STATE STUDY NO. 147

FINAL REPORT

Long-Term Effect of Lime-Fly Ash Treated Soils

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16. Abstract

In October of 2000 MDOT initiated State Study No. 147, entitled "Long-Term Effect of Lime-Fly Ash Treated Soils." The purpose of this study was to evaluate the long-term performance of LFA stabilized soil as a base course material. Part of the impetus for this study was the premature failure of several pavements. These failures were attributed to the saturation of the LFA and soil blend of the base layer before this layer had experienced an adequate degree of curing.

This study entailed the following to accomplish the above objective:

(1) FWD tests on both newer and older pavements;
(2) Coring pavement at each FWD location to visually observe the condition of the layers, to obtain pavement thicknesses for the computational procedures, and for obtaining LFA cores for unconfined compressive strength (UCS) testing;
(3) Computational procedures for the analyses of the deflection basins were employed to obtain in-situ HMA and LFA stabilized soil layer modulus values, and to evaluate the in-situ LFA structural layer coefficient values for comparison to MDOT's current design value of 0.20.

Based upon visual observation, backcalculated modulus, and in-situ structural layer coefficient values, it is concluded that MDOT LFA stabilized soil base courses possess highly variable material properties. There is also significant variation in the in-situ LFA stabilized soil base layer thickness within the majority of the pavements cored for this study. Recommendations were made to increase the average LFA material property values and reduce the spread in these values, as follows:

(1) A significant increase in the required level of field compaction of the LFA stabilized soil base layer to 96 percent modified Proctor effort was recommended to increase the average values.
(2) In the area of field construction, two potential methods to reduce variability are (1) improving the current method of field-mixed-in-place, and (2) plant mix with placement of the blended material via a paver.
(3) For the field-mixed-in-place method adjustment of the field moisture content, via the method of nursing, prior to spreading the lime and fly ash and spreading of the lime and fly ash with a Vane Feeder Spreader

The calculations related to the in-service loading condition supported the conclusion that the routine design thickness for LFA stabilized soil base layers should be increased from 6 inches to 8 inches and an in-situ LFA Proctor UCS value of 400 psi should be required in a field QC/QA program to provide for a Perpetual Pavement LFA base layer.

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In summary, this study provides a broad overview of the design and construction of a LFA stabilized soil base course in Mississippi. The resulting recommendations correlate the mix design, pavement layer design, construction, and QC/QA efforts with the objective of effecting a substantial improvement in the performance of this pavement layer construction material.

ACKNOWLEDGMENT

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During the period of this study, the Executive Director of MDOT was Mr. Hugh Long, followed by Mr. Larry "Butch" Brown, followed by Mr. Harry Lee James (Interim). The Deputy Executive Director/Chief Engineer was Mr. James Kopf, followed by Mr. Harry Lee James.

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Chapter 1 -- Introduction

In 1981 the Mississippi Department of Transportation (MDOT) was introduced to the concept of using lime and fly ash to chemically stabilize granular materials for use in base course construction. At that time the Federal Highway Administration (FHWA) was promoting the use of fly ash in highway construction as a method for utilizing this waste product. As a result, MDOT participated in the FHWA Demonstration Project No. 59, entitled “The Use of Fly Ash in Highway Construction” (Crawley, 1990). Two factors provided the impetus for Mississippi to try using lime-fly ash (LFA) stabilized soils for base course construction. These included problems with shrinkage cracking in cement stabilized soil base layers and potential savings in construction costs.

From the 1950s until the mid-1980s MDOT used cement treated bases (CTB) extensively in pavement structures. One negative characteristic of CTB is the significant shrinkage cracking that this material experiences subsequent to construction. The cracking poses no problems in a concrete pavement since this pavement type will bridge over the cracks. An asphalt pavement, however, reflects these cracks, which leads to water infiltration, spalling of the crack faces, and other problems producing an unacceptably rough riding surface.

The rate of chemical reaction between the lime and fly ash to effect strength gain in the stabilized material is slower than the rate experienced with soil cement stabilization. It was postulated that the relatively slow rate of strength gain acts to retard shrinkage or environmental cracking, thus constituting one of the two factors for trying LFA stabilized material. Another consequence of the relatively slow rate of reaction of LFA mixtures is that should shrinkage cracking occur, the continued chemical reaction would tend to heal these shrinkage cracks in a process referred to as autogeneous healing.

Cost was the second factor considered for the use of LFA stabilized material. The cost of an LFA base course was less than that for an equivalent load carrying asphalt base course, thus MDOT could experience a savings in construction expenditures.

MDOT's involvement with the Demonstration Project No. 59 included three phases of construction that constituted the first projects to be completed in Mississippi using LFA stabilized material for a base course layer. The project locations were as follows:

- SR 63 in Jackson County
- US 84/98 in Adams County
- US 98 by pass around Lucedale in George County

The Phase One project was built in 1982-1983 on SR 63 in Jackson County. This project consisted of adding two new lanes parallel to the existing two lanes of the highway. The project began at SR 613 and extended north for 15.262 miles to the George County line. The pavement design approach was to incorporate the lime and fly ash into the top 6 inches of the subgrade. If sufficient strength were developed in this layer, it would be considered a base course layer in the pavement structure. If not, then this layer would serve as a construction platform. The lime and Class F fly ash were blended into the subgrade soil with a single transverse shaft rotary road mixer. W.E. Blain & Sons, Inc. was the prime contractor on this project. Subsequent Dynaflect deflection testing indicated that this stabilized subgrade layer provided comparable structural capacity to the pavement as other materials currently in use for base course construction (Crawley, 1984). Data obtained from MDOT's Transportation Management Information System (TMIS) indicates that no overlay was performed on this route until 1998.

The Phase Two project was built in 1985-1986 on US 84/98 in Adams County. This project also consisted of adding two new lanes parallel to the existing two lanes of the highway. The project began at U.S. 61 and extended east for a distance of 5.911 miles. Based upon the good performance of the LFA stabilized material observed on SR 63, the LFA stabilized material for this project was designed for use as a pavement base course at the inception of the design process. In this project the lime, Class F fly ash and a bank-run sand gravel aggregate material were blended in a central mixing plant and then transported to the project site for placement on the roadbed. Numerous problems were experienced with the mixing plant, primarily with the fly ash and lime proportioning, which resulted in some loads of LFA material that did not have the proper amounts of lime and fly ash. Dickerson and Bowen, Inc. was the prime contractor for this project. Subsequent coring of the completed pavement indicated problems with both the LFA stabilized base course and the overlying asphalt courses (Crawley, unpublished field evaluation report, 1990). Data obtained from TMIS indicates that the top 1.5 inches of the existing pavement was hot in-place recycled, with between 3 and 4.5 inches of overlay placed in 1991. This project serves as a prime example of the detriment to long-term pavement performance resulting from extreme variability in pavement material properties.

The Phase Three project located in George County was the US 98 bypass around Lucedale built in 1987-1988. This 10-mile long project was a two-lane facility on new location (Ferguson, 1990). This phase of the evaluation involved road mixing Class F fly ash and lime with a low plasticity sand topping. Minor shrinkage cracking was observed in several places in the LFA base course and was attributed to higher values of plasticity index (PI) of the topping in those areas. Bush Construction Company, Inc. was the prime contractor for this project. Data obtained from TMIS indicates that about 6 miles of this project were overlaid with 1.5 inches of asphalt in 1994, and that the rest of the project was overlaid with between 1.5 and 3.5 inches of asphalt in 2000.

