures. However, in certain situations a lower level of cement is used to achieve a target increase in compressive strength and/or modulus for structural performance reasons. In that case target quantities of Portland cement should be added to the aggregate material and the compressive strength or modulus of the cement-soil mixture should be evaluated. As in the discussion of lime treatment of aggregate bases, it is assumed that the candidate aggregate base material is of at least moderate quality, otherwise, the material should be treated as a soil. Moderate quality is defined as: (1) not more than 20 percent finer than the no. 40 sieve (0.425 mm or 0.0165 in.), (2) a maximum plasticity index of 12 percent, and (3) a maximum liquid limit of 40 percent.

Since in aggregate base courses, the fine material (smaller than about 75 μm) comprises no more than about 10 percent of the of the entire mixture by weight, the amount of cement used by weight of the total aggregate base will be considerably less than that used in soils. As with lime, adding cement to the fines matrix will decrease plasticity as well as increase strength, and it can generally be surmised that if an acceptable target strength is achieved, that the plasticity of the fines will be appropriately altered as well. However, it is prudent to test the plasticity of the minus no. 40 sieve fraction with the target cement content to verify the impact of cement on the plasticity of the fines.

The steps for stabilization of a base course are: (1) add the appropriate target percentages of cement (generally starting with 1 percent by weight of the entire mixture and increasing in 1 percent increments to 3 percent), (2) determine moisture density relationships for each aggregate-cement blend following AASHTO T-99 or AASHTO T-180 based on agency requirements, and (3) determine unconfined compressive strengths of the cement-aggregate blends following most cure for 7-days followed by 4 hours soak as recommended by ASTM D 1633. The compressive strength testing procedures and the target compressive strength requirements should be based on specifications defined by the user agency.

Fly Ash Stabilization for Coarse Grained Soils and Aggregates

Fly ash typically contains at least 70 percent glassy material with particle sizes varying from 1μm to greater than 1 mm. Based on AASHTO M 295 (ASTM C 618), fly-ash can be classified into two groups: class C and class F. Class C refers to as a self cementing or cementitious fly ash that has enough available calcium to react with soil in the presence of water. Most of the calcium in class C fly ash is combined with the silica and/or alumina so that when water is added, a hydration reaction similar to the hydration reaction in Portland cement occurs. Some free lime is produced in the hydration process, as it is in the hydration of Portland cement. This free lime can participate in the pozzolanic reaction process between silica and/or alumina released from clay or silica and/or alumina from the fly ash, which are not combined with calcium. Class C fly ash is a by-product of burning lignite or sub-bituminous coal in power plants. Class F fly ash on the other hand is more of a pure pozzolan, with a low concentration of available calcium. Therefore stabilization with class F fly-ash requires the use of an activator like lime or cement to initiate hardening processes during stabilization (5). Low lime ash or class F fly ash is formed during burning of anthracite or bituminous coal. Although these fly ash types are known to induce cementitious reactions in stabilized soils, mix properties cannot be predicted solely from chemical composition of the ash. Due to the complex nature of ash hydration, the utility of fly ash for stabilization applications must be based on physical properties of ash treated materials.

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Mix Design Considerations

Prior to stabilization, the cementitious properties of fly ash should be characterized following ASTM D 5239-04. But, it should be noted that ASTM D 5239-04 does not evaluate the interaction between fly ash and soil or aggregate which must to be verified separately based on mix design procedures outlined in the following paragraphs.

Self Cementing Fly Ash

Class C fly ash can be used as a stand alone material. At present there are no standard test procedures available for design of materials stabilized with self-cementing fly ash. The American Coal Ash Association recommends using moisture density and moisture strength relationships for developing effective mix designs in soils (*5*).

Design Considerations

For self cementing fly ash, one of the primary design considerations is the rate at which fly ash hydrates upon exposure to water. Hydration reactions can start immediately on exposure to water and hence the time delay in mixing and compaction of the specimens needs to be accounted for and included in laboratory mix designs (*5*). As hydration progresses, soil particles are bonded in a loose state and a portion of the compaction energy used in densification is lost in breaking bonds in the mix. Maximum dry density achieved for a given compaction energy therefore decreases with increase in compaction delay. In addition, compaction delay can cause a significant reduction in compressive strength. This is most likely due to the inability to maximize the impact of cemetitious and/or pozzolanic product development at lower densities. In other words, if a soil mass is under-compacted, the cemetitious/pozzolanic product does not have the same opportunity to develop "bonds" among soil particles (or agglomerates of particles) as they would if the soil mass were compacted to within a reasonable range of target density. This effect is much more likely to be significant in class C fly ash mixtures due to the faster rate of reaction.

An additional design consideration when selecting the optimal fly ash content is to determine the optimum moisture content at which maximum strength gain is achieved. Optimum moisture content for strength gain may typically be 1–8 percent below optimum moisture content needed to attain maximum dry density. This value may vary with soil type and the mineralogy of ash particles.

Mix Design

Addition of fly ash alters the compositional characteristics of soils and hence the moisture density relationship must be established for each soil type and fly ash content. These can be measured based on adaptations of ASTM C 593 and ASTM D 1633.

Once the optimum moisture content for the mix is determined, the moisture-strength relationship is evaluated. In order to evaluate strength, specimens are prepared by blending soil, fly ash, and water and molded after the specified compaction delay. Test specimens are compacted at different moisture levels below optimum to determine the moisture content that will produce the maximum compressive strength. Test specimens are cured for 7 days at $38^{o}C$ ($100^{o}F$) in accordance with ASTM C 593 before compression testing. ASTM C 593 recommends moisture conditioning for 4 hours after curing period where the specimens are allowed to cool down to room temperature and are then immersed in water for 4 hours. However, as with lime mixtures, the authors recommend an alternative moisture conditioning regime of capillary soak until the moisture front ceases to migrate or for a minimum of 24-hours. The strength requirements

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typically vary based on objectives and requirements specified by the agency and these requirements should be followed in selecting the mix design for field application.

Non Self Cementing Fly Ash

For stabilization with non self cementing fly-ash, the addition of activators such as lime or cement is required to initiate stabilization reactions. These materials typically continue to gain strength after a curing period due to pozzolanic activity. The slow strength gain in these materials helps reduce shrinkage cracking and improves healing of micro cracks forming in the stabilized layers.

The methodology given below is adapted from coal ash association mix design procedure. Typical fly ash contents in granular mixes vary from about 10-15 percent with activator contents varying from about 2-8 percent lime by weight of the mixture. These materials are similar to cement stabilized base in production, placement and in appearance. Strength development depends on curing time and temperature and is typically measured after accelerated curing of 7 days at 38^0C (100^0F) (*30*).

Selection of Optimal Fly Ash Content

The first step in selecting the optimal fly ash content is to determine the utility of the stabilized product and the target level of strength required based on the utility of the product. The purpose of using fly ash in soils can broadly be divided into two categories: to achieve maximum strength for the mix or to achieve a target level of strength for the mix. If an aggregate base course is to be stabilized and the goal is to achieve maximum strength and durability then the strategy is to fill the voids with fly ash to achieve maximum density, then to determine the moisture density relationship for this optimal blend. This is followed by the addition of the amount of activator that will produce the maximum level of strength. If the goal is to achieve a target level of strength for either base courses or soils, then the strategy is different. In this case, experience or a trial and error process is required to identify trial fly ash percentages and activator contents. These estimates are used to establish moisture density relationships and to determine compressive strengths.

Five different samples are prepared with varying fly ash proportions typically starting at about 6 percent and ranging up to as high as about 20 percent by weight of the coarse aggregate fraction. Mixes are molded at estimated optimum moisture content in accordance with ASTM C 593 to determine the dry density of each mix. A two percent fly ash concentration above the proportion that gives the maximum dry density is selected as the optimum content for the mix. Optimum moisture content and maximum dry density are determined for the selected blend.

Sample Preparation for Selection of Optimal Activator Content

Determination of the optimal activator content is best achieved on a trial and error basis realizing that the required lime content or Portland cement content to activate the fly ash is typically between one part lime to three parts fly ash (1:3 ratio) to one part lime to four parts fly ash (1:4 ratio). Compressive strengths of the resulting mixture should then be compared to target values in order to judge whether or not the blend produces acceptable strengths for loading and environmental conditions.

