EVALUATION OF STRUCTURAL PROPERTIES OF LIME STABILIZED SOILS AND AGGREGATES

VOLUME 1: SUMMARY OF FINDINGS

PREPARED FOR THE NATIONAL LIME ASSOCIATION

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January 5, 1999

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SUMMARY

This study has two objectives. The first is to define the structural and performance characteristics that can be expected of lime stabilized subgrades and bases. The second is to define a laboratory mixture design and testing protocol to assure that properties necessary to meet these structural demands are achieved as part of a mechanistic design/analysis approach. Such an approach is expected be to a part of the 2002 AASHTO Pavement Design Guide.

The material properties and characteristics of lime stabilized pavement layers can be divided into four categories that have been identified as critical or key to performance. These categories are: (1) strength and permanent deformation, (2) resilient properties, (3) fracture and fatigue and (4) durability. These categories were identified based on a careful synthesis of the literature as well as unpublished research and performance information, and based on a review of the NCHRP 1-37 work plan and the 1996 AASHTO Workshop on Pavement Design.

Lime stabilization causes a significant improvement in soil texture and structure by reducing plasticity and by providing pozzolanic strength gain. A significant level of long-term strength improvement in lime stabilized soils and aggregates is possible and probable. This level of strength improvement can meet typical specifications required by various user agencies. This strength improvement has been verified not only by extensive laboratory testing but also extensive field testing. These tests define that, when lime is added to a reactive soil or aggregate, strengths in excess of about 1,400 kPa are expected. This strength level has been identified as one that provides significant structural benefit to the pavement. In some soils ultimate compressive strength values of as high as 7,000 to 10,000 kPa can be reached.

The level of strength improvement developed through lime stabilization of pozzolanically reactive soils is directly associated with a substantial reduction in the potential of the stabilized material to permanently deform (or rut) under repeated traffic loading, a critical issue in pavement design methods. Since many untreated soils and aggregates which form pavement layers possess unacceptable strength and deformation resistance, lime stabilization is an attractive option to improve the structural attributes of these materials when used in new pavements and in the reclamation of existing soil and aggregate layers.

Resilient or stiffness properties of pavement layers define their efficiency to distribute load-induced stresses within the pavement system , a key part of a mechanistic - empirical design approach. Lime stabilization often induces a 1,000% or more resilient modulus or stiffness increase over that of the untreated soil or aggregate. This level of resilient modulus improvement offers a significant structural contribution to the pavement system. Laboratory resilient modulus testing (AASHTO T-274 or similar methods) has established that this level of modulus improvement occurs over the range of expected field moisture contents. Values of back calculated

(from field FWD testing) resilient moduli typically fall within a range of from 210 MPa and 3,500 MPa. This is considered structurally effective in terms of stress distribution but yet not so stiff as to induce excessive shrinkage cracking distress.

Field studies in Texas, Australia, Kentucky and North Carolina, for example, verify that lime stabilized subgrades provide a strong support below unbound aggregate bases. The presence of the stabilized subgrade as found to enhance the performance of both unbound aggregate bases and full depth asphalt layers.

Significant research has been performed in the area of fracture and fatigue properties of lime stabilized soils and aggregates. These fatigue properties can be efficiently and reliably approximated based solely on unconfined compressive strengths of the stabilized materials and by knowing the tensile flexural stresses induced in the stabilized pavement layer. This process can be incorporated into a mixture design/pavement analysis protocol and into a mechanistic-empirical pavement design protocol. It is a relatively simple task to design lime stabilized pavement layers that are resistant to damage due to load-induced fatigue. This is done by properly assigning stabilized layer thicknesses according to the strength properties of the stabilized layers.

All types of pavement layers have suffered from the effects of the environment and any pavement layer is susceptible to the deleterious effects of these factors. However, the literature and available performance data demonstrate that well-designed stabilized layers can effectively resist these effects and can perform well for many years. One study (Kelley, 1976) identified lime stabilized layers that have performed extremely well and have maintained excellent strength properties for over 40 years. Extensive laboratory work by Thompson and Dempsey (1968, 1969) and by Little (1995) has demonstrated that the rate of strength loss due to moisture cycling and freeze-thaw cycling in soils and aggregates is usually substantially improved through the process of lime stabilization.

A pressing need is to establish a mixture design protocol for lime-soil and lime-aggregate mixtures that will assure the design of a reliable and durable pavement layer. This report suggests such a protocol based on the following steps: (1) select a soil or aggregate that is mineralogically reactive with lime, (2) establish optimum lime content based on pH testing and compressive strength development (accounting for the effects of moisture - density relationships), and (3) evaluate resistance to moisture-induced damage through a capillary suction test in which the surface dielectric value of the cured, lime-treated sample is measured.

Based on the unconfined compressive strength determined in this protocol, the designer can approximate design resilient modulus (a function of mixture strength and natural subgrade support), fatigue damage potential (based on a ratio of load-induced flexural stress to flexural strength) and permanent deformation potential (based on a ratio of load-induced shear stress to shear strength).

Volume I of this report summarizes studies (both laboratory and field) on lime stabilized layers regarding the following key properties: (1) strength and deformation, (2) resilient properties, (3) fracture and fatigue, and (4) durability. An appendix to this volume contains eight tables that summarize these and other studies. Volume II of this report provides a fuller description of the studies presented in Volume I. It also contains a brief discussion of life cycle analyses, pointing out the need for the AASHTO 2002 Guide to properly take into account the structural improvements of lime-stabilized subgrades and bases when performing such analyses.

INTRODUCTION

Lime stabilization of subgrades can provide significantly improved engineering properties. There are essentially two forms of improvement: modification and stabilization. Modification occurs to some extent with almost all fine-grained soils, but the most substantial improvement occurs in clay soils of moderate to high plasticity. Modification occurs primarily due to exchange of calcium cations supplied by the lime ($Ca(OH)_2$ or hydrated lime) for the normally present cation adsorbed on the surface of the clay mineral. Modification is also caused as the hydrated lime reacts with the clay mineral surface in the high pH environment promoted by the lime-water system. In the high pH environment the clay surface mineralogy is altered as it reacts with the calcium ions to form cementitious products. The results of the mechanisms are: plasticity reduction, reduction in moisture holding capacity (drying), swell reduction, improved stability and the ability to construct a solid working platform.

Stabilization occurs when the proper amount of lime is added to a reactive soil. Stabilization differs from modification in that a significant level of long-term strength gain is developed through a long-term pozzolanic reaction. This pozzolanic reaction is the formation of calcium silicate hydrates and calcium aluminate hydrates as the calcium from the lime reacts with the aluminates and silicates solubilized from the clay mineral surface. This reaction can begin quickly and is responsible for some of the effects of modification. However, research has shown that the full term pozzolanic reaction can continue for a very long period of time - even many years - as long as enough lime is present and the pH remains high (above about 10). As a result of this long-term pozzolanic reaction, some soils can produce very high strength gains when lime treated. The key to pozzolanic reactivity and stabilization is a reactive soil and a good mix design protocol. The results of stabilization can be very substantial increases in resilient modulus values (by a factor of 10 or more in many cases), very substantial improvements in shear strength (by a factor of 20 or more in some cases), continued strength gain with time even after periods of environmental or load damage (autogenous healing) and long-term durability over decades of service even under severe environmental conditions.

Although the modification process is geared to provide construction expediency, it can produce very important structural improvements such as significant bearing capacity improvements as, for example, measured by the CBR. In France and England where lime is widely used to improve capping layers, CBR improvements in wet soils in cool climates from as low as 1% (natural soil) to between 15% and 25% (modified soil) are typical. Such improvements provide reduced moisture sensitivity of the subgrade, a stronger subgrade support layer (higher modulus) and the ability to produce a better base layer through better compaction and support capacity of the subgrade. However, the real benefit of lime stabilization from a pavement structural standpoint occurs when a sound mixture design protocol is followed whose goal is to provide durable and permanent stabilization. In this case the improvements in resilient modulus properties and shear strength can provide significant structural benefits throughout the pavement structure. Even a weak clay after stabilization can become a rut resistant, strong supporting layer with good resilient properties. Such a layer can provide improved support for aggregate bases. This is important as aggregate bases are stress sensitive and depend on the supporting ability of the underlying subgrade for the development of the confinement necessary to promote good resilient and strength properties. This can be demonstrated theoretically and has been demonstrated practically in field projects.

The performance of lime stabilized subbases or bases has been somewhat hard to assess in the current AASHTO design protocol as the measure of structural contribution in that system, the structural layer coefficient, cannot be directly measured. However, indirect attempts to determine layer coefficients for lime stabilized subbases and bases have lead to structurally significant values. This need to indirectly assess structural properties for lime stabilized layers is obsolete with the change to the mechanistic - empirical (M-E) approach. In this methodology, measurable engineering and material properties such as resilient modulus will be used in a finite element or layered elastic pavement model to assess stress and strain distributions under load within the stabilized layer and throughout the pavement structure. The potential for damage or distress will be evaluated based on transfer functions which will relate the stresses and strains to performance through empirically developed relationships. In the case of the lime stabilized subbases and bases, for example, rutting potential within the layer can be assessed by means of repeated load triaxial tests which define the rate of accumulated permanent strain at a selected stress level or by means of a relationship between induced stress within the pavement layer under load and the shear strength of the material. The effect of the stress state improvement provided for the overlying base and asphalt surface layers by the stronger stabilized layer can also be assessed in the M-E approach. In other words, the effect of the stabilized layer is seen in the response of the surrounding and interacting layers.

In order to provide a reliable structural layer, a sound mixture design approach is essential. This approach must assess the lime content required to provide permanency, adequate moisture resistance, adequate strength, adequate resilient properties and adequate fatigue properties. This report presents a protocol for mixture design and testing geared to assure the establishment of such properties.

Lime has an important role in stabilization of new materials and in reclamation. Lime has been effectively used to upgrade or reclaim not only clay soils, but also clay contaminated aggregate bases and even calcareous bases which have little or no appreciable clay. Work in the U. S., South Africa and France has established the benefits of lime stabilization of calcareous bases which results in significant strength improvements, moisture resistance improvement and resilient modulus improvements without transforming the calcareous bases into rigid systems, which could be susceptible to cracking and shrinkage.

When lime is not adequate to achieve the desired strength and improvement, lime in combination with fly ash may provide the needed improvement. Recent research has demonstrated that moderate levels of lime and fly ash can achieve significant strength improvements in reclaimed soil and aggregate systems without producing extremely rigid and shrinkage sensitive systems. Generally, a target strength can be achieved through a sound mixture design process which identifies a lime - fly ash combination which will achieve desired strength and resilient modulus properties.

The main objective of this study is to define the structural and performance characteristics which can be expected of lime stabilized subgrades and of lime stabilized bases. The secondary objective is to define a laboratory mixture design and testing protocol which can be used to help assure that the properties necessary to meet structural demands are achieved.

The report attempts to address properties of stabilized layers that have been identified as critical by members of two important teams of experts: (1) the 1996 Workshop on the AASHTO Design Guide and (2) the 1997 Brainstorming session directed by the National Cooperative Highway Research Program (NCHRP) project 1-37. These properties include: (1) Strength properties, including the development of strength over time; (2) Resilient properties including the development of resilient modulus over time; (3) Deformation or potential to develop accumulated damage under repeated loading; (4) Fatigue and fracture potential under traffic loads and due to non-traffic load associated stresses; (5) Moisture susceptibility of the stabilized layer and (6) Relationships between laboratory and field materials properties - particularly resilient modulus and strength.

THE AASHTO DESIGN SCHEME

BACKGROUND

The AASHTO Guide for the Design of Pavement Structures (Guide) is the primary design approach used in the United States. The original Guide was issued in 1961. At that time the major objective of the Guide was to provide information that would continue to be used to develop pavement design criteria and pavement design procedures. Therefore, the road study that forms the background for the various versions of pavement design was always envisaged as a data base for development of updated and improved design protocols as technology developed.

In 1972 a second version of the Guide was released which included only a few changes to the original version including an overlay design approach. The 1981 edition of the Guide included updated criteria for portland cement concrete (PCC) pavement design. By far the most significant revision in the Guide was made in 1986 when 14 major changes were incorporated. The changes with the most impact were the incorporation of reliability concepts and the selection of the resilient modulus as the means of characterizing subgrade soil support. The 1986 Guide also placed more emphasis on the use of the resilient modulus to assign a structural layer coefficient value to the asphalt concrete surface, the unbound aggregate base and the unbound aggregate subbase. The impact, in terms of structural layer coefficients, for portland cement stabilized bases, fly ash stabilized bases and bituminous stabilized bases was also considered. It is interesting to note that neither lime stabilized subgrades nor lime stabilized bases were addressed in terms of structural significance in the 1986 Guide. Other important upgrades in the 1986 Guide were the considerations of serviceability loss due to non-traffic environmental effects including frost heave and swelling clays. The 1986 Guide also included a section on rehabilitation strategies other than overlays. This is significant as the emphasis is now on rehabilitation and not new pavement construction.

The 1986 Guide also included a state-of-the-art review and a position statement on mechanistic design. In essence the 1986 Guide, although not based on mechanistic design, includes some aspects of quasi-mechanistic design and sets the stage for future development in this direction. The most recent update to the Guide was in 1993 when the section on overlay design was updated and additional support for overlay design was provided in the form of detailed appendices. Otherwise, the 1993 Guide is virtually the same as the 1986 Guide.

OVERVIEW OF THE PROCESS

Performance Equation

The flexible pavement portion of the AASHTO Road test conducted between 1958 and early 1960 near Ottawa, Illinois, was a full factorial experiment. The experiment was designed to assess the effect of various combinations of pavement layer thicknesses on the ability of the pavement to carry the traffic with a high level of serviceability. Since the Road Test was conducted at one general location, the experiment was limited to one subgrade type and one climate. In fact great effort was taken to assure that the silty clay subgrade (a deep fill) was consistent throughout the Road Test site. The climatic conditions were limited to that of northern Illinois, and the full effects of the environment over a normal pavement design period (approximately 20 or more years) were not encountered. This is because the approximately 1.2 million axle load applications applied to each pavement test section occurred over approximately a (short) two year period.

The Road Test consisted of seven test loops subjected to different types (single or tandem axle) and different levels of axle load. The performance equation written in terms of the number of 18,000 pound single axle load applications is as follows:

$$\log_{10} W_{18} = Z_R * S_O + 9.36*\log_{10}(SN + 1) - 0.20 + \log_{10} \{[\Delta PSI/(4.2-1.5)]/[0.40 + 1094/(SN + 1)^{5.19}]\} + 2.32*\log_{10} M_R - 8.07 \quad \text{[equation 1]}$$

In this equation, the term W_{18} represents the number of 80 kN single axle loads, Z_R is the standard normal deviate, S_o is the standard deviation of the data, ΔPSI is the loss of serviceability, M_R is the subgrade resilient modulus and SN is the structural number of the pavement in question. In this equation, the first two terms account for the level of reliability required for the design, and the subgrade resilient modulus is an attempt to adjust the performance equation for subgrade types different from the one subgrade encountered at the Road Test. The loss of serviceability, ΔPSI, encountered during the design life of the pavement is entered at the discretion of the designer and represents the level of serviceability loss the designer is willing to accept due to traffic loads. The remaining term, SN, describes the effect of the pavement structure on the performance loss due to load. The structural number is the sum of the products of the thickness of each layer and the structural layer coefficient, a_i. The layer coefficient is actually a regression constant that allows the performance equation (equation 1) to fit the data with the least error.

The Structural Layer Coefficient

The pavement structure (SN) is determined for a given average annual subgrade support modulus, M_R; a selected level of reliability; a selected loss of serviceability, ΔPSI; and a selected level of equivalent single axle loads (ESAL's). Therefore, the required SN can be achieved either by adjusting the pavement layer thicknesses or the layer coefficients or both. This process emphasizes the importance of accurate assessment of the layer coefficient in the design process. The major problem that designers face when using the AASHTO design approach is selecting realistic layer coefficients. Since the pavement structure at the Road Test consisted of a hot mix asphalt concrete surface, a crushed limestone base and a gravel subbase, these are the only materials for which layer coefficients were directly determined on the basis of the full factorial experiment. All other layer coefficients were indirectly determined by various methodologies. Although some of the approaches used to determine layer coefficients for various materials are innovative and well-founded, they are still indirect methods and subject to controversy by the very nature of the layer coefficient which is not a material property but a statistical parameter. Fortunately, the required shift from an empirical performance equation to a mechanistic-empirical design approach offers a much more reasonable approach to assigning structural significance to materials other than those specifically used at the Road Test.

