CONSIDERATION OF LIME-STABILIZED LAYERS IN MECHANISTIC-EMPIRICAL PAVEMENT DESIGN

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PURPOSE OF MANUAL

Pavement foundations are treated with lime for a variety of reasons: for construction facilitation, treatment of expansive soils, and to provide structural support for the pavement system. Many studies have shown that well engineered and constructed lime stabilized soil layers provide strong and durable support to pavement structures, improving their long term performance. This manual deals with the use of lime stabilized layers as structural components of a pavement system.

Specifically, the objective of this manual is to provide highway pavement engineers with supplemental guidance on how to consider and include lime stabilized layers in designing flexible pavements using mechanistic-empirical (M-E) design procedures. The manual focuses on the mechanistic-empirical (M-E) design approach developed under NCHRP Project 1-37A: *Development of the 2002 Guide for the Design of New and Rehabilitated Pavements: Phase II*, but is also applicable to other M-E design procedures. Although the Guide is being distributed in draft form, many organizations are already gearing up to use the new design approaches and collecting data on their pavements.

The discussion in this document is presented in two sections:

- Section I: Introduction – Provides definitions of terms, overview of important soil lime reactions, discussion on typical usage of lime stabilized layers and potential benefits.
- Section II: M-E Design Considerations for Lime Stabilized Layers – Provides information on the steps and issues that must be considered when incorporating lime stabilized layers into pavement structural design including feasibility assessment, mixture design, materials testing for strength and durability, input selection, and economic benefit.

The design methodology adopted in NCHRP Project 1-37A and presented in this manual will be reviewed under NCHRP Project 1-40. The extent or nature of changes from this review regarding lime stabilized mixtures is unknown at this time. Once the design methodology is finalized, this report will be revised to reflect any changes and expanded to include design examples.

SECTION I. INTRODUCTION

Background

Modification and stabilization of highway and airport pavement subgrades using lime and is a well-established, time-tested practice in the United States. The addition of lime to reactive fine-grained soils has beneficial effects on their engineering properties, including reduction in plasticity and swell potential, improved workability, increased strength and stiffness, and enhanced durability. In addition, lime has been used to improve the strength and stiffness properties of unbound base and subbase materials.

Lime can be used to treat soils to varying degrees, depending upon the objective. The least amount of treatment is used to dry and temporarily modify soils. Such treatment produces a working platform for construction or temporary roads. A greater degree of treatment--supported by testing, design, and proper construction techniques--produces permanent structural stabilization of soils.

A vast amount of literature covering the construction and performance of pavement structures incorporating lime stabilized materials is available through reports by the National Lime Association (NLA), the National Cooperative Highway Research Program (NCHRP), the Transportation Research Board (TRB), State highway agencies (SHAs), and other organizations. The available information includes mixture design methodologies, strength and load-response behavior of lime–soil and lime–aggregate materials, construction-related issues, durability studies, and guidelines for design of pavement with lime-stabilized layers.

In general, most of the laboratory and field-based pavement design studies involving lime additives demonstrate that, when proper attention is paid to materials design, durability, structural design, and construction, lime-stabilized subgrades outperform sections that do not incorporate lime. Improved performance is seen in terms of both load-response characteristics and number of truck traffic loads carried to failure.

The mechanistic–empirical (M–E) design procedure developed under NCHRP Project 1-37A (referred to henceforth as the M-E Design Guide procedure) represents a major improvement and shift from existing empirical design procedures (e.g., AASHTO 1993). Through the use of comprehensive input data (including actual axle weights and distribution, improved material characterization, and hourly climatic data), mechanistic principles for structural response calculations, and calibrated performance prediction models, the new design procedure can produce reliable and cost-effective designs.

The types of flexible pavements considered in the M-E Design Guide approach include conventional hot mix asphalt (HMA), deep-strength HMA, full-depth HMA, and semi-rigid pavements. The allowable pavement cross sections include pavement structures with both conventional layering (decreasing material quality with depth), as well as sandwich structures (unbound aggregate layer placed between two stabilized layers).

One of the major considerations in the design of flexible pavements is the characterization of chemically stabilized layers, including those stabilized with lime and lime–flyash. The materials properties used in structural calculations and performance prediction for these layers include strength, stiffness, and Poisson's ratio. These inputs are considered in the M-E Design Guide using a three-level hierarchical scheme. Level 1 requires a thorough knowledge of the input properties for a given design scenario obtained through laboratory testing or field evaluations. Level 3 implies a relatively poorer knowledge of the actual design values and relies on historical records or engineering judgment. Level 2 represents a balance in the level of effort required to configure the inputs and relies on some basic laboratory testing and the use of correlations.

Recommendations are made in the M-E Design Guide on the approaches to obtain materials parameters at each of the three hierarchical levels for lime-stabilized pavement layers. Using these inputs, the M-E design process predicts pavement performance indicators, such as fatigue cracking, permanent deformation, and fracture of chemically stabilized layers. Such an approach considers the beneficial effect of lime-treated soils and base materials on pavement design.

Terms and Definitions

Like many other specialized subject areas, pavement subgrade and base stabilization has its own unique set of terms and expressions. Some of the terms, on occasion, are not well understood or spawn different interpretations. Also, given the continual advances in pavement and stabilizer technologies, the collection of terms used in practice steadily change or become expanded.

The terms and definitions given in Appendix A serve as a reference to those who are expecting to use the M-E Design Guide to design pavement structures containing lime-stabilized layers. They include both new terms brought about by the development of the M–E design procedure as well as traditional terminology.

Production and Usage

The term lime refers specifically to calcium oxide (quicklime) and calcium hydroxide (hydrated lime), which are both burned forms of limestone (calcium carbonate). Lime for stabilization should not be confused with inert carbonates, such as limestone, agstone, or aglime, which are used in agricultural applications. Typical properties of commercial varieties of quicklime and hydrated lime are summarized in Table 1.

As shown in Figure 1, the commercial manufacture of lime takes place at some 65 plants located throughout the U.S. These plants sell more than 19 million short tons (nearly 18 million metric tons) of lime on an annual basis (Miller, 2004).

Table 1. Properties of commercial limes—quicklime and hydrated lime (NLA, 1988).

Quicklime				
Constituent	**High Calcium Range, %** [a]		**Dolomitic Range, %** [a]	
CaO	92.25 - 98.00		55.50 - 57.50	
MgO	0.30 - 2.50		37.60 - 40.80	
SiO_2	0.20 - 1.50		0.10 - 1.50	
Fe_2O_3	0.10 - 0.40		0.05 - 0.40	
Al_2O_3	0.10 - 0.50		0.05 - 0.50	
H_2O	0.10 - 0.90		0.10 - 0.90	
CO_2	0.40 - 1.50		0.40 - 1.50	
Specific Gravity	3.2 – 3.4		3.2 - 3.4	
Specific Heat at 100°F (38°C)	0.19 BTU/lb	442 J/kg	0.21 BTU/lb	488 J/kg
Bulk Density, pebble lime	55 - 60 lb/ft^3	880 - 960 kg/m^3	55 - 60 lb/ft^3	880 - 960 kg/m^3

Hydrated Lime				
Principal Constituent	**High Calcium** $Ca(OH)_2$		**Dolomitic (Monohydrated)** $Ca(OH)_2 \cdot MgO$	
Specific Gravity	2.3 - 2.4		2.7 - 2.9	
Specific Heat at 100°F (38°C)	0.29 BTU/lb	674 J/kg	0.29 BTU/lb	674 J/kg
Bulk Density	25 - 35 lb/ft^3	400 - 560 kg/m^3	25 - 35 lb/ft^3	400 - 560 kg/m^3

[a] Percentage by weight.

Lime has many different manufacturing and environmental applications. Its largest construction-related use is for stabilization of foundation soils and materials that underlie highway and airfield pavements, building structures, drainage canals, and earth dams. In 2003, more than 1.6 million metric tons of lime were used for soil stabilization in the U.S. (Miller, 2004).

Lime is used alone or in conjunction with other stabilizing agents to treat troublesome subgrade soils and substandard base/subbase layer aggregates. Such treatment usually has a significant beneficial effect on the engineering properties of the soil (reduction in plasticity and swell potential, strength gain, improved workability, and enhanced durability) or aggregate (increased strength and stiffness). These effects not only help facilitate the pavement construction process, they have a significant positive impact on the performance of the pavement structure.

Many State highway agencies (SHAs) use lime for stabilization of pavement foundation materials and soils. The results of two separate State practice surveys—one sponsored by the National Cooperative Highway Research Program (NCHRP) and the other by the American Concrete Pavement Association—conducted in the mid to late 1990s, indicated that nearly two-thirds of SHAs specify and/or allow use of lime for stabilizing subgrade soils (shaded States in Figure 2). About one-quarter of the States specify and/or allow its use in treating base/subbase granular materials (checkmarked States in Figure 2).

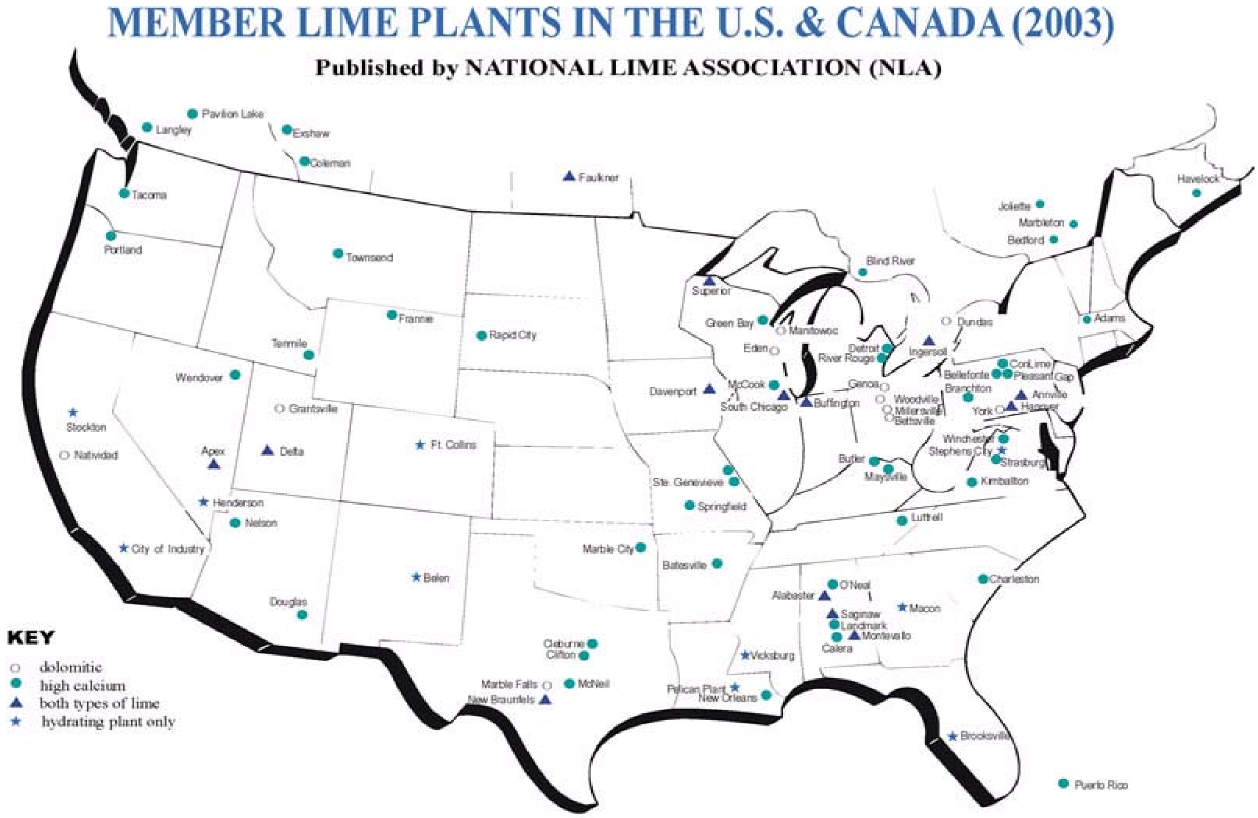

Figure 1. Map of lime plants in the U.S. and Canada (NLA, 2003).

