EXAMPLE PROBLEM ILLUSTRATING THE APPLICATION OF THE NATIONAL LIME ASSOCIATION MIXTURE DESIGN AND TESTING PROTOCOL (MDTP) TO ASCERTAIN ENGINEERING PROPERTIES OF LIME-TREATED SUBGRADES FOR MECHANISTIC PAVEMENT DESIGN/ANALYSIS

By: Dallas N. Little and F. A. M. Shafee Yusuf

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This document presents an example of mechanistic design and analysis using a mixture design and testing protocol (*Reference 1*) to address structural properties of lime-treated subgrade, subbase, and base layers

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PURPOSE

This document presents an example of mechanistic design and analysis using a mix design and testing protocol (*Reference 1*). More specifically, it addresses the structural properties of lime-treated subgrade, subbase, and base layers through mechanistic design.

INTRODUCTON

This design example is based on three Mississippi Department of Transportation (MDOT) widening projects on state highways in Kemper, Lowndes, and Washington counties. In all three, the lime-treated subgrade (LTS) layers are being evaluated as structural components in the design. The design options considered are

- using hot mix asphalt (HMA) directly over the LTS and
- using HMA with a lime-fly ash stabilized base over the LTS.

Both options have previously been used successfully in other parts of the State.

In fact, MDOT has used lime stabilization in clay soils for over 30 years to improve performance and as a platform for pavement structures. A large percentage of Mississippi's land area contains soils that respond well to lime stabilization—as evidenced by improved engineering properties and long-term performance as roadway sublayers.

This example presents three elements of the mixture design and testing protocol (MDTP):

- Laboratory Testing,
- Field Testing, and
- Mechanistic Analysis.

First, <u>laboratory testing</u> of soils is conducted using the procedures described in *Reference 1*. The soil samples evaluated were determined to be representative of the subgrade soils to be stabilized based on county soil reports and boring samples.

Also presented is <u>field testing</u> of existing pavements with LTS layers in the same soil series as those in the widening project. The engineering properties of the newly lime-treated soils are compared to in situ properties of lime stabilized soils that have been in service for 15 to 20 years. The field pavements selected for this comparison were

- US highway 45N (17 years of service) in Kemper County,
- US highway 82W (20 years of service) in Lowndes County,
- US highways 61N (15 years of service) in Washington County, and
- 82E (20 years of service) in Washington County.

In addition, the structural effectiveness of the LTS layer is analyzed by a mechanistic analysis. The properties of the LTS layer (as determined by in situ testing) are used in a layered elastic model to calculate maximum tensile flexural stress in the HMA and compressive strain at the top of the subgrade. These values are then placed in the appropriate transfer function to evaluate the structural effectiveness of the LTS layer. Specifically, the projected lives of the pavements with and without the LTS layer are compared, in terms of equivalent single axle loads (ESALs).

Finally, although not part of the MDTP, the quality of the pozzolanic reaction in the laboratory-cured LTS samples is confirmed by x-ray diffraction (XRD) and scanning electron microscopy (SEM)—see Appendix.

OUTLINE OF MIXTURE DESIGN AND TESTING PROTOCOL (MDTP)

The MTDP (*1*,*2*) is designed to produce a mixture that possesses the desired structural properties and durability in a pavement layer. The procedure measures engineering properties that are critical to the performance of the lime-stabilized mixture as a structural layer in a pavement system. The MDTP is comprised of the following steps:

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

Determine whether the soil has at least 25% passing the 75-μm (micron) sieve and has a plasticity index (PI) of at least 10 (*3*). The reactivity of lime with soil is predicated on the type and the amount of clay minerals present in the soil. Soils with organic contents exceeding 1% by weight are difficult to stabilize or may require uneconomical quantities of lime in stabilization. The screening criteria also limit soluble sulfates to less than 0.3% by weight in a 10:1 water-to-soil solution (*4*). High sulfate concentrations can cause deleterious reactions among lime, soil minerals, sulfate ions, and water. This can lead to loss of stability and heave.

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

Perform the Eades and Grim pH test (ASTM D 6276) to determine lime demand. This test identifies the lime content required to satisfy immediate lime-soil reactions and still provide significant residual calcium and a high system pH (about 12.4 at 25^0C). This is necessary to provide proper conditions for the long-term pozzolanic reaction that is responsible for strength and stiffness development.

Step 3: Moisture/Density Relationship for Lime-Treated Soil, and 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; ASTM D 698, D 1557; Texas Method 113A, etc. A modified compaction effort (or some reasonable percentage thereof, e.g., 95% of AASHTO T-180) is recommended for fabricating the samples. This level of compaction is usually achievable with conventional field equipment.

Samples are prepared for strength testing and moisture sensitivity testing at optimum moisture content with a tolerance of ±1%. All samples are cured for 7 days at 40^0C in sealed plastic bags (in order to retain sufficient moisture during curing). Previous studies have demonstrated that this period of accelerated cure promotes a strength that is representative of a long-term cure [13]. Furthermore, 40^0C represents high temperatures that can be experienced in the field.

Following curing, the samples are subjected to capillary soak for 24-hours (depending on soil plasticity) prior to strength testing. The capillary soak protocol consists of placing the sample wrapped in a wet absorptive fabric on a porous stone in water. The water level reaches the top of the porous stone, so that both the porous stone and the fabric wrap are in contact with water throughout the capillary soak process. Extensive laboratory testing has demonstrated that untreated clayey soils will typically degrade to a compressive strength of less than 70 kPa (about 10 psi) following capillary soak (*1,2*). Hence the capillary soak moisture-conditioning phase is considered an effective method of assessing moisture damage potential.