If lime kiln dust (LKD) or cement kiln dust (CKD) is used as an activator, then higher activator ratios are required based primarily on the CaO content of the kiln dust. ASTM C 593 requires

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preparation of three replicate samples for compressive strength testing for each blend of fly ash and activator.

Curing of Samples for Compression and Durability Testing

Fly-ash soil mixes are cured for 7-days in sealed containers. Samples prepared with lime and kiln dust activators are cured at 37.8^{o}C (100° F) for seven days. Portland cement activator fly ash mixes are cured at a 100 percent relative humidity environment at 22.8^{o}C (73° F) for seven days. ASTM C 593 recommends moisture conditioning following curing period in which the samples are subjected to 4 hour soak after cooling to room temperature. Then the compressive strength of the samples is measured. However, the authors recommend the NLA capillary soak described under the NLA recommendations for lime mixtures as an alternative moisture conditioning regime.

Compression and Durability Testing

The three replicates prepared are tested for compressive strength testing should be subjected to vacuum saturation or strength testing without moisture conditioning as recommended in ASTM C 593. Durability testing in fly-ash soil mixes can also be performed in accordance with AASHTO T 136/ASTM D 560. But the issues discussed earlier regarding the effectiveness of AASHTO T 136/ASTM D 560 are applicable in this case also. In areas where there is no freeze-thaw effect, durability testing may be waived in accordance with local practice.

Acceptability Criteria

A 7-day compressive strength of 400 psi is considered acceptable for field applications (*30*). A mix that attains the required properties with the lowest percentage activator is selected as the design mix for use in field.

Lime-Fly Ash Treatment of Soils to Achieve a Target Strength

Lime and fly ash may be used to achieve mixtures with a target strength instead of in an attempt to optimize strength of a mixture. This approach may be applied to any soil (coarse-grained or fine-grained). In this case various ratios of lime and fly ash should be tried until the target strength is achieved. A reasonable guideline is to begin with is to use four percent fly ash and increase the ash content in two percent increments for various trials. The initial trial activator ratio (lime content) added to each should be one part lime to two parts fly ash as a general rule, but this can be varied based on experience.

Approximately six trial ratios of lime and class F fly ash (three ash contents and two activator contents per ash content) should be used. A moisture-density relationship should be developed for each ratio to determine optimum moisture for each blend. Samples should be prepared at the target moisture content following ASTM C 593. Strength testing on each candidate mixture should be used to establish the acceptable mixture design. The authors recommend the same curing regime as described for lime stabilized soils.

On occasion, the goal is for fly ash to provide a strength increase to lime treated, fine-grained soils that are not sufficiently pozzolanically reactive. This normally occurs in clay soils where the lime is effective in reducing plasticity and improving workability but not in providing the target strength. In this case an acceptable approach is to determine the lime content required based on the Eades and Grimm pH test. This content should provide sufficient lime to modify the soil and still provide sufficient residual lime to provide pozzolanic reaction. Next trial

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quantities of fly ash should be added to the blend beginning with four percent fly ash and increasing in two percent increments until acceptable strength is achieved. A separate moisture-density relationship is required for each blend.

SUMMARY

The process of chemical modification or stabilization with calcium-based chemicals requires a basic understanding of the mechanisms of reaction. Each calcium-based stabilizer contains some amount of free lime (CaO or Ca(OH)$_2$) that reacts pozzolanically with the fine particles (clay and some silt). Normally, lime is 90 percent or more CaO or Ca(OH)$_2$ and therefore provides the most available free lime over the longest period of time in the treatment process of any of the traditional stabilizers: lime, Portland cement, and fly ash. However, Portland cement and class C fly ash also release lime during the hydration of calcium silicates and calcium aluminates. This lime participates in the pozzolanic reaction mechanism with soil fines (primarily clay).

As against lime, Portland cement and class C fly ash possess calcium-silicate and calcium-aluminate compounds that hydrate upon the addition of water. The resulting cementitious products bond soil particles together and develop strength. The kinetics of these cementitious reactions is rapid. This can be a draw back in the treatment of plastic clay soils because these soils require time for diffusion of calcium into the soil matrix and mechanical manipulation in order to properly modify or stabilize them. The rapid cementitious reactions cut down on the time available to manipulate the soil-stabilizer blend. This available "mellowing" period makes lime a preferred additive, by many, when dealing with moderately and highly plastic clay soils.

The standard of practice that will result from this study defines the following steps in modification or stabilization: selection of stabilizer based on soil index properties (primarily percent fines, smaller than 75 μm, and plasticity index), method of soil exploration and investigation required to support stabilizer selection and mixture design, and verification of stabilizer selection. However, the fundamentals of the mechanisms of stabilization should also be considered to recognize the variability among soils during treatment with the traditional additives. Soil exploration steps identify how readily available resources such as the National Resource Conservation Service (NRCS) soil surveys and U. S. Geological Survey reports can be used to plan a soil exploration program to support the stabilization process and to help deal with potentially problematic conditions such as high moisture contents due to fluctuating water levels and problematic composition such as high sulfate contents and high organic contents. If high sulfate contents are encountered, then the Standard on Stabilization of Sulfate-Bearing soils, also a product of this study, should be used.

The verification process varies depending whether the goal of treatment is modification or stabilization. Modification refers to treatment of the soil with the selected stabilizer in order to reduce plasticity and improve workability. The associated reactions for modification must be relatively rapid. The verification process is then simply to add the amount of the selected additive that will achieve the properties desired. Stabilization, on the other hand, requires not only immediate improvement but also long term strength and durability. Stabilization requires a more detailed and sophisticated verification protocol for which a structured mixture design protocol is included. The mixture design protocol for each stabilizer includes an initial approximation of the appropriate stabilizer content either based on an empirical database or a screening test, such as the Eades and Grimm pH test. This is followed by strength testing where the critical conditions expected in the field are simulated in the testing laboratory. Since it is normally beyond the scope of stabilizer selection and testing to mimic moisture and environmental variations over the year, a critical condition is normally simulated by partially saturating the sample. The method and degree of this "moisture conditioning" process is based

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on experience and varies among design agencies. A capillary suction diffusion process or "capillary soak" is favored by the authors for lime-soil mixtures as recommended by the National Lime Association. The normal moisture conditioning protocol for Portland cement stabilization includes moist curing and a 4-hour soak before strength testing. A similar soaking protocol is suggested by the American Coal Ash Association for fly ash stabilization of aggregates. However, when lime and fly ash are used in combination to treat soils, the authors recommend capillary soak is required for lime-soil-fly ash mixtures.

The authors recommend strength testing after moisture conditioning as the appropriate metric of determining strength and durability for all stabilization processes.

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Standard Recommended Practice for

Stabilization of Subgrade Soils and Base Materials

AASHTO Designation: R Draft (2008)

American Association of State Highway and Transportation Officials
444 North Capitol Street N.W., Suite 249
Washington, D.C. 20001

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Standard Recommended Practice for

Stabilization of Subgrade Soils and Base Materials
AASHTO Designation: R Draft (2008)

INTRODUCTION

Engineering design of pavement structures relies on the assumption that each layer in the pavement possesses the minimum specified structural quality to support and distribute the super imposed loads. But the available earth materials do not always qualify to be used directly as a construction material, but instead may require modification to improve their engineering properties in order to achieve the target strength requirements set for pavement materials. An economical way of addressing these strength deficiencies can be through chemical modification or stabilization. Successful stabilization of soils depends on the physio-chemical properties of soil as the soil-stabilizer interactions can vary with soil composition. This is particularly important if the treatment is performed with the intent of achieving long term benefits. Soil-stabilizer interactions are complex and may vary among soils due to heterogeneity in soil composition, differences in micro and macro structure of soils, heterogeneity of geologic deposits and differences in physical and chemical interactions between soil particles and additives.

Stabilization projects are site specific and require integration of standard test methods, analysis procedures and design steps to develop acceptable solutions. This recommended practice provides a simplified protocol to be followed in selecting the appropriate calcium-based stabilizer and achieving a viable and economic design for the use of the selected stabilizer for a specific subgrade soil or base material.