A review of the three projects constructed in Mississippi in conjunction with the FHWA Demonstration Project No. 59 indicated that the road mixing method for blending the lime, fly ash and granular material produced excellent results and was very cost-effective (Ferguson, 1990). As a result, MDOT allows the use of the road mixing method for all LFA base course construction.

Class C fly ash was introduced for LFA base course construction in 1989 (Ferguson, 1990). A research project was conducted in conjunction with the construction of the 5.218 miles of new alignment of SR 7 in Yalobusha County. The project began at a point north of Coffeeville on SR 7 and extended north to the connection with the existing two-lane section at Water Valley. The Lehman-Roberts Company was the prime contractor for this project. The construction and post-construction observations and review of test data indicated that Class C fly ash was a viable option along with Class F fly ash for this type stabilization (Ferguson and Avent, 1993). Both classes of fly ash are currently allowed for MDOT road construction.

Data obtained from TMIS indicates that no rehabilitation or overlay has been performed on this project location. However, cracking is observed in the pavement surface. At the time that this pavement was constructed, just prior to placement of the first lift of asphalt, a crack survey was performed on a segment of the base course. As part of the data collection for the current study, another crack survey was performed in the same section as previously surveyed to ascertain the extent of reflection cracking. A comparison of these two surveys showed that 82 percent of the cracks in the LFA base course had reflected through the overlying asphalt. As will be discussed in more detail in Chapter 11, this project provides an excellent example of the need for a construction platform in new pavement construction.

During the 1980s a total of eight projects were constructed using an LFA stabilized soil base course (Crawley, 1990). A significant increase in projects constructed with this material occurred during the 1990s. To date, MDOT has constructed over 100 projects using LFA stabilized soil as either a chemically stabilized subgrade or base course layer.

Reflection cracking has already been noted as one problem encountered with the use of this material. Another problem is the saturation of compacted LFA and soil mixture base layers before the occurrence of significant strength gain in this material. Since LFA stabilized material requires time and temperature for effective strength gain to occur, this is an important consideration for late season LFA construction given the relatively cool temperatures of late fall and winter.

Given the relatively large number of projects constructed utilizing LFA stabilized granular soil as a base course construction material, and some problems associated with its use, MDOT initiated State Study No. 147, entitled “Long Term Effect of Lime-Fly Ash Treated Soil,” in October of 2000. MDOT uses the AASHTO Interim Guide for the Design of Rigid and Flexible Pavements – 1972 for its flexible pavement design methodology. The structural layer coefficient is the primary input parameter reflecting the quality of the pavement materials in this design procedure. In this study the basis of evaluation for the LFA material is the development of in-situ LFA structural layer coefficients. Based upon the results of this evaluation and mechanistic analyses, recommendations have been developed which address various facets of LFA base course design, construction and quality control.

A total of nine different project sites were selected for this study. The long-term performance of this material is evaluated via four of these nine projects. The other five projects were selected to evaluate the quality of this material after about two years of field curing.

Chapter 2 -- Project Locations and Pavements Considered in Study

A total of nine different project sites have been used in the MDOT study, with locations dispersed throughout the State to facilitate a statewide evaluation of the LFA stabilized material. The route, county, project number(s), location and other pertinent information for each of these projects are listed in Table 1.

The long-term performance of this material is evaluated via four of the nine projects shown in Table 1 under the heading “Older Projects”. Note that the LFA stabilized soil had been in place from 8.5 to 11 years at the time that these pavements were tested for this study. An estimated traffic loading, from the completion of construction until the time of pavement testing, was obtained from the MDOT Planning Division. The estimated traffic loading and the design loading were used to obtain an estimate of the percent of design traffic loading placed on each of these older pavements.

The lower half of Table 1 lists five projects under the heading “Newer Projects”. The strength gain of LFA stabilized material occurs at a relatively slow rate compared to cement stabilized material. This gain in strength continues after the construction of the pavement and the pavement is open to traffic. One facet of this study is to determine what range of moduli values, or stiffness, and strength that the LFA stabilized material achieves prior to the material’s subsequent degradation due to long-term traffic loading.

Note that the age of the LFA stabilized material, at the time of testing, was approximately two years for all of the newer projects. Pavement cores are typically removed from a pavement with a drill rig that uses water to keep the core barrel cool during the coring operation. Previous experience with LFA stabilized material has shown that a sufficient curing time must be allowed in order to extract intact LFA cores using this coring method (George and Uddin, 2001). Two summers of field curing was considered a sufficient length of time to obtain a high percentage of core retrieval, but also minimize damage to this material from traffic.

Table 2 is a summary of the project design pavement layer thicknesses and foundation soils. A criterion used in the selection of the five newer projects was that the pavement structure at each of them can be characterized as a three-layer system; e.g., asphalt, LFA stabilized soil base and unstabilized subgrade. This criterion was used because the backcalculation analyses of the deflection basins provide a more reliable estimate of the LFA layer moduli given a three-layer system as opposed to a four or more layer system. This topic is considered in greater detail in Chapter 4.

The heading "Embankment" includes both the basement and design soils comprising the foundation soils for the overlying pavement. Basement soil is the soil placed on the original soil profile up to an elevation 3 feet below the subgrade. The design soil is the three feet of soil located directly beneath the pavement structure. It is either placed and compacted soil on top of the basement soil, or the top three feet of in-situ soil.

The embankment designations were obtained from the plans for each project. It was not determined for any of the projects which plan material was actually used at any of the project test locations. An estimate for a pavement design subgrade soil CBR for the test section/s in each of the five newer projects is shown in the last column of Table 2. These estimated values differ from the CBR values used in the original pavement design for each project (except for the Wilkinson County project) since the original design CBR values were based on the subgrade soil for the entire project length, as opposed to the subgrade soils located directly beneath the individual test sections. These estimates were obtained using the following equation, which is one of the models included in the 2002 AASHTO Pavement Design Guide:

$$M_r = 2555 \text{ CBR}^{0.64} \quad \text{Equation 1}$$

Chapter 4 includes a discussion of the backcalculation technique employed in this study and includes the results of the backcalculations performed for each test location of each test section. For the current discussion, let it suffice that the backcalculated modulus, E_{back}, subgrade soil values obtained from the test locations within a given test section were used to evaluate the 10th percentile subgrade soil E_{back} value for that given test section. This unique E_{back} value for each test section was then corrected to an equivalent laboratory modulus value, M_r, by multiplying each E_{back} value by a factor of 0.52 (Von Quintus and Killingsworth, 1998). This resulting unique M_r value for each test section was then substituted into Equation 1 to obtain a corresponding "design" CBR value for that given test section. The magnitude of these "design" CBR values compare favorably with those typically encountered in MDOT pavement designs, thus substantiating the method used to obtain these values from backcalculation results. Note that the 0.52 factor is a function of pavement and layer type; therefore, it is not a unique value for use in all cases.

Table 3 includes the LFA mix designs used for each of the nine projects. Several projects include more than one mix design, depending on the number of borrow pits used as sources of soil or the number of fly ash sources. It was not determined for any of these projects which design was used for a given test location.

With the exception of the Smith County project, all of the soil used in the mix designs was nonplastic (NP). Except for the Forrest/Perry project, all of the soil consisted of less than ½-inch size material. A-2-4 is the predominant soil classification of these soils. A 1:4 lime to fly ash ratio was used in 12 of the 15 mix designs and a 1:3 ratio was used for three of the designs. The predominant soil classification and these ratios are consistent with those from the review of 182 mix designs discussed in Chapter 8.