Step 4: Compressive Strength Testing to Determine the Unconfined Compressive Strength (UCS) of Lime-Soil Mixture

Unconfined compressive strength is determined using ASTM D 5102. UCS can be used to approximate design parameters such as flexural strength, deformation potential and stiffness (resilient modulus) when these data are not available:

- Tensile strength can be conservatively estimated as 10 percent of the UCS, and flexural strength can be conservatively estimated to be twice the tensile strength or approximately 20 percent of UCS. (*2*)
- Several correlations have been developed between UCS and resilient modulus. One of the most conservative for lime-stabilized fine-granular subgrades was developed by Thompson (*5*) in which the resilient modulus $E_R = 0.124$ (UCS) + 9.98, where UCS is presented in psi and E_R in ksi. Comparisons in this example have shown this relationship to be overly conservative by approximately 50 percent.

Step 5: Resilient Modulus Test to Determine the Resilient Properties

Resilient properties define the ability of the lime soil mixture to distribute load/pressure developed under heavy wheel loads so that those wheel loads will not over-stress the weaker pavement layers. Resilient modulus is determined using AASHTO T 294-94 or a rapid triaxial tester (RaTT) (which can be used instead of the more time-consuming and material-intensive AASHTO T 294-94 test) (*1*).

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

The dielectric value (DV), measured by a Tube Suction Test (*6*), is a measure of how much moisture a base/subbase/subgrade will absorb through capillary rise and the state of bonding of the absorbed moisture. Low dielectric values indicate the presence of tightly absorbed and well-arranged water molecules. Scullion and Saarenketo (*6*) have established DV selection criteria for subbase layers. The results of this test are used to assess resistance of the stabilized material to moisture damage.

LABORATORY TESTING

All the in situ materials are oven dried for 24-hours, before they are pulverized to make samples for laboratory testing.

Eades and Grim pH Test

Dry soils are screened through a No. 40 sieve. All the soils are tested with 0, 2, 3, 4, 5, 6 and 7% of hydrated lime in accordance with ASTM D 6276. Special attention is given to maintain the room temperature at 25^0C, as pH of lime-soil mixture is temperature dependent.

The natural soil from Kemper County (US 45N) had a pH of 4.77. All other soils had pHs between 6.5 and 7.6. According to the Eades and Grim pH test results, 5%, 6%, 6%, and 4% of lime were selected as optimum lime content for the soils of pavements from US 61N, US 82E, US 82W and US 45N, respectively.

Atterberg Limits Test

Liquid limits, plastic limits and plasticity indices are determined on all unstabilized soils following ASTM D 4318. After the pH test, the Atterberg limits test is performed again on all the soils with optimum lime content. Soil-water-lime mixtures are allowed to mellow for one-hour before testing.

Atterberg limits measured on unstabilized and stabilized soils from the four pavements demonstrated the ability of lime treatment to reduce plasticity and improve workability:

- US 61N, PI was reduced from 29 to 15;
- US 82E, PI was reduced from 32 to 16;
- US 82W, PI was reduced from 28 to 9; and
- US 45N, PI was reduced from 17 to 9.

This represents a substantial PI reduction after only one-hour of mellowing. Continued plasticity reduction occurred with curing (24-hours) due to early pozzolanic reaction. Plasticity indices were reduced to 10, 5, 5, and 6, respectively.

Unconfined Compressive Strength Test

UCS test specimens are prepared for unstabilized and stabilized soils at three moisture contents that bracket the optimum moisture contents for each soil and lime-soil. For stabilized materials, lime is mixed with the dry soil at the optimum lime percentage determined by the pH test. After the soil-lime mixtures are thoroughly mixed with water, they are placed in plastic zip-lock bags for one hour. After the mellowing period, specimens are compacted according to ASTM D 1557 to produce 64-mm diameter and 127-mm high samples.

Specimens are divided into two sets for curing, each consisting of three specimens. One set of specimens is cured for 7-days at 40^0C and the other set is cured for 30-days at 25^0C. All the specimens are cured in the zip-lock bags. Samples are subjected to 24-hour capillary soak prior to compressive strength testing. An identical set of replicate samples is tested without capillary soak to evaluate the effect of moisture conditioning.

The unconfined compressive strengths for unstabilized and stabilized specimens in the dry condition and after 24-hour capillary soak are shown in Table 1. All the data show substantial strength increase due to lime stabilization. The relationships between dry and soaked compressive strength for unstabilized and stabilized soils are shown in Figure 1.

Rapid Triaxial Tester (RaTT) Resilient Modulus Test

Two sets of test specimens (dry and capillary soaked) are prepared with unstabilized soils and stabilized soils in accordance with the Feed Back Controlled RaTT Stress Stage Resilient Modulus Test (Test No: 042) that embodies the requirements of the AASHTO T 294-94 specification. The design of the RaTT hardware allows the testing of cylindrically shaped specimens of nominal 150-mm diameter and 150-mm height. After a mellowing period, specimens are molded with an automatic compactor at optimum moisture content. After the curing period of 7-days at 40^0C, all specimens are tested to measure the resilient modulus at dry condition (not subjected to moisture conditioning). One set of specimens is subjected to a 24-hour capillary soak before determination of resilient modulus. These are referred to as soaked samples. Specimens made of unstabilized soils (except US 45N) swelled and cracked significantly during capillary soak, preventing them from being able to be tested in the RaTT device.

The RaTT (Test No. 042) resilient modulus testing protocol for subgrade soils consists of a conditioning period followed by determining resilient moduli at various deviatoric stresses (ranging from 14 to 69 kPa) and for confining pressures of 41, 28 and 14 kPa, respectively. A deviatoric stress of 41 kPa is typical within the subgrade and was used to identify a single resilient modulus. The resilient moduli at 41 kPa deviatoric stresses for unstabilized and stabilized soils are summarized in Tables 1. Figure 2 compares these resilient moduli for different soils.