SHORTCOMINGS OF THE CURRENT AASHTO DESIGN GUIDE PROCESS

The current design process using equation 1 has a number of limitations. Some of the major shortcomings are:

1. Only one subgrade type is reflected in the Road Test data base. The adjustment to the performance equation made by the last two terms of the performance equation is an indirect assessment of the effect of other subgrade types based on very limited data at the Road Test.
2. The Road Test is based on the application of about 1.2 million load applications over a period of just over two years. Pavements are now subjected to axle load applications that are orders of magnitude larger than those applied at the Road Test.
3. Distress at the Road Test was defined in terms of pavement roughness, rutting in the wheel path and load-associated fatigue cracking in the wheel path. The environmental effects related to thermal cracking, and thermally-induced volume change were not addressed. The environmental effects of swelling clays and frost action were only indirectly addressed in the 1986 Guide.
4. Pavement layers were characterized by a structural layer coefficient indirectly related to performance through an empirically-derived regression equation rather than by means of material properties of the pavement layers.

FUTURE DIRECTION AND DEVELOPMENT

In 1998, the AASHTO Joint Task Force on Pavements (JTFP) and others in the pavement design community believe that the technology is now available and the theory is well enough defined to embark on a different approach to pavement design. This is the *mechanistic design* approach. In this approach, the pavement structure is modeled as a mathematical system, and important engineering parameters, such as normal stresses and strains and shear stresses and strains, are calculated under simulated traffic loading. These parameters are then related to performance through empirical correlations developed in practice. Hence, the new approach is not totally mechanistic, but is mechanistic-empirical.

The foundation for the current National Cooperative Highway Research Program (NCHRP) research project to develop a mechanistic-empirical (M-E) AASHTO Design Guide was established at the "Workshop on Improved Pavement Design." This workshop was held March 24-26, 1996. The workshop identified a series of needs that must be addressed in the development of a comprehensive M-E Guide: (1) synthesis of available information on mechanistic and mechanistic-empirical design issues; (2) the development of a loading characterization method consistent with modern traffic streams and with the mechanistic design approach; (3) the development and validation of various transfer functions relating pavement distress to performance, both structural and functional; (4) the development of more thorough data bases of inputs for existing models, especially in the environmental area and; (5) development of an improved means of characterizing in situ paving materials for rehabilitation.

CONCEPT OF A CALIBRATED MECHANISTIC-EMPIRICAL DESIGN APPROACH

Structural Model of the Pavement

In a calibrated mechanistic design approach the pavement structure is generally represented by either a layered elastic model (LEM) or a finite element model (FEM). Each model uses elastic theory to calculate stresses and strains induced within the pavement layers due to traffic and environmentally induced loads. The mathematical and numerical models used to represent the pavement structure are sophisticated and have evolved to a very reliable level over the past 15 years. It is a virtual certainty that either a LEM or a FEM will be selected as the structural model used for the 2002 Guide.

During the last 10 years methodologies have been developed which realistically account for the non-linearity of the elastic properties of the granular pavement layers such as the aggregate base course, the aggregate subbase, the granular subgrade and the cohesive subgrades as well as for stabilized layers. Simplified methodologies have also been developed for the realistic treatment of the time and temperature (viscoelastic) dependency of asphalt-bound pavement layers.

Material Characterization of Pavement Layers

In order to appropriately model the pavement structure, the correct material properties must be used to describe each layer. In a LEM or a FEM structural model, these properties include a measure of layer stiffness or resilient modulus and Poisson's ratio. The resilient modulus is essentially defined as the ratio of applied stress (repeated stress or transient stress under a moving wheel load or due to an environmental cycle) to the strain induced by the transient load.

The resilient modulus for granular materials and stabilized materials can be determined according to AASHTO method T-294-94. This method allows one to account for the stress sensitivity and moisture sensitivity of the materials. The load applied during the test protocol mimics the load duration and magnitude applied in the field. Recent work by Lytton (1994) demonstrated that the determination of the ratio of lateral strain to longitudinal strain under load, Poisson's ratio, is also highly stress sensitive and may vary considerably more than previously considered. Tutumleur (1998) and Tutumleur and Thompson (1997) clearly verified the importance of considering cross-anisotropy or the fact that unbound granular layers have substantially different strength and stiffness properties in the horizontal than in the vertical direction (direction of compaction). When this anisotropy is accounted for properly, it establishes how unbound layers transfer load by means of shear stresses and do not develop high tensile stresses predicted by linear elastic analysis based on the assumptions of isotropy. Lytton (1998) has further established that stress-sensitive values of "Poisson's ratio" for such granular layers must also be accounted for in the analysis and that significant dilation of granular layers can lead to even more significant deviations from expected performance.

It may be proven that cross-anisotropic effects must be accounted for in stabilized pavement layers as well as in unbound layers. If this is the case, then testing protocols such as AASHTO T-294-94 will need to be adjusted in order to make the necessary measurements such as radial strains as well as axial strains. This is being evaluated in phase 2 of this study.

All material characteristics must be able to be efficiently and accurately measured and must be able to be input into a computer model, such as an FEM model.

Climatic Models

Temperature and moisture have a very significant impact on the material properties of the pavement layers. Thus, a realistic pavement structural model must account for the effects of the temperature gradient within the pavement layers. An example of such an approach was presented by Dempsey and Thompson (1970) to evaluate frost action in a multilayered pavement system. The model is a one-dimensional heat flow model which accounts for the change of temperature, T, as a function of time, t, and depth, z, within the pavement as a function of thermal diffusivity, α.

Similarly, a moisture equilibrium model is required to account for the water accumulated in the subgrade and granular layers as a function of capillary moisture movement. A successful approach to this was used by Little et al. (1997) in determining the moisture content in granular bases as a function of suction (or the energy to hold water) within the pavement layer. This approach is based on a unique relationship between moisture content and suction for a specific soil or aggregate type. Since the resilient modulus of soil and granular layers is highly dependent on the moisture content, the effect of stabilization of soil and aggregate layers with lime will be of considerable interest as will the effect of a lime stabilized subgrade as a moisture cutoff or capillary break.

A critical component of mix design for stabilized layers is to ensure that the stabilizer and amount of stabilizer selected effectively reduces the moisture sensitivity and that realistic evaluation of this criterion has been established.

Distress Models

Distress models are sometimes called transfer functions which relate structural responses to various types of distress. This is the "weak link" in the mechanistic-empirical method. Project 1-37A will look at this aspect very carefully in a concerted attempt to calibrate and verify existing transfer functions or perhaps establish more reliable distress predictions based on existing data such as the Long Term Pavement Performance (LTPP) data base established as part of the Strategic Highway Research Program (SHRP).

The most widely used transfer functions are those that relate tensile strain calculated within the hot mix asphalt surface layer under traffic loading to fatigue cracking in the asphalt surface. A second widely used model relates compressive strain at the top of the natural subgrade under traffic loading to rutting in the wheel path and to pavement roughness. Models have also been developed and have been widely used which relate compressive and shearing stresses within the hot mix layer and within the base layer to rutting or permanent deformation of the pavement.

There are two forms of thermal cracking models in the hot mix asphalt pavement (HMA) layer: low-temperature cracking and thermal fatigue cracking. In the low temperature cracking model, thermal stresses are induced due to a temperature drop as a result of the thermal coefficient of volume change of the asphalt layer. If the thermally induced stress exceeds the tensile strength of the HMA at a specific temperature, then a crack will result. If the thermal stress is severe enough, the crack may propagate completely through the HMA in one or two cycles. Less severe stresses may require a larger number of cycles to cause the crack to propagate through the layer or the thermal crack may propagate through the layer due to stresses induced at the crack tip from traffic loads or other loads.

A successful environmental model must also account for the thermal and moisture effects on the granular layers whether or not stabilized with lime or other stabilizers. This was demonstrated by Little et al (1997) in research project 1432 for the Texas Department of Transportation (TxDOT). This research clearly demonstrated the moisture susceptibility of aggregate bases. This moisture susceptibility can and does lead to strength and stability loss within the base layer leading primarily to permanent deformation. Moisture sensitivity of aggregate bases in areas subject to freeze-thaw environments are particularly susceptible to thermal cracking originating in the base course layer and propagating through the HMA surface. Little et al. (1997) showed the importance of stabilization with lime and lime fly ash (and other stabilizers) in reducing moisture and thermal sensitivity of aggregate bases.

CRITICAL ISSUES FACING PROJECT 1-37A REGARDING LIME STABILIZATION
Workshop on Improved Pavement Design - March, 1996

The breakout group on subgrade characterization at the Workshop on Improved Pavement Design in March, 1996, identified several critical issues for materials stabilized with portland cement, asphalt, lime or lime fly ash which are essential for the development of a viable 2002 Guide. These issues are as follows:

1. ***Determine the stiffness (or resilient modulus) of the stabilized layer as a function of time***. This implies that not only the immediate but also the ultimate stiffness with extended curing is important to the design scheme. Lime and lime fly ash stabilized materials cure much slower, in general, than portland cement stabilized layers. It is crucial to know or be able to approximate the rate of stiffness gain, the ultimate level of stiffness and a realistic target of stiffness. The layer must be stiff enough to properly distribute load, yet it may be better to limit the ultimate stiffness of the layer in order to reduce the potential of the layer to develop wide shrinkage cracks which can and do result in loss of load transfer and severe pavement distress.
2. ***Determine the fatigue characteristics of the stabilized layer as a function of time***. This concept is interrelated to item 1 as the stiffness of the layer and the fatigue characteristics are interrelated. Very stiff, rigid layers often contain wide shrinkage cracks with poor load transfer. At these severe transverse cracks, load-induced fatigue cracking is accelerated because the load-induced tensile stresses are much higher at the transverse cracks with poor load transfer.
3. ***Characterize strength-time relationship***. Once again this issue is strongly interrelated with items 1 and 2.
4. ***Characterize moisture susceptibility***. This is primarily related to the amount and mineralogy of the fines fraction. The properties of the fines are substantially impacted by stabilization.
5. ***Define and characterize non-load associated cracking***. Cracking is impacted by the moisture and thermal sensitivity of aggregate/soil fines which are significantly impacted by stabilization.
6. ***Consider the field stiffness as opposed to laboratory-determined stiffness considering especially the effects of in situ cracking***. This requires a correlation between lab and field testing.

1-37 Project Staff Meeting, April, 1997

The research team and consultants of project 1-37 met on April 25-26, 1997, and identified critical issues that must be successfully addressed in order to provide a successful product - the 2002 Guide. The following discussion is limited to the critical issues associated with stabilized layers. These critical issues are divided into categories and are summarized in Table 1 for convenience and clarity of presentation.

Table 1. Summary of critical issues identified at the April 25-26, 1997, brainstorming meeting of the 1-37 project staff.

Category	*Issue*
General	1. New pavement models should not be developed; rather existing models should be validated with field data. 2. Use of a mechanistic, distress-based approach is reasonable for the 2002 Guide, but linking distress to performance may be difficult. 3. Need to focus on rehabilitation and not new design. 4. 2002 Guide must be understandable. 5. Liaison with LTPP for calibration/validation data is essential.
Flexible pavement design concepts	1. Need to model environmental effects.
Analysis techniques	1. Layered elastic theory is acceptable for flexible pavement design/analysis. 2. Need to retain/expand back-calculation techniques for rehabilitation design. 3. Flexible pavement analysis must account for: - tensile stresses in granular bases - stress-dependent moduli

Category	*Issue*
Non-destructive testing (NDT)	1 A unified NDT approach must be addressed. 2. Need to decide if techniques other that the Falling Weight Deflectometer (FWD) will be allowed in NDT. 3. Need to define the inputs for rehabilitation design. 4. Need to address discrepancies between laboratory-measured and back-calculated moduli. 5. Determine whether NDT should be combined with coring or means of material verification (i.e., Dynamic Cone Penetrometer (DCP). 6. Need to determine if NDT can be used to determine subsurface distress. 7. Need to consider seasonal effects on NDT data. LTPP data should be valuable here. 8. The 2002 Guide should emphasize the importance of having a pavement engineer do the design/evaluation (not a technician).
Environment	1. A thorough treatment of differential heave is essential including tests for identifying susceptible soils, interpretation of test results, mitigation recommendations, available models, etc. 2. Knowledge of seasonal water content variations are essentially meaningless unless transfer functions are available to predict variation in strength. 3. The stress-ratio should be evaluated/considered rather than specifying a subgrade strain criterion.
Subgrade soils/Materials	1. Emphasis should be on back-calculation techniques rather than measurement of resilient moduli. 2. Problems exist with resilient moduli testing. 3. Characterization of modulus of cracked treated materials is extremely difficult, if not impossible with current laboratory test methods. 4. Poisson’s ratio should be assumed. Do not attempt to measure it. 5. More emphasis should be placed on drainage in the 2002 Guide relative to the ‘86/’93 Guides. 6. Need good transfer functions.
Life cycle cost analysis (LCCA)	1. LCCA should aid the engineer in selecting pavements based on structural attributes, not social/external costs. Only initial, major maintenance, user and delay costs as well as salvage value should be included. 2. It is imperative that LCCA be an integral part of the design process, not a separate module.

A review of the critical issues as articulated at the 1996 “Workshop” and at the “Brainstorming session” in 1997 reveals the importance of determining pertinent properties of pavement layers (including lime stabilized layers) which are essential to the M-E design process. These include both laboratory and field derived strength and resilient properties. The authors of the new Guide will probably not have the time nor resources to develop new pavement structural models. Instead the 2002 Guide will most probably rely on the best available model. Furthermore, the material characteristics used in this model for subgrade and aggregate layers (both stabilized and unstabilized) will be characterized, at least in part, by resilient properties including resilient moduli and Poisson’s ratio. The experts at the Workshop and Brainstorming Session (hereafter referred to as “experts”) consider the most reliable assessment of the resilient moduli to be an in situ assessment determined through NDT using the FWD. This is particularly true for existing layers which must be upgraded in a rehabilitation or recycling process, which will be a major emphasis of the 2002 Guide. In fact many experts agree that the current NDT methodology for back-calculating in situ properties of existing layers must be extended and upgraded to provide the needed input and reliability for rehabilitation and recycling design.

Not only do the experts place considerable emphasis on NDT moduli, but they also place emphasis on seasonal variations of these in situ moduli. A key to assessing realistic values of NDT determined in situ moduli and the seasonal variation of these moduli is the LTPP data base which incorporates NDT deflection data and back-calculated moduli for selected pavement sections across the U.S.

The experts realized that laboratory measurements of material properties such as strength and resilient modulus are critical to the design and quality control of pavement materials. However, the experts believe that the laboratory material tests must be correlated with in situ, back-calculated properties. However, the process of deriving reasonable resilient moduli of the in situ pavement layers is itself a demanding task with different states and agencies using different techniques and approaches. The fact is that a unique solution to back-calculated moduli does not exist, and a solution must be based on good and reasonable decisions made by the supervising engineer. In order to ensure reasonable values of back-calculated moduli it may be necessary to evaluate pavement cores or correlate back-calculated moduli with in situ strength testing such as the Dynamic Cone Penetrometer (DCP). The Texas Transportation Institute (TTI) routinely uses the DCP in conjunction with the FWD to assess in situ pavement layer properties. The DCP is used to verify pavement layer thicknesses and strength properties.

The experts addressed the need to consider the effects of cracking of stabilized layers. Little et al. (1994) identified the nature of the deleterious effects of rigidly stabilized bases in the Houston District of TxDOT. Although the moduli of these rigidly portland cement stabilized layers were very high between cracks, the load transfer across cracks was very poor, leading to severe distress. A substantial need exists to account for the deleterious effects of loss of load transfer efficiency across shrinkage cracks. This has been addressed by Little et al. (1997). This potential of a stabilized layer to shrink and crack - resulting in transverse shrinkage cracking is directly associated with the level of stiffness or rigidity of the stabilized layer. Therefore, a reasonable concept is that a target stiffness or resilient modulus (or a modulus “window”) of the stabilized layer should be sought in lieu of a minimum or threshold modulus. In other words, the layer should be stiff enough to successfully distribute traffic loads without damaging the pavement structure but not so stiff and rigid as to suffer from excessive shrinkage cracking which will propagate through the HMA surface. Therefore, it is likely that a window of acceptability will be determined for resilient modulus. An associated window of acceptability of design compressive strength is also likely to improve pavement design.

Although not addressed by the experts associated with the 1996 Workshop or the 1997 Brainstorming session, a critical need for lime stabilized layers is a good mixture design to ensure durability of the stabilized layer. The design of the lime stabilized material must not only consider strength and resilient properties but also the effect of the stabilizer on the moisture sensitivity of the material. This concept will be addressed later in this report. The moisture sensitivity of subgrade soils and of aggregates bases and subbases can be dramatically affected by lime and lime fly ash stabilization.