One of the original objectives of this study was to evaluate the relative performance of Class F and Class C fly ashes. Unfortunately, insufficient data is available to make this evaluation. Note that a conditioned fly ash was used for the Forrest/Perry project.

Table 4 provides the gradation of the soil used in each of the LFA mix designs for the nine projects, and of the unstabilized granular soil subbase of the three older projects that included this layer. With the exception of the Forrest/Perry project, the soils were composed of material that passed the No. 10 sieve. The Forrest/Perry project did not have a much higher quality of soil, with only 8 percent uncrushed material retained on the 0.5-inch sieve and 18 percent retained on the No. 10 sieve. For the three older projects that included the unstabilized granular subbase layer, the soil comprising these subbase layers was the same soil as that stabilized with LFA to construct the overlying base course. Table 4 also includes the pH of the soil used in the LFA mix designs for the five newer projects. Soil pH data was not available from the mix designs for the four older projects.

MDOT's Transportation Management Information System (TMIS) includes information from a network level survey conducted on the State's highway system every two years. A pavement condition rating (PCR) of each pavement is determined as part of each survey. Figure 1 illustrates the PCR vs. survey date for each of the four older pavements. The first PCR shown for each project corresponds to the first survey taken after completion of that project. The next to last PCR for each project corresponds to the year 2000 survey, which is the same year that the pavements were tested for this study. The year 2002 survey data has also been included in this figure to illustrate the continued performance of these pavements.

Many factors affect the performance of a pavement, so it is difficult to isolate the effect of one variable. Given this consideration, two observations are made regarding Figure 1. The first observation is that use of a construction platform is very important for the long-term performance of an LFA stabilized base course. The Yalobusha County project did not include a construction platform, and it can be seen that the PCR had dropped to 72 in the year 2000 survey after experiencing only 38 percent of its design traffic loading. Chapter 11 includes a discussion of the benefit of a construction platform.

The second observation is the importance of good field construction quality control to ensure that the design LFA layer thickness is actually constructed in the pavement structure. The George County project had a design LFA stabilized soil base course thickness of 6 inches. The average in-situ layer thickness, as determined by coring of the pavement, was only 4.7 inches, with a maximum layer thickness of 5.5 inches and a minimum of 3.8 inches. The 2000 survey PCR of this pavement was 83 after experiencing only 26 percent of its design traffic loading. The topic of determining LFA base course thickness is included in the next chapter.

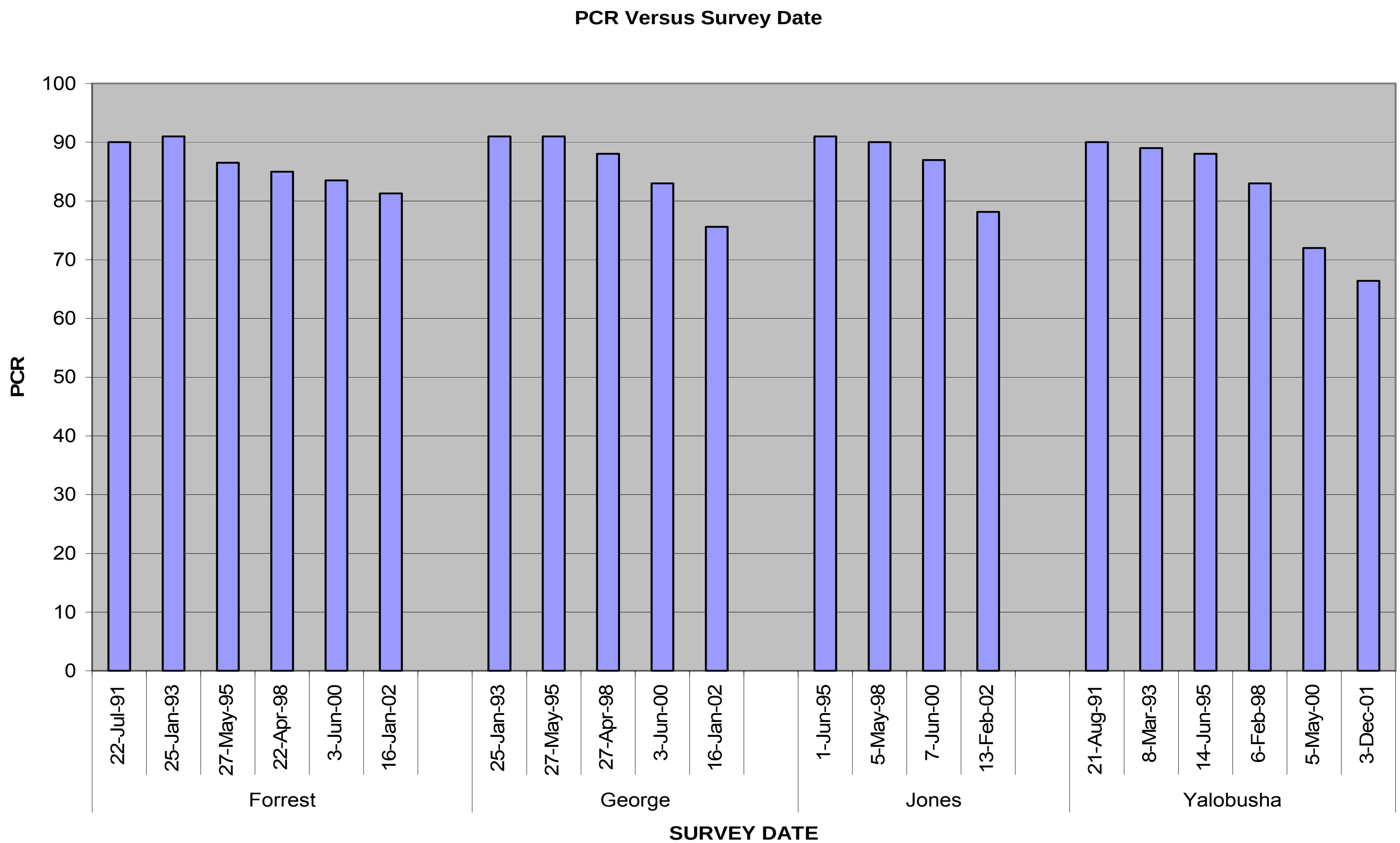


Figure 1. Pavement Condition Rating (PCR) vs. Condition Survey Date

Table 1. Nine Project Sites Used in LFA Study

Route	County	Project Number	Location	LFA Placed	Age of LFA at Time of Testing (Years)	Estimated Percent Design Traffic Loading on Pavement at Time Of Testing, %
			Older Projects			
U.S. 98	Forrest & Perry	88-0103-00-803 88-0103-00-804	From Ralston to New Augusta	Fall 1989 & Spring 1990	11	42
U.S. 98	George	96-0014-03-047-10	Add 2 Lanes and Median to U.S. 98 at Lucedale	Summer 1992	8.5	26
U.S. 84	Jones & Wayne	16-0015-02-042-10	Between Laurel and Just East of Jones/Wayne County Line	Summer & Fall 1992	8.5	26
Ms. Hwy. 7	Yalobusha	11-0019-02-022-10	Between Coffeeville and Water Valley	Fall 1989	11	38
			Newer Projects			
U.S. 61	Bolivar	97-0009-03-056-10	From Merigold to Shelby	Fall 1998 & Spring 1999	2	Negligible
U.S. 45	Clarke	17-0002-02-015-10	From Clarkco State Park to 1.0 Mile South of Lauderdale County Line	Fall 1998 & Spring 1999	2	Negligible
Ms. Hwy 35	Smith	46-0023-01-052-10	Between Raleigh and Smith/Scott County Line	Fall 1998 & Summer 1999	2 1.5	Negligible
U.S. 72	Tippah	42-0007-01-064-10	From Benton County Line to Walnut	Summer 1998	2.5	Negligible
U.S. 61	Wilkinson	97-0009-01-104-10	From State Road 563 to Sta. 538+00	Fall 1998	2	Negligible