Tube Suction Test/Swell Test

Specimens prepared for RaTT resilient modulus test are used for tube suction testing. Specimens are dried at 40^0C for 4-days before they are placed on porous stones with a deionized water level reaching almost the top of the porous stones. During the capillary soak, a dielectric probe is used to measure the surface DV of the compacted samples. Swell potential for each sample is also monitored. Capillary soak is continued for 10-days, or until the DV achieved an ultimate or asymptotic value.

Comparisons of DV, moisture content, and swell potential for the four Mississippi untreated and lime-treated soil are shown in Figure 3. Dielectric values greater than 16 indicate the presence of substantial "free" moisture and are considered poor in terms of moisture damage potential. A DV below 10 is considered excellent (*6*). However, these criteria were developed for untreated aggregate/soil systems. The effect of higher cation concentration (electrolyte concentration) on osmotic section and correspondingly DV is not precisely known. The existing criteria are considered conservative for treated soils.

FIELD TESTING

In this study, field tests were performed to evaluate in-situ properties of MDOT pavements with LTS.

Ground Penetrating Radar (GPR)

GPR operates by transmitting short pulses of electromagnetic energy from an antenna into the pavement. These pulses are reflected back to a receiving antenna. The reflected energy is collected and displayed as a waveform. The large peaks of the waveform are the energy reflected from the surface and interfaces between layers. The time interval between peaks is the time the radar wave needs to travel from the top of the layer to the bottom and back (twice the layer thickness). The speed with which the electromagnetic radar wave travels in a particular layer is related to the dielectric constant of that layer. In pavements, the parameter that most influences the dielectric properties of materials is the moisture content (*6*).

The layer thicknesses as determined by GPR are used to validate actual layer thicknesses, which are in turn used to more accurately back-calculate layer moduli. Dielectric values from GPR data are used to assess layer moisture sensitivity of the layers and durability.

Software called COLORMAP was used to analyze the radar signals. This software measures the amplitude and time delay of each radar trace received and applies the signal processing to calculate layer dielectric constants and layer thicknesses (*7*). Layer thicknesses and dielectric constants for all the MDOT pavements are summarized in Table 2.

Dynamic Cone Penetrometer (DCP)

DCP provides a log of resistance to penetration under an impact load that has been effectively correlated to in situ California Bearing Ratio (CBR) and modulus by the U.S. Army Corps of Engineers (*8*). This process provides not only in situ strength and stiffness data but also a log of the thickness of various paving layers to be used together with GPR determined thicknesses in modulus back calculations using FWD data.

DCP tests were performed in three locations in each pavement. The average values of these tests for each pavement are presented in Table 2.

Falling Weight Deflectometer (FWD)

FWD delivers a transient force impulse to the pavement surface that simulates a moving wheel load in both magnitude and duration. By varying the amount of weight and the height of drop, different impulse forces can be generated. The deflection data obtained from the FWD testing are used for evaluation of the in-situ stiffness of individual pavement layers (*9*).

A program called MODULUS, developed at Texas A&M University (Texas Transportation Institute), was used to determine layer moduli by back-calculation method. MODULUS stores many generated deflection basins and corresponding moduli values in a database for a given layer configuration. When a measured deflection basin is analyzed, the database is screened, and interpolations are used to find a deflection basin that best represents the measured basin (*10*). Although the moduli values for all the layers of the pavements were back-calculated from the field data, only the LTS and subgrade moduli are shown in Table 2.

COMPARISON OF LABORATORY AND FIELD DATA

The following laboratory test properties to field-test properties comparisons were made:

(1) Laboratory unconfined compressive strengths (UCS) were compared to field dynamic cone penetrometer (DCP) strength measurements.

(2) Laboratory-determined resilient moduli of the lime-stabilized soils were compared to back-calculated resilient moduli based on Falling Weight Deflectometer (FWD) deflection basins.

(3) Laboratory assessment of moisture sensitivity and durability of the lime-treated soils based on the dielectric value measurements and swell tests were compared to field Ground Penetration Radar (GPR) measurements.

Comparison of Laboratory UCS with Field DCP

From Figure 1, it is apparent that soaked UCS tests for all Mississippi LTSs are substantially higher than their untreated counterparts. In fact, the stabilized-to-unstabilized strength ratios shown in Table 1 are all greater than 45. Furthermore, the value of UCS of the soaked soils cured 7-days at 40^0C are all above about 1,700 kPa (about 250 psi) which are considered by Thompson (*5*) as values acceptable for assignment of structural significance as a base layer. These values correlate well with field DCP values (Table 2). These values show that the back-calculated CBR of the LTS layers are between 12 and 33 times the strength of the in situ natural subgrade.

Comparison of Laboratory Resilient Moduli with Field FWD Back-Calculations

Figure 2 summarizes laboratory determined resilient moduli for the four Mississippi LTSs. There is a striking difference between the resilient moduli for stabilized and unstabilized soils following soaking. For US 45N, the stabilized layer is 6.7 times greater than its unstabilized counterpart. The other unstabilized samples could not be tested due to disintegration during soaking. Furthermore, the soaked resilient moduli values for the four treated soils are all above about 200 MPa or about 30,000 psi. This is the level normally correlated with structural significance, e.g., AASHTO 1986 (*11*) assigns a structural layer coefficient of 0.14 to subbases with a resilient modulus of

200 MPa. It should also be noted that the soaked resilient moduli of the LTS layers for US 61N, US 82W, and US 45N are considerably higher than 200 MPa, ranging from about 260 MPa to about 415 MPa. These laboratory measured moduli are considerably lower than the FWD back-calculated moduli, but are consistent with the back-calculated values in the sense that both lab and field moduli of the LTS measurements are considerably higher than their unstabilized counterparts. Note that the back-calculated moduli are all above about 425 MPa and that the ratios of LTS to natural subgrade moduli for the field conditions at the time of testing demonstrate that the stiffnesses of the LTS layers are between 4.4 and 20.7 times greater than the stiffnesses of the natural subgrade.