SCOPE

This standard of practice discusses the techniques to successfully achieve the required engineering properties for individual soil groups. This recommended practice also addresses the basic mechanism(s) of stabilization when using lime, Portland cement and fly ash; soil exploration and sampling techniques; guidelines and techniques for identifying an effective additive for individual soil types; and techniques for validating the selection of the stabilizer through mixture design and testing. Construction practices are not addressed in this document.

This standard may involve hazardous materials, operations and equipments. This standard does not address all the safety problems associated with their use. It is the duty and responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

REFERENCED DOCUMENTS

AASHTO Standards:

- M 295, Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
- R 13, Conducting Geotechnical Subsurface Investigations

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- T 27, Sieve Analysis of Fine and Coarse Aggregate
- T 87, Dry Preparation of Disturbed Soil and Soil Aggregate Samples for Test
- T 89, Determining the Liquid Limit of Soils
- T 90, Determining the Plastic Limit and Plasticity Index of Soils
- T 99, Moisture Density Relationship of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop
- T 134, Moisture Density Relations of Soil-Cement Mixtures
- T 136, Freezing and Thawing Tests of Compacted Soil-Cement mixtures
- M 145, Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes
- T 180, Moisture Density Relationship of Soils Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop
- T 248, Reducing Samples of Aggregates to Testing Size
- T 290 (modified), Determining Water-Soluble Sulfate Ion Content in Soil
- R(Draft), New AASHTO Standard Recommended Practice for Stabilizing Sulfate Rich Soils

ASTM Standards:

- E 105, Standard practice for Probability Sampling of Materials
- E 122, Standard Practice for Calculating Sample Size to Estimate, With Specific Precision, the Average for a Characteristic of a Lot or a Process
- E 141, Standard Practice for Acceptance of Evidence Based on the Results of Probability Sampling
- D 559, Standard Test Method for Wetting and Drying Compacted Soil-Cement Mixtures
- D 560, Standard Test Method for Freezing and Thawing Compacted Soil-Cement Mixtures
- C 593, Standard Specification for Fly Ash and Other Pozzolans for Use With Lime for Soil Stabilization
- D 1632, Standard Practice for Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory
- D 1633, Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders
- D 2974, Standard Test Method for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils
- D 3551, Standard Practice for Laboratory Preparation of Soil-Lime Mixtures Using Mechanical Mixer
- D 3665, Standard Practice for Random Sampling of Construction Materials
- D 5102, Standard Test Method for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures
- D 5239, Standard Practice for Characterizing Fly Ash for Use in Soil Stabilization
- D 6276, Standard Test Method for Using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization

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TERMINOLOGY

- Soil - All deposits of loose material on the earth's crust that are created by weathering and erosion of underlying rocks.
- Coarse-grained soils - Soils with less than 50 percent of the mass passing 75 μm sieve.
- Fine-grained soils - Soils with more than 50 percent of the mass passing 75 μm sieve.
- Kinetics - Rate of progress of a chemical reaction.

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- ppm - Parts per million or milligram per liter; concentration of solute, sulfates, in milligrams per liter of solvent, water.

- Isomorphous substitution - Substitution of one atom by another of similar size within a crystal lattice and without disrupting the crystal structure of the mineral.

- Short-term strength - Strength derived immediately, within about 7-days of compaction.

- Modification - Improvement that occurs in the short term, during or shortly after mixing (within hours). This modification reduces the plasticity of the soil (improves the consistency) to the desired level and improves short-term strength to the desired level. Even if no significant pozzolanic or cementitious reaction occurs, the textural changes that accompany consistency improvements normally result in measurable strength improvement.

- Stabilization - A longer term reaction that is derived from the hydration of calcium-silicates and/or calcium aluminates in Portland cement or class C fly ash or due to pozzolanic reactivity between free lime and soil pozzolans or added pozzolans. A strength increase, when treated with a stabilizer, of at least 50 psi greater than that of the untreated soil fabricated and cured under the same conditions is used in this document to consider soils to be a stabilized material.

- Soil cement - Stabilized soil in which the coarse aggregate, sand size and larger, is surrounded and bonded by a matrix of cement paste and fine soil particles.

SIGNIFICANCE AND USE

Although stabilization is an effective alternative for improving soil properties, the engineering properties of individual soils vary widely with changes in soil composition. Soil-stabilizer interactions also change with soil composition as does the extent of improvement of soil properties. Therefore, the ability to identify the most appropriate stabilizer for use with an individual soil and a thorough understanding of the techniques needed to validate the selection of a stabilizer are critically important to the success of a stabilization project. This recommended practice provides the information needed for stabilizer selection and validation of that selection through mixture design and field testing.

BACKGROUND

Soil texture is largely related to its appearance and is dependent on the size, shape and distribution of particles in the soil matrix. Soil particle sizes may vary from boulders or cobbles, roughly a meter in diameter, to very fine clay particles, roughly a few microns in diameter.

Strength and stiffness development in coarse-grained soil fractions is primarily dependent on physical interlocking of particles and varies in degree with the size, shape and relative amount of coarse-grained particles present.

The behavior of finer fractions, silts and clays, is influenced more by electro-chemical and physio-chemical properties and varies with the quantity and type of individual minerals present in the soil. This is largely due to the high specific surface area of the fine grained fractions.

Weathering processes impact soil composition creating variability in surface soils. These processes primarily influence the composition of fine fractions in soil as these fine fractions are easily transported away from parent rock formations. These transport processes often result in mixing of soil minerals and may

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introduce salts or organic material of a variety of species and concentrations. These processes create differences in soil compositions within soil groups which can influence the behavior of individual soils.

Due to the higher specific surface area of fine soil fractions, they are more reactive in the presence of chemical modifiers when compared to coarse fractions in the soil.

Among these fine fractions, the specific surface area of clay particles is orders of magnitude larger than that of silt fines in soil. This difference is part of the reason that clay particles are more reactive than silt particles.

Clays also exhibit varying levels of consistency and engineering behavior and demonstrate various levels of plasticity and cohesiveness in the presence of water. Clay minerals have a unique sheet particle structure and a crystalline layer structure that is amenable to significant isomorphous substitution. As a result of this isomorphous substitution, clay mineral surfaces carry a significant negative surface charge that can attract positively charged ions and dipolar water molecules.

The cumulative effect of high surface area and surface charge makes clay particles particularly reactive with water, and is the root cause of the propensity of clay particles to shrink and swell depending on the availability of water.

Due to a higher reactivity of fine fractions in soils, altering their physio-chemical properties by using chemical stabilizers/modifiers is often a more effective form of durable stabilization than mechanical stabilization in subgrade soils where the concentration of finer fractions is dominant. The fines content can become significant when as little as 10 percent of the soil is comprised of fines, smaller than about 75 μm.

Soil characteristics including mineralogy, gradation and physio-chemical properties of fine-grained particles can all influence soil-additive interactions. Hence stabilizer selection should be based on the effectiveness of a given stabilizer in improving the physio-chemical properties of the selected soil.

MECHANISMS INVOLVED IN CHEMICAL STABILIZATION

Chemical stabilization using traditional, calcium-based stabilizers involves mixing or injecting the soil with chemical compounds such as Portland cement, lime or fly ash.

Traditional stabilizers generally rely on pozzolanic reactions and cation exchange to modify and/or stabilize soil properties.

Lime-soil reactions are complex and involve multiple, synergistic processes. These reactions can broadly be grouped into two parts: initial and longer-term. The initial reactions involve cation exchange and flocculation/agglomeration of soil particles that result in textural and plasticity changes in the soil. These

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processes also make the soil more friable and workable. Longer-term reactions involve interactions between free lime ($Ca(OH)_2$) and soil particles. These interactions are referred to as pozzolanic as they involve pozzolans, the alumina and silica made available from the soil by the high pH lime-water solution. When these pozzolans react with free lime and water, a cementing effect among particles as well as an alteration of surface mineralogy occurs. These pozzolanic reactions contribute to an increase in strength which can be considerable depending on the degree of pozzolanic reaction in lime-soil mixtures. These pozzolanic reaction products, calcium-silicate-hydrates and calcium-aluminate-hydrates, are similar to the cementitious products formed when Portland cement hydrates.