Table 2. Summary of Project Design Pavement Layer Thicknesses and Foundation Soils

Route	County	Upper & Lower Lifts	Base	Subbase	Embankment [d]	Subgrade[e] CBR
			Older Projects			
U.S. 98	Forrest & Perry	6.0 " HMA	6 " LFA	3 " Cl 9 Gr A	B-9	
U.S. 98	George	4.5 " HMA	6 " LFA	13.5 " Cl 9 Gr A	Unclassified, B-7	
U.S. 84	Jones & Wayne					
	Section I [b]	4.5 " HMA	6 " LFA	10 " Cl 10 Gr A	B-7, ESFE	
	Secton II [c]	6 " HMA	7 " LFA	7.5 " Cl 10 Gr A	B-7, ESFE	
Hwy. 7	Yalobusha	4.5 " HMA	6 " LFA		B-9	
			Newer Projects			
		Upper & Intermediate Lifts	Lower Lift	Base		
U.S. 61	Bolivar	4.5 " HMA	3.5 " HMA	6 " LFA	Unclassified, B-9, B-9-6	4
U. S. 45	Clarke	3.5 " HMA	2.25 " HMA	6 " LFA	B-4	6
Hwy. 35	Smith	4.25 " HMA	3 " HMA	6 " LFA	Unclassified, B-17, B-15	8
U.S. 72	Tippah	4.5 " HMA	3 " HMA	8 " LFA	Unclassified, B-9, B-9-6, B-18	2 to 3
U.S. 61	Wilkinson	2.5 " HMA [a]	3 " HMA	6 " LFA	Unclassified, B-9	5

[a] Lower lift in place at time of field testing
[b] Design subgrade CBR = 15
[c] Design subgrade CBR = 3
[d] Embankment material called for in the plans
[e] Revised design subgrade CBR based on 10th percentile subgrade modulus in test section

Table 3. LFA Mix Designs for the Nine Projects

Route	County	Laboratory Number	Class Soil Stabilized	Soil PI	Percent Retained on 1/2" Sieve	AASHTO	Percent Lime	Company Supplying Lime	Percent Fly Ash	Class Fly Ash	Company Supplying Fly Ash	Power Plant Supplying Fly Ash
Older Projects												
U.S. 98	Forrest & Perry	9459342	Cl 9 A	NP	8	A-3	4		16[a]	F	Monex Resources	Purvis, Ms.
U.S. 98	George	9544489	Cl 9 A	NP	None	A-2-4	4	Falco	12		Monex Resources	Leroy, Ala.
		9548064	Cl 9 A	NP	None	A-2-4	4	Falco	12		Monex Resources	Leroy, Ala.
U.S. 84	Jones & Wayne	9549732	Cl 10 A	NP	None	A-4	3	Falco	12		Monex Resources	Leroy, Ala.
		9549733	Cl 10 A	NP	None	A-2-4	3	Falco	12		Monex Resources	Leroy, Ala.
		9550091	Cl 10 A	NP	None	A-4	3	Falco	12		Monex Resources	Purvis, Ms.
		9550092	Cl 10 A	NP	None	A-2-4	3	Falco	12		Monex Resources	Purvis, Ms.
Hwy. 7	Yalobusha	9485377	Cl 9 C	NP	None	A-2-4	3	Falco	12	C		White Bluff
Newer Projects												
U.S. 61	Bolivar	9683320	Cl 9 C	NP	None	A-2-4	4	Falco	12	C	Fly Ash Products	Pine Bluff
U. S. 45	Clarke	9683906	Cl 9 A	NP	None	A-2-4	3	Falco	12		Boral	
		9684195	Cl 9 A	NP	None	A-2-4	3	Falco	12		Boral	
Hwy. 35	Smith	9690060	Cl 9 A	3	None	A-2-4	3	Falco	12		Fly Ash Products	
		9689795	Cl 9 A	3	None	A-2-4	3	Falco	12		Boral	
U.S. 72	Tippah	9677123	Cl 9 C	NP	None	A-2-4	3	Falco	12	C	Fly Ash Products	Pine Bluff
U.S. 61	Wilkinson	9685437	Cl 9 C	NP	None	A-2-4	3	Falco	12	C	Bayou Ash	Big Cajun

Table 4. Soils Data Corresponding to LFA Mix Designs and Unstabilized Subbase Layers

Older Projects

County	Forrest Perry	George	George	Jones Wayne	Jones Wayne	Yalobusha
Lab Number	9459342	9544489	9548064	9549732	9549733	9485377
Sieve Size	Percent Passing					
1"	100					
0.5"	92					
No. 4	85					
No. 10	82	100	100	100	100	100
No. 40	53	85	91	94	95	78
No. 60	15	56	58	87	85	25
No. 200	7	18	16	43	17	11
No. 270	7	18	10	27	16	10
% Silt	1	3	4	10	5	2
% Clay	5	15	6	17	11	8

Newer Projects

County	Bolivar	Clarke	Clarke	Smith	Tippah	Wilkinson
Lab Number	9683320	9683906	9684195	9689795	9677123	9685437
Sieve Size	Percent Passing					
No. 10	100	100	100	100	100	100
No. 40	97	96	99	63	61	95
No. 60	63	82	71	31	27	62
No. 200	6	26	12	19	26	26
No. 270	4	22	11	21	11	21
% Silt	2	5	1	1	1	6
% Clay	2	17	10	20	10	15
Soil pH	6.77	4.15	4.96	4.38	5.49	6.1

Chapter 3 -- Pavement Coring Operations and LFA Core UCS

FWD testing and pavement coring operations were conducted in conjunction with the field operations of this study. At each location where an FWD test was performed an attempt was made to obtain an intact LFA core. A discussion of the pavement coring operations and UCS testing of LFA cores are the topics of this chapter, and the FWD testing and backcalculation of pavement layer moduli are addressed in Chapter 4. The purposes of conducting the coring operations include (1) a visual observation of the condition of both the asphalt and LFA pavement layer materials, (2) extraction of an intact LFA core for UCS testing, and (3) determination of pavement layer thicknesses to facilitate the backcalculation of pavement layer moduli.

The age of the LFA material at the time of coring is an important consideration since LFA stabilized soil properties change with time. The long-term performance of the LFA material can be evaluated via the selection of older pavements, but the timing for coring relatively new pavements entails a couple of considerations, rate of strength gain and testing before cracking occurs.

The first consideration is that the strength gain of LFA stabilized material occurs at a relatively slow rate compared to that of cement stabilized material. This is usually considered a benefit since a reduced rate of strength gain is generally attributed to a reduction in the development of shrinkage cracks; however, this aspect also affects how soon after construction that intact LFA cores can be obtained from the pavement. For this study the pavement cores were cut with a drill rig that uses water to keep the core barrel cool during the coring operation (Figure 2). A wire was used to facilitate the removal of the cores from the pavement (Figure 3). Based on previous experience two complete summers of field curing is a sufficient length of time to obtain a high percentage of core retrieval using this coring procedure (George and Uddin, 2001).