Soil–Lime Reactions

Lime is used extensively to change the engineering properties of fine-grained soils and the fine-grained fractions of more granular soils. It is most effective in treating plastic clays capable of holding large amounts of water. The particles of such clays have highly negative-charged surfaces that attract free cations (i.e., positive-charged ions) and water dipoles. As a result, a highly diffused water layer shown in Figure 3 forms around the clay particles, thereby separating the particles and causing the clay to become weak and unstable. The extent to which this occurs depends on the amount of water present and the morphology and mineralogy of the clay (Little, 1987; NLA, 2004).

The addition of lime to a fine-grained soil in the presence of water initiates several reactions. The two primary reactions, cation exchange and flocculation–agglomeration, take place rapidly and produce immediate improvements in soil plasticity, workability, uncured strength, and load-deformation properties.

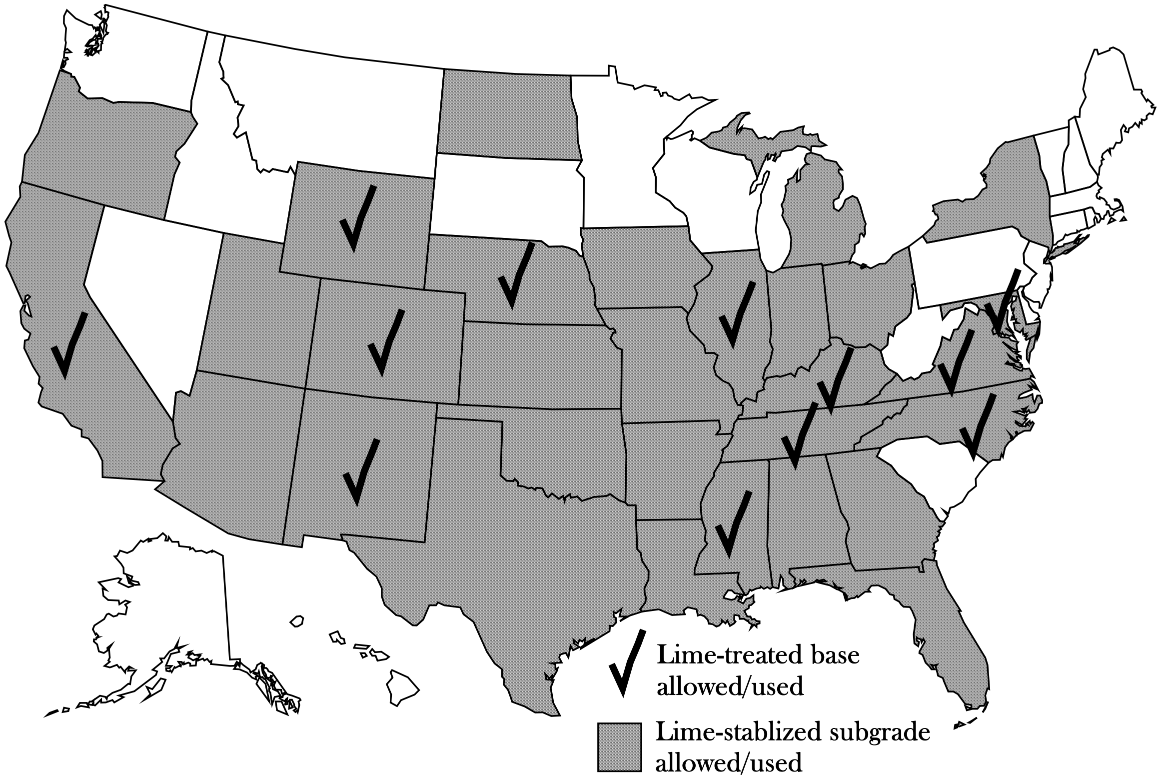

Figure 2. State use of lime in highway pavement construction (Jiang et al., 1996; ACPA, 1998).

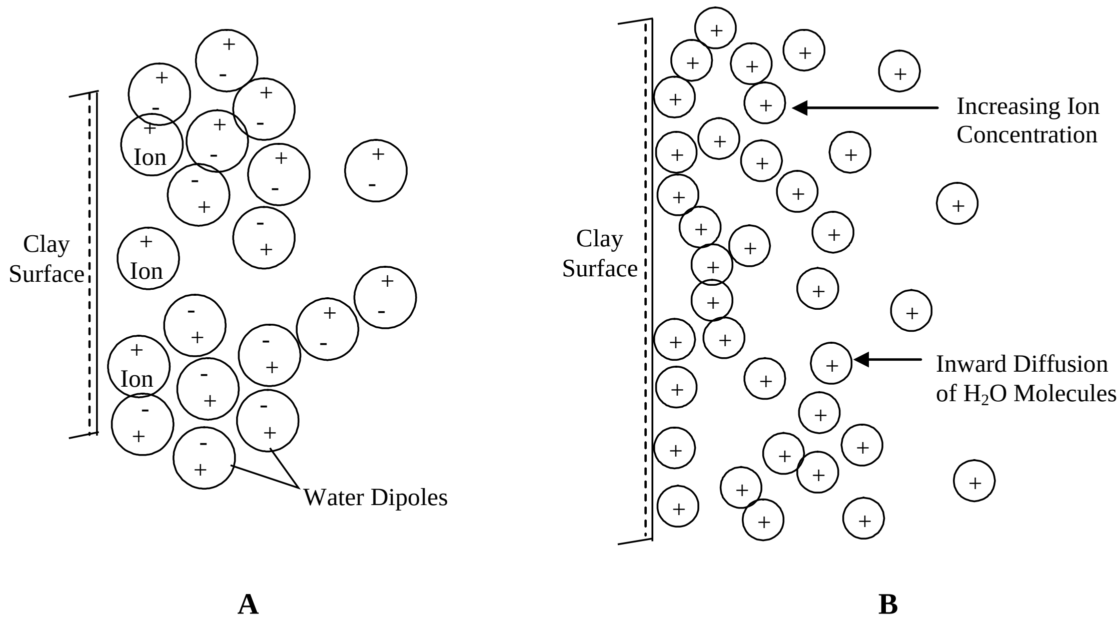

Figure 3. Formation of a diffused water layer around clay particle (Little, 1987).

Depending on the characteristics of the soil being treated, a pozzolanic reaction may also occur, resulting in the formation of various cementing agents that further increase mixture strength and durability. Pozzolanic reactions are time and temperature dependent. Therefore, given appropriate temperatures, strength development is gradual but continuous for long periods of time. Temperatures less than 55 to 60°F (13 to 16°C) retard the reaction, while higher temperatures accelerate the reaction.

A fourth reaction which may occur in the lime is carbonation, in which lime reacts with atmospheric carbon dioxide to form a relatively insoluble carbonate. This chemical reaction is detrimental to the stabilization process. It can be avoided by properly expedited and sequenced construction procedures that avoid prolonged exposure to the air and/or rainfall.

Cation Exchange and Flocculation–Agglomeration

Practically all fine-grained soils display cation exchange and flocculation–agglomeration reactions when treated with lime in the presence of water. The reactions occur quite rapidly when the soil and lime are intimately mixed.

Assuming equal concentrations, the general order of replaceability of the common cations is given by the Lyotropic series, $Na^+ < K^+ << Mg^{++} < Ca^{++}$ (Grim, 1953). In general higher valence cations replace those of lower valence, and larger cations replace smaller cations of the same valence. The addition of lime to a soil in a sufficient quantity supplies an excess Ca^{++} which replaces the weaker metallic cations from the exchange complex of the soil. The exchange of cations causes a reduction in the size of the diffused water layer, thereby allowing clay particles to approach each other more closely, or flocculate.

Flocculation and agglomeration produce an apparent change in texture, with the clay particles "clumping" together into larger-sized "aggregates" (Terrel et al., 1979). The flocculation and agglomeration are caused by the increased electrolyte content of the pore water and as a result of ion exchange by the clay to the calcium form. The net result of cation exchange and flocculation–agglomeration is soil modification (Little, 1987):

- Substantial reduction and stabilization of the adsorbed water layer.
- Increased internal friction among the agglomerates and greater aggregate shear strength.
- Much greater workability due to the textural change from a plastic clay to a friable, sand-like material.

Pozzolanic Reaction

The reactions between lime, water, soil silica, and alumina that form various cementing-type materials are referred to as pozzolanic reactions. The cementing products are calcium–silicate–hydrates and calcium–aluminate–hydrates, the same hydrates formed during the hydration of Portland cement (Terrel et al., 1979). Although a wide variety of hydrate forms can be obtained, the basic pozzolanic reaction is illustrated in the following equations:

$$Ca(OH)_2 \rightarrow Ca^{++} + 2(OH)^-$$
$$Ca^{++} + OH^- + SiO_2 \text{ (soluble clay silica)} \rightarrow \text{calcium–silicate–hydrate (CSH)}$$
$$Ca^{++} + OH^- + Al_2O_3 \text{ (soluble clay alumina)} \rightarrow \text{calcium–aluminate–hydrate (CAH)}$$

Possible sources of silica and alumina in typical fine-grained soils include clay minerals, quartz, felsdspars, micas, and other similar silicate or alumino-silicate minerals, either crystalline or amorphous in nature. The clay minerals and amorphous materials are the only important sources in most soils.

When a significant quantity of lime is added to a soil, the pH of the soil–lime mixture is elevated to approximately 12.4, the pH of saturated lime water. This is a substantial pH increase for natural soils. The solubilities of silica and alumina are greatly increased at these elevated pH levels. Thus, as long as enough residual calcium from the lime remains in the system and the pH remains high enough to maintain solubility, the pozzolanic reaction will continue (Little, 1987).

The extent to which the soil–lime pozzolanic reaction proceeds is influenced primarily by natural soil properties. Soils exhibiting a strength increase of greater than 50 lb/in^2 (345 kPa^2) after a 28-day curing period at 73°F (23°C) are considered "reactive," whereas those with strength increases lower than 50 lb/in^2 (345 kPa^2) are deemed non-reactive. Several soil properties and characteristics influence the lime reactivity of a soil, which include the following:

- Soil pH.
- Organic carbon content.
- Natural drainage.
- Excessive quantities of exchangeable sodium.
- Clay mineralogy.
- Degree of weathering.
- Presence of carbonates.
- Extractable iron.
- Silica-sesquioxide ratio.
- Silica-alumina ratio.