These pozzolanic reaction processes are slow when compared to flocculation/agglomeration reactions in soil. Therefore, mellowing periods, normally about one-day, but up to about 4-days, are sometimes prescribed to maximize the effect of short term reactions in reducing plasticity and increasing workability. The mellowing period also affords time for re-mixing after initial reactions have taken place. This can result in more intimate mixing and a more thorough degree of modification prior to compaction. However, the pozzolanic reaction process may progress relatively quickly in some soils depending on the reactivity of the soil minerals with lime or if a cementitious hydration reaction, such as with Portland cement or some class C fly ashes, accompanies the pozzolanic reaction. In this case, the pozzolanic reaction as well as cation exchange contributes to plasticity reduction. In fact, long term plasticity reduction is primarily due to pozzolanic effects.

The extent of formation of pozzolanic reaction products depend primarily on the rate and degree of dissolution of the soil minerals from the soil matrix. The pozzolanic reaction process can therefore be modest or quite substantial depending on the mineralogy of the soil. Maintaining a high enough pH condition, generally agreed to be above 10.5, is required in order to solubilize soil pozzolans that participate in these reactions.

Portland cement is comprised of calcium-silicates and calcium-aluminates that hydrate to form cementitious products. This cementitious reaction is the primary mode of strength gain in soil cement. Free lime, $Ca(OH)_2$, produced during the hydration process can comprise up to about 25 percent of the cement paste (cement and water mix) on a weight basis. This free lime can produce a concomitant pozzolanic reaction between the lime and soil, which can continue as long as the pH is high enough to solubilize the soil minerals.

Cement hydration is rapid and causes immediate strength gain in stabilized layers. Therefore, a mellowing period is not typically allowed between mixing and compaction. The general practice is to compact soil cement before or shortly after initial set, preferably within 2 hours of mixing.

An extended mellowing period, beyond 2 to 4 hours, may be acceptable if an improved uniformity of the mix is required. The soil-cement mixture, in this case, should be

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remixed after the mellowing periods to achieve a homogeneous mixture before compaction. The ultimate strength of a soil-cement mixture with an extended mellowing period may be lower when compared to mixtures where compaction is achieved before initial set. It is important to realize that intimate mixing after extended mellowing may require equipment with more power than is normally used in mixing due to the cementitious effect.

Traditional stabilizers such as fly ash and by-product stabilizers like lime kiln dust and cement kiln dust also rely on pozzolanic reactions and cation exchange to modify and/or stabilize soil properties. Each of these by products may be highly variable.

Based on AASHTO M 295, fly ashes can be classified as either class C (self-cementing) or class F (non-self cementing) fly ash.

Class C fly ash contains a substantial amount of lime, CaO, most of which is combined with glassy silicates and aluminates in the ash. Upon mixing with water, hydration reactions produce free lime that either can combine with other unreacted pozzolans, silicates and aluminates, available within the fly ash, or may react pozzolanically with silicates and aluminates available in soil to form cementitious reaction products. Formation of these cementitious products contributes to strength gain in fly ash stabilized soils. Pozzolanic reactions between soil particles and free lime released from the ash during the hydration process can alter soil properties and increase strength just as they do in soil cement.

Class F fly ash contains very little lime, CaO, when compared to a class C fly ash and most of the glassy silica and/or alumina exists as pozzolans in the ash. Activation of these pozzolans requires additives such as Portland cement or lime, which provide a sufficient source of free lime. The pozzolanic reactions that occur when this fly ash-activator blend is mixed with water form the products that bond soil grains or agglomerates the soil particles together to develop strength within the soil matrix.

Kinetics of the cementitious reactions and pozzolanic reactions that occur in fly ash stabilized soils vary with the type of ash and with the composition of ash used in stabilization. Therefore, the allowable compaction time for fly ash soil blends vary with the type of ash and depends on whether or not an activator is used. However, the standard practice is to compact the mixture within 6 hours of initial mixing.

Lime kiln dust and cement kiln dust are by-products formed during the production of lime and Portland cement. As such they may be highly variable.

Lime kiln dust (LKD) normally contains between about 30 to 40 percent lime which may either be free lime or combined with pozzolans in the kiln. LKDs may be somewhat cementitiously and/or pozzolanically reactive because of the presence of pozzolans or they may be altogether non reactive due to the absence of pozzolans or due to the low quality of the pozzolans contained in the LKD.

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Cement kiln dust (CKD) generally contains between about 30 and 40 percent lime and about 20 to 25 percent pozzolanic material. CKD is more likely to contain reactive calcium-silicates and/or calcium-aluminates and/or pozzolans when compared to LKD and therefore may be able to support some level of cementitious and/or pozzolanic reactivity.

SOIL EXPLORATION, SAMPLING AND CLASSIFICATION

Soil exploration involves assimilating information regarding conditions of the underlying strata that can affect the performance of pavement structures. This also involves recovery of representative soil samples for classification and testing purposes. Successful soil exploration requires careful consideration of certain selected features as listed and discussed in the following sections and subsections.

Preliminary data collection involves acquiring all pertinent information that can influence the outcome of a stabilization project. Geological and pedological information from the location provides the basis for differentiating earth materials and to identify problem areas.

A pedological approach to soil classification can be used as a basic approach in assessing the impact of the soil profile on the stabilization project. The reactivity of soils with calcium-based stabilizers is known to vary with depth within the pedological profile or with the soil horizons based on the changes in mineralogy and/or soil chemistry within these profile horizons. Furthermore, pedological profile data provides pertinent information for assessing the presence and form of minerals, such as sulfates or certain sulfide forms that might deleteriously affect the modification or stabilization process.

Estimates of the soil compositional characteristics based on pedological profiles can be acquired from the National Resources Soil Conservation Service (NRSCS) county soil survey reports. Data from these reports can be used as a guideline for sampling efforts and to identify the required depth and frequency of sampling as well as to establish the expected results of the sampling process.

Geological data, which is available from the NRSCS, the U.S. Geological Survey (USGS) and the State Geologic Survey reports, is valuable as it provides a basis from which to interpret the impact of land forms and to identify materials below the soil layer that might impact the stabilization process via migration or diffusion of ions with moisture fluctuations. Data from geological documents along with the soil surveys are particularly useful in identifying the presence of sulfate seams below the surface that can provide a source for sulfate diffusion into the stabilized layer.

If potential sources of sulfates are identified during preliminary data collection, a risk assessment should be made prior to undertaking the stabilization process and

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> *should be performed in accordance with the AASHTO standard recommended practice for stabilizing sulfate rich soils.*

Subsurface investigations should be guided by the purpose, requirements, and geographical settings of the project location. This involves reviewing all available data, described in Section 7.2, regarding the project location prior to beginning the field investigation and collecting and reviewing all pertinent information from investigation reports from adjoining projects, if available.

A comprehensive exploration plan should be developed as a part of subsurface investigation to communicate the intent and level of testing required for the project.

Subsurface investigation should be conducted in accordance with AASHTO R 13. The success of a subsurface investigation depends primarily on the effectiveness of the geotechnical engineers and technicians involved in field operations and therefore should only be performed by responsible, well-trained and experienced people.

Direct observation of subsurface conditions and retrieval of field samples can be achieved by examination of soil formations from accessible excavations, such as shafts, tunnels, test pits, or trenches, or by drilling and sampling to obtain cores or cuttings.

A properly designed sampling plan should be developed to minimize sampling error and optimize sampling efficiency. Samples should be taken in a manner that minimizes the bias of the person taking the samples. The sampling plan is required to randomize sampling locations.

Sample units of roadway materials should be selected randomly in accordance with ASTM D 3665. The number of field samples to be collected depends on the level of confidence required by project specifications. Guidance in determining the number of samples required to obtain the desired confidence levels are detailed in ASTM test methods E 105, E 122 and E 141.

A sufficiently large quantity of soil should be sampled to allow adequate testing to determine engineering properties of the soil that are pertinent to the proposed design (e.g. mix design).