The second consideration concerning the timing for coring relatively new pavements is testing the pavement before any significant cracking has occurred in the pavement test sections. This consideration is important for two reasons. First, the HMA modulus temperature correction equation discussed in Chapter 5 was developed using intact pavement sections and is not considered applicable for use in cracked sections. Second, cracks in the LFA stabilized material reduce the modulus of this pavement layer; therefore, the backcalculated moduli from intact sections more accurately depict the initial developed stiffness of this material.

Traffic loading and environmental effects are primary factors that cause pavement cracking. The damage to the pavement due to these two factors is cumulative with time. Pavement testing performed soon after the construction of the pavement minimizes the impact of these factors and allows for the evaluation of a relatively intact pavement section.

Pavement testing conducted after the LFA material has had sufficient time for strength and stiffness development, but prior to degradation due to traffic loading and environmental effects, provides estimates of in-situ pavement layer moduli which can be utilized in flexural stress – flexural strength comparison calculations. Calculations using deflection data from intact sections provide estimates of developed in-situ LFA structural layer coefficients for comparison to the design LFA structural layer coefficient.

The five newer project sites were selected to evaluate properties of the LFA material after approximately two summers of curing under field conditions. While each of these newer projects had experienced minimal traffic loading prior to testing, due to the elapsed time between the turning of traffic on the finished pavement and the time of testing, the tests were performed between the wheel paths to further minimize the effect of traffic loading.

Test locations on the four older projects were located transversely at each station in the outside wheel path. With these older projects the objective was to evaluate the long-term performance of the LFA material including the effects of traffic loading. Several stations in the Forrest/Perry project were an exception to this transverse location scheme due to difficulty in obtaining an intact core from within the wheel path for UCS testing.

Tables 5 and 6 include data for the newer and older pavements respectively. The stations listed correspond to the FWD test and coring locations for each project. A minimum of eight test locations were selected for each project. The 1993 AASHTO Guide for Design of Pavement Structures includes a discussion on typical limit of accuracy curves in section 3.6.4. Eight test locations were considered a reasonable estimate corresponding to the lower end of Zone II as illustrated in Figure 3.19 of the Guide. Several of the projects had 16 test locations. With each of these projects eight of the test locations were located in segments of the road constructed near the end of one construction season, and the remaining eight located in sections of the road that were constructed during the following construction season. The intent of this effort was to determine if late season placement of LFA material adversely affects the performance of this material. Chapter 10 addresses this issue.

The general condition of the pavement surface is noted within the proximity of each test location by observation for the presence and type of surface cracking. Rut depth measurements are included for each of the wheel paths of the tested lane (Figure 4). The asphalt cores were visually inspected and rated following the guidelines provided in Appendix A of this report. The design and in-situ asphalt thicknesses are included in Tables 5 and 6, which provide information on the deviation from design, and the variability of, layer thickness for a material placed with a paver. These can then be compared to data obtained for material mixed and compacted in place as is the procedure followed for LFA base course construction. A significant difference is noted and further discussed in Chapter 9.

No granular subbase layer was included in the pavement design of the five newer projects; however, three of the four older projects included this layer. During construction of these three older projects the total design thickness of granular material for both the subbase and base course layers was placed, and then the upper portion of this material was stabilized with LFA to create the base course. The total design thickness of the granular material placed for both courses minus the measured LFA base course thickness was considered the thickness of the untreated granular subbase layer. In retrospect, a dynamic cone penetrometer should have been employed at each test location to determine the actual thickness of untreated granular material as this is a significant value in the calculation of LFA in-situ structural layer coefficients.

Tables 7 and 8 include a visual assessment of the in-situ condition of the LFA material and results of UCS testing of the LFA cores for the newer and older projects respectively. Note under the column heading “Coring Location Station” that more than one coring attempt was made for various FWD test locations. For example, in Bolivar

County an FWD test was performed at Station 340+00, and a coring attempt was performed at Stations 339+98 and 340+00. For this FWD test location an attempt was first made to extract an intact core for UCS testing from Station 340+00, but the extracted LFA material was not suitable for testing. Another attempt was made to extract a testable core from Station 339+98. The core extracted from this second coring attempt was suitable for testing.

An LFA core rating scheme was developed and used to visually classify the relative quality and suitability for UCS testing of the LFA material on a scale from one to six. Relative quality refers to how intact the material appears and provides an indication of the degree of compaction and cementation of this material. Table 9 provides a description of each of these six classifications. As seen from these descriptions, the relative quality and/or suitability for UCS testing of the LFA material decreases as the numbers progress from one to six. Figures 5 through 12 illustrate these classes.

Note the grooves cut in the cores shown in Figures 8 and 11. The probable cause of these grooves is stones caught between the core barrel and LFA core as the coring progressed. The sources of stone include an overlying layer of HMA, Figure 13, or stone contained within the LFA material that broke loose during the coring operation. One way to minimize this problem is to make sure the top of the LFA layer is free of any loose aggregate prior to coring through the LFA layer. Figure 14 illustrates the use of a shop wet vacuum for this purpose.

Table 9 provides a summary of the visual examination of the LFA material as it was observed at the FWD test stations as opposed to the final cored stations within the given FWD test locales. Using the same example in Bolivar County, an FWD test was performed at Station 340+00, and coring attempts were performed at Stations 339+98 and 340+00. The classification of the LFA material at Station 340+00 was used in the summary statistics of Table 9. In the Forrest/Perry project approximately half of the coring was performed between the wheel paths instead of in the wheel paths to obtain a testable core. These locations were excluded from the Table 9 statistics since the FWD testing was performed in the wheel paths at all locations in this project.

Table 9 indicates that the LFA material in 62 percent of the tested locations within both the newer and older pavements were in excellent condition. Based on a summation of percentages corresponding to classifications one through three, 74 and 68 percent of the FWD test locations within the newer and older pavements respectively produced UCS testable cores. Two out of the 63 newer pavement test locations, both located in the same project, had very poor LFA material present in the pavement, whereas no LFA material in any of the older pavements had a six classification.

The LFA in-situ layer thickness is recorded for each of the cored locations in Tables 7 and 8. An evaluation/attempt was made to obtain a specimen for UCS testing from each of the extracted cores. The material on the ends of the extracted LFA cores was often cracked or poorly cemented and was removed to expose the intact LFA core material for testing. Possible causes of poorly cemented core end material include inadequate curing of the top of the LFA layer or insufficiently mixed material at the bottom of the layer. Cracks in the LFA material could be either pre-existing or created during the coring operation. Specific details on how the cores were prepared and tested for UCS, including capping where required, are included in Appendix B. Note that at some locations a significant difference exists between the LFA in-situ layer thickness and the height of the tested core.

The core UCS is the tested core strength of the 4-inch diameter cores. A direct comparison of core UCS test results cannot be made because of the variation in tested core heights. The Proctor UCS is the core UCS divided by a correction factor which normalizes the tested core strengths to the strength of a core having a 1.15:1 height to diameter ratio. This ratio corresponds to a standard four-inch diameter Proctor mold, thus the reference to a Proctor UCS. For example, the tested core UCS of 241 psi recorded for Station 290+00 in the Bolivar County project has an equivalent "Proctor UCS" of 255 psi. Standard four-inch diameter Proctor size specimens are used in LFA stabilized soil design. Normalization of Core UCS results to Proctor UCS values allow comparison of in-situ LFA UCS to the LFA laboratory design UCS of 500 psi used for base course layers.