It is often difficult to develop unique layer moduli based on back-calculation techniques in pavement with multiple layers. However, in this analysis the subgrade modulus was assigned based on the deflection the sensor located approximately 225 cm from the point of load impact (Sensor #7). The modulus of the HMA surface was assigned based on the mixture properties, the impact properties of the falling weight and the HMA temperature at the time of testing. These data were placed in Witczak's equation (*12*) to estimate the HMA modulus. Since the HMA and subgrade moduli were already determined, the modulus of the LTS was the only unknown in each back-calculation.

Comparison of Laboratory Dielectric Value to Field GPR

Figure 3 shows DV results of unstabilized and stabilized soils from the four Mississippi sites. These samples are allowed to imbibe water through capillary suction. Figure 5 clearly demonstrates the differences between unstabilized soils and their stabilized counterparts. For moderately plastic clay soils (US 82E and US 82W), the differences are more obvious as the stabilized samples are intact after 12-hours of soak; whereas the unstabilized samples have swelled significantly and have cracked, particularly at the horizontal interface separating compaction lifts. The correlation between visual evidence of moisture uptake and DVs is not as good in these soils as for samples from US 61N and US 45N. This is mainly due to the interruption of flow due to horizontal fracture at compaction zone interfaces. However, as shown in Figure 5, the degree of moisture absorption and swelling is substantially and consistently higher in the unstabilized than in the stabilized samples for all of the four pavement soils.

Field DVs derived from the GPR are consistent with the laboratory results as all the field DVs are well below the threshold value of 16 which separates an acceptable pavement base or subbase from a marginal or poor quality base or subbase. In fact, the DVs shown in Table 2 indicate a sublayer of good quality in terms of its ability to resist moisture damage effects. Considering that these are stabilized clay layers, this is a significant statement.

MECHANISTIC ANALYSIS

Mechanistic analysis was performed for the Mississippi pavements with and without considering the LTS layer. ELSYM5, a layered elastic computer program, was used to determine four parameters:

(1) tensile strain (ε_t) at the bottom of the hot mix asphalt (HMA) layer which is related to fatigue cracking in the HMA layer,

(2) vertical compressive strain (ε_v) at the top of the subgrade which is related to deep layer rutting,

(3) flexural stress (σ_t) at the bottom of the LTS layer which is related to fatigue within the LTS, and

(4) vertical compressive stress (σ_v) at the top of the subgrade which is also related to deep layer rutting due to the accumulation of deformation within the subgrade.

The pavement structure in the layered elastic model was based on GPR data, which defined layer thicknesses and the back-calculation of moduli values (E) of different layers from FWD data (Table 2). These values are presented in Table 3.

Resistance of HMA and LTS Layers to Flexural Fatigue

The allowable number of load repetitions (N_f) to control fatigue cracking in the HMA layer for the pavements with and without the LTS layer were calculated using the well-known transfer function (*15*):

$$N_f = 0.0796(e_t)^{-3.291}|E|^{-0.854} \tag{1}$$

A typical fatigue-response relationship for LTS layers between the ratio of load-induced flexural stress to flexural strength (stress ratio, S) and the number of load applications to failure (N) for different pavements was developed by Thompson and Figueroa (*16*):

$$S = 0.923 - 0.058 \log N \qquad (2)$$

This relationship was used to assess fatigue damage due to load-induced flexural stresses in the LTSs.

The flexural stress is approximately 0.20 to 0.23 times the UCS (*13*). The four Mississippi DOT pavements were respectively modeled using thicknesses verified by GPR testing and moduli from FWD back-calculations. Pavement design lives were calculated from layered elastic analysis and the results are cataloged in Table 3. The stress ratio was calculated as the maximum flexural stress induced in the LTS divided by 0.29 x UCS. This value was then used to determine N in equation 2.

Permanent Deformation Potential

The allowable number of load repetitions (N_d) to control permanent deformation for the pavements with and without LTS layer were calculated using the familiar transfer function (*15*):

$$N_d = 1.365 \times 10^{-9} (e_v)^{-4.477} \qquad (3)$$

The effect of the LTS on deep-layer rutting potential can also be assessed by comparing the vertical stress at the top of the subgrade to the compressive strength of the subgrade layer. The analysis of many different UCS tests (*17*) has demonstrated that the stress-strain plot derived during the UCS tests becomes non-linear at about one-half the stress required for failure. In other words, at stress levels below about 0.5 times the UCS, strains are recoverable, but at values of vertical compressive stress at the top of the subgrade that exceed about 0.5 times the UCS, some permanent strain occurs that accumulates with loading cycles. This can result in deep-layer or subgrade rutting. Therefore, the vertical compressive stress at the top of the natural subgrade was computed for each pavement and compared to the soaked subgrade strength to assess the potential for the accumulation of permanent strain or deep-layer rutting.