Undisturbed soil samples are not usually required to evaluate the efficacy of soil stabilization as stabilization operations involve mixing and compaction operations that destroy the original soil fabric. Therefore, any mechanical disturbance of samples during extraction does not normally compromise the quality of the sample or its acceptability for testing.

Frequency of sampling should be based on the continuity of soil and rock formations observed during subsurface investigations. Sample should be collected every time there is a change in observed physical characteristics of the soil.

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Subsurface conditions identified at the individual test pits, boring holes or by examining open cut sections during subsurface investigations should be used as a guideline to decide the frequency of sampling.

The uniformity of the soil and the potential for the soil profile to contain minerals that may cause a deleterious reaction with the stabilizer form the basis for determining the frequency of sample collection. A general recommendation regarding frequency of sampling based on varying soil conditions is given in Table below.
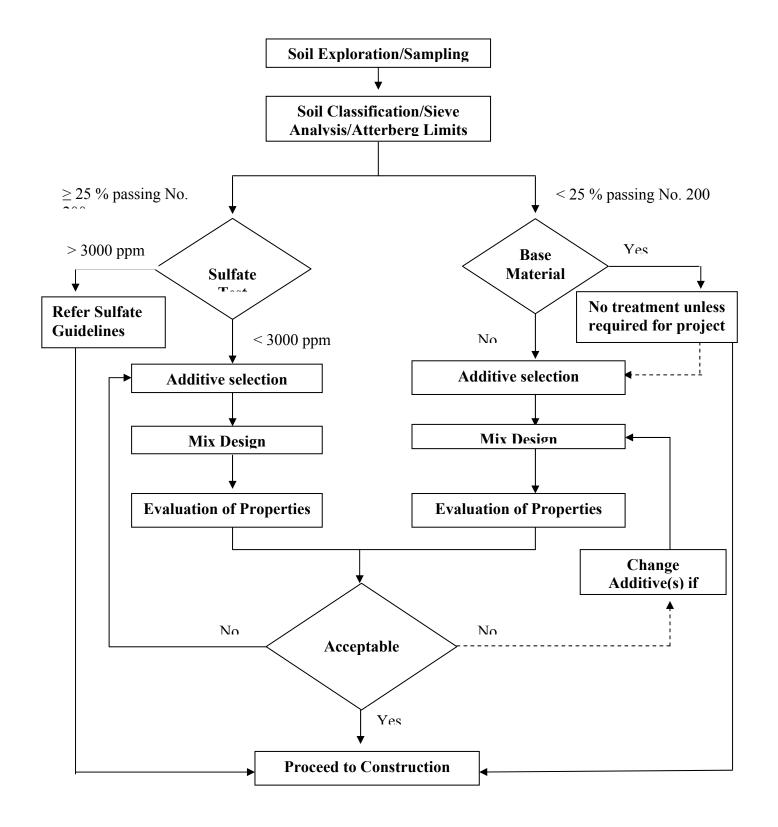
Soil condition	Frequency of sampling
Uniform	0.5 to 1.0 mile
Non-Uniform	0.25 to 0.5 mile
Highly variable	1,000 ft to 0.25 mile
Sulfate bearing	500 ft

Sampling should be deep enough to identify all strata that can significantly influence the outcome of the stabilization.

AASHTO R-13 recommends that the depth of exploratory borings or test pits for road beds be at least 5 feet below the proposed subgrade elevation.

For cuts exceeding these depths, sampling should be done to the road bed depth plus an additional 2 feet.

The spacing requirements and boring depths mentioned above should not be considered as either a minimum or a maximum, but instead should be used as a guide. The final decision regarding sampling should be based on field conditions and expertise of the geotechnical engineer such that the sampling operation and collected soil samples provide a basis for capturing all pertinent information regarding the engineering and hydro geologic properties relevant to the project design.

GUIDELINES FOR SOIL STABILIZATION

Soil-stabilizer interactions vary with soil type and so does the extent of improvement in soil properties as a result of stabilization. Hence the efficacy of using a stabilizer must be evaluated prior to the treatment. A generalized flowchart detailing the steps to be followed in evaluating the effectiveness of a stabilizer is presented below.

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The key decision making factor prior to deciding stabilization techniques is classification of selected soil type.

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Classification of soils should be done in accordance with AASHTO M 145.

As a candidate for stabilization purposes, the soil should first be classified as either a subgrade category or base category material. This is based on the fraction passing No. 200 sieve. If 25 percent or more of the soil mass passes the No. 200 sieve, then the soil is classified as a "soil". Otherwise, it is classified as a base for stabilization purposes.

Base materials must also satisfy plasticity and gradation requirements and restrictions which vary among agencies. Base materials that qualify can be used directly for field applications without stabilization (Note 1).

> Note 1. *In this standard, it is assumed that the candidate aggregate base material is of at least moderate quality, otherwise, the material should be treated as a soil. Moderate quality is defined as: (1) not more than 40 percent finer than the no. 40 sieve (0.425 mm or 0.0165 in.), (2) a maximum plasticity index of 12 percent, and (3) a maximum liquid limit of 40 percent.*

The second factor to be considered when using traditional soil stabilizers is the presence of and the concentration of sulfate salts and organic materials in the soil.

Water soluble sulfate levels in soils should be identified in accordance with the modified version of AASHTO T 290 or equivalent test methods. Soils with sulfate levels above 3,000 ppm have the potential for formation of significant levels of expansive minerals, like ettringite and/or thaumasite, which may result in disruptive volume changes within the stabilized layer. The standard practice for stabilization of high sulfate soils should be consulted if the soluble sulfate level exceeds 3,000 ppm.

Screening for organic contents in soil should be done in accordance with ASTM D 2974. Soils with an organic content of greater than 1 percent as determined by ASTM D 2974 may be difficult to stabilize or uneconomical quantities of additives may be required in order to stabilize them. However, the impact of the organic content varies considerably with the type of organic present, and a full testing regime is usually required to assess the impact.

TECHNIQUES AND GUIDELINES FOR STABILIZER SELECTION

Characteristics of fine-grained soils including mineralogy, gradation and physio-chemical properties influence the soil-additive interaction.

The preliminary selection of the appropriate additive(s) for soil stabilization should therefore be based on soil index properties (i.e., particle size (sieve) analysis and Atterberg limit data) and should be identified following the test procedures detailed below.

Soil samples should be prepared following AASHTO T 87. The initial processing of most soils requires thorough air drying or assisted drying at a temperature not exceeding

60ºC (140ºF). Aggregations of soil particles should be broken down into individual grains to the extent possible.
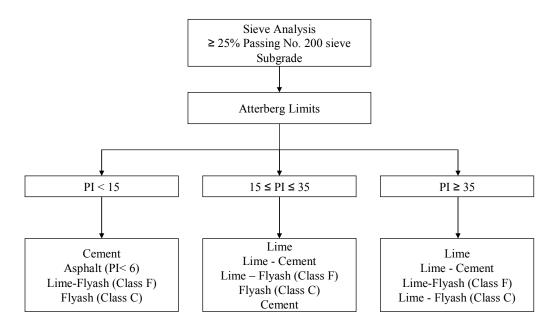
A representative soil fraction should then be selected for testing following AASTHO T 248 which should be used to determine index properties of the soils.

Sieve analysis should be performed following AASHTO T 27.
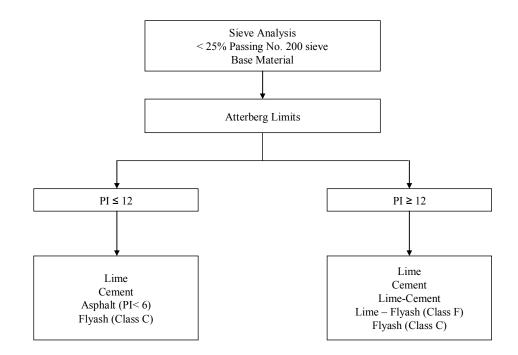
Liquid limit testing should be performed following AASHTO T 89 and plastic limit and plasticity index testing should be measured following AASHTO T 90.