Carbonation

Lime carbonation is an undesirable reaction that may also occur in soil–lime mixtures. In this reaction, lime reacts with carbon dioxide to form calcium carbonate instead of the cementitious CAHs and CSHs.

Soils Suitable for Lime Stabilization

Since the beneficial effects of lime stabilization are the result of various reactions between the fines portion of the soil and lime, fine-grained soils, such as clay and silty-clay, respond most favorably. A minimum clay content of approximately 10 percent and a plasticity index (PI) greater than 10 are desirable, although benefits have been noted for lower PI silty soils containing less clay.

For low PI sands and non-plastic soils, a pozzolan additive is needed to produce the necessary lime-silica reaction. Flyash, volcanic ash, and expanded shale fines are examples of pozzolans that have been successfully incorporated.

Applications and Benefits

The effects of lime treatment or stabilization on pertinent soil properties can be classified as immediate and long-term. Immediate **modification** effects are achieved without curing and are of interest primarily during the construction stage. They are attributed to the cation exchange and flocculation–agglomeration reactions that take place when lime is mixed with the soil. Long-term **stabilization** effects take place during and after curing, and are important from a strength and durability standpoint. While these effects are generated to an extent by cation exchange and flocculation–agglomeration, they are primarily the result of pozzolanic strength gain.

Construction Facilitation (Subgrade Soil **Modification**)

The treatment of subgrade soils with lime facilitates construction work in three major ways. First, the addition of lime to the soil decreases the liquid limit and increases the plastic limit, which results in a significant reduction in the PI. This reduction in PI means a marked increase in workability, which, in turn, expedites manipulation and placement of the treated soil. The manner in which lime influences the plasticity characteristics is illustrated in Figure 4 and Table 2.

The second way in which construction is facilitated is through a change in the moisture–density relationship of the soil, as a result of more and more lime being added. The change reflects the new nature of the soil and is marked by a decrease in the maximum dry density and increase in the optimum moisture content. Figure 5 illustrates this phenomenon for a clay–loam soil.

Whereas prior to the application of lime, a soil in a fairly saturated state would not be capable of reaching maximum density, the same soil with lime may require additional moisture to achieve the new maximum density. The excess water is expended in the soil–lime reactions, thereby creating a less mud-prone construction platform.

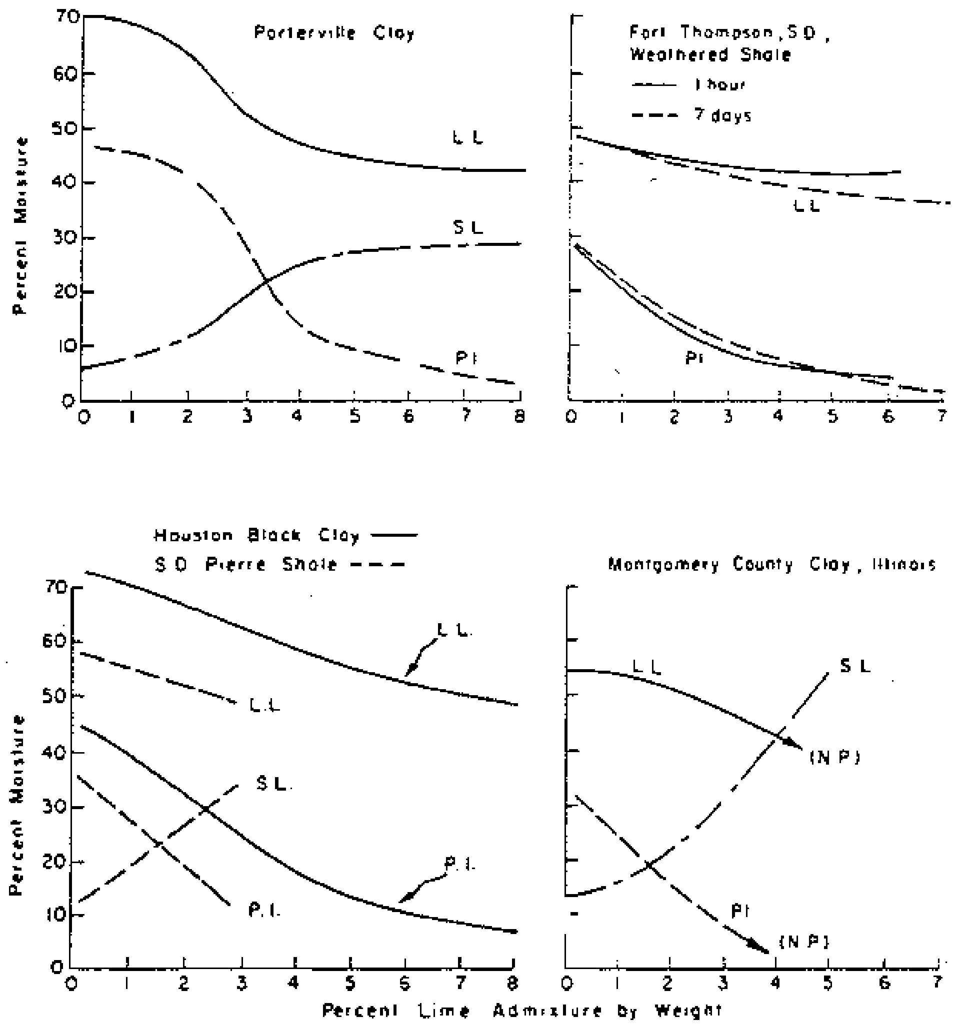

Figure 4. Effect of lime on plasticity characteristics of montmorillonite clays (TRB, 1987).

Table 2. Influence of lime on plastic properties (TRB, 1987).

Soil	AASHO Class	LL or PI by Percentage of Lime					
		None		3		5	
		LL	PI	LL	PI	LL	PI
Bryce B	A-7-6(18)	53	29	48	21	NP	NP
Cisne B	A-7-6(20)	59	39	NP	NP	–	–
Cowden B	A-7-6(19)	54	33	47	7	NP	NP
Drummer B	A-7-6(19)	54	31	44	10	NP	NP
Elliott B	A-7-6(18)	53	28	42	19	NP	NP
Fayette B	A-7-5(17)	50	29	NP	NP	–	–
Hosmer B	A-7-6(11)	41	17	NP	NP	–	–
AASHO Road Test	A-6(18)	25	11	27	6	27	5
Huey B	A-7-6(17)	46	29	40	9	NP	NP
Sable B	A-7-6(16)	51	24	NP	NP	–	–

Note: LL = liquid limit, PI = plasticity index, and NP = nonplastic.

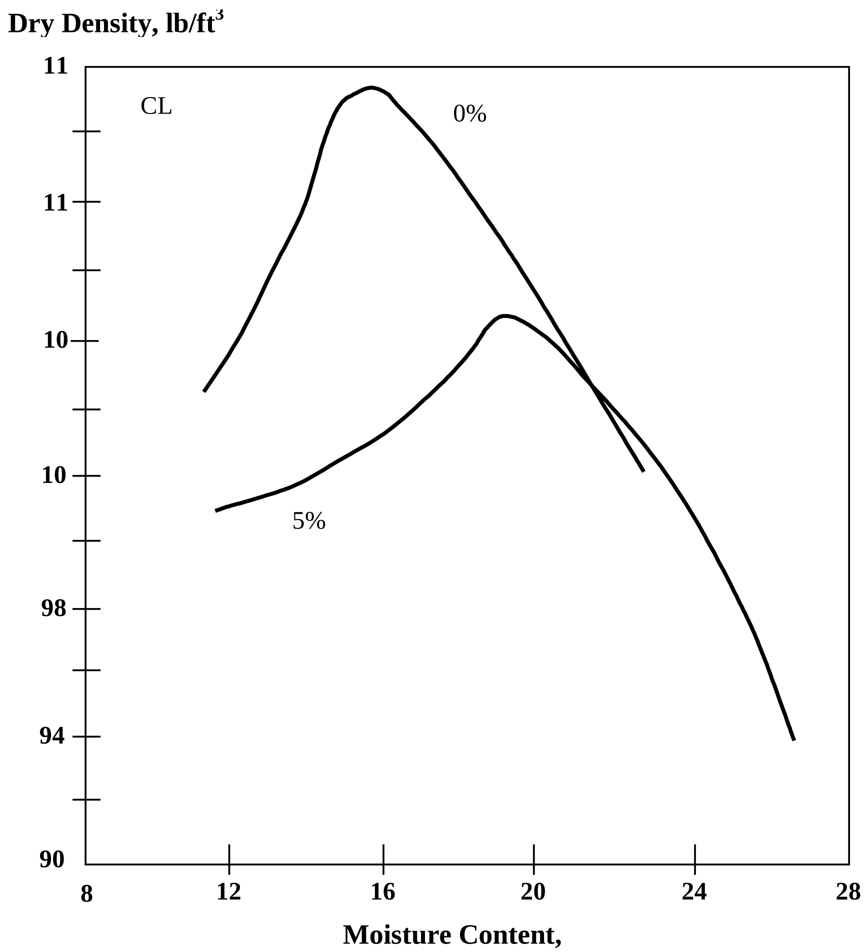

Figure 5. Shift in the moisture–density relationship of a soil, as a result of adding lime (Little, 1995).

The third major way in which construction is facilitated by the addition of lime to a soil is through immediate increases in soil strength and deformation properties. These improvements, which are largely the result of the flocculated particle structure, increase the mobility of wheeled vehicles involved in construction operations and they help provide a stable working platform for all construction equipment. Examples of the immediate increases in three specific strength parameters—unconfined compression strength, cone index, and the California Bearing Ratio (CBR)—are shown in Figure 6. As indicated by this figure, the strength increases in some cases can be much more than two-fold.

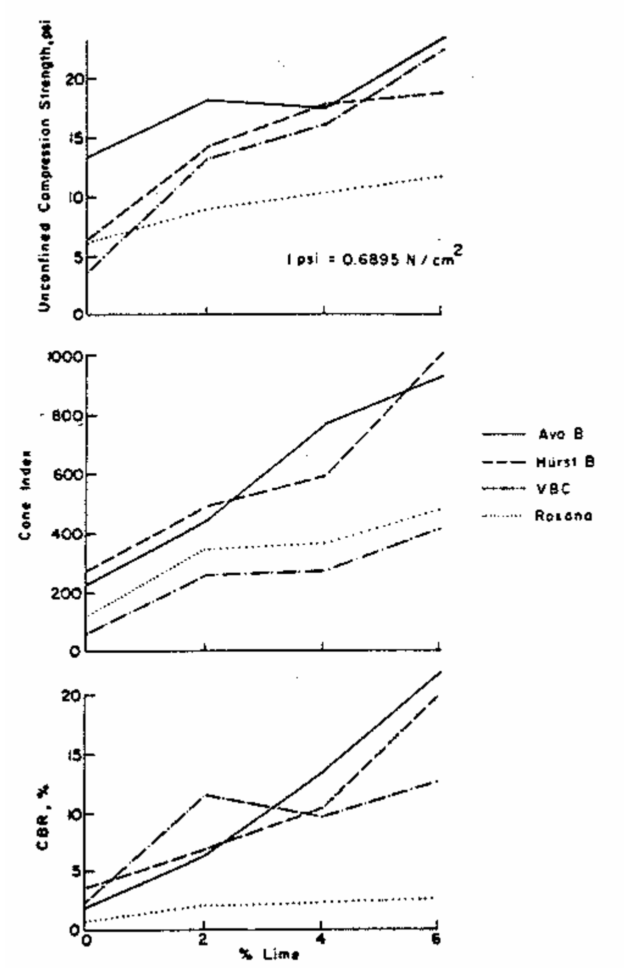

Figure 6. Immediate effects of lime treatment on soil strength (TRB, 1987).