The equation for the correction factor is based on studies that indicate that Proctor strengths are generally 30% higher than that of samples with a height-to-diameter ratio of 2:1, which is the standard ratio used for determination of material UCS (George, 2001). The following equation calculates a unique correction factor for each core of variable height by assuming linear interpolation between the ratios of 2:1 and 1.15:1:

$$\text{Correction Factor} = 0.77 + (0.27*(2 - H/D)) \quad \text{Equation 2}$$

Where: H/D = height-to-diameter ratio of the tested core

Note that no test location, given the typical 4-inch core diameter, design LFA pavement layer thicknesses of 6 and 8 inches, and reduction in core height due to trimming of the core ends, yielded testable cores having a 2:1 height to diameter ratio.

Note the strength recorded at many of the test locations is “795+” (psi). The UCS test device, illustrated in Appendix B, had an upper loading limit of about 10,000 pounds, which corresponds to 795 psi for the 4-inch diameter cores. Quite unexpectedly, the strength of many of the cores exceeded the loading capacity of this testing device. Despite this limitation, the use of this device was continued due to its capability to record both load and deformation readings. These readings were subsequently used to develop stress/strain plots for determination of Young’s Modulus of the LFA material. Young’s Modulus values are recorded in Tables 11 and 12 of Chapter 4.

The continued use of the UCS test device did not allow for calculation of either the average or coefficient of variation in in-situ LFA strength; however, the upper loading limit of this device did allow the applied stress to exceed the 500 psi LFA base design value. Table 10 includes the percent of FWD test locations that exceeded this design strength, both for each project and collectively for the newer and older pavements. For the Forrest/Perry project only the cored locations within the wheel path were included in the summaries. The in-situ strength of 41 percent of the LFA stabilized material in the newer pavements and 56 percent in the older pavements exceeded the design value. The in-situ strength of 21 percent of the LFA stabilized material in the newer pavements and 31 percent in the older pavements exceeded 795 psi. The greater percentages associated with the older pavements are attributed to the continuing strength gain of LFA stabilized material with time.

Table 10 shows that percent core retrieval is less in the older projects than in the newer projects. This is not surprising since coring was performed within the wheel path of these older pavements that had been subjected to years of traffic loading. Trafficinduced cracking rendered more of the base material unsuitable for UCS testing as compared to the newer pavements.

Figure 2. Drill Rig Used for Obtaining LFA Cores

Figure 3. Method Used to Extract LFA Cores from Pavement

Figure 4. Obtaining Rut Measurements

Figure 5. Example of Visual Classification 1

Figure 6. Example of Visual Classification 2

Figure 7. Example of Visual Classification 3

Figure 8. Example of Visual Classification 3—Grooves Cut in Core

Figure 9. Example of Visual Classification 4

Figure 10. Example of Visual Classification 5

Figure 11. Example of Visual Classification 5—Grooves Cut in Core

Figure 12. Example of Visual Classification 6

Figure 13. Stripped HMA as Potential Source of Stone for Grooves Cut in LFA Cores

Figure 14. Shop Wet Vacuum Used to Remove Stone from Top of LFA Layer Prior to Coring

Table 5. Pavement Condition and Layer Thickness Data for the Newer Projects

County	Station	Pavement Surface Cracking Severity/Type	Rut Depth Inside WP 1/16" Incr.	Rut Depth Outside WP 1/16" Incr.	Lane Location of FWD Testing	HMA Core Rating	HMA [a] Design Thickness (inches)	HMA In-Situ Thickness (inches)	LFA Base Design Thickness (inches)	LFA Base In-Situ Thickness (inches)	Granular [b] Subbase Thickness (inches)
Bolivar	290+00 SBOL	None	0	0	Center	3	8	7.75	6	9.5	0
	295+00 SBOL	None	0	0	Center	3	8	7	6	6	0
	300+00 SBOL	None	0	0	Center	3	8	7	6	5.75	0
	305+00 SBOL	None	0	0	Center	3	8	7.5	6	5.75	0
	335+00 SBOL	None	0	0	Center	3	8	8.5	6	6	0
	340+00 SBOL	None	0	0	Center	3	8	8	6	6.5	0
	345+00 SBOL	None	0	0	Center	3	8	9	6	6.5	0
	350+00 SBOL	None	0	0	Center	3	8	8	6	7.5	0
	713+00 NBOL	None	0	0	Center	3	8	8	6	7	0
	718+00 NBOL	None	0	0	Center	3	8	7.63	6	6.25	0
	723+00 NBOL	None	0	0	Center	3	8	8	6	7.5	0
	728+00 NBOL	None	0	0	Center	3	8	7.75	6	7.5	0
	733+00 NBOL	None	0	0	Center	3	8	8.5	6	6	0
	738+00 NBOL	None	0	0	Center	3	8	7.5	6	8	0
	743+00 NBOL	None	0	0	Center	3	8	7.5	6	5.5	0
	748+00 NBOL	None	0	0	Center	3	8	8	6	7.5	0

[a] Polymer modified HMA in surface and intermediate HMA lifts in Bolivar County project
[b] Design thickness

SBOL - southbound outside lane
NBOL - northbound outside lane
WP - wheel path

Table 5 Continued. Pavement Condition and Layer Thickness Data for the Newer Projects

County	Station	Pavement Surface Cracking Severity/Type	Rut Depth Inside WP 1/16" Incr.	Rut Depth Outside WP 1/16" Incr.	Lane Location of FWD Testing	HMA Core Rating	HMA Design Thickness (inches)	HMA In-Situ Thickness (inches)	LFA Design Thickness (inches)	LFA In-Situ Thickness (inches)	Granular [b] Subbase Thickness (inches)
Clarke	39+50 NBIL	None	0	0	Center	3	5.75	5	6	5.5	0
	40+00 NBIL	None	1	2	Center	3	5.75	5.13	6	6.63	0
	40+50 NBIL	None	0	3	Center	3	5.75	5.25	6	6.75	0
	41+00 NBIL	None	2	1	Center	3	5.75	5.25	6	6	0
	41+50 NBIL	None	2	1	Center	3	5.75	5.25	6	6	0
	42+00 NBIL	None	1	1	Center	3	5.75	5.25	6	5.5	0
	42+50 NBIL	None	2	1	Center	3,10	5.75	5.75	6	5.75	0
	43+00 NBIL	None	3	2	Center	3,10	5.75	6	6	6	0
	752+00 NBOL	None	0	0	Center	3	5.75	5	6	5.75	0
	755+00 NBOL	None	0	0	Center	3	5.75	4.5	6	5	0
	758+00 NBOL	None	0	0	Center	3	5.75	4.5	6	6	0
	761+00 NBOL	None	1	1	Center	3	5.75	6.25	6	6.5	0
	764+00 NBOL	None	0	1	Center	3	5.75	5.25	6	5.5	0
	767+00 NBOL	None	1	1	Center	3,10	5.75	5.25	6	6	0
	770+00 NBOL	None	0	1	Center	3,10	5.75	6.75	6	6.5	0
	773+00 NBOL	None	3	1	Center	3,10	5.75	6.75	6	7	0

NBIL - northbound inside lane
NBOL - northbound outside lane
WP - wheel path

Table 5 Continued. Pavement Condition and Layer Thickness Data for the Newer Projects