The mechanistic analysis is summarized in Table 3. This analysis is the result of a layered elastic determination of four important parameters (ε_t, ε_v, σ_t, and σ_v) as previously discussed. Based on these parameters and the appropriate transfer functions or related damage models, relative performance predictions for the four Mississippi DOT pavements can be made. The HMA thicknesses are so thick for the pavements analyzed that the predicted life in terms of equivalent 80kN (18,000 pound) single axle loads (ESALs) are very large. Therefore, the effect of the LTS was judged in terms of the percentage increase in pavement life (increase in number of ESALs the pavement is able to carry during its design life). These increases are summarized as follows:

1. Based on the parameter ε_t, which is the controlling design parameter in each case, the pavements with LTS layers show an <u>increase in life</u> of
   - 900% for US 61N,
   - 30,000% for US 82E,
   - 3,000% for US 82W, and
   - 2,000% for US 45N.
2. Based on the parameter σ_t, which together with the flexural strength of the LTS defines the ability of the LTS to <u>resist flexural fatigue</u>, each LTS layer is capable of <u>easily supporting the ESALs</u> in the design life used in item 1.
3. Based on stress induced at the top of the subgrade, σ_v, the LTS <u>reduces stress</u> by
   - 47% for US 61N,
   - 55% for US 82E,
   - 48% for US 82W, and
   - 61% for US 45N.

   Because the subgrade UCSs are so weak after soaking, the stress reductions from lime treatment significantly reduce the potential for accumulation of permanent strain at the top of the subgrade (see Table 3).

RECOMMENDED ENGINEERING PROPERTIES FOR MECHANISTIC DESIGN

The MDTP generates unconfined compressive strengths and resilient moduli of lime-treated pavement materials following moisture conditioning (as described in *Reference 1*). The values produced by this testing protocol for these example pavements are summarized in Table 4. These design values are realistic and reasonably conservative.

In this design example, laboratory values were validated through field testing. This field testing was performed in the spring of the year (March through April), which is typically a wet period for the sections involved. Therefore, although field conditions can never be controlled, the field values should also be conservative.

This example validates the MDTP. It confirms that structural values to be used in mechanistic design can be derived from practical laboratory test procedures.

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APPENDIX: OTHER TESTS TO ASSESS LTS DURABILITY

Scanning Electron Microscopy (SEM) and Energy Dispersive X-Ray (EDX) Test

SEM/EDX analysis was performed at the Electron Microscopy Center of Texas A&M University. Sample preparation included mounting samples on carbon double-stick tape on aluminum stubs. The samples were coated with 300 Å of gold/palladium using a hammer I Sputter Coater.

The microstructure of the natural soils contained significant amounts of typical thin, platy clay structures that form the dominant feature in these micrographs. The pores in the matrix range from a few micrometers to 20μm in size. EDX analysis of several of these plates suggests that the clay is predominantly composed of aluminosilicates, with detectable amounts of potassium and iron in the lattice. The iron content was found to vary from one location to another. Calcium was either absent or present in a negligible amount.

The microstructures of the counterpart lime-stabilized samples were distinctly different. A much denser matrix with fewer micropores was observed in these samples. The presence of calcium (from lime) was detected on the clay particles. Additionally, calcium-silicate-hydrate (C-S-H) was identified in the matrix. The EDX analysis indicates that the pozzolanic reaction had already converted some of the clay minerals to C-S-H after a 7-day cure period (*13*). An elevated curing temperature used for these experiments most likely accelerated this reaction. The densification of the matrix is due to the pozzolanic reaction. The net effect was a significant increase in strength after 7-days.

X-ray Diffraction (XRD) Test

XRD analyses were performed at the Geology Department of Texas A&M University. Sample preparation included grinding of material. Approximately 1 gm of the clay size fraction was applied to a slide with acetone.

In order to explain the pozzolanic reaction that takes place in the lime-stabilized soil samples, X-ray diffraction (XRD) analysis was performed on samples of the as-received native soils as soon as they were mixed with lime, and on the lime-stabilized soil after 7-days of curing at 40^0C. Figure 4 (top), which represents the XRD pattern of the native soil from US 61N, clearly indicates that it is composed mainly of quartz, feldspars and clay minerals. In contrast, the characteristic peaks of hydrated lime or $Ca(OH)_2$ are visible in the XRD pattern of the same soil sample immediately after it was mixed with the lime, Figure 4 (middle). Reduction in the amount of hydrated lime after this lime-stabilized soil was cured for 40^0C is evident from the decrease in the peak heights of $Ca(OH)_2$, shown in Figure 4 (bottom). This decrease was calculated by peak integration and found to be 35% less than the original value. The $Ca(OH)_2$ is expected to have been used to form C-S-H; however, formation of C-S-H cannot be identified in the XRD pattern because of its amorphous nature.

Analysis of Pozzolanic Development

Each of the four soils (representing the four Mississippi LTS layers) is amenable to lime stabilization based on fines content and plasticity requirements. Furthermore, the Eades and Grim pH test results indicate that sufficient lime was used for stabilization. This indicates not only that enough lime was present for a pozzolanic reaction but also that the pH would remain high for a substantial period of time. Eades et al. (*14*) demonstrated that as long as the pH remains above about 10.5, the pozzolanically driven strength gain is likely to continue as is the concomitant autogenous healing. The SEM, EDX and XRD previously discussed further substantiate that a denser soil-lime matrix has been established that supports pozzolanic strength gain. The level of compressive strength gain and resilient modulus improvement, following capillary soak, indirectly validates this, as well.