FINDINGS

This study concentrated on both laboratory and field properties of lime stabilized subgrades and lime stabilized bases as the experts at the 1996 Workshop and at the 1997 Brainstorming session have strongly emphasized the importance of field or in situ properties. Work for the Texas Department of Transportation (TxDOT) over the past five years at TTI has provided a strong data base for comparing and correlating lab and field data, particularly resilient moduli. These Texas field data together with field performance data from the Siler City Road test in North Carolina, Kentucky DOT studies in the 1990's, an extensive study now underway in Mississippi, a similar study in Australia and the national LTPP data base provide guidance for developing these correlations and determining reasonable lab-to-field shift factors. However, it will generally be up to the user agency to develop these factors.

Besides the emphasis on field properties and lab-to-field correlations, the experts generally agree on characteristics of stabilized layers that are critical to performance. These are: (1) strength and deformation, (2) resilient properties, (3) fracture and fatigue and (4) durability. Each of these is addressed in the following sections.

The literature evaluated which substantiates the findings documented in this section is summarized in Appendix A. This appendix is comprised of eight tables (A1 through A8). These tables are placed in an appendix to facilitate ease of use and to prevent interruption of the flow of the material presented in this section (due to the length of the tables).

STRENGTH AND DEFORMATION PROPERTIES

General

The most obvious improvement in a soil or aggregate through lime stabilization is strength gain. Traffic-induced shear stresses within the base or subbase must not result in either shear failure or excessive accumulated strain (damage). The most direct approach is to evaluate the ratio of induced shear stress to shear strength. Increased shear strength due to lime-stabilization reduces the shear stress ratio and hence the susceptibility of the lime-stabilized layer to shear-induced damage.

Deformation properties are not normally measured for lime stabilized subgrades or for lime stabilized bases. This is because the testing is time consuming and the stress state within the deep structural layers is of much less concern than the stress state in the hot mix surface where most of the permanent deformation occurs. Normally surrogate strength tests replace tests designed to assess permanent deformation potential under repeated loading. However such testing may be adopted to ascertain the benefits of adding lime to upgrade marginal bases which have proven to be susceptible to permanent deformation under heavy load or high traffic levels.

This type of testing will probably be reserved for high level pavements.

Overview of Key Studies

Tables A1 summarizes information derived from the literature which verifies the effect of lime in the modification of the texture, plasticity and compaction characteristics of soils. This modification process is a first and significant step in influencing the ultimate structural performance of the pavement. Lime modification of texture, plasticity and compaction characteristics substantially alters and improves the performance potential of the lime-treated soil as a structural layer. Furthermore, if an improved and durable lime-treated layer can be designed and constructed, then a superior subbase is offered for the construction of the overlying base and surface layers.

Table A2 summarizes pertinent strength data gathered from the literature which has been derived from laboratory testing, and Table A3 summarizes strength data derived from field testing protocols. The information presented in Tables A1 through A3 is not exhaustive, but instead is representative of the types of data presented in the literature. A more detailed discussion of the studies and/or data presented in Tables A1 through A3 is provided in volume 2.

Methods of Measurement

The shear strength of lime-soil mixtures has been measured in the laboratory in a variety of ways: unconfined compressive strength, triaxial shear strength, indirect tensile (diametral tensile) strength, CBR and California R-value. The most common method of strength measurement is the unconfined compression test. There are many different protocols for performing this test, and the results vary widely depending on the protocol used. However, the important factor is the relative increase in shear strength due to lime stabilization. The literature provides substantial documentation of the effects of lime stabilization on shear strength and tensile strength increase for a wide variety of soil and aggregate types under a wide variety of testing and conditioning protocols.

A limited amount of repeated load permanent deformation testing has been accomplished. This type of testing is generally performed in a triaxial cell. Repeated loads are applied at a deviatoric stress that simulates the stress imparted by a moving heavy truck load. The permanent deformation is continuously measured during the test and is generally evaluated according to the maximum level of permanent strain and the rate of permanent strain.

Level of Structural Improvement Due to Stabilization

Laboratory

A review of Table A1 reveals that lime-treatment of fine-grained soils alters the physical properties of these soils resulting in:

7. Significant reduction in plasticity index.
8. Significant reduction in swell potential.
9. Textural change toward a more coarse-grained material.
10. Altered compaction characteristics demonstrating a lower maximum density and a higher optimum moisture content.
11. The reaction between lime and soil continues for extended periods of time, evidenced by the long-term maintenance of a high pH.

The magnitude of these effects, like all effects in lime-soil mixtures, is soil-dependent. Based on this soil and mineralogical dependency, it is difficult to offer a specific prediction of the level of improvement. Indeed this must be measured for each lime-soil mixture. However, the data do clearly show that very substantial and structurally significant changes are possible and are expected as lime substantially improves fine-grained soil physical properties in the large majority of applications. It is the role of the mixture design protocol to establish successful application of lime-treatment with a given soil and to assure that adequate proportions of lime are used to achieve a durable lime-soil mixture.

It is difficult to appreciate the effect of lime-treatment on soil physical properties without an understanding of the mechanisms of lime-soil reactions. Although an expanded discussion of lime-soil reactions is not within the scope of this report, an overview of these reactions are presented in Table A4 as a quick and abbreviated reference. These reactions are further explained and developed in Little (1995).

A substantial data base exists on the laboratory measured strength of lime stabilized soil and aggregates. There are two categories of strength upgrade: uncured and cured. Uncured strength enhancement refers to the shear strength improvement derived from reactions that occur quickly (within a few hours to several days). From Table A4, these reactions are cation exchange, molecular crowding and some level of rapidly occurring pozzolanic reaction. These reactions occur to some degree with most all fine-grained soils resulting in a substantial immediate strength improvement. This immediate strength gain may range from very modest to a strength gain of several hundred percent when compared to the untreated soil under similar conditions. Such improvement in uncured strength is usually associated with the benefits of providing an improved working platform and the ability to construct better base and surface layers due to a much improved (stronger and more uniform) construction platform.

Cured strength in lime-soil mixtures is due to the long-term pozzolanic process. To achieve and assure good long-term strength gain, it is necessary to utilize a proven mixture design protocol that identifies a soil or aggregate as lime-reactive and then provides a method for optimum lime content selection. As demonstrated in Table A2, long-term strength gains due to pozzolanic reactions are very much dependent on soil mineralogy. Thompson (1970) defined a reactive lime-soil mixture as one with a 350 kPa change in unconfined compressive strength after a 48-hour (45ºC) period of cure. Based on this definition, lime-soil mixtures can range from non-reactive to highly reactive with strength gains of over 10,000 kPa.

The strength requirements for using lime stabilized layers as structural layers in pavement systems vary considerably from agency to agency. Thompson (1970) defined a lime-soil mixture as acceptable for a structural base if the unconfined compressive strength exceeded about 1,050 kPa. The Texas Department of Transportation (TxDOT) requires a Texas Triaxial strength of at least 700 kPa when testing in accordance with TxDOT methods TEX 117-E and 221-E. The City and County of Denver, Colorado, MEGPEC (1998) specifications require an unconfined compressive strength of at least 1,200 kPa after 7-day, 38ºC cure and compacted to 95% of AASHTO T-99 compaction. The California Department of Transportation (CALTRANS) requires an unconfined compressive strength of 2,800 kPa when testing in accordance with CALTRANS methods. The literature (Table A2) demonstrate that these criteria can be met or substantially exceeded by lime stabilization.

The data in Table A2 address the effect of lime stabilization on unconfined compressive strength, tensile strength, flexural strength, California Bearing Ratio (CBR) and California R-Value. These data lead to the following salient points with respect to the role of lime-stabilization in altering the strength properties of soil and aggregate systems:

1. A significant level of long-term strength gain in lime stabilized soils and aggregates is possible and probable. The level of strength gain achieved can meet typical specifications required by various user agencies. Typical minimum unconfined compressive strength criteria for use as a structural base or subbase are between 700 kPa and 1,400 kPa.
2. The strength gain is time dependent and can continue for a very long period of time (many years in some situations) if adequate design procedures are followed.
3. The strength of lime-soil mixtures is less sensitive to fluctuations in compaction moisture content and in stress variations than are their untreated counterparts.
4. Lime-soil mixtures gain strength through pozzolanic reactions that are relatively slow when compared to portland cement hydration. However, ultimate strengths of lime-soil mixtures can be as high as 7,000 to 10,000 kPa or higher.
5. Lime stabilization can significantly reduce the potential for a soil or aggregate to accumulate permanent damage.

Field

Table A3 summarizes important field data on the in situ strength of lime-stabilized layers. These data include in situ CBR, Dynamic Cone Penetrometer (DCP), modulus of subgrade reaction and unconfined compressive strength from field cores. From these data, the following conclusions are drawn:

1. Very significant strength improvements were realized by lime stabilization in a variety of soil types and environmental conditions.
2. The level of strength improvement through lime stabilization of soils and aggregates was structurally significant. Marginal and unacceptable soil and aggregate strengths were improved to levels which meet structural requirements of subbases and bases.
3. Field measured strengths of lime-soil layers substantiate the level of strength gain measured in the laboratory.
4. Lime stabilization is a durable process and strength development can continue for an extended period of time (even years).

Relationship Between Laboratory and Field

Only limited research has been done with the specific objective of correlating laboratory and field values of strength and performance. One study, Hopkins et al. (1996) does provide data and empirically-based correlations based on these data. Despite the dearth of studies and data geared to correlate lab and field results, field data (Table A3) do generally verify and are consistent with laboratory derived data.

Impact of Lime Stabilization on Structural Performance

The magnitude of strength gain provided by lime-stabilization (Tables A2 and A3) allows one to design and construct subbases and bases with significantly improved resistance to shear failure and accumulation of permanent damage. The level of shear strength improvement through lime stabilization has been verified by laboratory and field testing.

In a mechanistic analysis, the resistance of a subbase or base course layer to plastic damage may be assessed by means of a number of approaches/models:

1. Repeated load permanent deformation models
   a. $\log \epsilon_p + a + b \log N$ - where ϵ_p is accumulated permanent strain, a and b are experimentally derived and N is the number of repeated load applications. Data for this model are obtained from triaxial testing with a

confining pressure typically ranging from 0 to 100 kPa. This response is dependent on the stress state under which the testing is conducted.

b. $\epsilon_p/N + AN^m$ - where A and m are experimental constants. This model is often referred to as the Ohio State model and was developed by Majidzadeh (1981) who found it to be successful in describing rutting development in all pavement layers including subgrade soils.

c. $RR = RD/N = A/N^B$ - where RR is the rate of rutting, RD is rut depth, N is the number of load applications and N and B are developed from lab or field calibration data. This model was developed by Thompson and Nauman (1993).

2. Stress ratio approach - Thompson (1990) found a good correlation between the ratio of deviator stress and shear strength as a means of differentiating between stable (low deformation potential) and unstable (high deformation potential) material. He found SSR to be a reliable parameter for assessing the potential of AASHTO Road Test materials to rut. If the SSR was below a threshold level, then rutting was not a problem. This approach has good potential. Thompson noted that low SSR values are related to low A-values in the equation in paragraph 1b above. In relative terms, low A-values are noted for reduced SSR's and large A's for increased stress ratios. Since stress ratio is a valid indicator of rutting potential, the factors influencing the stress state and strength of the in situ granular materials are important considerations.

Little (1995) used a similar method to evaluate deformation potential by suggesting that the compressive stress within the granular or cohesive layer in question should always be less than one-half of the compressive strength. This is because hundreds of compressive strength tests revealed that at stress values of about one-half of the compressive strength, the stress-strain curve becomes non-linear and accumulated strain or deformation occurs with loading.

Figure 1 illustrates representative stress-strain plots for a Burleson, Texas, clay (with and without lime stabilization). Figure 2 represents repeated load deformation plots for the same Burleson clay with and without lime stabilization. In the repeated load plots a deviatoric stress level of 140 kPa was applied per load cycle. As would be expected, the accumulated strain at this deviatoric stress level is well into the non-linear region for the unstabilized Burleson clay but is in the linear elastic region for the stabilized clay. Therefore, accumulated deformation is excessive for the natural Burleson clay at the 140 kPa stress level but is inconsequential for the 5% hydrated lime stabilized clay. Note also that at 20,000 loading cycles the natural clay is in a state of tertiary deformation whereas the stabilized clay is in a steady state situation.

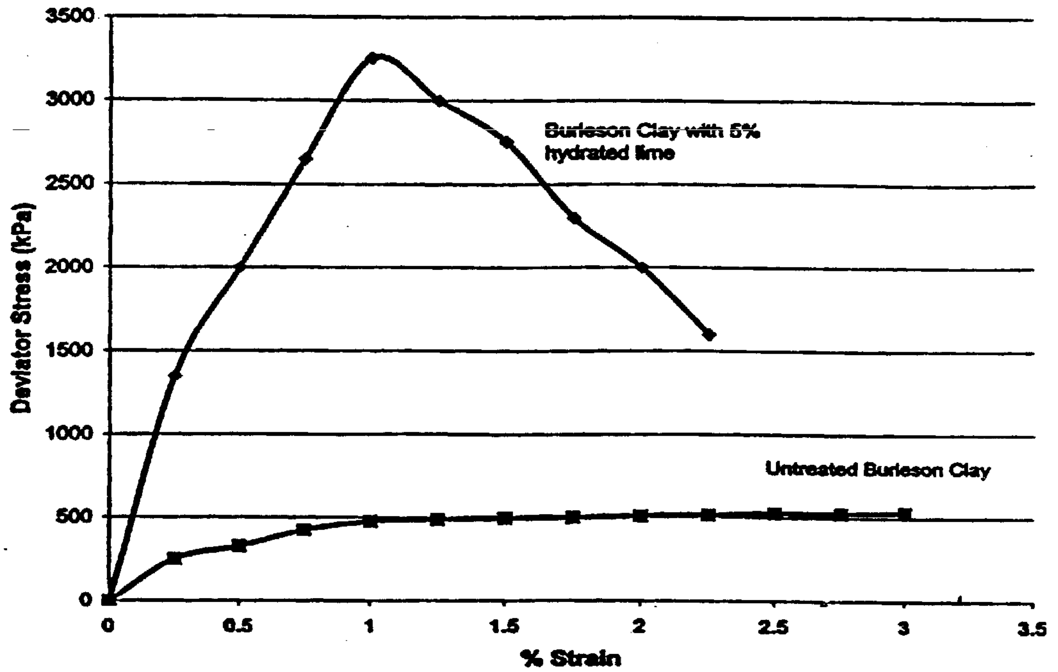

Figure 1. Typical stress-strain plots for Burleson soil with and without lime stabilization.

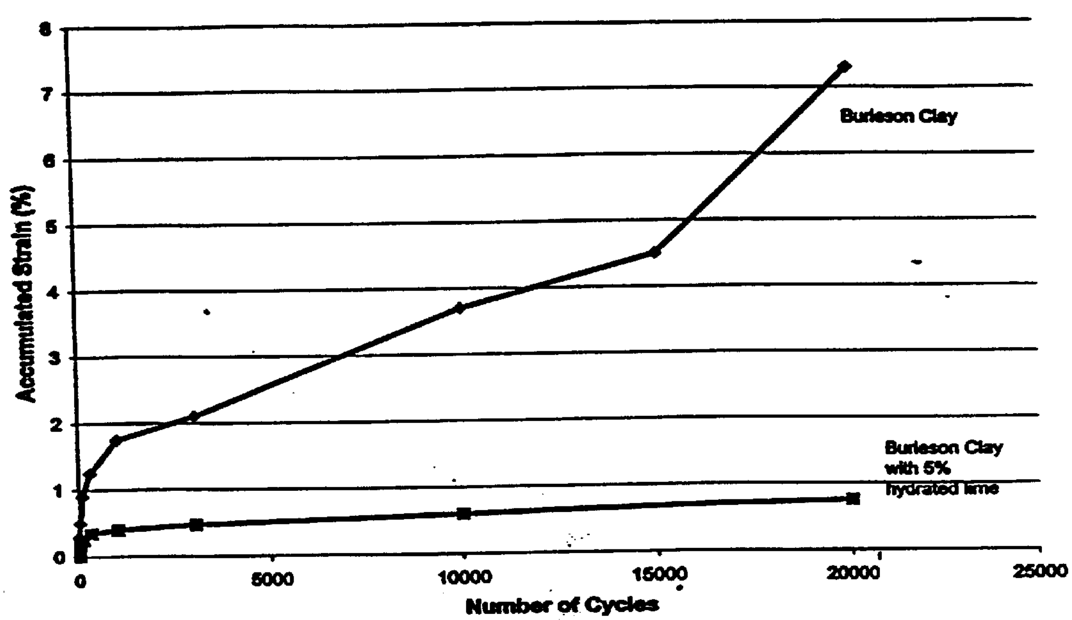

Figure 2. Repeated load deformation plots for the Burleson clay with and without lime stabilization.