Plasticity index (PI) and the percent passing the no. 200 sieve (percent smaller than 75 μm) are index properties that have been successfully used to identify the appropriate stabilizer for a given soil. This process should be performed following the decision tree for stabilizer selection presented in Sections 9.3 and 9.4. Individual agencies are encouraged to use these decision trees as guidelines and to incorporate local experience into revise and improve these decision trees.

The decision tree for selecting stabilizers for use in subgrade soils is given in figure below.

The decision tree for selecting stabilizers for use in Base materials is given in figure below.

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                    ┌─────────────────────────┐
                    │      Sieve Analysis      │
                    │ < 25% Passing No. 200 sieve │
                    │       Base Material      │
                    └─────────────────────────┘
                                 │
                                 ▼
                    ┌─────────────────────────┐
                    │      Atterberg Limits    │
                    └─────────────────────────┘
                       │                    │
              ┌────────┴──────┐      ┌──────┴────────┐
              │    PI ≤ 12     │      │    PI ≥ 12    │
              └───────────────┘      └───────────────┘
                     │                       │
                     ▼                       ▼
            ┌─────────────────┐    ┌─────────────────────┐
            │      Lime       │    │        Lime         │
            │     Cement      │    │       Cement        │
            │  Asphalt (PI< 6)│    │     Lime-Cement     │
            │  Flyash (Class C)│   │ Lime – Flyash (Class F) │
            └─────────────────┘    │   Flyash (Class C)  │
                                   └─────────────────────┘
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Figures in sections 9.3 and 9.4 identify more than one potential stabilizer for a given soil. Additional guidelines detailed in the following sub sections may also be used to refine the process of stabilizer selection prior to validation testing. These guidelines specify the stabilizer (lime, Portland cement, or fly ash) best suited for the soil in question.

Lime has been found to react successfully with medium, moderately fine and fine grained soils resulting in a decrease in plasticity and swell potential of these soils. Lime is an appropriate stabilizer for most cohesive soils but the level of reactivity depends primarily on the type and amount of clay minerals in the soil.

Soils with a plasticity index of 10 or greater and with a minimum of 25 percent passing the No. 200 sieve are typically considered to be candidates for lime stabilization.

Lime is generally considered to be a suitable, if not the most suitable, stabilizer for soil types that belong to AASHTO classifications A-4, A-5, A-6, A-7 and some of A-2-6 and A-2-7 soils.

Cement can be successfully used to stabilize a wide range of soils. However, it is particularly well suited to stabilize well graded soils that contain sufficient amount of fines to effectively fill the available voids space and float the coarse aggregate particles. Silty soils (A-2-4 to A-4) have been documented to derive the highest degree of improvement (when stabilized with Portland cement) among the soils amenable for cement stabilization. Most soil types, except those with high organic content, highly plastic clays and poorly reacting sandy soils, are amenable to stabilization with Portland cement.

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Portland cement is generally considered to be a good candidate stabilizer for soils with less than 35 percent passing the no. 200 sieve.

General gradation specifications limit the nominal maximum aggregate size at 2-inches with at least 55 percent passing the no. 4 sieve. For uniformly graded materials, the addition of non plastic fines like fly-ash, aggregate screenings, cement and lime kiln dust may help fill the voids in the soil structure and help reduce the required cement content.

For fine-grained soils, the general consistency guidelines are that the plasticity index (PI) should be less than 20 and the liquid limit (LL) should be less than 40 in order to ensure proper mixing of cement and soil. A more specific guideline defining the upper limit of PI for soils is given in equation below:

$$P.I \leq 20 + \frac{50 - (\% \ smaller \ than \ 0.075 mm)}{4}$$

However, depending on the efficiency of mixing equipment and expectations of the stabilization process, soils with PIs above 20 percent may also be stabilized with Portland cement.

The ability to stabilize soils with plasticity indices above about 20 using cement is based on the ability to intimately mix cement with the soil to a degree that will produce a reasonably homogeneous and continuous stabilized matrix of the agglomerates. This requires a certain efficacy of mixing, which is in turn associated with the energy imparted to the soil by the mixing equipment and by the time span over which mixing occurs.

Fly ash can be used effectively to stabilize coarse grained particles with little or no fines. In these soils, fly ash generally acts as a pozzolan and/or filler to reduce the void spaces among larger size aggregate particles to float the coarse aggregate particles.

Fly ash may be effective in silty soils or soils that have low clay content or when the clay is not pozzolanically reactive. Fly ash can help enhance the pozzolanic reactivity of these fine-grained soils.

The presence of organic matter and sulfates in the soil to be treated or stabilized must be evaluated as part of the stabilizer selection process.

Techniques for measuring sulfate contents in soils and recommendations for stabilizing soils with high sulfate contents are detailed in Sections 8.3.1 and 7.2.4.

Soil organic content should be measured following procedures detailed in Section 8.3.2. Additional steps and considerations for soils with higher organic content are detailed in the mix design procedure for each specific stabilizer.

It is important to understand that the decision trees and the guidelines presented in previous sections of the standard are only the first step towards stabilizer selection. Once the stabilizer is selected, specific laboratory tests are required

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in order to determine whether the strength and performance characteristics of the stabilized soil are satisfactory. Specific test methods required to validate the use of a selected stabilizer are discussed in later sections of this standard.

VALIDATION TECHNIQUES FOR STABILIZER SELECTION: LABORATORY TESTING METHODS

Stabilization projects are site specific and the soil-stabilizer interactions vary with soil types. Therefore the extent of improvement in soil properties is dependent on the interactions of the selected stabilizer with the soil. The following sections and sub-sections detail the techniques for validating the use of individual stabilizers and outline the requirements for selecting individual stabilizer type for field applications.

The preliminary stabilizer selection process is outlined in Section 9.

In the lime stabilization process, the extent of lime-soil interactions depends on the type and amount of clay minerals present in the soil. The mix design protocol given below is designed to optimize the potential for long-term strength gain and durability of lime stabilized soils.

The candidate soil for lime treatment should be identified following the steps detailed in Section 9.2.

Organic content in soil should be determined by following steps detailed in Section 8.3.2 (Note 2).

> **Note 2.** If the organic content of the soil is above one percent, additional compression strength samples should be prepared and tested with higher lime contents, i.e., at least one to two percent above optimum lime content. The purpose of this testing is to determine whether the strength and durabilty of the lime-soil mixture can be enhanced with additional lime and that the additional lime compensates for the loss of free calcium due to adsorption of calcium by organic functionalities, which interupt cation exchange and pozzolainc reactons between calcium and the soils being treated.

Water soluble sulfate levels in the soil to be treated with lime must be evaluated following steps detailed in Section 8.3.1. Soils with sulfate contents above 3,000 ppm may be considered problematic and should be addressed separately as detailed in the AASHTO Standard Recommended Practice for Stabilizing Sulfate Bearing Soils.

The trial stabilization lime content should be determined following ASTM D 6276. For reliable test results, the lime used in the pH test should be the same as that to be used in construction and this lime should be carefully stored to avoid carbonation.

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Lime-soil mixtures should be fabricated following ASTM D 3551 for moisture density and compressive strength testing. For compressive strength testing, samples are not required to mellow before fabrication unless it is difficult to achieve satisfactory homogeneity during laboratory mixing. If so, mellowing periods of up to 24 hours can be included to simulate field mellowing.

The moisture-density relationship of the lime-soil mixtures should be evaluated for the amount of lime identified by the Eades and Grimm pH test, ASTM D 6276. The moisture-density relationship of lime-soil mixtures should be determined in accordance with AASHTO T 99.

Triplicate samples should be prepared for compressive strength testing following ASTM D 5102, procedure B with the lime content determined following ASTM D 6276 (Note 3). Samples should be fabricated at optimum moisture content (OMC) ± 1 percent.

Note 3. It is recommended to prepare and test additional mixtures with lime contents one to two percent higher than the optimal lime content as determined following ASTM D 6276. Preparation and testing of these samples should also follow the procedures detailed in Sections 10.3.5 and 10.3.6.

After compaction the test specimens should be wrapped in a plastic wrap and stored in an air tight moisture proof bag with about 10 ml of free water to ensure proper moisture for pozzolanic reactions.