<u>Treatment of Expansive Soils</u>

Soil swell potential and swelling pressure are normally significantly reduced by lime treatment (Little, 1995). In fact, the reduction in PI associated with virtually all fine-grained soils upon the addition of lime is a significant indication of the reduction of swell potential due to lime stabilization. One particular relationship developed between PI and swell potential is illustrated in Figure 7. In this plot, the percent swell is defined as volume change incurred in the soil as the moisture content increases from optimum moisture level to saturation level.

CBR swell values of lime-treated soils vary, but it is not uncommon to decrease swell to less than 0.1 percent, compared to values of 7 to 8 percent for untreated soil. Table 3 presents CBR and CBR swell values reported for several Illinois and Texas soils. Typical expansive pressures are shown in Figure 8 (TRB, 1987).

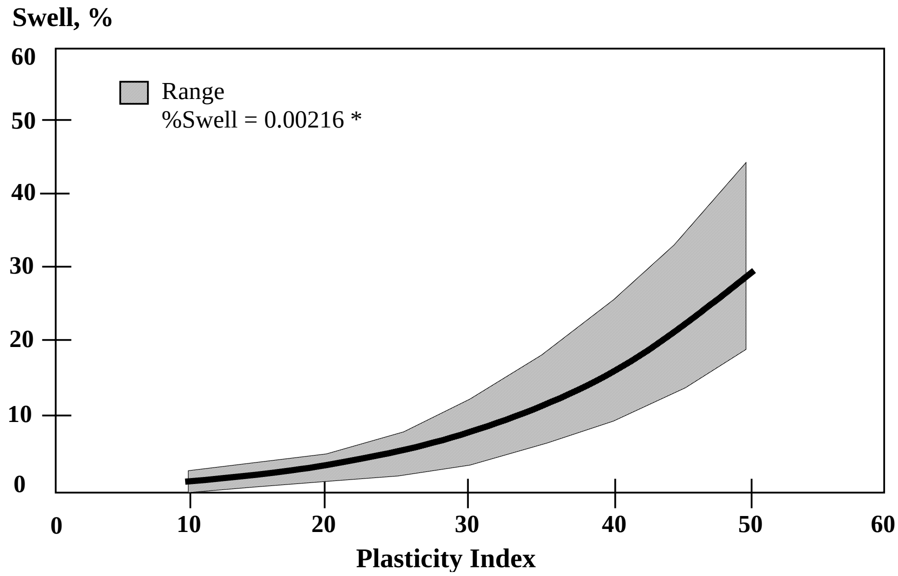

Figure 7. Swell potential as a function of plasticity index (Little, 1995).

Pavement Performance (Lime-Stabilized Subgrades and Base/Subbase Layers)

The material properties of both lime-stabilized soils and lime-stabilized aggregates, as related to their impact on overall pavement performance, can be divided into four categories (Little, 1999):

- Strength—The most obvious improvement in a lime-reactive soil or aggregate is strength gain over time. The various strength parameters impacted by the pozzolanic reactions that occur include unconfined compressive strength, tensile strength, flexural strength, and CBR. Examples of long-term strength increases for various lime-treated soils can be seen in Figure 9 and in Table 3. In both exhibits, the strength changes effected by lime percentage and time can be seen. Long-term field performance of lime-stabilized pavements has also been validated (Kentucky Transportation Center, 2002).

- Resilient modulus/stiffness—Concurrent with the strengthening of a soil brought about by pozzolanic reactions, are changes in the stress–strain relationship of the material (Little, 1999). Lime-stabilized soils fail at much higher deviator stresses than their nonstabilized counterparts, and at a much lower strain (typically about 1 percent strain for the stabilized mixture versus about 3 percent for the nonstabilized material). Materials tested in the laboratory (repeated-load triaxial and indirect tensile tests) and in the field (impulse deflection testing, vibrational testing) both confirm significant increases over time in the resilient properties of lime-treated materials. For instance, as shown in Figure 10, as the curing time for a lime-stabilized clay increases, the compressive stress at failure increases dramatically and the strain at failure correspondingly decreases.

Table 3. CBR values for selected soils and soil–lime mixtures (TRB, 1987).

Soil	Natural Soil		Percentage of Lime	Soil–Lime Mixtures			
				No Curing [a]		48-hr Curing @ 120°F	
	CBR, %	Swell, %		CBR, %	Swell, %	CBR, %	Swell, %
Reactive Soils							
Accretion Gley 2	2.6	2.1	5	15.1	0.1	351.0	0.0
Accretion Gley 3	3.1	1.4	5	88.1	0.0	370.0	0.1
Bryce B	1.4	5.6	3	20.3	0.2	197.0	0.0
Champaign Co. till	6.8	0.2	3	10.4	0.5	85.0	0.1
Cisne B	2.1	0.1	5	14.5	0.1	150.0	0.1
Cowden B	7.2	1.4	3	–	–	98.5	0.0
Cowden B	4.0	2.9	5	13.9	0.1	116.0	0.1
Cowden C	4.5	0.8	3	27.4	0.0	243.0	0.0
Darwin B	1.1	8.8	5	7.7	1.9	13.6	0.1
East St. Louis clay	1.3	7.4	5	5.6	2.0	17.3	0.1
Fayette C	1.3	0.0	5	32.4	0.0	295.0	0.1
Illinoian B	1.5	1.8	3	29.0	0.0	274.0	0.0
Illinoian till	11.8	0.3	3	24.2	0.1	193.0	0.0
Illinoian till	5.9	0.3	3	18.0	0.9	213.0	0.1
Sable B	1.8	4.2	3	15.9	0.2	127.0	0.0
Nonreactive Soils							
Fayette B	4.3	1.1	3	10.5	0.0	39.0	0.0
Miami B	2.9	0.8	3	12.7	0.0	14.5	0.0
Tama B	2.6	2.0	3	4.5	0.2	9.9	0.1

[a] Specimens were placed in 96-hr soak immediately after compaction.

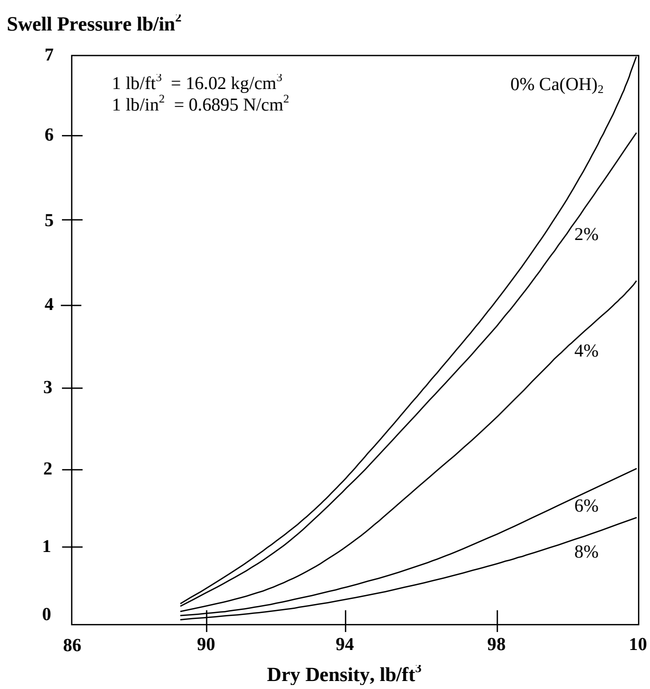

Figure 8. Swell pressure–density relationships for lime-treated Porterville clay (TRB, 1987).

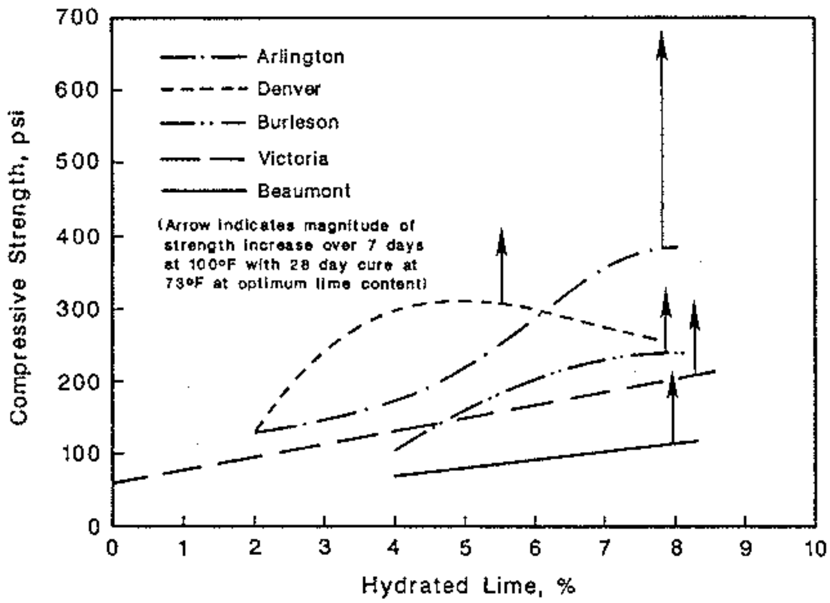

Figure 9. Long-term strength gain effects of various lime-treated soils in TX & CO (Little, 1995).

- Fracture and fatigue—Flexural fatigue strength is related to the number of loads that can be carried by a material at a given stress level, and it is an important consideration in the evaluation of lime–soil and lime–aggregate mixtures. The strength-gain effects produced by pozzolanic reactions are often substantial for reactive soils. The response curves of various cured lime–soil mixtures in Figure 11 are analogous to those obtained for some cement-treated aggregates. The strengths of these mixtures at 5 million stress applications varied from 41 to 66 percent of the ultimate flexural strength.

- Durability—The ability of lime-stabilized materials to resist the detrimental effects of moisture and freeze-thaw cycling over time has been evaluated in several ways, in both the laboratory (e.g., soaking in conjunction with strength/stiffness tests, cyclic freeze-thaw tests) and the field. The results of these evaluations have often shown only slight detrimental effects of environment on the levels of strength/stiffness produced by the addition of lime. As an example, an Illinois study found that the ratio of soaked to unsoaked compressive strength of lime–soil mixtures is quite high, at approximately 0.7 to 0.85 (Thompson, 1970). The soaked specimens seldom achieved 100 percent saturation and, in most cases, the degree of saturation was in the range of 90 to 95 percent.