The AASHTO 2002 Design Guide will be based on a Mechanistic-Empirical (M-E) approach. As such it is important to determine a testing approach for subgrade, subbase and base materials (unstabilized and stabilized) which is effective and compatible with M-E approaches. An excellent illustrative model is the Illinois (IL) M-E procedure. In this model granular base rutting is controlled by establishing minimum thicknesses for the asphalt concrete surface to limit the stress state within the granular layer to a level that will not promote rutting. The factors that influence the rutting susceptibility of the granular layer are material shear strength and the moisture sensitivity - loss of shear strength as a result of moisture increase. Subgrade rutting in the IL M-E approach is considered by limiting the deviator stress or subgrade stress ratio (SSR) at the pavement structure - subgrade interface to acceptable levels (generally in the range of 0.75 to 0.40). Increased rutting is sometimes permitted for lower design traffic ESAL's. In the IL M-E procedure the SSR criterion is < 0.5 (Thompson, 1998). Asphalt surface thickness design is typically governed by the tensile strain within the asphalt and the permissible stress state within the granular base.

Although resilient properties are important to the assessment of the stress state in the mechanistic analysis, it is the aggregate, soil or stabilized layer shear strength that dictates resistance to deformation and stability in the pavement. Thus it is important to characterize these properties in the testing process. The IL M-E testing approach (Thompson and Smith, 1987) includes repeated triaxial testing (for evaluating permanent deformation and resilient modulus) and a rapid shear test (100 kPa confining pressure) on the triaxial specimen previously subjected to repeated loading. In some cases, rapid shear tests are performed over a range of confining pressures on unconditioned samples to define the Mohr-Coulomb shear strength parameters of cohesive intercept and angle of internal friction (Thompson, 1998).

Thompson (1998) states that the SHRP-46 protocol is similar to the IL M-E procedure and includes a shear test on the conditioned specimen. However, the AASHTO T-294-94 procedure does not include shear testing of the "conditioned" specimen. Thompson (1998) further states that the "conditioning" stress state (1000 repetitions at 100 kPa deviator stress and 140 kPa confining pressure) in SHRP-46 and AASHTO T 294-94 for granular materials is not large enough to establish rutting potential. The IL protocol is a "conditioning" stress state (1000 repetitions at 315 kPa and 100 kPa confining pressure) and is adequate to differentiate among aggregates or soils with "excellent" to "inadequate" rutting resistance (Thompson, 1998).

Other, more sophisticated models to assess the permanent deformation response of granular materials in flexible pavements have recently become available. One of these was developed by Bonaquist and Witczak (1998). This method uses constitutive relationships based on the flow theory of plasticity to limit permanent deformation in granular layers. In this approach, a yield function is used to differentiate between elastic and plastic behavior. This yield function is a function of the stress state and may change on loading and unloading to describe cyclic hardening behavior. This approach requires a relatively sophisticated finite element model and triaxial laboratory testing. This method has good potential for specific pavement categories.

Recent work at the University of Illinois (Tutumluer, 1998) has shown that unbound aggregates typically exhibit anisotropic behavior due to compaction and subsequent load induced stiffening in the vertical direction. Both the effects of the anisotropic resilient stiffness and the dilative behavior under a single wheel have been successfully modeled by a nonlinear cross-anisotropic constitutive relationship in the granular material. Unlike the commonly used isotropic model, a cross-anisotropic representation has different material properties assigned to the horizontal and vertical directions. Correct modeling of stress states considering the existing locked-in residual stresses is essential, not only for describing the dilative behavior but also for reducing/eliminating the significant tensile stresses often predicted in aggregate or soil layers. Accurate assessment of anisotropic properties of granular layers may be an important consideration in the AASHTO 2002 Guide. The effects of the stabilized subbase or the effects of base stabilization on these anisotropic properties should be investigated in the AASHTO study

Any of (or a variation of) the aforementioned models can be used to assess permanent deformation potential in a mechanistic - empirical analysis. Obviously, permanent deformation potential is substantially improved through stabilization. This is evident in models 1a, 1b and 1c as, at a given stress level and moisture state, stabilization reduces the potential to accumulate damage. Furthermore, at a given level of deviatoric stress, the stress ratio is substantially reduced as is the potential to accumulate damage (model 2).

Tensile strength properties which are important in understanding and predicting the shrinkage cracking potential and flexural fatigue potential of lime-soil mixtures can be approximated from strength tests. Thompson (1966) determined that the indirect tensile strength of lime-soil mixtures is approximately 0.13 times the unconfined compressive strength. The 1987 TRB State of the Art Report No. 5 states that the flexural tensile strength of lime-soil mixtures is approximately 0.25 times the unconfined compressive strength. Because the fatigue life of lime soil mixtures is defined as a function of the stress ratio (ratio of induced tensile flexural stress to flexural strength), the stress ratio and hence tensile strength properties provide an excellent indication of a lime-soil mixtures resistance to fatigue cracking failure and can hence provide valuable information in an M-E analysis.

RESILIENT PROPERTIES

General

The pavement experts at the 1996 Workshop and at the 1997 Brainstorming session recommended that, to the extent possible, resilient properties be determined from NDT testing in lieu of laboratory testing. However, this does not mean that laboratory testing should be abandoned. On the contrary, the 2002 Guide must define and describe a protocol for measuring the required properties of the various paving layers. Resilient properties of the lime stabilized subgrade and/or base are critically important properties to the design process. It may be appropriate to determine resilient properties of lime stabilized materials in accordance with AASHTO T-294-94 or variations thereof to accommodate the stabilized nature of the materials. This protocol identifies the stress sensitivity of the material. Laboratory tests are necessary to establish the influence of stress state and moisture content. Laboratory tests are also needed to verify field-derived properties which may not produce unique solutions.

Overview of Key Studies

Key laboratory and field studies regarding the resilient properties of lime-stabilized soils and aggregate layers are summarized in Tables A5 and A6, respectively. As the lime-soil pozzolanic reaction takes place strengthening the soil, a stiffening process concomitantly occurs. This process significantly alters the stress-strain relationship of the material. Lime-stabilized soils fail at much higher deviatoric stresses than their unstabilized counterparts and at a much lower strain (typically about 1% strain for the stabilized mixture versus about 3% for the unstabilized counterpart). As a result, lime-stabilized mixtures are typically 10 to 25 times stiffer than their untreated counterparts. The resilient modulus is the key mechanistic property required to define the pavement layer's ability to distribute load. According to layered elastic theory, the efficiency of stress dissipation with depth within the layered elastic system is dependent on the elastic modulus. Pavement layers are usually non-linear elastic (stress dependent) or nonlinear viscoelastic. Therefore, it is more realistic and practical to characterize the layer in question in terms of a resilient modulus which is measured under a level of stress, temperature and duration of loading that duplicates as closely as possible that which occurs in the field. Lime stabilization substantially improves the strength of lime-soil mixtures through long-term pozzolanic effects. As strength increases, so does the stiffness. Resilient modulus is a measure of stiffness under a defined set of testing conditions. A more detailed discussion of resilient properties of lime-stabilized materials is presented in volume 2.

Methods of Measurement

The effects of lime-stabilization on stiffness have been documented in the laboratory by monotonically loaded unconfined compression tests and indirect tensile tests. In these tests the linear slope of the change in deviatoric stress versus the induced strain defines the modulus of the mixtures. Resilient properties of lime-soil mixtures as well as their untreated counterparts have been measured in repeated load triaxial tests (i.e., AASHTO T-294-94) and in repeated load diametral or indirect tensile tests (i.e., ASTM D-4123).

Resilient properties and deflection sensitivity in the field have been evaluated and documented by vibratory testing, seismic analysis, impulse deflection testing (using direct methods and back calculations), steady state deflection testing (using direct methods and back calculations) and correlations with strength tests such as the Dynamic Cone Penetrometer (DCP), Benkleman Beam testing and plate load testing.

Level of Structural Improvement Due to Stabilization

Laboratory

The review of pertinent literature on stiffness and resilient improvements to soils and aggregates made through lime stabilization summarized in Table A5 provide information from which the following conclusions are drawn:

1. Lime-stabilization often induces a 1,000% or more stiffness increase and a significant reduction in the strain at failure compared to untreated soils.
2. As illustrated by Little (1996), moderate to highly plastic Denver, Colorado, soils responded vigorously to the effects of lime. The AASHTO T-274 resilient modulus increased from 800% to 1,500% over resilient moduli of untreated soils prepared and tested in an identical manner. Resilient moduli in the range of 210 to 400 MPa were easily achieved after only 5-days of curing at 38 ^{o}C. This level of stiffness is typically consistent with a good quality base material.
3. Little (1996) demonstrated by means of ASTM D 4123 tensile modulus testing that the lime-stabilized soils were much less sensitive to the effects of moisture than were their untreated counterparts.
4. As with strength properties, resilient properties of lime-soil mixtures are very sensitive to level of compaction and molding moisture content.
5. Resilient properties of both reactive clays and of non-reactive fine-grained soils can be substantially improved with lime stabilization. Furthermore, the resilient properties of both reactive soils and non-reactive soils are much less sensitive to damaging environmental conditions (i.e., freeze-thaw effects) than their untreated counterparts.

Field

A review of pertinent field data based on vibrational testing, impulse deflection testing, plate load testing and DCP testing is summarized in Table A6. Based on these studies, the following conclusions are drawn:

1. Elastic moduli of lime stabilized layers subgrades calculated from vibratory testing and from back calculations from Falling Weight Deflectometer (FWD) testing demonstrate stiffness or resilient modulus increases of typically between 5 and 15 fold when compared to stiffnesses or resilient moduli of the same untreated soil.
2. Field resilient moduli have been documented to increase with time as have field determined compressive strengths. This modulus increase as a function of time indicates that long-term pozzolanic reactions often continue with time and can result in improved pavement performance, autogenous healing and improved fatigue and rutting resistance.
3. Back calculated moduli of lime stabilized subbases typically fall within a range (between 210 MPa and 3,500 MPa) which is considered to be structurally effective in terms of excellent stress distribution within the pavement system but yet not so stiff as to cause excessive shrinkage cracking.
4. Lime-treatment of reclaimed aggregate bases or lime-treatment of new aggregate bases has proven to be very effective in altering back calculated resilient moduli of the treated new material or reclaimed material to levels consistent with excellent performance. The studies of Little et al. (1994) and Syed (1998) are referenced as examples.
5. The effect of low percentages of hydrated lime to effectively upgrade the (in situ determined) resilient properties of calcareous bases was verified by back calculated resilient moduli. These findings verify laboratory experiments reported earlier which document the effect of low percentages of lime in improving the shear strength of calcareous bases.
6. The use of lime stabilized subgrades to provide a strong support below unbound aggregate bases was verified by Koshla et al. (1996). Their study demonstrated that composite moduli of sections with aggregate bases over lime stabilized subgrades were generally higher than composite moduli of aggregate bases without lime stabilized subgrades. Lime stabilized subgrades were found to enhance the performance of pavements containing both unbound aggregate bases and full depth asphalt layers. The back calculated moduli of lime-stabilized subgrades were generally equal to or higher than those of aggregate bases.

Relationship Between Laboratory and Field

It is important to establish a relationship between laboratory and field-derived resilient properties. However, this is a difficult and complex process and requires a large data base. Consequently, specific lab-to-field relationships will probably be determined by specific user agencies.

Impact of Lime Stabilization on Structural Performance

In a mechanistic - empirical design, the pavement layers must be accurately characterized in terms of their resilient properties. The resilient modulus of lime-stabilized subgrades and bases must be determined as a function of stress state and moisture state. The AASHTO T-294-94 protocol is probably the most applicable existing method. However, testing is currently underway to determine the importance of anisotropic or orthortropic conditions in lime-stabilized layers. Tutumluer (1998) and Lytton (1998) have demonstrated both empirically and theoretically the importance of orthotropic characterization of unbound aggregate bases.

Any mechanistic-empirical approach demands a good and accurate characterization of the resilient properties of all pavement layers.

FRACTURE AND FATIGUE PROPERTIES

General

Generally laboratory fracture testing is not performed on lime stabilized materials. It probably will not be specified in the 2002 Guide. This is because the tensile strength and fracture properties are directly related to the compressive strength and can be successfully predicted from compressive strength tests. The ability of the lime-stabilized layer to resist fatigue cracking can be assessed by means of a stress ratio approach in which the critical tensile flexural stress induced under the load should not exceed a specified percentage of the flexural tensile strength of the material.

It is well known that the shrinkage properties of lime stabilized materials are directly related to the ultimate strength and the ultimate modulus of the mixtures. Research for TxDOT at TTI (TxDOT studies 1287 (Little et al., 1994), 1432 (Little et al., 1997), and Scullion and Zollinger (1998) has established target limits on compressive strength and resilient modulus that can be used to control the level of shrinkage cracking and fracture damage in stabilized bases.

Overview of Key Studies

When pavement base and subbase layers are stabilized with Portland cement, lime , lime - fly ash or asphalt, the stiffness enhancement of these layers substantially changes the distribution of stresses within the pavement system. However this stiffening effect may place these layers in jeopardy of flexural fatigue. Fatigue occurs when flexural tensile stresses are induced under load within the stabilized layer. Although the stress levels are seldom high enough to produce failure in one load application, unless the pavement is severely under designed, repeated load applications at a significantly high stress level induce damage that results in crack initiation and propagation. Ultimately, with extended traffic applications, the level of fatigue damage will increase to a level deemed unacceptable.

Swanson and Thompson (1967) did ground breaking research on the fatigue characteristics of lime-stabilized mixtures. They established the flexural fatigue behavior of lime-soil mixtures by performing flexural fatigue beam testing at various stress levels. They established a relationship between stress ratio (the ratio of the tensile flexural stress induced within the stabilized material to the flexural tensile strength of the material) and the number of load cycles or applications until failure. Based on this research a design algorithm relating stress ratio to the number of load applications required to cause failure was developed as follows:

$$S = 0.923 - 0.058 \log N \qquad \text{[equation 2]}$$

where S is the stress ratio and N is the number of load applications.

This relationship is a conservative one based on flexural fatigue testing of several lime-stabilized soil mixtures. In this testing the strengths at 5-million stress applications varied from 41% to 66% of the ultimate flexural fatigue strength with an average of 54%.

Moore and Kennedy (1971) performed fatigue testing using the indirect tensile or diametral testing configuration. They established the effect of long-term curing on stress ratio reduction and improved fatigue properties as the lime-stabilized pavement layer ages. Little (1996) performed indirect tensile testing on nine different Colorado soils and on three different Texas soils over a wide range of molding moisture contents and established a substantial improvement in tensile strength characteristics (compared to the untreated counterparts) over a wide range of molding moisture contents.

Table A7 summarizes pertinent studies on fracture and fatigue. A more detailed discussion of fracture and fatigue properties is presented in volume 2.

Method of Measurement

Flexural fatigue has been characterized by testing beams under controlled-stress, third point repeated loading. In this method the number of load applications are related to the stress conditions applied. The stress conditions are normally characterized in terms of the ratio of the flexural stress applied to the flexural tensile strength. Fatigue testing can also be done in the diametral mode. In this case controlled-stress, repeated loading is accomplished and, as in the flexural beam fatigue testing, a correlation is established between stress ratio and number of cycles or load applications to failure.

Indirect or diametral (ASTM D 4123) tensile testing provides insight to the ability of lime-stabilized materials to function in an environment that produces tensile stresses. These tensile stresses may be load-induced or non-load induced. Such tensile stresses may be induced by environmental means.

Level of Structural Improvement Due to Stabilization

Lime-stabilization substantially increases shear strength and, concomitantly, tensile strength. This strength increase provides a stiffer layer with improved load distributing capabilities. However, as the stiffness of the layer increases through the development of cohesion within the stabilized layer, the layer becomes more susceptible to load-induced tensile stresses that can lead to fatigue failure unless proper design steps are taken to reduce the potential of load induced damage. This is generally accomplished by ensuring that the layer thicknesses are such as to insure the development of acceptable flexural stresses within the stabilized layer. Typically the design parameter is the flexural tensile stress ratio.

Impact of Lime Stabilization on Structural Performance

Lime-stabilization can provide increased shear strength, increased stiffness and increased tensile strength. Pavement layer thicknesses must be adjusted in order to keep flexural tensile stresses within tolerable levels. This is usually done by adjusting pavement layer thickness (particularly that of the stabilized layer) to a level that will accommodate design traffic without unacceptable fatigue cracking damage. This is normally done by keeping the stress ratio in check. The stress ratio is correlated with number of loading applications until fatigue failure by means of empirically derived relationships between stress ratio and number of cycles or load applications until fatigue failure.