The specimens should be cured at $40^o C$ ($104^0 F$) for 7 days before compression testing (Note 4).

Note 4. *Since the accelerated cure is not always a good approximation of strength gain at long term, normal cure, it is appropriate to subject one set of lime soil samples to normal cure for 28 days before compression testing.*

After the specified curing period, the specimens should be removed from the storage bags and plastic wraps and then wrapped with a wet absorptive fabric or geotextile and placed on a porous stone for capillary soak (Note 5). A soaking period of at least 24 hours is recommended. But capillary soaking should continue for as long as it takes for the moisture front to move to the top of the sample or until the moisture front ceases to move.

Note 5. **During capillary soak, the water used in soaking should never come in direct contact with the specimen. The water level should be maintained at the top of the porous stone and kept in contact with the fabric wrap.**

Samples prepared for compression testing may also be used to evaluate volume changes in lime stabilized expansive soils (Note 6). Vertical and circumferential measurements of samples before and after soaking should be made to calculate volume changes between the dry and soaked conditions. A three dimensional volumetric expansion of up to 2 percent is typically regarded as acceptable.

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*Note 6. **This step is applicable only for expansive clay soils and sulfate bearing soils.***

If the expansion in the treated soil is higher than the recommended value, then 1 to 2 percent additional lime should be added and procedures detailed in Sections 10.3.5 to 10.3.11 should be repeated.

Swell test can also be used to determine the extent of swelling in sulfate bearing soils. But the period of exposure to moisture required for sulfate bearing soils is considerable longer and should be continued until swelling ceases. Additional samples should therefore be prepared for swell testing in case of sulfate bearing soils. These samples should be prepared in accordance with steps discussed in in Section 10.3.5 and 10.3.6.

Following capillary soaking, the specimens should be subjected to unconfined compression strength testing in accordance with ASTM D 5102 procedure B. The suggested minimum requirements for lime stabilized soils are given in table below.

Anticipated Use of Stabilized layer	Compressive strength recommendations for different anticipated conditions (psi)			
	Extended Soaking for 8 Days	Cyclic Freeze-Thaw		
		3 Cycles	7 Cycles	10 Cycles
Sub-base Material				
Rigid Pavements/Floor Slabs/Foundations	50	50	90	120
Flexible Pavement (>10 in.)	60	60	100	130
Flexible Pavement (8 in. - 10 in.)	70	70	100	140
Flexible Pavement (5 in. - 8 in.)	90	90	130	160
Base Material				
	130	130	170	200
For cyclic moisture conditioning the samples should be made to reach 80 percent saturation upon "wetting" followed by 50 percent saturation upon "drying".				

The strength guidelines in section 10.3.12 are minimum vales. If more than one lime content is considered in compression testing, then the lime content that produces the highest strength and durability, and of course that meets minimum strength requirements, should be selected as optimum. The compressive strength values in Table 10.3.12 may vary depending on the purpose of stabilization, exposure conditions, expected number of freeze-thaw cycles and the insulation effect of pavement layers over the stabilized layer.

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The purpose of adding lime as a stabilizer for base materials is for lime to interact with the fine material to form a matrix and provide improved strength, stiffness, and durability for the aggregate base. Since the fine material (smaller than 75 μm material) comprises no more than about 10 percent of the of the entire mixture by weight, the amount of lime used by weight of the total aggregate base will be considerably less than that used in soils. The following steps should be followed for base course stabilization with lime.

Add incremental concentration of lime to the base materials, generally starting with 1 percent by weight of the entire mixture and increasing in 1 percent increments to a maximum of 4 percent.

Moisture density relationships for each aggregate-lime blend should be determined following either AASHTO T-99 or AASHTO T-180 based on agency requirements.

Determine unconfined compressive strengths of the lime-aggregate blends following curing for 7-days at 40^0C (104^0F) followed by capillary soak. Procedures described in the Section 10.3.10 should be used to moisture condition the samples prior to compressive strength testing.

The compressive strength testing procedure and target compressive strength requirements should be based on specifications defined by the user agency. In the absence of such guidelines, the residual strengths in the table in section 10.3.12 can be applied. The lowest lime content that meets the compression strength requirement is considered as the required lime content for stabilization purposes.

Steps involved in developing mix designs for Portland cement stabilized base materials are given in the following sub sections.

Soils must be screened for organic content following Section 8.3.2. Soils with more than two percent organic content are usually considered unacceptable for cement stabilization.

If the presence of organic mater is detected in soils, a pH test should be conducted to verify if organic matter will interfere with the hydration processes in soil.

In the pH test, a soil-cement paste should be prepared by mixing 10 parts soil to one part cement (by weight). Determine the pH of the paste after 15 minutes of mixing.

If the pH is 12.0 or higher after 15 minutes of mixing, then it is probable that organics will not interfere with the normal cement hydration process. If not, then a higher cement content than those recommended based on AASHTO soil groups given in Section 10.4.3.1 may be required for stabilization.

Soils must also be screened for sulfate content prior to using Portland cement for stabilization.

Sulfate contents in soil should be determined in accordance with AASHTO T 290 (modified) test method or equivalent test methods.

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If the soluble sulfate concentration of the soil to be treated is greater than 3,000 ppm, then the standard practice that deals with stabilization of high sulfate soils should be consulted before proceeding. The standard practice for stabilization of high sulfate soils identifies the risk associated with stabilization of these soils and describes steps to reduce the risk of treatment of these soils.

As a first step towards identification of cement content for stabilizing a given soil, classify the soil based on AASHTO M 145.

A preliminary estimate of the cement content should then be made based on the following table.

AASHTO Soil Groups	Usual Range in Cement Requirement		Estimated Cement Content, Percent by Weight
	Percent by Volume	Percent by Weight	
A-1-a	5-7	3-5	5
A-1-b	7-9	5-8	6
A-2	7-10	5-9	7
A-3	8-12	7-11	9
A-4	8-12	7-12	10
A-5	8-12	8-13	10
A-6	10-14	9-15	12
A-7	10-14	10-16	13

These cement contents are only preliminary estimates and are proportioned on a weight basis in terms of the percent of oven dry soil. These estimates should further be verified or modified based on additional testing detailed in the following sections.

It is important to understand that the estimated cement contents given in the table in section 10.4.3.1 are based on durability tests, ASTM D 559 and D 560, and that many soils can be successfully stabilized with considerably lower cement contents.

After the required amount of cement is added to the soil, the blend should be mixed thoroughly until the color of the mixture is uniform.

Soil samples used to establish the moisture density relationship should be fabricated following AASHTO T 134.

The median cement content as estimated in Section 10.4.3.1 should be used to determine moisture density relationships of soil cement mixtures. However, if it is expected that acceptable treatment can be achieved with considerably lower cement contents, then that cement content should be used to determine the moisture-density relationship.

Three cement contents should be used to determine the compressive strength of soil cement mixtures. Compression test samples should be prepared with the median cement content and with cement contents two percent above and below the median content. Sample should be prepared following AASHTO T 134.

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Preparation and curing of samples for compressive strength testing should follow ASTM D 1632.

The samples should be moist cured through out the curing periods and immersed in water for four hours prior to compression testing. Compressive strength testing should follow ASTM D 1633.

Typical acceptable ranges of unconfined compressive strength criteria of moisture conditioned soil cement specimens for various soil classifications are given below.

Soil Type	AASHTO Classification	Soaked Compressive Strength (psi)	
		7 Days	28 Days
Sand and	A-1, A-2, A-3	300-600	400-1,000
Silty	A-4, A-5	250-500	300-900
Clayey	A-6, A-7	200-400	250-600

Strength requirement for stabilized layers may vary considerably from agency to agency. A range of compressive strengths that are typically considered to be acceptable lower limits are presented in the table below.

Purpose of Stabilized Layer	Minimum 7 day Unconfined Compressive Strength (psi)	
	Flexible Pavement	Rigid Pavement
Base Course	300 - 750	300 - 500
Sub base, select material or subgrade	250	200

The lowest cement content in the mixture design that meets the requirements in Section 10.4.11 or that satisfies the agency requirements should be used as the design cement content. If the tested samples does not confirm to these minimum requirements, then higher cement contents may be added to soil and Sections 10.4.4 to 10.4.9 should be repeated until the strength values meet minimum strength and durability requirements.