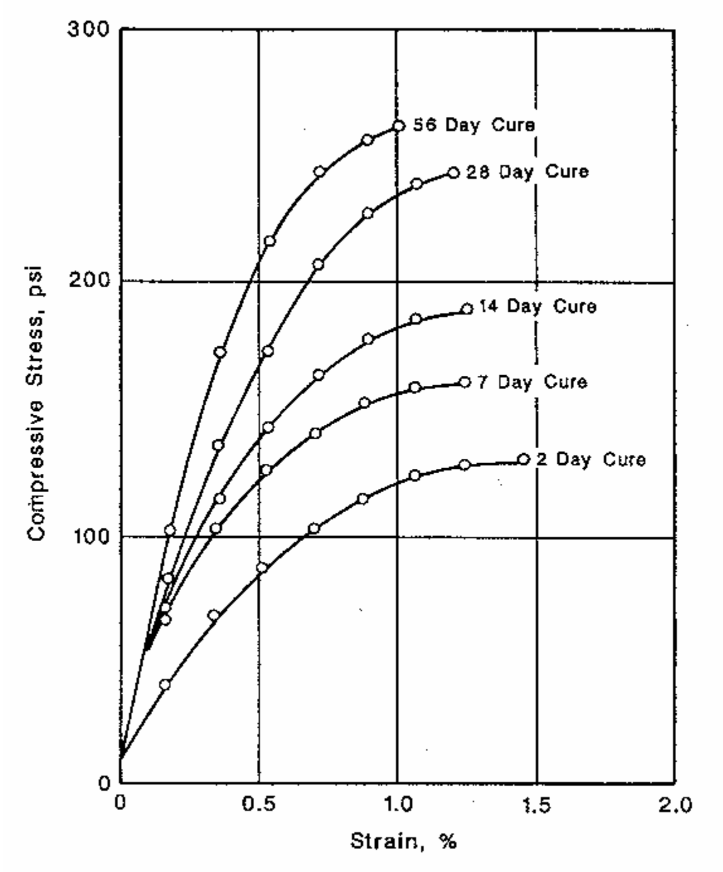

Figure 10. Change in stress-strain characteristics over time for a reactive, lime-stabilized soil (Goose Lake clay) (Little, 1995).

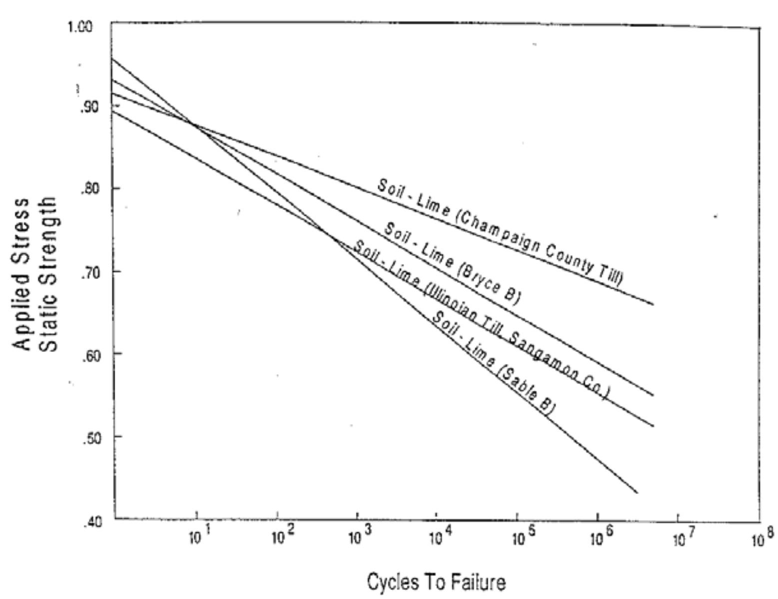

Figure 11. Flexural fatigue response curves for various lime-stabilized soils in Illinois (Swanson and Thompson, 1967).

SECTION II. M-E DESIGN CONSIDERATIONS FOR LIME STABILIZED LAYERS

This section provides detailed guidance on the rational incorporation of lime stabilized layers into flexible pavement cross-sections using the M-E Design Guide including: feasibility assessments of using lime for soil stabilization purposes, materials inputs required for lime stabilized layers in the M-E design approach; soil-lime materials characterization for new and rehabilitation design; and construction variability and ways to handle it during the design phase. This discussion expands and supplements the information presented regarding soil-lime mixtures in PART 2, Chapter 2 and PART 3, Chapters 3 and 6 of the M-E Design Guide.

Overview of the M-E Design Guide's Flexible Pavements Design Procedures [1]

The following flexible pavement types are considered in the M-E Design Guide approach:

- Conventional flexible pavements—i.e., thin HMA layer over granular base/ subbase materials.
- Deep strength HMA pavements—i.e., thick HMA layers over granular layers.
- Full-depth HMA pavements—i.e., sections consisting only of HMA layers (HMA surface and HMA stabilized bases).
- "Semi-rigid" pavements—i.e., sections having some type of a chemically stabilized layer.

The allowable pavement cross sections include both conventional layering (decreasing material quality with depth) and sandwich structures (unbound aggregate layer placed between two stabilized layers).

The M-E Design Guide requires an iterative hands-on approach by the designer. The designer must first select a trial design and then analyze the design in detail to determine if it meets the performance criteria. Performance criteria considered in the M-E Design Guide for flexible pavements include permanent deformation in each pavement layer and in the foundation soil (rutting), fatigue cracking (both bottom-up and top-down), thermal cracking, and smoothness (International Roughness Index or IRI). For semi-rigid pavements, fatigue cracking of the chemically stabilized base layer is an additional performance consideration. If the trial design does not satisfy the performance criteria, the design is modified and reanalyzed until the design satisfies all criteria. The designs that meet the applicable performance criteria are then considered feasible from a structural and functional viewpoint and can be considered in further evaluations, such as life cycle cost analysis and constructability issues.

The main steps in the iterative design process for asphalt pavements are summarized below. The term "asphalt pavement" refers to any new, reconstructed, or rehabilitated pavement system that has HMA as the surface layer.

[1] Lime stabilized layers can also contribute to the performance and longevity of rigid pavement systems.

1. Assemble a trial design for specific site conditions—define subgrade support, HMA and other paving material properties, traffic loads, climate, pavement type, and design and construction features.
2. Establish criteria for acceptable pavement performance at the end of the design period (i.e., acceptable levels of rutting, fatigue cracking, thermal cracking, and IRI).
3. Select the desired level of reliability for each of the applicable performance indicators (e.g., select reliability levels for rutting, cracking, and IRI).
4. Compute structural responses (stresses and strains) using multilayer elastic theory for each axle type and load and for each damage-calculation increment throughout the design period.
5. Calculate accumulated distress and/or damage at the end of each analysis period for the entire design period.
6. Estimate key distresses (rutting, bottom-up/top-down fatigue cracking, thermal cracking) at the end of each analysis period throughout the design life using the calibrated mechanistic-empirical performance models provided in the Guide.
7. Calculate smoothness (IRI) as a function of initial IRI, distresses that accumulate over time, and site factors at the end of each analysis increment.
8. Evaluate the expected performance of the trial design at the given reliability level.
9. If the trial design does not meet the performance criteria, modify the design and repeat the steps 4 through 8 above until the design does meet the criteria.

Lime stabilized subgrade soils can be incorporated as subbase layers for of conventional and deep strength asphalt pavements. The performance indicators or distresses of interest for such pavements include rutting, fatigue cracking, thermal cracking, and smoothness. Well-designed and constructed lime stabilized layers can minimize the development of these pavement distresses and are therefore a viable alternative to be considered in structural design of pavements using the M-E Design Guide approach.

Under certain special circumstances, such as low-volume roads, lime stabilized subgrade layers can function as a rigid base layer, i.e., the wearing course can be constructed directly on top of these layers. When used in this manner, however, proper attention should be directed to providing sufficient cover thickness, selecting minimum modulus values, and addressing durability issues. An additional performance criterion of interest in such systems is the fatigue cracking of the lime stabilized layer.

Feasibility Assessment

The first step in incorporating lime stabilized layers into the pavement cross-section is to confirm that the desired soil-lime reactions will occur for the target materials. Experience has shown that lime will react with medium, moderately fine, and fine-grained soils to produce decreased plasticity, increased workability, reduced swell, and increased strength and stiffness. Generally speaking, those soils classified by the Unified Classification System as CH, CL, MH, SC, SM, GC, SW-SC, SP-SC, SM-SC, GP-GC, and GM-GC are potentially capable of being either modified or stabilized by lime (Little, 1995). Similarly, aggregates with plastic fines, such as clay-gravel, "dirty" gravels and limestone, caliche and other marginal bases which contain an excess amount of material passing the No. 40 sieve, will react with lime favorably to produced base or subbase layers with improved pavement properties (Little, 1995).

Figure 12 presents a procedure to quickly determine the feasibility of using lime to either modify or stabilize soils or marginal aggregates based on soil index properties such as minus No. 200 material and plasticity index (PI) (Currin et al., 1976).[2] Based on this preliminary assessment, a more rigorous laboratory testing procedure should be established to confirm that the beneficial effects will be obtained for a specific soil-lime mixture. The amount of laboratory testing is directly proportional to the objective of lime treatment – modification or stabilization.

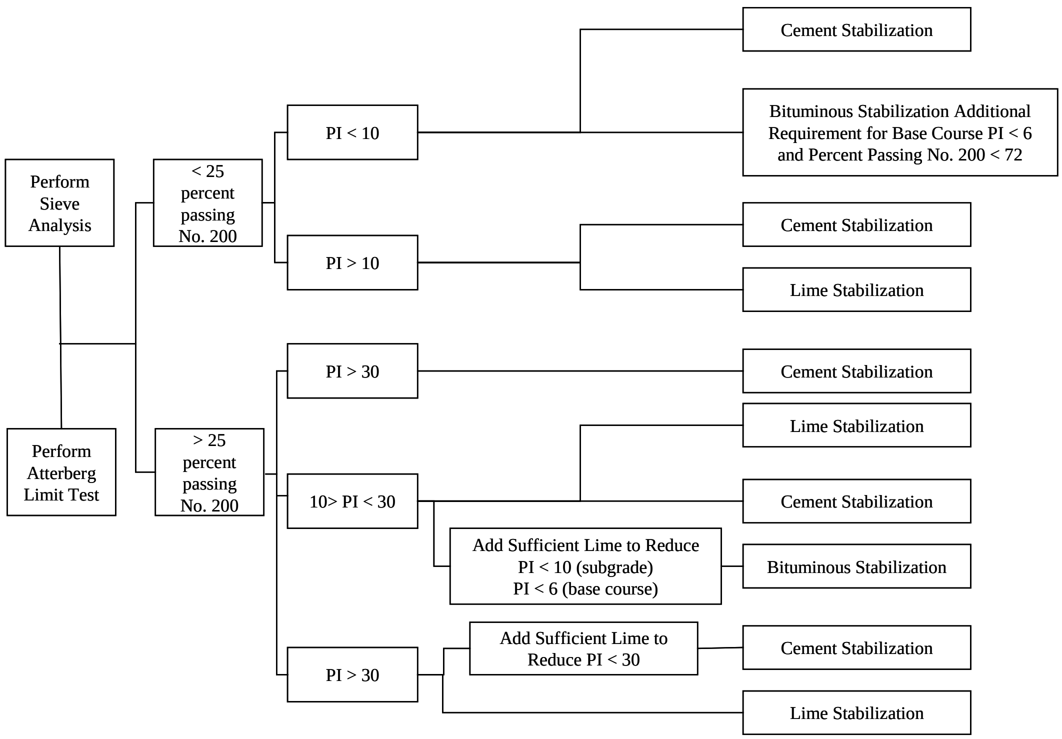

Figure 12. Determination of appropriate stabilizer type based on minus No. 200 content and plasticity index (after Currin et al., 1976).