County	Station	Pavement Surface Cracking Severity/Type	Rut Depth Inside WP 1/16" Incr.	Rut Depth Outside WP 1/16" Incr.	Lane Location of FWD Testing	HMA Core Rating	HMA Design Thickness (inches)	HMA In-Situ Thickness (inches)	LFA Design Thickness (inches)	LFA In-Situ Thickness (inches)	Granular[b] Subbase Thickness (inches)
Smith	493+00 NB	None	0	1	Center	3,10,99	7.25	7.75	6	4.75	0
	498+00 NB	None	0	0	Center	3	7.25	7.25	6	4.5	0
	503+00 NB	None	0	0	Center	3	7.25	7.25	6	6	0
	508+00 NB	None	0	0	Center	3	7.25	7	6	6.75	0
	518+00 NB	None	0	1	Center	3,10	7.25	6.875	6	5.5	0
	522+00 NB	None	0	2	Center	3,10	7.25	7	6	6.5	0
	528+00 NB	None	3	2	Center	3,10	7.25	7.5	6	6	0
	610+00 SB	None	2	1	Center	3	7.25	7.375	6	7.125	0
	613+00 SB	None	0	0	Center	3	7.25	7.75	6	6.25	0
	616+00 SB	Low/Longitudinal	1	1	Center	3	7.25	8.25	6	7.25	0
	619+00 SB	None	1	1	Center	3	7.25	7.5	6	7.75	0
	622+00 SB	None	1	1	Center	3	7.25	7.5	6	5.5	0
	625+00 SB	None	0	0	Center	7,10	7.25	7.25	6	4.75	0
	628+00 SB	None	0	1	Center	7,10	7.25	8.75	6	6	0
	631+00 SB	None	0	0	Center	7,10	7.25	7.25	6	6	0

NB - northbound
SB - southbound
WP - wheel path

Table 5 Continued. Pavement Condition and Layer Thickness Data for the Newer Projects

County	Station	Pavement Surface Cracking Severity/Type	Rut Depth Inside WP 1/16" Incr.	Rut Depth Outside WP 1/16" Incr.	Lane Location of FWD Testing	HMA Core Rating	HMA [a] Design Thickness (inches)	HMA In-Situ Thickness (inches)	LFA Design Thickness (inches)	LFA In-Situ Thickness (inches)	Granular [b] Subbase Thickness (inches)
Tippah	163+00 WBOL	None	0	0	Center	3	7.5	7.5	8	6.5	0
	167+00 WBOL	None	0	0	Center	3	7.5	6.75	8	7	0
	171+00 WBOL	None	0	0	Center	3	7.5	7.5	8	8.5	0
	175+00 WBOL	None	0	0	Center	3	7.5	7.5	8	7.25	0
	179+00 WBOL	None	0	0	Center	3	7.5	8.5	8	7	0
	183+00 WBOL	None	0	0	Center	3	7.5	7	8	8.25	0
	187+00 WBOL	None	0	0	Center	3	7.5	7.75	8	7.25	0
	191+00 WBOL	None	0	0	Center	3	7.5	7.5	8	7	0
Wilkinson [c]	164+00 SBOL	Low/Fatigue	0	0	Center	3	7.5	5.5	6	5.5	0
	169+00 SBOL	None	0	0	Center	3	7.5	4.8	6	7	0
	174+00 SBOL	None	0	0	Center	3	7.5	5	6	5.3	0
	179+00 SBOL	None	0	0	Center	3	7.5	5.5	6	9	0
	184+00 SBOL	None	0	0	Center	3	7.5	5	6	4.3	0
	189+00 SBOL	None	0	0	Center	3	7.5	6	6	4.3	0
	195+00 SBOL	None	1	0	Center	3	7.5	6	6	5.5	0
	200+00 SBOL	None	0	0	Center	3	7.5	6	6	6.5	0

[a] Polymer modified HMA in surface and intermediate HMA lifts for both Tippah and Wilkinson County projects
[c] 2" Surface course not placed yet at time of coring in Wilkinson County

WBOL - westbound outside lane
SBOL - southbound outside lane
WP - wheel path

Table 6. Pavement Condition and Layer Thickness Data for the Older Projects

County	Station	Pavement Surface Cracking Severity/Type	Rut Depth Inside WP 1/16" Incr.	Rut Depth Outside WP 1/16" Incr.	Lane Location of FWD Testing	HMA Core Rating	HMA Design Thickness (inches)	HMA [b] In-Situ Thickness (inches)	LFA Base Design Thickness (inches)	LFA Base In-Situ Thickness (inches)	Granular [a] Subbase Thickness (inches)
Forrest/Perry	288+00 EBOL	Low/Transverse	1	1	OSWP	3	4.5	7	6	5	4
	293+00 EBOL	Low/Transverse	2	1	OSWP	3	4.5	5.5	6	5.5	3.5
	298+00 EBOL	Low/Transverse	1	1	OSWP	3	4.5	5.5	6	5	4
	303+00 EBOL	Low/Transverse	2	3	OSWP	4,10	4.5	7.5	6	5	4
	330+00 EBOL	Low/Transverse	2	1	OSWP	3	4.5	6	6	6	3
	335+00 EBOL	Low/Transverse	1	1	OSWP	3	4.5	5.5	6	6	3
	340+00 EBOL	Low/Transverse	1	1	OSWP	3	4.5	6.25	6	5	4
	345+00 EBOL	Low/Longitudinal	1	3	OSWP	3	4.5	6.25	6	6	3
	516+00 EBOL	Low/Longitudinal	3	6	Center	3	4.5	6	6	6.25	2.75
	519+00 EBOL	Low/Longitudinal	7	8	Center	4,10	4.5	6.5	6	5.5	3.5
	524+00 EBOL	Low/Longitudinal	3	4	Center	3	4.5	5.5	6	6.25	2.75
	528+00 EBOL	Low/Trans.&Long.	2	4	Center	3	4.5	5.5	6	6.5	2.5
	530+00 EBOL	Low/Longitudinal	3	3	Center	3	4.5	5.25	6	6	3
	535+00 EBOL	Low/Longitudinal	6	6	Center	3	4.5	6	6	5	4
	540+00 EBOL	Low/Longitudinal	3	3	Center	3	4.5	5.5	6	5	4
	545+00 EBOL	Low/Longitudinal	0	3	Center	3	4.5	5.5	6	5	4

[a] 9" Design thickness of granular material less LFA base thickness

[b] HMA In-Situ thickness exceeds design thickness. The construction plans required varying thicknesses of HMA for segments of the roadway. The segments of road containing the test sections required 4.5" of HMA, but abutting segments required 6". It is assumed that as a construction expediency, the 1.5" additional HMA was included throughout the segments of road containing the test sections.

EBOL - eastbound outside lane
OSWP - outside wheel path
WP - wheel path

Table 6 Continued. Pavement Condition and Layer Thickness Data for the Older Projects

County	Station	Pavement Surface Cracking Severity/Type	Rut Depth Inside WP 1/16" Incr.	Rut Depth Outside WP 1/16" Incr.	Lane Location of FWD Testing	HMA Core Rating	HMA Design Thickness (inches)	HMA In-Situ Thickness (inches)	LFA Base Design Thickness (inches)	LFA Base In-Situ Thickness (inches)	Granular [c] Subbase Thickness (inches)
George	84+96 EBOL	Low/Longitudinal	6	2	OSWP	3,10	4.5	4.5	6	5.5	12.5
	124+04 EBOL	Low/Longitudinal	5	5	OSWP	3	4.5	4.5	6	5.5	12.5
	242+45 EBOL	Low/Fatigue	5	6	OSWP	19	4.5	4	6	5	13
	272+44 EBOL	None	9	4	OSWP	3	4.5	5.25	6	5.5	12.5
	315+00 EBOL	None	3	4	OSWP	3	4.5	5	6	4.25	13.75
	351+36 EBOL	Low/Transverse	2	2	OSWP	19	4.5	4	6	4	14
	390+21 EBOL	Low/Longitudinal	7	5	OSWP	3	4.5	3.5	6	3.75	14.25
	424+57 EBOL	Low/Longitudinal	6	5	OSWP	3,10	4.5	4	6	4.25	13.75