PROPERTIES OF LIME-STABILIZED MIXTURES RELATING TO DURABILITY

Overview of Key Studies

The positive impact of lime-stabilization on pavement performance due to the shear strength gain and stiffness gain over time has been addressed in preceding sections. This section addresses the ability of the lime-stabilized layer to maintain these desirable properties over time and particularly their ability to resist the effects of moisture and freeze-thaw cycling. Several key studies on the durability of lime-stabilized mixtures and pavement layers are summarized in Table A8. A more detailed discussion of the studies and information summarized in Table A7 is offered in volume 2.

Methods of Measurement

Resistance of lime-stabilized pavement layers to the effects of moisture and freeze-thaw conditions have been monitored by evaluating the effects of water exposure through soaking, cyclic freeze-thaw, strength recovery during rest periods (autogenous healing), dielectric value measurement, long-term strength retention and monitoring of chemical and physical property changes in lime-soil mixtures subjected to the effects of leaching.

Impact of Lime Stabilization on Pavement Performance

All types of pavement layers have suffered from the effects of the environment and any pavement layer is susceptible to the deleterious effects of these factors. However, the literature provides insight to the ability of well-designed lime-stabilized layers to resist and withstand the deleterious effects of moisture and freeze-thaw cycling. Both laboratory and field evidence is provided. The key points of the literature review are summarized as follows:

1. Prolonged exposure to water through soaking has only slightly detrimental effects on strength loss. Soaked unconfined compressive strength (UCCS) values measured by Thompson (1970) were between 0.7 to 0.85 of the unsoaked UCCS. The effect of soaking is dependent on the level of strength or pozzolanic reaction achieved prior to the beginning of the period of soaking. Little (1998) demonstrated that once a significant level of pozzolonic reaction takes place, the effects of soaking are not substantial (less than 10% UCCS loss). However, when soaking occurs prior to significant pozzolanic strength development or without significant strength development in the same soils, deleterious effects of soaking can be much more detrimental (strength loss of up to 40% of dry UCCS).

2. Dempsey and Thompson (1968) describe the average rate of strength decrease in a well stabilized lime-soil layer as typically 60 kPa and 1,220 kPa per freeze-thaw cycle, respectively, for 48-hour and 96-hour (48.9°C) curing.
3. Thompson and Dempsey (1969) demonstrate the considerable strength development during periods of favorable environmental conditions even following periods of damage. This phenomenon called autogeneous healing identifies the importance of designing for and insuring long-term pozzolanic activity on performance.
4. Robnett and Thompson (1976) demonstrated that lime-stabilization of both highly lime reactive and lowly lime reactive soils benefit from lime stabilization, and lime-stabilization very substantially improves resistance to freeze-thaw damage.
5. Little et al. (1998) demonstrated the effect of lime-stabilization in reducing moisture susceptability of marginal aggregate bases in Texas (including caliche and gravel aggregates). Little used the dielectric value in these deteminations.
6. Several long-term field studies document the durability of lime and lime-fly ash mixtures.
7. McCallister and Petry's study (1990) of the sensitivity of lime-soil mixtures to leaching with water containing various levels of salts demonstrated that loss of stabilization may occur if the soil is understabilized but is not likely to occur if proper stabilization is achieved. This underscores the importance of a sound mixture design protocol.

Impact of Lime Stabilization on Structural Performance

In order for designers to assign structural significance to lime stabilized layers they must be confident that the layer will be durable. The literature, as summarized in the previous paragraphs, provides evidence of this durability. A pressing need is to define a lime-stabilized mixture design and testing protocol which accounts for durability. This is addressed later in this document. A test of moisture ingress based on surface dielectric value is incorporated in the protocol with strength testing to help insure durability.

DESIGN OF RELIABLE LIME-STABILIZED PAVEMENT LAYER

Lime has long been effectively used to modify subgrades to provide a working platform and as a construction expedient. Lime has proven to be very effective for this purpose and is widely used for this purpose. A new direction must be taken when using lime to stabilize subgrade soils and base course aggregates to achieve structural benefit. The soil or aggregate must be sufficiently altered to achieve the required resilient properties, permanent deformation properties and strength properties for the purpose intended. Little (1995) recommends the Thompson method of mixture design for lime stabilized mixtures. This method uses the pH test to determine the level of lime required to satisfy the demand of the soil for lime and to still provide enough residual lime and a high enough pH to drive the all important pozzolanic reaction which is responsible for the strength and stiffness development. The Thompson method verifies the design lime content through strength testing.

The literature (Thompson, 1970, Petry and McAllister, 1990, and Little 1995) demonstrates that mixtures can be effectively altered with lime to reduce plasticity, reduce swell potential and improve strength without using the amount of lime required to optimize strength properties and to assure durability. This study recommends an extended mixture design protocol to be incorporated in the 2002 Guide to assure the durability and permanency of lime stabilized subgrades and bases.

TTI has developed a testing protocol to assess moisture sensitivity quickly and accurately. The test measures the dielectric value of the surface of a compacted cylindrical sample subjected to capillary absorption of moisture for a period of between 24 and 250 hours. While the long-term test is preferred to establish the equilibrium dielectric value, the short term test can be substituted if time is not available for the long-term test. TTI (Little et al., 1998) has established criteria for the dielectric value test and has established the beneficial effects of stabilizers (including lime, lime - fly ash and portland cement) in reducing the dielectric value to within acceptable or good levels for most soil and aggregate systems which are highly susceptible to moisture damage without lime.

A lime-stabilized pavement layer that will provide structural benefit must begin with a mixture design protocol that will insure optimal design. Following this design it is necessary to be able to assign realistic properties to the stabilized layer that can be achieved in the field. This section of the report outlines such a mixture design protocol and method to approximate pertinent engineering properties of the in situ lime-stabilized pavement layer. This protocol is being verified in phase 2 of this study.

MIXTURE DESIGN

Step 1: Soil Classification as Assessment for Suitability for Lime Stabilization

Lime is an appropriate stabilizer for most cohesive soils. The reactivity of lime with soil is predicated on the type and amount of clay mineral present in the soil. Although lime has been effective in the stabilization of a wide range of soils, certain index properties can be used to assess whether or not lime is an appropriate stabilizer. A study in the l970's sponsored by the U. S. Air Force (1976) identified plasticity index and percent fines (minus 75 micron material) as simple and effective indices to select candidate soils for lime stabilization. According to this approach, a candidate soil for lime stabilization should possess at least 25% minus 75 micron material and have a PI of at least 10.

Soil-lime reactions and the stabilization process is not only affected by mineralogy but also by the presence of other compounds within the soil including organics and salts. As a general rule, soils with organic contents in excess of one percent may be difficult to lime stabilize or may require uneconomical quantities of lime to stabilize. High salt concentrations may also interfere with or affect stabilization. The most important salts are sulfate salts (sodium, magnesium or calcium sulfates). This is because high sulfate concentrations can lead to deleterious reactions among the lime, the soil minerals, the sulfate ions and the water of construction or water within the soil. These deleterious reactions can lead to loss of stability and heaving. As a general rule it is important to consider the presence of sulfate salts by investigating local information, referring to county soil reports or geological or geotechnical reports or by performing soluble sulfate tests. If the total soluble sulfate level is greater than about 0.3% in a ten to one water to soil solution, then additional precautions to guard against sulfate reactions, such as swell tests, may be warranted (Little, 1997).

Step 2: Perform Eades and Grim pH Test (ASTM D-977) to Determine Approximate Optimum Lime Content

The optimum lime content is approximated using the Eades and Grim pH test as explained in ASTM D 977. This test will identify the lime content required to satisfy immediate lime-soil reactions and still provide the level of calcium and residual high pH necessary to provide the proper conditions for the long-term pozzolanic reaction. This reaction is necessary for stabilization and durability. The lime content established by the pH test is an indicator of optimum. It is not necessarily the optimum content and must be verified by strength testing.

Step 3: Determine Moisture/Density Relationship for the Lime Treated Soil and Determine Moisture Sensitivity and Strength Gain Following Accelerated Curing

Determine the moisture/density relationship according to the appropriate protocol defined by the user agency, i.e., AASHTO T-99, T-180, Texas Method 113A, etc., of the lime-soil mixture at the approximate optimum lime content (from the Eades and Grim protocol). Then compact samples at three moisture contents (optimum, 1% above optimum and 1% below optimum) for each of three lime contents (Eades and Grim optimum and ±1% of optimum). This represents 9 samples. Cure the samples for 5 days at 38ºC in plastic bags so the water of reaction in the pozzolanic process will not be lost. Following curing subject the samples to 10-days of capillary soak. The capillary soak protocol consists of placing the sample on a porous stone with a water level at the top of the porous stone. During the period of capillary soak, use a dielectric probe to measure the surface dielectric value of the compacted sample. Record the plot of dielectric value (DV) versus time of capillary soak. Capillary soak should continue for 10-days or until the DV achieves an ultimate or asymptotic value. This value should not exceed 16 for acceptability to moisture resistance. A DV of 10 is considered excellent (Saarenketo and Scullion, 1996). Table 2 provides examples of the effect of hydrated lime on the Texas Triaxial compressive strength and the moisture susceptibility of selected Texas aggregates of marginal quality.

The DV is actually a quick and continuous way to measure moisture content. The DV of free water is 81 whereas the DV of ice (structured water) is only 4. Therefore, as the DV reaches a high, equilibrium value when measured at the top of a sample, it demonstrates a high potential for the sample to attract, hold and transport water. The test is not a fundamental test because the time required to achieve an equilibrium value is dependent on the size of the sample (since size dictates the time of capillary rise). However, once an agency adopts a convenient sample size, DV criteria can be established and verified.

For convenience, at this point, a 100 mm diameter by 110 mm high sample (as fabricated by AASHTO T-99 or T-180 or ASTM D-5102) is recommended. However, the sample size can easily be adjusted to mesh with other testing protocols such as the Texas Triaxail Method (TEX 117E).

Table 2. Change in DV due to stabilizers for selected high fines, moisture sensitive aggregates.

Aggregate Base Material	Dielectric Value of Natural Material		Texas Triaxial Shear Strength, kPa	
	Untreated	Lime Treated	Untreated	Lime Treated
Abilene Clements - caliche with high PI fines	14	8 (1.5%)	320	1,700 (1.5%) 2,100 (3.0%)
Abilene Johnson - caliche with high PI fines	36	13 (1.5%) 11 (3.0%)	250	1,650 (1.5%) 1,550 (3.0%)
Abilene Tubbs - caliche with moderate PI fines	22	23 (1.5%) Lime was ineffective in affecting DV	310	450 (1.5%)
Amarillo Buckles - caliche with high PI fines	35	6 (1.5%) 24 (3.0%)	315	650 (1.5%) 1,050 (3.0%)
Amarillo Jordan - river gravel with moderate PI fines	6	3 (1.5%)	175	650 (1.5%) 750 (3.0%)
Yoakum Victoria - river gravel with high PI fines	18	5 (1.5%) 5 (3.0%)	210	610 (1.5%) 750 (3.0%)
Lufkin - Iron ore gravel , low PI	7	5 (1.5%)	400	500 (1.5%)

Following capillary soak, perform unconfined compressive strength testing in accordance with ASTM D 5102 and compare against the criteria for acceptance in Table 3, for example. If the lime-soil mixture is not reactive (i.e., a strength of at least 700 kPa is not achieved) then the lime-soil mixture may be considered to be a lime modified system instead of a stabilized system. In this case the structural benefit may be based on improved CBR, R-value, etc.

Note that accelerated cure is not always a good approximation of the strength which can be gained by long-term normal cure. Thus, as a reference to long-term strength development at nominal pavement subgrade temperatures, cure at least one set of lime-soils samples at Eades and Grim optimum (three moisture contents bracketing optimum - 3 samples) for 28 days at 20ºC.

Table 3. Soil-lime mixture compressive strength requirements. (After Thompson, 1970).

Anticipated Use	*Residual Strength Requirement (kPa)[a]*	*Strength Requirements for Various Anticipated Service Conditions*			
		Extended Soaking for 8 Days (kPa)	*Cyclic Freeze-Thaw*		
			3 Cycles (kPa)	*7 Cycles (kPa)*	*10 Cycles (kPa)*
Modified Subgrade					
	140	350	350	630	840
Subbase					
Rigid Pavement	140	350	350	630	840
Flexible Pavement- 254 mm	210	420	420	700	910
Flexible Pavements- 200 mm	280	490	490	700	980
Flexible Pavement - 130 mm	420	630	630	910	1,120
Base					
	700	910	910	1,190	1,400

Note: a - Min. anticipated strength following first winter exposure.
b - Strength required at termination of field curing following construction to provide adequate residual strength.
c - Number of freeze-thaw cycles expected in the soil-lime layer during the first winter of exposure.
d - Total pavement thickness overlying the subbase.

DETERMINE DESIGN MODULUS

General

In order to be able to assign structural significance to a stabilized subgrade the designer must be reasonably confident that the stabilization is permanent and that the structural contribution is significant. Although permanency or durability cannot be absolutely assured, it is possible to provide a high level of reliability by following the mixture design procedures in the following section.

Procedure

In 1995, Little et al. recommended a process by which to assign structural significance to lime-stabilized layers. The first step is to assign a realistic approximate resilient modulus to the layer. This can be done by either laboratory resilient modulus testing or from pre-existing field data. If laboratory testing is selected, then the resilient modulus (K1) should be determined in accordance with AASHTO T-294-94 after curing for 5-days at 38ºC. If laboratory testing facilities for such testing is not available, the resilient modulus can be approximated from unconfined compressive strength testing when compressive strength testing is performed in accordance with ASTM D 5102 or Texas Method TEX-121-E following a curing period of 5-days and at a temperature of 38ºC. Then, based on an empirical relationship between laboratory derived unconfined compressive strength, determine the approximate resilient modulus of the lime-stabilized layer.

If seasonal field deflection data and back calculated modulus data are available, then the average annual resilient modulus should be calculated using the weighted average annual modulus calculation described in the 1986 AASHTO Pavement Design Guide.

Little et al. (1995) also recommended tempering this value by considering the level of subgrade support. In other words, they found that the modular ratio of the lime stabilized layer (E_{LSS}) to the natural subgrade (E_{SUB}) had a strong effect on the E_{LSS}. The approach is amplified in the following paragraphs.

Estimation of Stabilized Subgrade Modulus

An estimate of the design resilient modulus of a lime-stabilized subgrade can be determined based on the 5-day unconfined compressive strength determined in accordance with ASTM D 5102 at a test temperature of 38°C and an estimate of the average annual subgrade modulus based on FWD data modulus back calculations.

A review of work by Suddath and Thompson (1975), Thompson and Figueroa (1989) and Little et al. (1995) supplemented by testing (Little et al., 1995) reveals a relationship between the unconfined compressive strength of the lime-soil mixture and the resilient modulus of the mixture.

Figure 3 presents a relationship between unconfined compressive strength and flexural modulus (based on data from Thompson and Figueroa (1989)), unconfined compressive strength and back calculated field moduli (determined from FWD data from the 1287 study) and unconfined compressive strength and compressive moduli (based on data from Thompson and Figueroa (1989)). From this figure, it can be seen that the relationship between unconfined compressive strength and flexural modulus and between unconfined compressive strength and field (FWD back calculated from study 1287) modulus are in reasonable agreement. This relationship was further verified by CTL/Thompson (1998) for a variety of Denver, Colorado, soils. The compressive modulus approximated from unconfined compressive strength data appears to be a conservative approximation of the modulus of the lime stabilized layer. Based on the findings summarized in Figure 3, a realistic and conservative approximate modulus for the lime-stabilized layer that can be used in design approximations is presented by the dashed line in Figure 3. For clarity, this relationship is reported in Figure 4.

It is reasonable that the resilient modulus of the stabilized subgrade should also be affected by the level of support provided by the natural subgrade. Figure 5 is a plot of subgrade resilient modulus versus the ratio of modulus of the lime-stabilized subgrade (from FWD back calculations) to modulus of the natural subgrade (from FWD back calculations). These data indicate that for natural subgrade moduli below about 50 MPa, the modulus ratio is typically 10 or above. For subgrade moduli between 50 MPa and 200 MPa, the modulus ratio is between 5 and 10, and for subgrade moduli exceeding 200 MPa, the modulus ratio is less than about 5. This concept is reasonable in that one would expect that the in situ modulus ultimately provided by the stabilized layer is influenced by the supporting subgrades. For example, a very stiff stabilized layer over a soft subgrade will crack leading to a reduction in the response modulus of the layer.