The extent to which soil-lime pozzolanic reactions occur is influenced by the type and clay mineral present in the soil. Other significant factors that influence the soil-lime reactions and the modify-cation or stabilization process include the presence and amount of organics and salts in the soil.[3]

When considering a lime stabilized layer as a structural layer in a pavement (e.g., base or subbase layers), it is important that the enhanced strength and stiffness properties of the soil are sustained over the design life. This implies that the soil-lime mixtures are durable over several wet-dry and freeze-thaw cycles and are resistant to leaching.

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[2] Lime-fly ash stabilization can also be considered for low to moderate plasticity soils.

[3] As a general rule, soils with organic contents in excess of 1 percent may be difficult to lime stabilize or may require uneconomical quantities of lime to stabilize (Little, 1999). High concentrations of sulfate salts (sodium, magnesium, or calcium sulfates) may also interfere with or affect stabilization through deleterious interactions with lime, soil minerals, and water (Little, 1999). As a general rule, if the total soluble sulfate level is greater than about 0.3 percent in a 10:1 water to soil solution, then additional precautions to guard against sulfate reactions, such as swell tests and special construction procedures, may be warranted (NLA, 2001).

Sources of Information for Feasibility Assessment

An important and good source of information about soils and their potential for lime stabilization is the United States Department of Agriculture's County Soil Reports (Little, 1995). The information contained in county soil reports includes:

- Typical gradations, Atterberg limits, soil classifications (UCS and AASHTO), pH values, permeabilities, cation concentrations, drainage characteristics, and mineralogical descriptions of the various soil types in the county.
- Aerial photographic maps of the county with soil types superimposed to aid in the location of soil types for a given project.
- Summary of the basic geology of the area.
- Discussion of soil development or pedology of the area and each soil type.

This information can be used for the feasibility assessment.

Materials Inputs Required for Lime Stabilized Subgrade Layers in the M-E Design Guide Approach

Introduction

This section describes the material inputs required for lime stabilized subgrade soils for new and rehabilitated pavements in the M-E Design Guide. It should be noted that this discussion pertains only to lime stabilized subgrades which qualify as structural subbase layers in conventional and deep strength asphalt pavements. In the M-E Design Guide, the lime stabilized subgrades are treated as moisture insensitive materials with the modulus being constant over time.

The materials inputs required for the M-E Design Guide approach generally fall under three subcategories.

- Material properties required for computing pavement responses.
- Materials properties required for climatic effects modeling.
- Materials properties required for distress/transfer functions.

Pavement Structural Response: Lime stabilized soils are treated as being linearly elastic in the M-E Design Guide. Resilient modulus (M_r) and Poisson's ratio (μ) are required to characterize these layers for calculating structural response.

Climatic Effects: Lime stabilized layers are assumed to be moisture and frost insensitive in the M-E Design Guide. However, unit weight, specific heat, and thermal conductivity of lime-stabilized materials are required in calculating the moisture, temperature, and frost regime throughout the pavement and subgrade soil.

Distress Prediction: Lime stabilized material can be used as a base layers in low volume road applications, In such cases, the lime stabilized base layer must resist flexural and tensile stresses to prevent the occurrence of fatigue cracks. Thus, its fatigue strength needs to be considered. In such instances, the number of allowable load applications is calculated in accordance with equation 1.

$$N_f = 10^{\left(\frac{k_1\beta_{c1} - \left(\frac{\sigma_s}{MR}\right)}{k_2\beta_{c2}}\right)} \qquad (1)$$

where,

N_f	=	Number of repetitions to fatigue cracking.
σ_s	=	Tensile stress at the bottom of the lime-stabilized layer, psi.
MR	=	Flexural strength of the lime stabilized layer, psi.
k_1, k_2	=	Regression coefficients from national calibration representative of material properties.
β_1, β_2	=	Regression coefficients from national calibration representative of material properties.

The required material input, in addition to M_r and μ for distress prediction, is the modulus of rupture or flexural strength (MR) of the lime stabilized layer. Further the regression coefficients in equation (1) are also material inputs to the distress/transfer functions that describe the fatigue characteristics of the material. A set of default values is embedded in the M-E Design Guide software. These default values have not been fully confirmed through the calibration process.[4] Regional or state-wide values should be developed for local use.

If the lime stabilized layers are only used as a high modulus, moisture insensitive material, then fatigue damage and the additional materials characterization steps are not needed. This will be the case in most common applications of lime stabilization, since the layer is usually placed deeper in the structure where fatigue failure and the associated reflective cracking is not an issue.

Hierarchical Input Levels

The general approach for selecting or determining design inputs for materials in the Design Guide is a hierarchical (level) system. In its simplest and most practical form, the hierarchical approach is based on the philosophy that the level of engineering effort exerted in the pavement design process should be consistent with the relative importance, size, and cost of the design project.

Level 1 is the most detailed procedure, normally involving comprehensive laboratory or field tests. In contrast, Level 3 allows the designer to estimate the design input value based on experience or historical records. Level 3 inputs require the least amount of testing, while Level 2 inputs

[4] There were few test sections with sufficient materials test data for all pavement layers, including lime stabilized layers, included in the calibration and validation process of the distress prediction models developed under NCHRP Project 1-37A. For those LTPP test sections with lime-stabilized materials, most of the inputs had to be estimated for the calibration process. The default values can be changed using the "expert input mode" to be more representative of the specific material type being modeled.

Laboratory flexural fatigue tests can generate material-specific values, but these tests are difficult, expensive, and time consuming. Thus they are not expected to be performed on a project by project basis.

represent a compromise between the testing and field evaluation requirements for input Level 1 and using a "best estimate" value for input Level 3. The advantages of this hierarchical approach include the following:

- Provides the engineer with greater flexibility in selecting an engineering approach consistent with the size, cost, and overall importance of the project.
- Allows each agency to develop an initial design methodology consistent with its internal technical capabilities.
- Provides a very convenient method to increase an agency's technological skills gradually over time.
- Ensures the development of the most accurate and cost-efficient design, consistent with agency financial and technical resources.
- Allows the agency to initially use the M-E Design Guide without major investment of personnel time and funds.

The selection of the hierarchical level while configuring inputs for a particular material type will be highly influenced by whether the design is for a new or rehabilitated pavement system. Existing pavement structures provide an in-situ "laboratory" of the exact state of damage and behavior of each material. For example, the moduli of existing pavement layers can be estimated from falling weight deflectometer (FWD) deflection basin data or through the use of dynamic cone penetrometer (DCP) testing. Thus, any hierarchical system for materials should discriminate between new and rehabilitated structures.

Estimation of Resilient Modulus

New or Reconstructed Pavement Design

The required modulus input is the 28-day resilient modulus (M_r) of the lime stabilized soil cured at room temperature (73°F). The stress state (deviatoric stress and confining pressure) at which the M_r should be estimated can be determined from structural analysis of the trial design (after properly accounting for overburden pressure). Note that configuring a trial pavement cross-section for a given set of site factors (traffic, subgrade, and climate) is a first step in pavement design using the M-E Design Guide approach. In the absence of the capability to perform structural analysis to estimate the stress state within the lime stabilized layer, an M_r value determined at a deviatoric stress of 6 psi is adequate for design purposes (after Little, 2000).

Level 1. At input level 1, M_r of the lime stabilized soils is determined in accordance with a mixture design and testing protocol (MDTP) (Little, 2000). The AASHTO T-307 test protocol or an equivalent test can be used to estimate the resilient modulus. The MDTP requires the resilient modulus test be conducted on lime-soil mixtures containing the target lime content and molded and conditioned at optimum moisture and maximum density. The soil-lime mixtures are cured for 7 days at 104 °F in plastic bags to retain sufficient moisture for the curing process. This is a reasonable period of accelerated cure and is representative of the 28-day curing at room temperatures.

Level 2. At input level 2, M_r of the lime stabilized soils can be estimated from unconfined compressive strength (q_u) of the cured soil-lime samples performed in accordance with the MDTP (Little, 2000). The 28-day q_u is measured and determined in accordance with ASTM D 5102 as part of the MDTP. The MDTP should be adopted for the determination of lime demand for the soil, fabrication, curing, and soaking prior to determining the unconfined compressive strength. M_r can subsequently be estimated from q_u as follows (Thompson, 1966):

$$M_r = 0.124(q_u) + 9.98 \tag{2}$$

where,
M_r = resilient modulus, ksi.
q_u = unconfined compressive strength, psi.

Level 3. At input level 3, M_r is estimated from experience or historical records. The literature demonstrates that resilient moduli of 30,000 to 60,000 psi can be readily achieved for reactive soils (with 25 percent passing No. 200 sieve and PI of at least 10). However, the exact extent of increase is a function of the soil mineralogy and lime content.

Rehabilitation Design with HMA Overlays

The stiffness of a lime stabilized layer can change over time. Concurrent with the increasing in lime-soil mixture strength and stiffness due to pozzolanic activity, the material also degrades to repeated application of freeze-thaw cycles and traffic loading. Further, shrinkage cracking can occur in lime stabilized layers further contributing to a loss in material integrity. For HMA overlay designs using the M-E Design Guide approach, the modulus estimates of all existing layers, including lime stabilized layers, must account for the damage already induced before HMA overlay placement.

Level 1. At input level 1, the modulus at the current damage level can be obtained from non-destructive evaluation of the pavement being rehabilitated using FWD. The layer moduli can be determined from the FWD deflection basin data using standard backcalculation programs. It is recommended that FWD testing be conducted over the entire project length to more accurately estimate the project mean inputs required for design. Since layer thicknesses are important inputs to the backcalculation process, it is recommended that select coring be performed to verify layer thicknesses. Alternatively, other nondestructive testing techniques such as the GPR can also be used to determine layer thicknesses. Further, it is recommended that limited testing be performed on cored lime stabilized soil specimens to verify or confirm the backcalculated values.

Based on the backcalculation and coring, if the lime stabilized layers are behaving as rigid layers, no adjustment is necessary to the modulus values derived to account for seasonal effects. Backcalculation of modulus values for layers less than 6 inches thick located below other paving layers can be problematic. When the lime stabilized layer is less than 6 inches, samples may be need to be recovered for laboratory testing.

Level 2. At input level 2, the modulus estimates of the lime stabilized layer can be derived by correlating the compressive strength of the retrieved cores to the resilient modulus using equation 2. Alternatively, the DCP can be used to obtain estimates of stiffness. The DCP provides a measure of resistance to penetration under an impact load that has been effectively correlated to in situ modulus in the M-E Design Guide approach.

Level 3. At level 3, the in situ resilient modulus of the lime stabilized layers is determined from the estimates of the intact modulus, E_{max}, and the current damage to the lime stabilized layer, d_{LS}. The intact modulus is obtained from correlation to compressive strength (equation 2) using typical compressive strengths based on material description. The current damage is obtained from a general condition rating of the pavement. Table 4 presents an example of how pavement condition can be related to damage. With the intact modulus and current damage level known, the pre-overlay modulus can be obtained from equation 3.

Table 4. Example of damage estimation for lime stabilized layers based on pavement condition rating.

Pavement Condition Rating	Estimated Damage, d_{LS}
Excellent	0.00 – 0.40
Good	0.40 – 0.80
Fair	0.80 – 1.20
Poor	1.20 – 1.60
Very Poor	> 1.60

$$E_{LS} = X + \frac{\left(E_{\max} - X\right)}{1 + e^{\left[-a+b\left(d_{LS}\right)\right]}} \qquad (3)$$

where,
E_{LS} = Modulus of lime stabilized layer, psi.
E_{max} = Intact modulus, psi.
d_{LS} = Damage in lime stabilized layer.
X, a, b = Regression constants.