In environments where significant freeze-thaw activity is expected, durability testing following AASHTO T 136 can be used. In this case the freeze-thaw activity is mimicked in the laboratory by freeze-thaw cycling as described in AASHTO T 136 but a residual strength criterion following the freeze-thaw activity is used in lieu of loss after brushing. The residual strength requirements should be established by the user agency, but in the absence of such criteria, the values in section 10.4.10 may be used.

In cement stabilization of base materials, the fine material (materials smaller than 75 μm) comprises no more than about 10 percent of the entire mixture by weight. Therefore, the amount of cement used by weight of the total aggregate base will be considerably less than that used in soils. Steps to be followed in successful stabilization of base materials are detailed below.

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Add incremental concentrations of cement to the base materials, generally starting with 1 percent by weight of the entire mixture and increasing in 1 percent increments to a maximum of 3 percent.

Moisture density relationships for each aggregate-lime blend should be determined following either AASHTO T-99 or AASHTO T-180 based on agency requirements.

Determine the unconfined compressive strengths of the cement-aggregate blends following moist cure for 7-days followed by a 4 hours soak as recommended by ASTM D 1633. The compressive strength testing procedures and the target compressive strength requirements should be based on specifications defined by the user agency.

The lowest cement content that meets the compression strength requirement is considered as the required cement content for stabilization purposes.

Fly ash can be effectively used to stabilize coarse grained soils or bases. In coarser aggregates where a substantial improvement is strength is the target, fly ash generally acts as a pozzolan and/or filler to reduce the void spaces among larger size aggregate particles and to float the coarse aggregate particles. In fine-grained soil, fly ash is typically used as a standalone stabilizer or in conjunction with lime or cement to enhance the reactivity of the fine-grained soil with lime or cement. Steps involved in developing mix designs for use of fly ash in soils and base materials are given in the following sub sections.

The fly ash to be used in stabilization should be classified as either class C fly ash or a class F fly ash following AASHTO M 295.

The cementitious properties of the fly ash to be used in stabilization should be characterized following ASTM D 5239 (Note 7).

Note 7. ASTM D 5239 does not evaluate the interaction between fly ash and soil or aggregate. This must be verified separately based on mix design procedures outlined in the following paragraphs.

Selection of the optimal fly ash content for soil modification or stabilization is a function of the purpose of stabilization; i.e., whether the purpose is to achieve maximum strength or to achieve a target level of strength.

To achieve maximum strength and durability, fill the voids in the matrix with fly ash to achieve maximum density and then determine the moisture density relationship for this optimal blend.

To achieve a target level of strength, experience or a trial and error process should be used to identify trial fly ash percentages and activator contents. These estimates are then used to establish moisture density relationships and to determine the compressive strength of the mixture.

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Steps for developing effective mix designs in soils using class C fly ash are detailed in sub sections 10.5.4.1 through 10.5.4.4.

Development of mix designs to meet the strength requirement is dependent on moisture density and moisture strength relationships. Compactive energies should be selected based on the intended application of these materials.

Mix and mold the samples to be used for moisture-density determination following ASTM C 593.

Determine the moisture-strength relationship for all the selected mixes. To determine the moisture-strength relationships, compact the samples at different moisture contents to determine the moisture content that will produce the maximum compressive strength (Note 8).

Prepare compressive strength specimens by blending soil, fly ash, and water and mold the specimens after the specified compaction delay. The compaction delay should ideally be kept below 2 hours. Sample preparation and testing should be done in accordance with ASTM C 593.

Cure the specimens for 7 days at $38^{o}C$ ($100^{o}F$) in accordance with ASTM C 593 before compression testing.

Note 8. Optimum moisture content for highest strength gain may typically be 1–8 percent below optimum moisture content needed to attain maximum dry density. This value may vary with soil type and the mineralogy of ash particles.

The strength developed by adding the class C fly ash may be sufficient. However, if substantially higher strength development is required, the addition of an activator may optimize the pozzolanic activity and substantially increase the strength and durability of the mixtures.

Add activator (lime or Portland cement) using a trial and error approach to the mixture of soil, fly ash and water. Add the activator in 2 percent increments beginning with an activator content of 2 percent. The goal is to determine the amount of activator required to maximize the pozzolanic reaction based on strength gain.

The activator content required to develop maximum pozzolanic activity may be as much as 20 to 40 percent of the ash, and hence it may be necessary to replace some of the fly ash with the activator.

The following recommendations are applicable to cases where class C or class F fly ash is added to fine-grained soils that are not pozzolanically reactive, meaning soils where lime is effective in reducing plasticity and improving workability but not in providing the target strength.

Determine the lime content required for soil stabilization based on the Eades and Grimm pH test, ASTM D 6276.

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Add increasing concentrations of fly ash to the soil with the designated amount of lime, as determined in the preceding section, starting with 4 percent by weight of the mixture and at increments of 2 percent.

Follow the steps outlined in sections 10.3.7 through 10.3.10 and 10.3.12 for compressive strength and durability testing (Note 9).

> ***Note 9.*** *Fly ash, activator, and soil samples may also be conditioned following agency directives prior to compression strength and durability testing. The conditioning processes vary from the severity of vacuum saturation as prescribed in ASTM C 593 to a capillary soak as described in section 10.3.10. A compressive strength test following moisture conditioning prescribed by the agency to mimic the climatic conditions of the region may also be considered acceptable.*

When fly ash, normally class F fly ash, is used to stabilize base course materials (pozzolanic stabilized mixture or PSM), the goal may be either to achieve maximum density and optimal strength or to achieve a target strength.

Determine the optimum fly ash content or target fly ash content for the mixture following a trial and error process to achieve a maximum dry density for the mixture. Determine the optimum moisture content of the selected mixture following ASTM C 593.

Determine the optimum activator content or target activator content for the selected blend by trial and error method (Note 10). Prepare at least three replicates for compression testing for each blend of fly ash and activator.

Note 10. Required lime or Portland cement contents to activate Class F fly ash are typically between one part lime/cement to three parts fly ash to 1 part lime/cement to four parts fly ash. If lime kiln dust (LKD) or cement kiln dust (CKD) is used as activator, then higher activator ratios may be required.

Cure the fly ash-lime-soil mixes for 7-days in sealed containers. Samples prepared with lime and kiln dust activators are cured at 37.8°C (100° F) for 7-days. Moisture conditioning prior to testing should be performed following the protocol presented in sub sections of 10.3.10 (Note 9).

Cure the fly ash-cement mixes at a 100 percent relative humidity environment at 22.8°C (73° F) for 7-days. The samples should be moisture conditioned by soaking in water for 4 hours prior to testing.

Determine the compressive strength of the blends following test methods ASTM C 593 and ASTM D 1633. The conditioning processes prescribed by the agency to mimic the climatic conditions of the region or capillary soak as described in section 10.3.10 may also be considered acceptable. Compare the compressive strength values with target values to determine if the blend produces acceptable strengths to function properly under the design loading and environmental conditions.

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Typically, a 7-day compressive strength of 400 psi is considered acceptable for field applications. But this requirement may vary with field conditions and among agencies. A mix that attains the required properties with the lowest percentage activator is selected as the design mix for use in field.

Durability testing should be performed at the end of curing periods based on specifications defined by the user agency (Note 9). In areas where there is no freeze-thaw effect, durability testing may be waived in accordance with local practice.

REPORT

The report for stabilization of soils and base materials should include:

Identification of sampling locations, details of locations of test pits and bore holes, and details of all other sampling sources used to obtain soil for test purposes.

Details of subsurface conditions identified at the individual test pits, boring holes or by examining open cut sections during subsurface investigations.

Approach used to select the stabilizer and method used to validate the selection of the stabilizer and perform mixture design.

Tabulation of test data supporting the stabilization decisions.

PRECISION AND BIAS

This standard provides qualitative data only; hence, precision and bias are not applicable.

KEYWORDS

Soil stabilization; mineralogy; traditional stabilizers; soil sampling

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