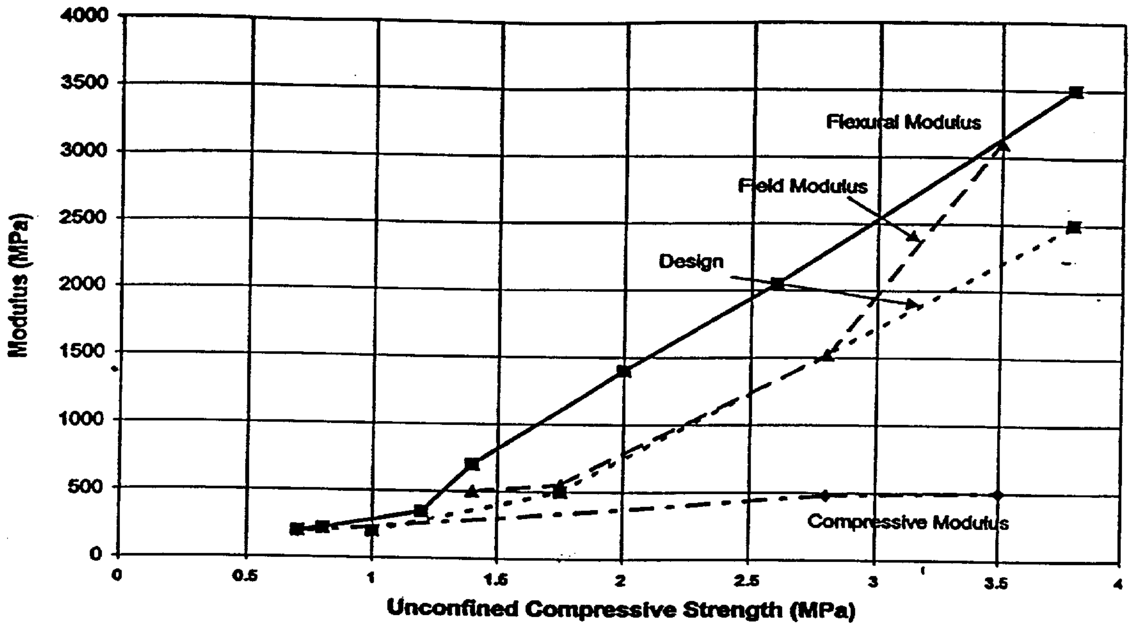

Figure 3. Relationships Between Unconfined Compressive Strength and Moduli of Lime-Stabilized Soils. (After Little et al., 1995).

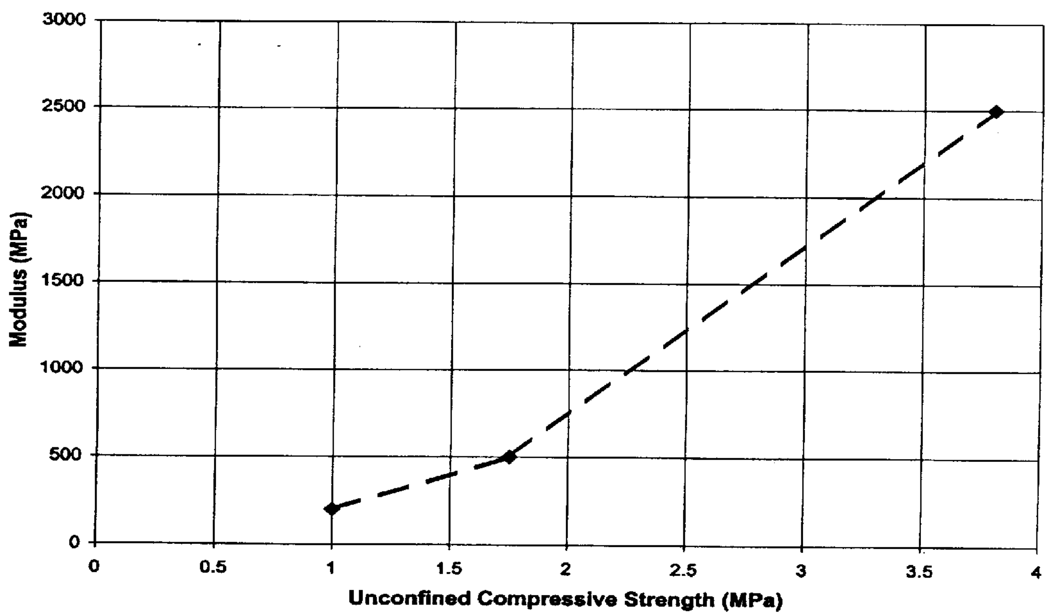

Figure 4. Selected Design Relationship Between Unconfined Compressive Strength and Resilient Modulus for Lime-Stabilized Subgrade Pavement Layers. (After Little et al., 1995).

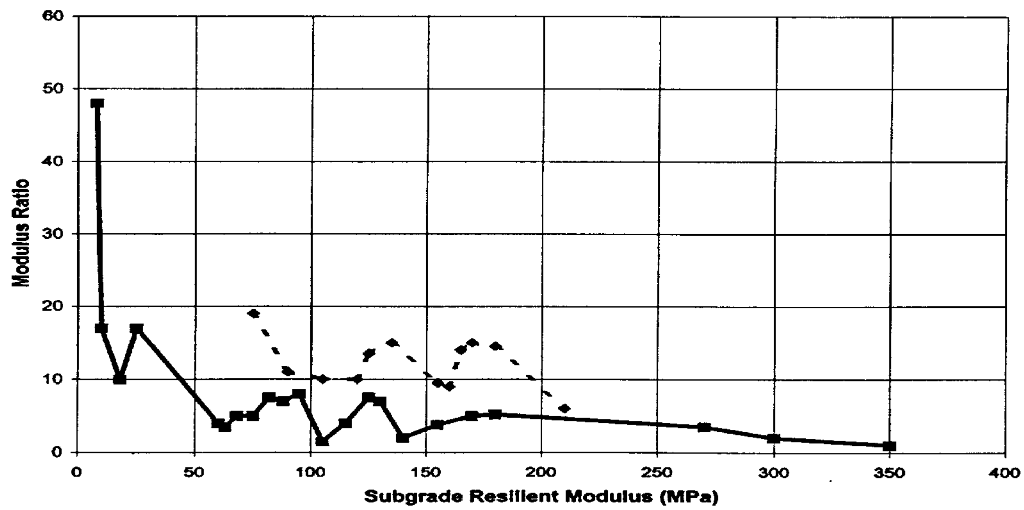

Figure 5. Relationship Between In Situ Modulus of the Natural Subgrade Soil as Determined by FWD Measurements and the Modulus Ratio (Lime-Stabilized Layer to Natural Subgrade Layer) as Determined by FWD Measurements. (After Little et al., 1995).

DETERMINE RESISTANCE OF LIME STABILIZED LAYERS TO FLEXURAL FATIGUE

Once the lime-stabilized soil mixture has been determined to be reactive, e.g., unconfined compressive strength of 700 KPa or greater and an increase in unconfined compressive strength of at least 350 KPa over that of the unstabilized soil, and the average annual roadbed modulus and stabilized layer moduli have been determined, evaluate the ability of the pavement structure to resist flexural fatigue.

Perform this evaluation using any layered elastic computer model. This evaluation is easily incorporated into computer models. In the absence of a computer model, assess the ability of the stabilized layer to resist fatigue damage by

1. Determining the critical radial tensile stress developed under load within the lime-stabilized layer and
2. Comparing the flexural strength of the stabilized layer with the critical flexural tensile stress developed within the stabilized layer.

As shown in Figure 6, the stress ratio, ratio of induced tensile flexural stress to flexural strength, should be less than 0.50 to insure a long (10^7 axle applications or greater) life or a fatigue resistant layer. Since the flexural strength is approximately 0.25 times the unconfined compressive strength and since the ratio of tensile strength induced within the stabilized layer should be less than 0.50, the critical flexural stress within the stabilized layer should not exceed 12 percent of the compressive strength.

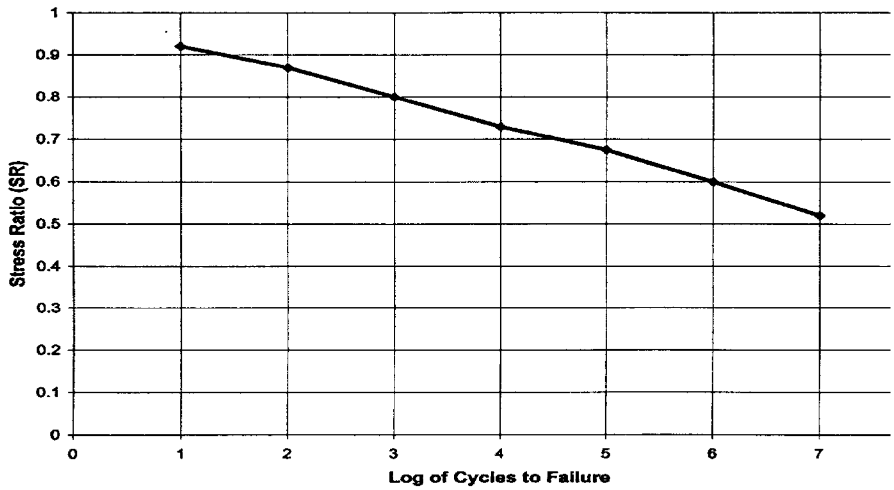

Figure 6. Stress Ratio Versus Cycles to Failure Fatigue Relationship (After Thompson and Figueroa (1989)).

DETERMINE PERMANENT DEFORMATION POTENTIAL

Resilient modulus is important in determining the stress distribution within pavement layers. However, the importance of resilient modulus to structural response and performance of granular pavement layers has been questioned by Elliot and Thompson (1985). They found shear strength and rutting potential to be the most important properties relating to performance for granular and subbase materials. This should also be the case for stabilized subbases and bases.

Probably the most effective method to accurately assess the permanent deformation potential and stability of lime stabilized layers is either a repeated load permanent deformation test or a rapid load shear stress test coupled with a stress ratio analysis.

If the first option is adopted, it can be tied to resilient modulus testing in accordance with AASHTO T-294-94. Repeated load testing at a stress amplitude deemed appropriate to evaluate accumulated permanent strain can simply be added to the AASHTO T-294-94 testing protocol following resilient modulus testing. The samples tested for resilient modulus and permanent deformation properties should be conditioned to achieve a representative moisture condition for the environment represented. One attractive approach to achieve this is to subject test samples to capillary soak (Little et al., 1998) for 10 days or until equilibrium moisture is reached. Moisture equilibrium may be noted as when the surface dielectric value (DV) achieves equilibrium (Little et al., 1998). Model 1a, page 20, may be used to characterize permanent deformation potential.

In the second and more time efficient option, the unconfined compressive strength of the sample can be determined either by subjecting the AASHTO T-294 specimen to compressive failure under a rapidly applied monitonic load or by determining the unconfined compressive strength on separate samples prepared and tested in accordance with ASTM D-5102 or similar method. As stated in the previous paragraph, a moisture state representative of equilibrium or design conditions for the environment in question should be established prior to testing. Acceptable performance in this approach is based on comparing the ratio of induced stress under traffic loading to shear strength to tolerable limits or a shear stress ratio criterion.

PHASE II VERIFICATION TESTING

Phase II research will assess the proposed mixture design protocol to establish desired engineering properties on a set of soils selected to represent typical candidates for lime stabilization.

The proposed mixture analysis protocol is designed to produce a mixture which will posses the structural properties desirable in a pavement layer of major significance. The procedure measures material engineering properties that are critical to the performance of the lime-stabilized mixture in a pavement structural layer. These include the resilient properties which define the ability of the mixture to dissipate pressures developed under heavy wheel loads so that those wheel loads will not over stress the weaker pavement layers. Also included are strength properties of the lime-stabilized mixture which define its ability to resist deformation and cracking. Finally the protocol will assess the ability of the of the mixture to resist the deleterious effects of moisture and will also assess the permanency of the mixture.

APPENDIX A

DATA AND LITERATURE SUPPORTING FINDINGS IN TABULATED FORMAT

Table A1. Summary of information verifying modification in texture, plasticity and compaction through the addition of lime.

Source of Information	Parameter Evaluated	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Thompson (1967) Holtz (1969) Little (1995) Eades et al. (1960)	Plasticity and work ability	Atterberg limit testing	Lime reduces PI and makes the soil more workable as the lime reacts with the clay surface. The reaction is mineralogy dependent, but almost all plastic soils show a plasticity reduction and work ability increase. Some very plastic soils (PI's of over 50) can be rendered non-plastic with lime. Information from a very large (world-wide) data base confirms.	Physical property improvements such as PI reduction can substantially reduce moisture sensitivity of strength and resilient properties which can be accounted for in environmental models of the pavement layers.
Thompson (1969) Goldberg and Klein (1952) Little (1995) Dempsey and Thompson (1968)	Volume change	Various methods including CBR and consolidometer	Data demonstrating swell potential reductions from 8 to 10% (untreated) down to less than 0.1% (treated) are common as lime reduces PI and swell potential. Swell and PI reduction effects are immediate but can substantially improve with time of curing and pozzolanic reaction.	Reduced swell potential can be accounted for in environmental effects models.

Source of Information	Parameter Evaluated	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Neubauer and Thompson (1972) Little (1995)	Compaction characteristics	AASHTO T-99 and T-180	The reaction between lime and soil causes a alteration of the moisture-density relationship that is soil dependent and is also dependent on the time of curing and amount of lime added. The density curve peaks at a higher moisture content and at a lower value of density with lime than without.	Improved compaction characteristics provides a subbase with better support for overlying layers - particularly unbound, granular layers. This will probably be accounted for in a mechanistic approach by an improved resilient modulus of the overlying layer.
Basma and Tuncer (1991)	Swell potential	One-dimensional swell test	Swell potential of a high PI clay with a swell pressure of 2,600 kPa was reduced to 1,700 kPa with 10% hydrated lime (immediately) and was further reduced to 0 kPa with 28-days of cure at only 4% hydrated lime.	Reduced swell potential can be accounted for in environmental effects model.
Basma and Tuncer (1991)	Textural change	Soil classification and material finer than 2 microns	A plastic clay's classification changed from CH to ML with the addition of only 3% hydrated lime. The percent smaller than 2 microns decreased (with the addition of hydrated lime)from 56% (untreated clay) to 40% (no cure), 10% (7-day cure) and 2% (28-day cure).	Textural changes affect the important material characteristics of resilient properties and strength properties.

Source of Information	**Parameter Evaluated**	**How Measured**	**Results and Practical Impact on Pavement Performance**	**Impact on Mechanistic Design**
Eades et al. (1963)	pH	pH test according to ASTM C-977 (appendix)	pH of the lime stabilized soil maintained a high level suitable for pozzolanic reaction (>10) for a period of over 3 years.	Maintenance of high pH indicates the ability for the pozzolanic reaction to continue over a long-term promoting further plasticity alteration and continued strength gain.
Bicysko (1996)	pH	pH test according to ASTM C-977 (appendix)	Lime stabilized layer maintained pH of over 10 for over 16 years.	

Table A2. Summary of strength derived through lime stabilization (determined in the laboratory).