Estimation of Flexural Strength

New or Reconstructed Pavement Design

The fatigue life of lime-soil mixtures is linked to the critical flexural stress induced within the stabilized layer as is evident from Equation 1. The required MR input for design purposes is the 28-day value of the lime stabilized soil cured at room temperature (73°F).

Leve1 1. At input level 1, MR should be estimated from laboratory testing of beam specimens of lime stabilized soils. However, historically, flexural fatigue testing of lime stabilized mixtures has rarely been conducted, and there is no AASHTO or ASTM method for flexural strength testing of lime stabilized mixtures. Therefore, hierarchical input level 1 is not applicable for this material parameter.

Level 2. At input level 2, MR can be estimated from unconfined compressive strength (q_u) testing of the cured soil-lime samples performed in accordance with the MDTP (Little, 2000). This test has been described previously. MR can be conservatively estimated as being 20 percent of the q_u (Little, 2000).

Level 3. At input level 3, MR is estimated from experience or historical records based on material description.

Rehabilitation Design with Asphalt Overlays

Level 1. Input level 1 is not applicable for the determination of MR of lime stabilized mixtures due to the lack of a nationally recognized flexural strength testing protocol.

Level 2. At input level 2, MR estimates of the lime stabilized layer can be derived by correlating the compressive strength of the retrieved cores to the flexural strength as described earlier ($MR = 0.2 * q_u$). Alternatively, the DCP can be used to obtain estimates of in situ strength which can then be used to estimate flexural strength.

Level 3. At level 3, the in situ MR of the lime stabilized layers is determined from estimates of typical compressive strengths based on material description and the current pavement damage level. The current damage is obtained from a general condition rating of the pavement as described in Table 4.

Estimation of Poisson's Ratio

Another important input required for structural analysis in the M-E Design Guide approach is the Poisson's Ratio of the lime stabilized layers. Although this parameter can be determined from laboratory tests, testing is not recommended by the M-E Design Guide. Typical values may be used for new, reconstruction, and rehabilitation design with overlays. Values between 0.15 and 0.2 are typical for lime stabilized soil layers. A value of 0.20 is recommended in the M-E Design Guide.

Estimation of Thermal Conductivity and Heat Capacity

Several materials-related inputs are required for the design of flexible pavements and asphalt concrete overlays in the M-E Design Guide approach. Among these properties are those that control the heat flow through the pavement system and thereby influence the temperature and moisture regimes within it. Changing temperature and moisture profiles in the pavement structure and subgrade over the design life of a pavement are fully considered in the Design Guide approach through a sophisticated climatic modeling tool called the Enhanced Integrated Climatic Model (EICM). The Enhanced Integrated Climatic Model is a one-dimensional coupled heat and moisture flow program which simulates changes in the behavior and characteristics of pavement and subgrade materials in conjunction with climatic conditions over several years of operation. This program has been developed over three decades of research conducted by several researchers (Dempsey et al., 1985; Guymon et al., 1986; Lytton, 1990; Larson and Dempsey, 1997; NCHRP 1-37A).

The specific material properties pertaining to each layer that enter the EICM moisture and temperature calculations include:

- Thermal conductivity, ***K***.
- Heat or thermal capacity, ***Q***.

Thermal conductivity, ***K***, is the quantity of heat that flows normally across a surface of unit area per unit of time and per unit of temperature gradient. The moisture content has an influence upon the thermal conductivity of lime stabilized. If the moisture content is small, the differences between the unfrozen, freezing, and frozen thermal conductivity are small. Only when the moisture content is high (e.g., greater than 10%) does the thermal conductivity vary substantially. The EICM does not vary the thermal conductivity with varying moisture content of the lime stabilized layers and other bound layers as it does with the unbound layers.

The heat or thermal capacity is the actual amount of heat energy ***Q*** necessary to change the temperature of a unit mass by one degree. Table 5 outlines the recommended approaches to characterizing ***K*** and ***Q*** at the various hierarchical input levels for lime stabilized layers.

Table 5. Recommended approach for thermal conductivity and heat capacity estimation of lime stabilized layers for EICM calculations.

Material Property	Input Level	Description
Thermal Conductivity, K	1	A direct measurement is recommended at this level (ASTM E1952).
	2	Not applicable.
	3	User selects design values based upon agency historical data or from typical values shown below: • Typical values for lime stabilized layers range 1.0 to 1.5 Btu/(ft)(hr)(°F). A typical value of 1.25 Btu/(ft)(hr)(°F) can be used for design.
Heat Capacity, Q	1	A direct measurement is recommended at this level (ASTM D2766).
	2	Not applicable.
	3	User selects design values based upon agency historical data or from typical values shown below: • Reasonable values range from 0.2 to 0.4 Btu/(lb)(°F). A typical value of 0.28 Btu/(lb)(°F) can be used for design.

Mixture Design and Testing Protocol (MDTP) for Lime Stabilized Soils

Relationship Between Mixture Design Procedures and Structural Design in the M-E Design Guide

The M-E Design Guide does not make specific recommendations regarding the mix design procedures to be adopted for each material being considered in the pavement structure. As noted in the previous section, the M-E Design Guide merely specifies the strength and stiffness inputs required for structural response calculation and for use in distress prediction along with recommendations for appropriate test protocols to arrive at these values. The designer, however, is encouraged to adopt the best mix design practices when characterizing pavement materials. The integration of good materials design practices with sound structural design concepts is in fact one of the strongest features of the M-E pavement design approach. The materials inputs required for lime stabilized materials in the M-E Design Guide for new and reconstructed pavements assume that proper laboratory characterization has been carried out beforehand to estimate the following:

- Assessment of suitability of the given project material for lime stabilization.
- Target lime content required to achieve stabilization objectives.
- Moisture/Density relationships.
- Durability of the mixtures and longevity of the lime-soil reactions.

The required strength and stiffness properties are estimated at the target lime contents, moisture contents, and compaction levels determined from mixture testing. The validity of the structural design inputs is predicated upon the fact that a successful mix design which satisfies the stabilization objectives is accomplished. In this context, the MDTP developed by Little (2000) is highly recommended to design lime-stabilized soil mixtures. The issues addressed by MDTP include:

- Classification of soil and determination of lime demand.
- Sample preparation.
- Testing procedures to determine strength and stiffness properties.

The resilient modulus estimation procedure for the M-E Design Guide follows the recommendations in the MDTP (Little, 2000).[5]

Overview of the MDTP

This section presents an overview of the steps involved in developing Level 1 or Level 2 inputs for a lime stabilized layer that possesses the desired structural properties and durability. A detailed outline and discussion of this procedure is provided through other reports (Little, 2000).

Step 1: Soil Classification and Assessment of Suitability for Lime Stabilization

1. Determine if the soil has at least 25 percent passing the No. 200 sieve and has a plasticity index of at least 10 (Currin et al., 1976).
2. Verify that the organic contents do not exceed 1 percent by weight.
3. Ensure that soluble sulfates are less than 0.3 percent by weight in a 10:1 water-to-soil solution. Note that if the sulfate contents exceed 0.3 percent by weight, special construction procedures are recommended.

Step 2: Perform Eades and Grim pH test to Determine Approximate Optimum Lime Content

Perform Eades and Grim pH test in accordance with ASTM D 6276 to determine lime demand and to ensure lime availability for immediate lime-soil reactions as well as to ensure a high system pH (about 12.4 at 77 °F). This is necessary to provide proper conditions for long-term pozzolanic reactions and strength and stiffness development. It also promotes longevity or permanency of lime treatment and minimizes the risk of leaching effects or strength loss.

Step 3: Determine Moisture/Density Relationships and Moisture Sensitivity and Strength Gain following Accelerated Curing

1. Determine moisture/density relationship according to the appropriate protocol defined by the user agency, e.g., AASHTO T-99 or T-180.
2. Prepare samples for strength testing and moisture sensitivity testing at optimum moisture content within a tolerance of $\pm$ 1 percent.

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[5] There is one difference: the M-E Design Guide recommends only nationally recognized testing standards, i.e., ASTM or AASHTO approved tests. The MDTP also discusses newer test protocols such as the Rapid Triaxial Tester (RaTT) for resilient modulus estimation and tube suction test (TST) for moisture sensitivity.

3. Cure samples for 7 days at 104 °F in sealed plastic bags.
4. Subject samples to capillary soak 24-hours prior to strength testing.

Step 4: Perform Strength Testing to Determine the Unconfined Compressive Strength

The unconfined compressive strength of the soaked sample is determined using ASTM D 5102. Note that this parameter is used to estimate the design M_r at hierarchical input level 2 in the M-E Design Guide approach.

Step 5: Perform Resilient Modulus Testing

The resilient modulus is a direct design input at hierarchical input level 1 in the M-E Design Guide approach. The resilient modulus of the soaked sample is determined using AASHTO T-307[6] or a Rapid Triaxial Tester (RaTT). In the Design Guide, AASHTO T-307 is recommended in lieu of the RaTT since it is a national standard.

Step 6: Perform Tube Suction Test (TST) to Evaluate Moisture Sensitivity

The dielectric value measured by a tube suction test is a measure of how much moisture a given material will absorb through capillary rise and the state of bonding of the absorbed moisture. Low dielectric values indicate the presence of tightly absorbed moisture. This test is recommended for lime stabilized layers to determine their moisture sensitivity. Higher affinity to moisture could lead to durability problems. Note that if the lime-stabilized mixture has a high sensitivity to moisture, the resilient modulus and flexural strength should be measured at the higher moisture contents and used in design.

Economic Benefits

Today's highway pavement designers are charged not only with developing a feasible set of design cross-sections for individual construction projects, they are responsible for selecting the most appropriate design based on full consideration of many factors. One factor that has received increased attention over the years is economic benefits, particularly as it relates to long-term investment return. Thus, to make the most of taxpayer's dollars, today's designers are challenged with identifying the design that best satisfies all engineering and constructability requirements, while yielding the lowest overall projected cost of building and maintaining the structure over many years. Through a process known as life-cycle cost analysis (LCCA), designers can assess the economic benefits associated with different design features, including the provision of lime-treated subgrades and/or subbases.

[6] Note that the resilient modulus testing protocol adopted by Little (2000) in the MDTP protocol was AASHTO T-294 which has since been replaced by AASHTO T-307.

Corresponding to the vast material property improvements that are capable of being made by lime when added to subgrade soils and base/subbase aggregates, there is generally a significant increase in the performance and/or service life of the pavement structure in which the lime is used. Although an additional up-front cost for purchasing and incorporating the lime into the designated foundation materials is to be expected, this cost can be offset by reduced thickness requirements for other pavement layers, reduced use of imported aggregates, and future savings associated with a delay in pavement rehabilitation and reductions in the frequency and amounts of pavement maintenance. Thus, in terms of total life-cycle costs, there are distinct advantages to using lime–soil and lime–aggregate mixtures over untreated materials, as well as other forms of treated materials.