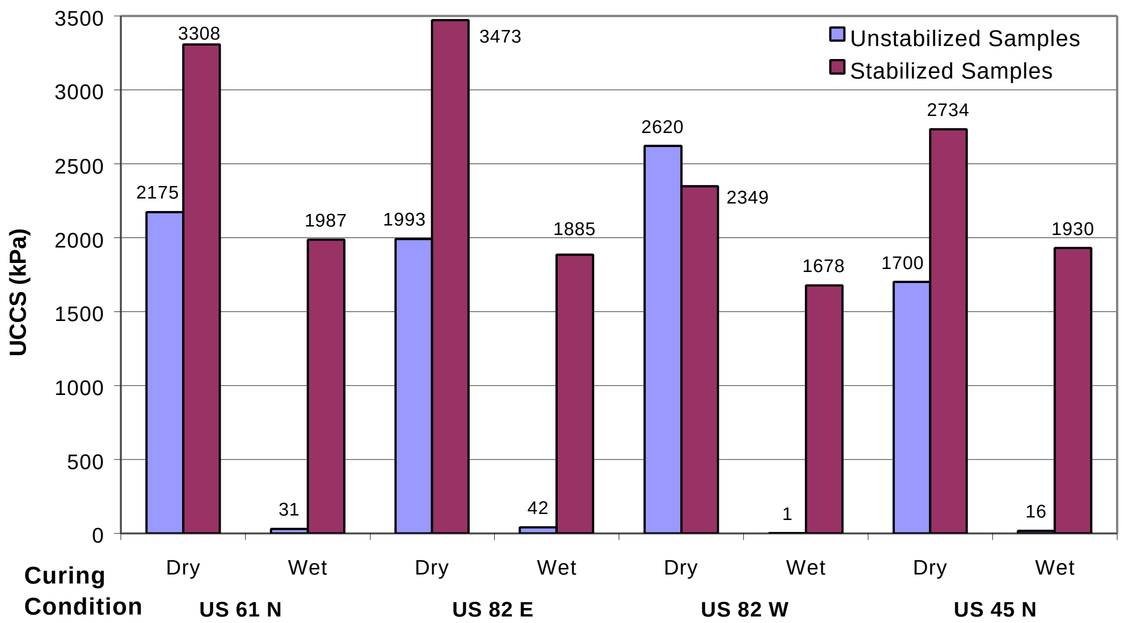

FIGURE 1 Relationship Between Dry and Soaked Unconfined Compressive Strength of Unstabilized and Stabilized Mississippi Soils.

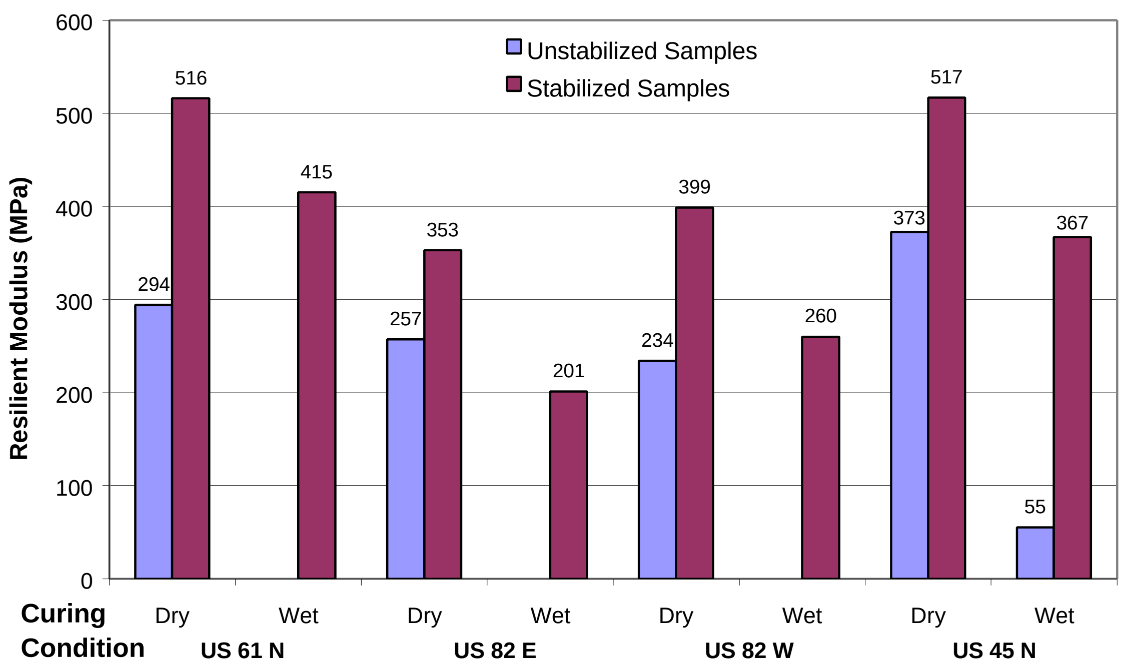

FIGURE 2 Comparison of Resilient Moduli of Unstabilized and Stabilized Mississippi Soils for Dry and Soaked Conditions.