Source of Information	**Strength Parameter Measured**	**How Measured**	**Results and Practical Impact on Pavement Performance**	**Impact on Mechanistic Design**
TRB State of the Art Report - 5 (1987) Neubauer and Thompson (1972)	Uncured strength	Unconfined Compressive Strength (UCCS), CBR or other methods	Immediate effects of lime in soil can promote reduction in plasticity, reduced moisture retention and improved compaction characteristics resulting in strength gains ranging from a modest level to several hundred percent. Working platform provided by lime can result in better construction of aggregate base and HMA surface layers.	Working platform can improve construction quality of unbound aggregate base and surface HMA layers which may be reflected in in-situ resilient and strength properties of those layers.
TRB State of the Art Report - 5 (1987)	Cured strength	UCCS	Many Illinois soils show strength increase of 700 kPa or more over natural soil strength following 28-day, 22^o C curing. Extended curing of the same soils at 22^{o}C resulted in UCCS increases of up to 4,375 kPa. Prolonged curing of the AASHTO test embankment soil (75-days at 48.9 ^{o}C) resulted in very high strengths (about 11 MPa). Field evidence indicates some soil-lime mixtures can continue to gain strength for in excess of 10-years.	Strength properties of lime-stabilized layer must be accounted for in mechanistic-empirical (M-E design).
Thompson (1969)	Cured strength	UCCS	Thompson reports a strong influence of compaction on UCCS. A 5% increase in compaction can increase UCCS by as much as 60%.	Same as above.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Eades and Grim (1963)	Cured strength	UCCS	Eades and Grim measured the cured UCCS of soils of six distinctly different mineralogies and found the ultimate level of strength gain to be dependent on the amount of lime added and the mineralogy of the soil. Their data demonstrated strength increases due to lime stabilization of 200% to 1,000%. The study illustrated the importance of mixture design to determine optimum lime content.	Same as above.
Doty and Alexander (1978)	Cured strength	CALTRANS Method	The California study evaluated strength gains through lime stabilization for 12 different soils and found 7-day cure at 38^{o} C to be roughly equivalent to 28-day 23^{o} C cure. Strength increased with time of curing and even some soils with low PI demonstrated very significant strength gains. All soils exhibited strength gains, and some soils exhibited strength gains in excess of 10 MPa after 360-days of curing. Significant strength gains were recorded between 180 and 360-days.	The time-dependent nature of pozzolanic strength development must be accounted for in M-E design.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Uddin et al. (1997)	Cured strength	UCCS	A plastic clay was subjected to curing for 180-days at lime contents ranging between 2.5% and 15%. Very substantial strength gains occurred between 60-days and 180-days. Strength gains were maximum at 10% lime and decreased with 15% lime. Optimum lime content produced an 1,100% strength gain (11,050 kPa).	The time-dependent nature of pozzolanic strength development should be accounted for in M-E design.
Evans (1998)	Cured strength	UCCS	Soils from two test projects in the highly plastic Queensland black clays were lime stabilized. Natural soils with PI's of near 40 were treated with 8% hydrated lime to reduce PI's to below 8 while increasing the 28-day strengths from 0.1 MPa to over 1.4 MPa. Long-term compressive strengths from cores 26-weeks old demonstrated UCCS's of over 4.5 MPa. As in previous data, the strength gain between 28-days and 26-weeks was very substantial demonstrating the long-term pozzolanic process.	Same as above.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Dunlop (1996)	Shear strength (UCCS)	New Zealand Methods	Lime is shown to react vigorously with many New Zealand clay soils where both lime and portland cement are widely used for stabilization depending on the classification of the soil. UCCS's produce strength increases of over 350 kPa (over that of the natural soil) after 14-days of cure at 20º C. Ultimate UCCS's of over 3,500 kPa after 14-days of curing at 20º C were measured. These levels of UCCS are associated with secant moduli ranging from 200 to 800 MPa.	New Zealand uses a M-E approach to pavement design containing stabilized layers. The lime or cement stabilized layer is evaluated in the design to insure that tensile stresses within the layer are not high enough to promote flexural fatigue and associated strength loss and that subgrade compressive strains are maintained within acceptable levels.
Holt and Freer-Hewish (1996)	Shear strength	UCCS (British Test Methods)	Research investigated the effect of the mellowing period on ultimate strength with shorter mellowing periods (12-hour v. 24-hours or longer) recommended. Appreciable strength gains due to lime stabilization were recorded demonstrating the potential for structural application.	Strength properties must be accounted for in M-E design.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
CTL/Thompson, Inc. (1998)	Shear strength	UCCS (ASTM D 5102)	CTL/Thompson, Inc. has developed a flexible pavement design protocol for Denver, Colorado, area soils. Lime is specified when swelling clay subgrades are encountered. The protocol in these soils is to moisture treat to a depth of 600 mm and to stabilize the top 200 mm with lime. The lime stabilized layer must achieve a UCCS of 1,120 kPa after 7-day 38º C cure when compacted at 95% of AASHTO T-99. This is a significant strength at this level of compaction. These strength requirements are routinely met in the Denver area. The swell must be below 1% (typically over 15% for the natural claystone) and the PI must be below 10%.	The CTL/Thompson approach is a good example of a locally developed structural specification for lime-stabilization.
Uddin et al. (1997)	Shear strength	Triaxial Testing	Internal friction and cohesive strengths were measured for lime contents ranging from 2.5% to 15% and for curing periods of up to two months. Substantial improvements in both parameters were demonstrated.	Strength and resilient material properties must be accounted for in M-E design.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little (1994) and Little (1998)	Shear strength	Texas Triaxial method	Little (1998) evaluated a large number of marginal Texas aggregates including caliche, gravel, iron ore and glauconite. 1% to 3% hydrated lime was added and significantly improved the Texas Triaxial strength. Typical strength increases were 50% to 150% of the natural material following capillary soak according to the Texas method (TEX 117-E)	Strength properties must be accounted for in M-E design.
Thompson (1966)	Shear strength	Triaxial testing	Major effect of lime is to produce a substantial increase in cohesive strength and a minor increase in internal friction. Shear strength increase can be substantial even in uncured lime-soil mixtures.	Improved shear strength renders the lime stabilized layer more resistant to shear failure and to accumulated damage or rutting.
Miller et al. (1970), Moore et al. (1971), Thompson (1966) and Tulloch et al., (1970) Thompson (1969)	Tensile strength	Indirect Tensile Strength and Flexural Tensile Strength Tests	Indirect tensile strength is typically about 0.13 of the UCCS (Thompson, 1966). A reasonable approximation of the flexural tensile strength is about 0.25 of UCCS (Thompson, 1969). Since lime can substantially improve UCCS in cured mixtures, it can substantially improve tensile strengths.	Tensile strength of lime-soil mixtures must be accounted for when evaluating shrinkage cracking through environmental effects model and when assessing fatigue damage potential.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Thompson (1969) Moore et al. (1971) Little (1995, 1996, 1997)	California Bearing Ratio (CBR)	CBR test on uncured and cured lime-soil mixtures	Thompson demonstrated that lime treatment of fine-grained Illinois soils produces increased CBR irrespective of the length of cure and lime-reactivity of the soil. Thompson also demonstrated the substantial improvement due to long-term curing in reactive soils. Moore et al. (1971) and Little (1995) have substantiated similar results for southeastern US and southwestern US and western US soils.	Time effects on strength gain and material properties of lime-soil mixtures must be considered in M-E design.
Eades et al. (1963)	Shear strength	CBR	Three different Virginia soils were evaluated (micaceous schist, plastic clay and weathered granite). Each demonstrated cured strength increases in soaked CBR of from less than 5% to near 100%. X-ray diffraction (XRD), and scanning electron microscopy (SEM) verified the presence of pozzolanic material responsible for the strength gain.	The durability of strength over time of materials used in M-E design should be established.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Puatti (1998)	Shear strength	CBR	Lime is used in Normandy, the Czech Republic, Poland and other locations in France as a capping layer. The natural clay silt soil is modified to depths of from 300 mm to one meter to modify and dry the soil. Soaked CBR's are increased through the addition of between 3% and 4% hydrated lime, from between 1% and 5%, and from 15% and 20%. This is considered to be an extremely effective and necessary process to provide a working platform to construct a quality pavement.	The use of lime to produce a less moisture sensitive and more stable working platform which allows construction of superior unbound aggregate base and surface HMA layers, should be considered in the in situ materials characterization of pavement layers for M-E analysis.
Perry et al. (1996)	Shear strength	British Methods	Low percentages of quicklime (up to 2.5%) were used as capping layers similar to the work documented by Puatti in Normandy, the Czech Republic and Poland. When added to wet soils (approximately 35% to 40% moisture in a clayey silt) 2.5% quicklime effectively dried the soil and increased the soaked CBR from 1.5% to 30% after 3-days of dry cure followed by 25-days of wet cure.	Same as above.
Little (1995)	Shear strength	R-value on uncured and cured lime-soil mixtures. Samples were subjected to capillary soak prior to testing.	Little (1995) tested 30 western US soils (TX, AZ, CA, UT, CO and ID) for R-value. He found substantial improvement using lime even in uncured situations. R-values of in excess of 90% were easily achieved in all soils tested.	Resistance to moisture induced damage must be considered in M-E design by means of environmental effects model.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little (1994)	Shear strength (Triaxial Testing)	AASHTO T-274 protocol (amended)	Little (1994) measured the resilient and permanent deformation properties of three Denver, Colorado, area soils which ranged form plastic (PI > 30) to moderately plastic (PI = 23). Lime-stabilization dramatically reduced moisture sensitivity, improved strength and resilient properties and reduced accumulated permanent strains during repeated high stress loading from between 0.8% and 4.0% for untreated soils to between 0.2% and 0.4% for the same soils treated with lime.	Accumulated damage must be accounted for in a M-E design approach.

Table A3. Summary of strength data derived through lime stabilization (determined in the field).

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little (1997) - Swift Transportation, Arizona	Dynamic Cone Penetrometer (DCP) correlated with in situ CBR	Procedure of the U.S. Army Corps of Engineers (USACOE)	DCP testing in lime stabilized plastic clay demonstrated strength increases of 450% to 1,300% after one-year of service. In situ CBR's of between 80% and 225% were measured.	Field verification of material properties including strength and modulus is important in M-E analysis.
Aufmuth (1970)	In situ shear strength CBR	In situ CBR	Aufmuth evaluated untreated and lime treated subgrade soils based on in situ CBR testing. Subgrades were tested in Arkansas, Texas, Virginia and Minnesota. Soils were tested in a wet season deemed to represent a critical period. Pavements ranged in age form 3 to 17 years with the average pavement about 9 years old. The lime-treated subgrade CBR averaged 65% while the untreated controls averaged 10%.	Long-term strength verification (durability) is important to establish in order to establish reliable material properties in M-E analysis.
Little et al. (1994), Texas Department of Transportation	DCP correlated with in situ CBR	USACOE	DCP testing on lime stabilized subgrades in the Bryan and Ft. Worth Districts compared estimated (from DCP) in situ CBR's of lime stabilized and natural subgrades after from 1 to 20 years of service (most between 5 and 16 years). Typical strength increases due to stabilization were in the range of 800% to 1,500%. Most estimated in situ CBR's were over 100%.	Same as above.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little (1995) - Sonora, Texas	DCP correlated with in situ CBR	USACOE	In situ CBR’s were measured in 1995 on a caliche base material and the same caliche reclaimed in 1984 with 4% hydrated lime. Both layers had achieved moisture equilibrium under a runway pavement. The stabilized caliche (1994) had an average CBR of 110% (30 measurements) while the natural caliche had an average CBR (approximately 30 measurements) of 20%. The reclaimed caliche had been in service for 10 years at the time of testing. The section was again reclaimed in 1995 with 3% lime and 3% Class F fly ash. The one year UCCS on this material was approximately 5.6 MPa.	In situ strength properties should be established in order to verify and validate laboratory properties.
Little (1997) - Mobile Modular	DCP correlated with in situ CBR	USACOE	Lime stabilized subgrade in a wet state (300 mm) in a silty clay soil in the Houston, Texas, area provided in situ CBR’s of over 100% while the in situ CBR of the untreated clay in a similar wet state was less than 10%.	Same as above.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
I-20	UCCS	Measured from field cores	Field cores from the lime stabilized subgrade demonstrated a continued strength gain for a period of 12 years.	Long-term strength properties should be established for lime-stabilized layers to insure confidence in assigning properties of structural significance to these layers.
Evans (1998)	UCCS and in situ CBR	Measured from field cores	The Eight Mile and Killarney to Freestone test roads in the plastic black clays of Eastern Queensland demonstrated CBR's of over 100% compared to approximately 10% in the natural, untreated soils.	Such measurements help verify laboratory testing.
Biczysko (1996)	Modulus of subgrade reaction	Plate loading tests	Plate loading tests on lime treated sections after demonstrated continuous strength gains over a 10-year period. The modulus of subgrade reaction at the end of six years averaged over 300 $kN/m^2/mm$ for the stabilized sections compared to 85 $kN/m^2/mm$ for the untreated sections.	Same as above.

Source of Information	Strength Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Hopkins et al. (1996) - Alexandria-Ashland	CBR and UCCS	In situ	In situ CBR's on this test section were measured in the CL to CH soil on sections that were left uncovered for the first winter after construction. The average CBR was 37% (range of 19 to 61%). This is compared to an average CBR of approximately 10% for the untreated soil. A companion UCCS study on the section before and after the winter period demonstrated substantial strength gains even during the winter period. Typical UCCS strength increases were from 630 kPa (average) prior to the winter period to approximately 1,050 kPa (average) following the winter period.	Such field testing helps to verify the effects of severe environmental effects.
Hopkins et al. (1996) - KY 11	CBR	In situ	In situ CBR values were charted for a seven year period on the untreated and lime treated soils of KY 11. The lime stabilized soils demonstrated a very substantial strength improvement with time and dramatically out-performed the natural (untreated) soils in the category of strength development and retention.	Approximations of undrained shear strength and design modulus were made based on these data. A 90-percentile field design strength of 333 kPa for lime treated subgrades was established with a design modulus based on the CBR of - $E_s = 17{,}914\ CBR^{0.874}$

Table A4. Summary of lime-soil stabilization mechanisms and the improvement derived from the stabilization mechanism.

Mechanism	*Benefit Derived*
Cation Exchange or Molecular Crowding - The adsorption of calcium cations (provided by the lime) at the surface of the clay particles in lieu of the commonly present cation in the natural clay. Some researchers believe that the calcium hydroxide molecule is adsorbed to the clay surface in lieu of the calcium cation (molecular crowding).	The saturation of calcium or of the calcium hydroxide molecule at the clay surface dramatically reduces the energy with which the clay surface attracts and holds water. As a result, the physical changes in the lime-soil system are: • Flocculation and agglomeration of particles into a larger effective particle size • Reduction in plasticity and swell potential • A drying effect as the water-holding potential of the soil is reduced • Improved work ability and compactability • Improved shear strength as a result of the cation or molecular crowding
Pozzolanic Reaction - The reaction between the calcium in the lime and the silica and alumina released from the soil (primarily the clay) in the lime stabilization process. When the required amount of lime is added to a soil, the pH increases to approximately 12.4 (at 25°C). At this pH, the soil alumina and soil silica (pozzolans) become soluble and can combine with free calcium and water to form cementitious products (calcium-silicate-hydrates or CSH) and calcium-aluminate-hydrates or CAH)	The high pH at the soil or clay surface allows the lime to actually solubilize silicates and aluminates from the clay surface. This "attack" alters the mineralogy of the clay causing a reduction in plasticity and an increase in strength which continues for an extended period of time - up to many years. As a result of this reaction, the following physical benefits are derived: • Further reduction in plasticity and swell potential as the time-dependent pozzolanic reaction proceeds • Further increase in effective particle size and improved work ability as the pozzolanic reaction proceeds • Substantial increase in shear strength and stiffness as well as a substantial improvement in resilient properties as the pozzolanic reaction proceeds

Mechanism	*Benefit Derived*
Carbonation Reaction - The reversion of calcium to calcium carbonate as free lime reacts with carbon dioxide from the atmosphere. This reaction is considered deleterious in the construction phase of the stabilization process as it depletes the system of free calcium needed in the stabilization process. Therefore, steps are taken to minimize carbonation in the construction process. However, work by Graves (1987) and Little et al. (1996) has demonstrated the benefits of the carbonation reaction over the long-term in the stabilization of calcareous aggregates.	In calcareous material, lime has been shown by Grave (1987) and Little et al. (1995) to enhance the growth of carbonate cement which bonds carbonate particles together resulting in a substantial shear strength and a substantial stiffness increase.