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APPENDIX A – TERMS AND DEFINITIONS

General Pavement Terms

- ***Surface***—The uppermost layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate.
- ***Base***—The layer or layers of specified or select material of designed thickness placed on a subbase or subgrade to support a surface course. For pavements with a Portland cement concrete (PCC) surface, the layer directly beneath the PCC slab is called the base layer (NCHRP 1-37A).
- ***Subbase***—The layer or layers of specified or selected materials of designed thickness placed on a subgrade to support a base course (NCHRP 1-37A).
- ***Subgrade***—The top surface of a roadbed upon which the pavement structure and shoulders are constructed (NCHRP 1-37A).
- ***Flexible Pavement***—A pavement structure that maintains intimate contact with and distributes loads to the subgrade and depends on aggregate interlock, particle friction, and cohesion for stability (NCHRP 1-37A).
- ***Conventional HMA***—Flexible pavement consisting of a thin hot mix asphalt (HMA) surface layer placed on a granular base/subbase.
- ***Deep-Strength HMA***—Flexible pavement consisting of a thick set of HMA surface and base layers placed on a granular subbase (NCHRP 1-37A).
- ***Full-Depth HMA***—Flexible pavement consisting only of HMA layers (e.g., HMA surface and HMA stabilized bases/subbases) (NCHRP 1-37A).
- ***Semi-Rigid Flexible Pavement***—Flexible pavement containing some type of a chemically stabilized layer (NCHRP 1-37A).
- ***Rigid Pavement***—A pavement structure that distributes loads to the subgrade, having as one course a PCC slab of relatively high-bending resistance (NCHRP 1-37A).

Pavement Materials Terms

- ***Soil***—Sediments or other unconsolidated accumulation of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter (ASTM D 18).
- ***Aggregate***—A granular material of mineral composition, such as sand, gravel, or crushed stone, used either in its natural state as a base or subbase layer or with a cementing medium to form a hardened or semi-hardened surface or base course.
- ***Soil Aggregate***—Natural or prepared mixtures consisting predominantly of stone, gravel, or sand, which contain a significant amount of minus 75-mm (No. 200) silt-clay material (ASTM D 8-97).
- ***Dense-Graded Aggregate***—An aggregate that has a particle size distribution such that when it is compacted, the resulting voids between the aggregate particles, expressed as a percentage of the total space occupied by the material, are relatively small (ASTM D 8-97).
- ***Open-Graded Aggregate***—An aggregate that has a particle size distribution such that when it is compacted, the voids between the aggregate particles, expressed as a percentage of the total space occupied by the material, remain relatively large (ASTM D 8-97).

- ***Asphalt***—A dark brown to black cementitious material in which the predominating constituents are bitumens which occur in nature or are obtained in petroleum processing (ASTM D 8-97).
- ***Asphalt Cement***—A fluxed or unfluxed asphalt specially prepared to a quality and consistency for direct use in the manufacture of asphalt pavements, and having a penetration at 25°C (77°F) of between 5 and 300, under a load of 100 g applied for 5 sec (ASTM D 8-97). Sometimes referred to as bitumen.
- ***Hot Mix Asphalt (HMA)***—A controlled mixture of asphalt cement and graded aggregate compacted to a dense mass. Also occasionally referred to as asphalt concrete (AC), hot-mixed asphalt concrete (HMAC), bituminous concrete (BC), and plant mix (PM) (NCHRP 1-37A).
- ***Portland Cement***—A commercial product which when mixed with water alone or in combination with sand, stone, or similar materials, has the property of combining with water, slowly, to form a hard solid mass. Physically, portland cement is a finely pulverized clinker produced by burning mixtures containing lime, iron, alumina, and silica at high temperature and in definite proportions, and then intergrinding gypsum to give the properties desired (ACPA website).
- ***Portland Cement Concrete (PCC)***—A composite material consisting of a portland or hydraulic cement binding medium and embedded particles or fragments of aggregate (NCHRP 1-37A).

<u>Material Property Terms</u>

- ***Liquid Limit (LL)***—The percentage of moisture at which a soil changes, with a decrease in moisture, from a viscous or liquid state to a plastic one (Oglesby and Hicks, 1982).
- ***Plastic Limit (PL)***—The percentage of moisture at which a soil changes, with decreasing wetness, from a plastic to a semisolid state (Oglesby and Hicks, 1982).
- ***Plasticity Index (PI)***—The numerical difference between a soil's liquid limit and its plastic limit.
- ***Modulus of Elasticity***—A measure of the stress-strain behavior of a material.
- ***Modulus of Rupture***—An indicator of tensile bending strength of concrete, is the maximum tensile stress at the bottom at rupture during a flexural test of a simply supported concrete beam (NCHRP 1-37A). Occasionally referred to as flexural strength.
- ***Modulus of Subgrade Reaction***—Westergaard's modulus of subgrade reaction for use in rigid pavement design (the load in lb/ft^2 on a loaded area of the roadbed soil or subbase divided by the deflection in of the roadbed soil or subbase) (NCHRP 1-37A)
- ***Compressive Strength***—The measured resistance of a concrete or mortar specimen to axial loading; expressed as lb/in^2 of cross-sectional area (ACPA website).

<u>Stabilization Terms</u>

- ***Soil Stabilization***—Chemical or mechanical treatment of a soil mass to increase its stability or improve its engineering properties.
- ***Base Stabilization***—Upgrading of the strength and consistency properties of aggregates, which may be considered unusable or marginal without stabilization (Little, 1995)
- ***Soil/Base Modification***—Improvement in the workability and constructability of soils and aggregates through the addition of small quantities of lime or lime–flyash (Little, 1995).

- ***Chemical Stabilization***——Alteration of soil or base material properties through the addition of certain chemical additives (lime, lime–flyash), resulting in an increase in the material's strength and bearing capacity and a reduction in its water sensitivity and volume change during wet–dry cycles.
- ***Mechanical Stabilization***——Alteration of soil or base material properties achieved through one of two means: changing the gradation of the soil by the addition or removal of particles, or densification by compaction (Carpenter et al., 1991).
- ***Lime***——A generic term embodying only the manufactured forms of lime—quicklime and hydrated lime, which are produced from the burning of calcitic or dolomitic limestone (Little, 1995). It includes all classes of high calcium lime, including quicklime (calcium oxide, CaO) and hydrated lime (calcium hydroxide, Ca(OH)2), as well as dolomitic lime (Ca×MgO, Ca(OH)2×MgO, Ca(OH)2×Mg(OH)2).
- ***Quicklime***——The product of calcination of limestone, it consists of the oxides of calcium and magnesium, and in the U.S it is available in three forms (NLA website):High calcium quicklime (CaO): derived from limestone containing 0 to 5% magnesium carbonate. Magnesian quicklime (MgO): derived from limestone containing 5 to 35% magnesium carbonate. Dolomitic quicklime (Ca×MgO): derived from limestone containing 35 to 46% magnesium carbonate.
- ***Hydrated Lime***——A dry powdered lime manufactured by treating quicklime with sufficient water to satisfy its chemical affinity for water, thereby converting the oxides to hydroxides. Depending upon the type of quicklime used and the hydrating conditions employed, the amount of water in chemical combination varies, as follows: High calcium hydrated lime (Ca(OH)2): high calcium quicklime produces a hydrated lime containing generally 72 to 74% CaO and 23 to 24% chemically combined water. Dolomitic hydrated lime (normal)--under atmospheric hydrating conditions only the CaO fraction of dolomitic quicklime hydrates, producing a hydrated lime of the following chemical composition: 46 to 48% CaO, 33 to 34% MgO, and 15 to 17% chemically combined water. Dolomitic hydrated lime (pressure): produced from dolomitic quicklime under pressure, which results in hydrating all of the MgO as well as all of the CaO, producing the following chemical composition: 40 to 42% CaO, 29 to 30% MgO, and 25 to 27% chemically combined water.
- ***Flyash***——A siliceous or alumino-siliceous material that in itself possesses little or no cementitious value, but that in finely divided form and in the presence of moisture will chemically react with alkali and alkaline earth hydroxides (such as lime) at ordinary temperatures to form or assist in forming compounds possessing cementitious properties (ASTM C 593-95).
- ***Lime–Flyash***——Blend of lime and flyash used to stabilize aggregates and some fine-grained soils (primarily silts). The chemical reaction between the calcium in lime and the pozzolan in flyash produces cementitious products.
- ***Lime–Cement–Flyash***——Blend of lime, cement, and flyash used primarily to stabilize aggregates. The chemical reaction between the calcium in both the lime and cement and the pozzolan in flyash produces cementitious products.

<u>Pavement Design and Performance Terms</u>

- ***Empirical Design***——A design approach that is based on the results of experiments or experience. Generally, it requires a number of observations to be made in order to ascertain the relationships between the variables and the outcomes of trials. It is not necessary to firmly establish the scientific basis for the relationships as long as the limitations are recognized (NCHRP 1-37A).

- ***Mechanistic Design***—A design approach that provides a scientific basis for relating the mechanics of structural behavior to loading. To quantify how load acting on a given structure is being distributed to its members, certain fundamental properties of the materials must be known, along with the geometric properties of the structure being loaded (NHI, 2002).
- ***Mechanistic–Empirical Design***—A design philosophy or approach wherein classical mechanics of solids is used in conjunction with empirically derived relationships to accomplish the design objectives (NCHRP 1-37A).
- ***Design Life***—The length of time for which a pavement structure is being designed, including the time from construction until major programmed rehabilitation (NCHRP 1-37A). Also referred to as performance period.
- ***Design Reliability***—The probability that a pavement section designed using the pavement design–performance process will perform satisfactorily over the traffic and environmental conditions for the design period.
- ***Hierarchical Input Level***— The proposed input level for the mechanistic-empirical design and analysis using the M-E Design Guide procedure. The M-E Design Guide facilitates materials, climatic, and traffic inputs at three different levels – levels 1, 2, and 3. The required level of effort for laboratory testing of field evaluation generally decreases from input level 1 to input level 3.

<u>Economic Terms</u>

- ***Life-Cycle Cost Analysis (LCCA)***—An economic assessment of an item, area, system, or facility and competing design alternatives considering all significant costs of ownership over the economic life, expressed in equivalent dollars (NCHRP 1-37A). The process used to compare projects based on their initial cost, future cost and salvage value, which accounts for the time value of money (ACPA website).
- ***Analysis Period***—The time period used for comparing design alternatives. An analysis period may contain several maintenance and rehabilitation activities during the life cycle of the pavement being evaluated (NCHRP 1-37A).
- ***Net Present Worth (NPW) Method***—A discounted cash flow analysis that involves the conversion of all of the present and future expenses to a base of today's costs. Also occasionally referred to as the net present value (NPV) method.
- ***Equivalent Uniform Annual Cost (EUAC) Method***—A discounted cash flow analysis that requires converting all of the present and future expenditures to a uniform annual cost. Also occasionally referred to as the annualized method.
- ***Discount Rate***—The time value of money used as the means of comparing the alternative uses for funds by reducing the future expected costs or benefits to present-day terms. Discount rates are used to reduce various costs or benefits to their present worth or to uniform annual costs so that the economics of the various alternatives can be compared (approximately equals interest minus inflation) (NCHRP 1-37A).
- ***Salvage Value***—The value (positive if a residual economic value is realized and negative if demolition costs are accrued) of competing alternatives at the end of the life cycle or analysis period. Can also be considered to represent the useful life of the pavement beyond the end of the analysis period.