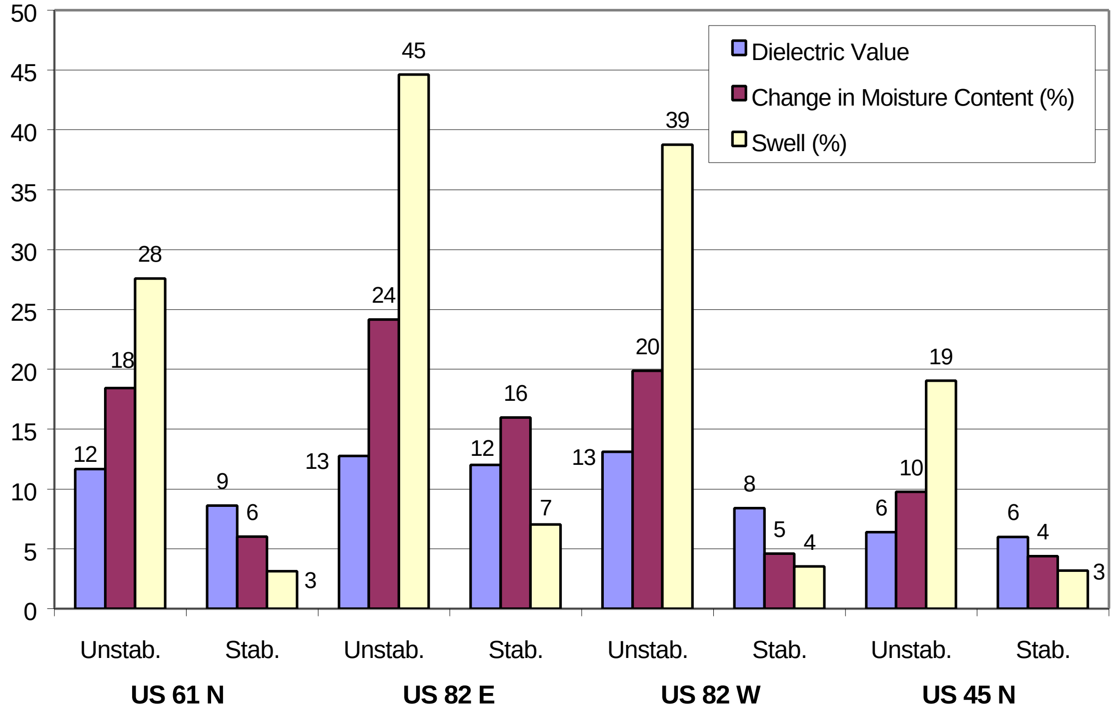

FIGURE 3 Comparison of Dielectric Value, Change in Moisture Content and Swell for Unstabilized and Stabilized Mississippi Soils after 8-days of Capillary Soak.

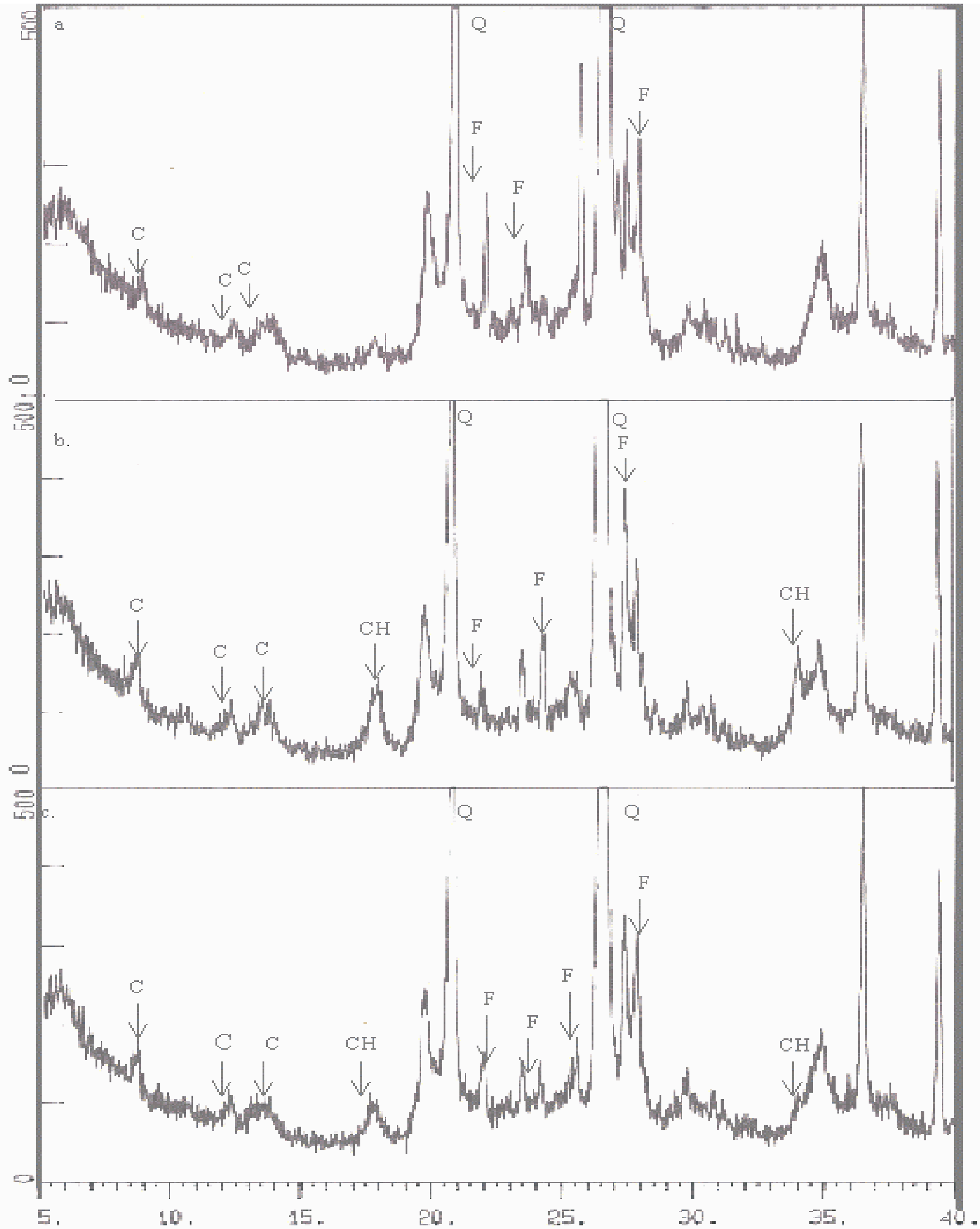

FIGURE 4 XRD Patterns of Soil, where C = Clay Minerals, Q = Quartz, F = Feldspar Minerals and CH = $Ca(OH)_2$ for: (top) As-received Native Soil; (middle) Immediately After Mixing with Lime (note the CH peaks at 22 of 18^0 and 34^0); and (bottom) After Curing for 7-days at 40^0C.