Table A5. Summary of laboratory derived stress-strain (stiffness) and resilient properties of lime soil mixtures.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Neubaur and Thompson (1972) Thompson (1966)	Stress-strain characteristics, modulus	Stress-strain plot of uncured lime-soil mixtures	Immediate (uncured) effects of lime treatment are apparent in stress-strain relationships which coincide with the strength improvements on uncured mixtures due to immediate lime-modification reactions. Very substantial stiffness or modulus increases are encountered.	Elastic properties and stress-strain constitutive relationships are a key element in M-E analysis.
Thompson (1966) Thompson (1969)	Stress-strain characteristics, modulus	Stress-strain plot of cured lime-soil mixtures	Thompson developed a generalized stress-strain plot for lime stabilized soils. The lime stabilization substantially stiffens the soil (modulus increase of 10-fold or better) and reduces the failure strain from around 2 or 3% to 1% or less. Thompson used UCCS and stress-strain data to approximate the elastic modulus from UCCS as: E(ksi) = 9.98 + 0.124 (UCCS, psi)	Stress-strain relationships are a key part of material characterization in M-E analysis.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little (1996) Little et al. (1994)	Resilient modulus under various stress states	AASHTO T-294-94	Little (1996) evaluated nine Colorado soils and six Texas soils and found the lime soil mixtures cured for 5-days at 38º C to be stress sensitive but to a much lesser degrees than the natural (untreated) soils. The resilient modulus was also much less moisture sensitive after lime treatment. Typical resilient modulus increases due to lime treatment were in the range of 800% to 1,500%. Resilient moduli in the range of 210 to 400 MPa were readily achieved. Little et al. (1994) used low percentages of lime (1 to 2%) to improve strength and resilient properties of calcareous aggregate bases. The lime enhanced both strength and stiffness by approximately 70% to 125%. Typical resilient modulus increases were from a range of 48 MPa for untreated caliche bases to 414 MPa at 7 kPa confinement and at a deviatoric level of 200 kPa and from 138 MPa to 862 MPa for a higher confining pressure of 140 kPa.	Resilient properties of lime-stabilized soils and aggregates are the basis of materials characterization in M-E analysis.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Puppala et al. (1996)	Resilient modulus	AASHTO T-294-94	Puppala et al. investigated the effects of compaction and confining pressure on lime treated and untreated soils. After a short 3-day curing period, resilient modulus increases of 30% to 50% (due to lime stabilization) were measured.	Same as above.
Uddin et al. (1996)	Stress-strain characteristics	Model of yield	Uddin et al. proposed a model of volumetric yield that consists of an initial pseudo-elastic phase, a work-hardening phase and a strain-softening phase.	This model may prove beneficial in modeling the deformation behavior of lime-soil mixtures in mechanistic models.
Thompson and Elliot (1985)	Resilient modulus	Triaxial testing	Thompson and Elliot characterized the non-linear resilient modulus relationship for fine-grained soils and divided fine-grained soils into categories of very soft, soft, medium and stiff based on resilient modulus properties. These data provide an excellent basis for comparison of the effects of lime-stabilization.	This model can be effectively used to account for stress sensitivity in fine-grained soils in a M-E analysis.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Robnett and Thompson (1979)	Resilient modulus	Resilient modulus as a function of stress state, moisture content, and levels of freeze-thaw damage	Robnett and Thompson compared the effects of lime to the natural soil resilient properties on over 50 Midwestern US soils. They found that lime stabilization substantially increased the resilient modulus and substantially improved freeze-thaw damage resistance potential of the soils evaluated, even a non-reactive (with lime) silty soil (Tama B). As an example, the Tama B soil (untreated) had a resilient modulus of approximately 25 MPa after 10 freeze-thaw cycles, while its lime treated counterpart had a resilient modulus of approximately 108 MPa following identical conditioning.	This study illustrates that damage can occur in spring break-up and the fact that stabilization can ameliorate the effects of this freeze-thaw damage.

Table A6. Summary of field deflection and resilient modulus data.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Maxwell and Joseph (1967)	Elastic modulus	Vibratory testing	Used field vibratory testing on clay-gravel subgrade. Elastic moduli calculated from this process were in the range of 1,150 MPa following construction to in excess of 3,900 MPa approximately 2 to 2.5 years after construction. Similar values computed for a lime stabilized subbase were 1,300 MPa after construction and over 7,000 MPa 2 to 2.5 years following construction.	In situ resilient properties are necessary for reliable M-E modeling.
Little (1990) - Mesa, AZ	Resilient modulus	Back calculated from Falling Weight Deflectometer (FWD)	A decomposed granite was stabilized with 1% hydrated lime. The result was a 500% to 1,200% increase in back calculated resilient modulus over that of the untreated material.	This provides an example of the level of modulus upgrade in base course materials that can be achieved through lime-stabilization.
Little (1997) - Scottsdale , AZ	Resilient modulus	Back calculated from FWD	In situ DCP and back calculated resilient moduli were determined on Arizona subgrade soils. The moduli were consistent with DCP data. Lime-stabilization improved both strength and modulus by a factor of approximately 500% to 700%.	The ability to achieve in situ strength and modulus values is critical to a M-E analysis.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little et al. (1994) - TxDOT Pavements	Resilient modulus	Back calculated from FWD	Thirty-seven pavement sections including lime stabilized subgrades and subbases were evaluated throughout Texas. The back calculated resilient moduli from FWD deflection measurements were calculated using the MODULUS deflection basin matching program. The back calculated modulus of the lime stabilized layer, E_{LSS} , was compared to the back calculated modulus of the subgrade stabilized, E_{SUB}. The analysis showed substantial improvement in the modulus of the stabilized layer when compared to the same, untreated soil or base. Modular ratios (E_{LSS}/E_{SUB}) averaged 7.5 with a standard deviation of 1.2. The average back calculated E_{LSS} was 620 MPa. The average E_{SUB} was 80 MPa.	This information provides a data base of expected, back calculated, field values of resilient modulus for lime stabilized layers.
Syed (1998) - Bryan District, TxDOT	Resilient modulus	Back calculated from FWD	Syed (1998) evaluated 16 pavements in the Bryan District of the Texas Department of Transportation. Each pavement consisted of an aggregate base course reclaimed with lime and combined with natural subgrade. The back calculated moduli of the reclaimed base averaged 4,190 MPa with a standard deviation of 1,250 MPa. The average back calculated modulus of the aggregate base prior to reclamation was 1,202 MPa with a standard deviation of 865 MPa.	This information provides a data base of expected, back calculated, field values of resilient modulus for lime stabilized, reclaimed aggregate bases.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little et al. (1994) - Caliche bases in south Texas	Resilient modulus	Back calculated from FWD	Hydrated lime is often added to caliche and lower quality limestone bases in south Texas at the rate of between 1% and 2% by weight. Lime considerably improves the shear strength (Table 2) and resilient modulus (laboratory - Table 4). Back calculated resilient moduli from FWD deflection data verify the effect of the 1% to 2% hydrated lime in improving in situ resilient moduli. Back calculated resilient moduli from eight pavement sections in south Texas averaged 1,544 MPa with a standard deviation of 325 MPa. Back calculated resilient moduli on two control sections with unstabilized caliche were 207 MPa and 27 MPa, respectively.	These data establish the level of effect that hydrated lime can have on the improvement of resilient modulus in aggregate bases. Lime-stabilization of this type may prove very valuable in reclamation and/or recycling efforts.
Evans (1998)	Resilient modulus	Back calculated from FWD	Test roads constructed with hydrated lime on the Killarney and Eight Mile to Freestone sections consisted of 200 mm of lime stabilized black clay subgrade in Queensland, Australia. Back calculated resilient moduli of the natural subgrade averaged 90 MPa while that of the lime stabilized (9%) black clay averaged approximately 800 MPa, and the average back calculated modulus of the unbound aggregate base overlying the stabilized subgrade was approximately 310 MPa. The roadways have been in service for approximately two years.	Value of in situ resilient moduli provide an informative data base for selecting realistic modulus values for mechanistic analysis and design.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Bicysko (1996)	Elastic modulus	Derived from DCP testing	Biczysko evaluated two projects in the Northamptonshire area. The projects were constructed in 1980 and 1982. DCP testing was performed and the results used to approximate elastic moduli on the 14 and 16 year old pavements. Back calculated moduli were: 480 MPa for the granular base resting on top of the lime stabilized soil, 241 MPa for the upper layer of lime stabilized soil, 158 MPa for the lower layer of lime stabilized soil and 55 MPa for the natural subgrade.	This type of information is important in mechanistic analysis as it provides verification of the level of modulus change offered through lime stabilization. The assignment of an expected modulus is a critical step in mechanistic analysis.
Texas GPS Sites	Resilient modulus	Back calculated from FWD deflections	FWD deflection data on twelve Texas GPS-1 sections were analyzed using MODULUS to determine back calculated resilient moduli. Back calculated moduli of the lime stabilized subgrade exceeded those of the natural subgrade in 9 of the 12 cases. The average E_{LSS}/E_{SUB} ratio was 2.9. The average E_{LSS} was 604 MPa while the average E_{SUB} was 208 MPa.	Lime-stabilized layers demonstrated a significant structural contribution in 9 of 12 pavements.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Texas GPS Sites (Little and Kim, 1998)	Deflection data	Evaluation of FWD deflection data to evaluate composite pavements.	Little and Kim (1998) compared FWD data for 14 Texas pavements with aggregate bases over natural subgrades against 12 Texas pavements with aggregate bases over lime-stabilized subgrades. The pavements were evaluated based on a composite thickness with a reference modulus of 2,800 MPa. The pavements with the stabilized subgrades had a significantly higher composite thickness than those without.	Lime-stabilized layers demonstrate a significant structural contribution.
Mississippi Study of Lime and Lime Fly Ash Pavements (1998)	Resilient modulus	Back calculated from FWD	Data in this study are under evaluation. However, preliminary evaluation shows a significant structural effect of lime-stabilized subgrades in 8 of 9 cases investigated.	These data are important in demonstrating structural effectiveness of lime stabilization in various climatic and geographical regions of the state.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Koshla et al. (1996)	Resilient modulus and performance	Back calculated from FWD data performance evaluations	Between 1986 and 1996, the North Carolina Department of Transportation (NCDOT) conducted a comparative study of the performance of different flexible pavements on a test facility constructed on US 412 near Siler City. Based on this study the following conclusions regarding the performance of lime stabilized subgrades were drawn: 1. Back calculated moduli of lime stabilized subgrades are generally larger than those of aggregate bases 2. Composite moduli of sections with aggregate bases over lime stabilized subgrades are generally higher than composite moduli of comparable thicknesses of aggregate bases without lime stabilized subgrades 3. Subgrade stabilization generally enhanced the performance of the overlying aggregate base. 4. Lime subgrade stabilization is enhances performance under full depth asphalt pavements as well as under conventional pavements. 5. Lime stabilization seems to be an important factor for reducing distress in flexible pavements.	Such studies provide field data on resilient moduli that help establish realistic design parameters in the M-E approach.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
MacDonald (1969)	Surface deflection	Benkleman Beam	MacDonald recorded pavement deflections in South Dakota on five different pavement types (secondary roads with low traffic, primary roads with medium traffic, IH-90, a thin mat HMA and a seal coat pavement). In each pavement a lime treated base was compared to an untreated base. The untreated base in each case was significantly thicker than the lime treated base, yet surface deflections following several years of service were significantly smaller in the pavements with the lime treated base. Furthermore, annual maintenance costs were significantly lower for the pavements with lime treated bases.	Such studies provide a data base of structural performance that can be used to establish reasonable M-E design properties.
Lund and Ramsey (1959)	Surface deflection	Benkleman Beam	This study near Tecumseh, Nebraska, compared surface deflections on pavements with and without lime stabilized subgrades and with and without lime stabilized bases. Thickness adjustments were made to evaluate the effect of stabilized layer thickness. As in the South Dakota study, the presence of a lime stabilized subgrade or a lime stabilized base resulted in significantly lower deflections compared to the control section without a stabilized layer. The study continued for a period of approximately 3.5 years.	Same as above.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Perry et al. (1996)	Effect of moisture on subgrade modulus	Plate bearing test	Perry investigated the effect of moisture on the subgrade modulus of London clay. 2.5% quicklime very substantially increased the subgrade modulus over the natural soil over the entire range of moisture contents at the time of compaction.	Same as above.

Table A7. Summary of data relating to fracture and fatigue properties of lime-soil mixtures.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little (1998)	Fracture tensile strength	Indirect tensile (IDT) strength	Little measured the indirect tensile strengths (7-day, 38 º C) of lime-soil mixtures for nine Colorado soils, Arizona soils, four California soils, four Texas soils and two Utah soils. The IDT strengths were measured over a range of molding moisture contents of from ±2% of optimum for compaction. The results demonstrated that a significantly reduced sensitivity to molding moisture and a 400 to 1,500% tensile strength increase due to stabilization. Each sample was subjected to 24-hours of capillary soak prior to testing.	The tensile strength of stabilized soils or aggregates is often related to fatigue performance through the stress ratio concept. If the tensile stress induced within the stabilized layer does not exceed a certain percentage of the tensile strength, then the fatigue life can be approximated. The effect of lime in tensile strength increase is an important design consideration.

Source of Information	Parameter Measured	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Swanson and Thompson (1967)	Flexural fatigue	Beam fatigue testing	Swanson and Thompson performed flexural beam fatigue testing on lime-soil mixtures. They found that the fatigue results are very similar to those of lime-fly ash and aggregate mixtures and for portland cement concrete. The fatigue strengths at 5-million stress applications varied from 41 to 66% of the ultimate flexural fatigue strength with an average of 54%.	Tensile strength of lime-stabilized soils and aggregates is an important material property in M-E fatigue and fracture analysis.
Moore and Kennedy (1971)	IDT fatigue	Indirect tensile test	Moore and Kennedy found that a repeatedly loaded IDT test can be effectively used in fatigue testing. They found that as curing of the lime-soil mixture continues, the stress ratio decreases and the fatigue life increases.	

Table A8. Summary of properties of lime-soil mixtures relating to durability.

Source of Information	Parameter Evaluated	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Thompson (1970) Dumbleton (1962)	Durability	Exposure to water	Prolonged exposure of lime-treated soils to water produces only slightly detrimental effects and the ratio of soaked to unsoaked UCCS is high at approximately 0.7 to 0.85. Lime stabilized soils seldom reach above about 90% saturation.	Moisture effects must be accounted for in M-E pavement layer characterization. Data such as these provide important knowledge and insight.
Dempsey and Thompson (1968)	Durability	Freeze-thaw induced volume change and strength loss	Average rates of strength decrease for typical lime-soil mixtures were 60 kPa per cycle and 120 kPa per cycle for 48-hour and 96-hour (48.9º C) curing, respectively.	These data provide a guide for evaluating the effects of freeze-thaw cycles induced within the lime-soil pavement layer. Dempsey and Thompson (1968) recommend a minimum UCCS prior to the first winter of freeze-thaw activity in order to withstand the damage induced within that season.

Source of Information	Parameter Evaluated	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Thompson and Dempsey (1969)	Durability	Autogenous healing	Lime-soil mixtures possess the ability to heal during periods conducive to pozzolanic activity. If a lime-soil mixture can achieve a substantial strength so that it can withstand the damage induced within the first winter following construction, and if the lime-soil mixture is properly designed with adequate lime for long-term pozzolanic reactivity, damage encountered within the first winter can be recovered due to subsequent pozzolanic reactivity. Thompson and Dempsey (1969) present data for Illinois soils demonstrating 350 kPa strength loss due to freeze-thaw activity followed by a 1,400 kPa strength gain in a period of no freeze-thaw and temperatures to accommodate additional pozzolanic activity.	Long-term strength gain (as influenced by autogenous healing) should be considered and accounted for in M-E analysis. The strength characterization versus time should be incorporated in the characterization.
McDonald (1969)	Durability	Autogenous healing	McDonald (1969) presented field data verifying autogenous healing.	Same as stated above.
Roads and Streets (1975) Gutschick (1975, 1985)	Durability	Visual	The base and sides of the Friant-Kern Canal in California were lime-stabilized. The Canal has functioned very well for over 25 years in the most trying of circumstances. The canal is subjected to periods of high flow and periods of dryness and dessication. The canal maintains slope integrity and resistance to erosion.	Durability of lime-stabilized layers should be established to provide confidence in design reliability.

Source of Information	Parameter Evaluated	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Little (1995)	Durability	Dielectric value and strength testing	Little demonstrated the effect of lime in improving Texas Triaxial shear strength and in reducing moisture susceptibility of 9 Texas aggregates.	Same as above.
Robnett and Thompson (1976)	Durability	Resistance to freeze-thaw damage	Robnett and Thompson demonstrated the effect of lime in maintaining strength and resilient modulus through several freeze-thaw cycles in two soils: a lime-reactive soil and non-reactive soil.	Same as above.

Source of Information	Parameter Evaluated	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
Kelley (1977)	Strength and performance	Compressive strength on cores and field observations	In 1977 Kelley investigated the performance of lime stabilization on five military bases in the United States. The following conclusions were drawn for the sections tested in 1977 (construction dates of lime-stabilized layers are shown in parentheses): 1. Fort Polk, Louisiana (1951) - lime-cement stabilization increased compressive strength to within the range of lean concrete (approximately 12.6 Mpa. 2. Fort Chaffe, Arkansas (1949) - Lime stabilized base reached a strength of approximately 12.8 MPa. 3. Fort Sam Houston, Texas (1953) - Excellent performance under heavy traffic for 24 years. 4. Fort Sill, Oklahoma (1949) - Excellent performance under heavy truck and tank traffic for 28 year period. Low level of maintenance required. 5. Fort Hood, Texas (1953) - Excellent performance over 24 years under heavy truck traffic. 6. General - UCCS's of lime stabilized layers on these sites often approached that of lean concrete. About 2/3 of the amount of lime used could have produced adequate strength.	Same as above.

Source of Information	Parameter Evaluated	How Measured	Results and Practical Impact on Pavement Performance	Impact on Mechanistic Design
McCallum and Petry (1990)	Chemical and physical properties of lime-stabilized soils after leaching with various water-salt solutions	Various methods	This experiment focused on Texas clay soils and demonstrated that using too little lime can result in loss of stabilization effects. Stabilization effects were often reversible when too little lime was used. However, when enough lime was used to produce optimum property changes, the stabilization effects were generally resistant to the effects of leaching.	Same as above.

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