FIGURE 5 Unstabilized and Stabilized Soil Samples of (top) US82W and US82E; (bottom) US45N and US61N after 12-hours of Soaking.

TABLE 1 Unconfined Compressive Strength and Resilient Moduli of Unstabilized and Stabilized Mississippi Soils (NT = Not Tested).

Soil I.D.	Curing Condition	Unstabilized Soil		Stabilized Soil		Ratio (Stabilized / Unstabilized)
		Individual	Average	Individual	Average	
Unconfined Compressive Strength (kPa) of Dry Specimens						
US 61 N Washington Co.	7-days @ 40^0C	2342	2175	3056	3308	1.52
		2008		3559		
US 82 E Washington Co.	7-days @ 40^0C	1912	1993	3661	3473	1.74
		2074		3284		
US 82 W Lowndes Co.	7-days @ 40^0C	2604	2620	2349	2134	0.81
		2636		1919		
US 45 N Kemper Co.	7-days @ 40^0C	1740	1700	3008	2734	1.61
		1660		2460		
Unconfined Compressive Strength (kPa) of Soaked Specimens						
US 61 N Washington Co.	30-days @ 25^0C	2	14	1585	1704	125
		10		1461		
		29		2067		
	7-days @ 40^0C	25	31	1827	1987	63
		32		2030		
		37		2104		
US 82 E Washington Co.	30-days @ 25^0C	20	8	1249	1506	188
		2		1650		
		2		1620		
	7-days @ 40^0C	41	42	2080	1885	45
		58		1803		
		26		1773		
US 82 W Lowndes Co.	30-days @ 25^0C	1	1	1198	1356	1017
		1		1298		
		2		1573		
	7-days @ 40^0C	1	1	1729	1678	1678
		1		1650		
		1		1654		
US 45 N Kemper Co.	30-days @ 25^0C	4	8	1158	1445	188
		6		1486		
		13		1690		
	7-days @ 40^0C	15	16	1788	1930	118
		26		2072		
		8		1931		
Resilient Modulus (MPa) of Dry and Soaked Specimens						
US 61 N Washington Co.	Dry	288	294	530	516	1.76
		300		502		
	Wet	NT		415	415	
US 82 E Washington Co.	Dry	217	257	377	353	1.37
		297		329		
	Wet	NT		201	201	
US 82 W Lowndes Co.	Dry	252	234	404	399	1.70
		216		393		
	Wet	NT		260	260	
US 45 N Kemper Co.	Dry	340	373	520	517	1.39
		405		514		
	Wet	55	55	367	367	6.67

TABLE 2 GPR, FWD and DCP Results for LTS and Unstabilized Subgrade Soils for Four Mississippi Pavements.

Pavement	GPR Results		DCP Results				FWD Results		
	Layer Thickness (mm)	Dielectric Constant	Layer Thickness (mm)	Subgrade CBR	LTS CBR	Ratio (Stabilized/ Unstabilized)	Subgrade Moduli (MPa)	LTS Moduli (MPa)	Ratio (Stabilized/ Unstabilized)
US 61 N Washington County	HMA: 250 LTS: 150	9 - 13	LTS: 125	15	500	33.33	97	425	4.38
US 82 E Washington County	HMA: 325 LTS: 150	6 - 8	LTS: 125	12	150	12.50	119	2466	20.72
US 82 W Lowndes County	HMA: 363 LTS: 150	7 - 10	LTS: 150	4	47	11.75	123	1350	10.98
US 45 N Kemper County	HMA: 250 LTS: 250	No Data	LTS: 275	10	133	13.30	125	1482	11.86

TABLE 3 Mechanistic Analysis for Mississippi Pavements With and Without Considering the LTS Layer.

Soil ID	Pavement Structure	UCCS (kPa)	E (HMA) (MPa)	E (LTS) (MPa)	E (Subgrade) (MPa)	ε_t (mm)	ε_v (mm)	σ_t (kPa)	σ_v (kPa)	N (no.)	N_f (no.)	N_d (no.)
US61N	w LTS	1987	1151	425	97	0.0040	0.0062	91	40	6E+12	9E+06	2E+07
	w/o LTS	31	1151		97	0.0070	0.0086	449	76	0E+00	1E+06	5E+06
US82E	w LTS	1885	1627	2466	119	0.0006	0.0017	208	20	2E+08	3E+09	6E+09
	w/o LTS	42	1627		119	0.0034	0.0041	314	44	0E+00	1E+07	1E+08
US82W	w LTS	1678	1434	1350	123	0.0011	0.0021	126	21	5E+10	6E+08	3E+09
	w/o LTS	1	1434		123	0.0031	0.0037	245	40	0E+00	2E+07	2E+08
US45N	w LTS	1930	4420	1482	125	0.0011	0.0013	70	15	3E+13	2E+08	2E+10
	w/o LTS	16	4420		125	0.0024	0.0033	680	38	0E+00	1E+07	3E+08

TABLE 4 Recommended Design Values for Mechanistic Analysis.

Material	Unconfined Compressive Strength, kPa	Resilient Modulus, MPa
US 61 N	1900	400
US 82 E	1600	200
US 82 W	1600	250
US 45 N	1900	300