[c] 18" Design thickness of granular material less LFA base thickness

EBOL - eastbound outside lane
OSWP - outside wheel path
WP - wheel path

Table 6 Continued. Pavement Condition and Layer Thickness Data for the Older Projects

County	Station	Pavement Surface Cracking Severity/Type	Rut Depth Inside WP 1/16" Incr.	Rut Depth Outside WP 1/16" Incr.	Lane Location of FWD Testing	HMA Core Rating	HMA Design Thickness (inches)	HMA In-Situ Thickness (inches)	LFA Base Design Thickness (inches)	LFA Base In-Situ Thickness (inches)	Granular [d] Subbase Thickness (inches)
Jones/Wayne	102+00 EBOL	Low/Transverse	1	2	OSWP	3	6	6.5	7	8.25	6.25
	107+00 EBOL	None	3	2	OSWP	3	6	6	7	7.5	7
	112+00 EBOL	Low/Longitudinal	3	5	OSWP	1	6	5.5	7	8	6.5
	169+00 EBOL	Low/Longitudinal	1	3	OSWP	3	4.5	5.25	6	6.25	9.75
	170+00 EBOL	Low/Fatigue	1	2	OSWP	3	4.5	5.25	6	6	10
	171+00 EBOL	Low/Transverse	1	4	OSWP	3	4.5	4.5	6	6.5	9.5
	172+00 EBOL	Low/Fatigue	1	3	OSWP	3	4.5	4	6	6.5	9.5
	376+00 EBOL	Low/Longitudinal	3	2	OSWP	3	6	6.75	7	6	8.5
	381+00 EBOL	None	2	2	OSWP	3	6	6	7	6	8.5
	386+00 EBOL	None	3	1	OSWP	3	6	6	7	6	8.5
	411+00 EBOL	Low/Fatigue	1	1	OSWP	3	6	5.75	7	6	8.5
	416+00 EBOL	Low/Fatigue	3	2	OSWP	3	6	5	7	6.75	7.75
	421+00 EBOL	Low/Fatigue	2	1	OSWP	3	6	5.5	7	7	7.5
	456+00 EBOL	Low/Transverse	3	2	OSWP	3	6	6.63	7	7	7.5
	461+00 EBOL	Low/Longitudinal	4	4	OSWP	3	6	5.5	7	7.5	7
	466+00 EBOL	None	5	5	OSWP	1	6	6.25	7	7.25	7.25

[d] 18" Design thickness of granular material less LFA base thickness

EBOL - eastbound outside lane
OSWP - outside wheel path
WP - wheel path

Table 6 Continued. Pavement Condition and Layer Thickness Data for the Older Projects

County	Station	Pavement Surface Cracking Severity/Type	Rut Depth Inside WP 1/16" Incr.	Rut Depth Outside WP 1/16" Incr.	Lane Location of FWD Testing	HMA Core Rating	HMA Design Thickness (inches)	HMA In-Situ Thickness (inches)	LFA Base Design Thickness (inches)	LFA Base In-Situ Thickness (inches)	Granular [e] Subbase Thickness (inches)
Yalobusha	340+04 NB	Low/Block	2	2	6' off c/l	3	4.5	4.75	6	6	0
	341+96 SB	Low/Block	3	3	7' off c/l	3	4.5	4	6	5.5	0
	360+04 NB	Low/Block	5	3	6' off c/l	3	4.5	4.5	6	5.8	0
	361+96 SB	Low/Block	3	3	8' off c/l	3	4.5	4	6	7.5	0
	402+04 NB	Low/Block	2	3	6' off c/l	3	4.5	4.5	6	7.5	0
	403+96 SB	Low/Block	3	4	6' off c/l	3	4.5	3.5	6	6	0
	420+04 NB	Low/Block	3	3	6' off c/l	3	4.5	4.25	6	6	0
	421+96 SB	Low/Block	3	4	6' off c/l	3	4.5	5	6	6.8	0
	455+04 NB	Med/Fatigue	4	4	6' off c/l	3	4.5	5	6	8	0
	456+96 SB	Low/Block	4	6	6' off c/l	3	4.5	4.5	6	9	0
	488+04 NB	Med/Fatigue	3	2	6' off c/l	3	4.5	4	6	7.8	0
	488+96 SB	Med/Fatigue	3	3	6' off c/l	3	4.5	4.25	6	7	0
	492+04 NB	Low/Block	2	3	6' off c/l	3	4.5	4.75	6	8	0
	493+96 SB	Low/Block	2	4	6' off c/l	3	4.5	4	6	7	0
	507+04 NB	Low/Block	1	3	6' off c/l	3	4.5	4.5	6	7.8	0
	508+96 SB	Low/Block	4	3	6' off c/l	3	4.5	4.5	6	8	0

[e] Shown on plans but not considered part of the pavement structure when calculating LFA structural layer coefficients

NB - northbound
SB - southbound
WP - wheel path

Table 7. LFA Visual Information and Tested Core Data for the Newer Projects

County	Coring Location Station	Month/Year LFA Placed	Field LFA Core Rating	LFA In-situ Thickness (in.)	Tested Core Height (in.)	Core Density (lbs./ft^3)	Percent Proctor Density (%)	Moisture Content after 48 hr. Soak, (%)	Core UCS (psi)	Proctor UCS (psi)
Bolivar	290+00	Oct-98	1*	9.5	5.34	100.8	88.7	22.2	241	255
	295+00	Oct-98	1*	6	4.79	104.3	91.8	20.6	601	609
	299+98	Oct-98	5							
	300+00	Oct-98	4*							
	305+00	Oct-98	1*	5.75	4.36	112.9	99.3	16.0	795+	795+
	335+00	Oct-98	3*	6	4.81	105.2	89.6	19.0	384	390
	339+98	Oct-98	3	6	4.02	105.0	89.4	19.6	161	155
	340+00	Oct-98	5*							
	345+00	Oct-98	1*	6.5	5.66	112.5	95.8	16.8	795+	795+
	350+00	Oct-98	1*	7.5	6.42	114.1	97.2	14.3	795+	795+
	713+00	May-99	1*	7	5.64	113.7	97.3	14.9	453	490
	718+00	May-99	1*	6.25	4.95	104.0	89.1	19.5	506	521
	723+00	May-99	2*	7.5	4.66	105.1	90.0	19.0	574	579
	728+00	May-99	1*	7.5	5.06	109.9	94.1	16.6	661	685
	733+00	May-99	4*							
	733+02	May-99	2	6	4.33	109.7	93.7	16.3	462	455
	738+00	May-99	2*	8	5.57	110.5	94.4	16.4	623	668
	743+00	May-99	2*	5.5	4.16	99.7	85.1	23.1	368	359
	748+00	May-99	5*							
	748+02	May-99	3	5.5	3.97	107.8	92.0	17.4	492	473
Clarke	39+50	Mar-99	1*	5.5	4.73	101.3	88.4	19.5	795+	795+
	40+00	Mar-99	1*	6.63	6.01	103.0	89.9	18.5	796	884
	40+50	Mar-99	1*	6.75	5.66	106.4	92.9	16.3	795+	795+
	41+00	Mar-99	5*							
	41+02	Mar-99	3	6	4.27	108.2	94.5	16.0	795+	795+
	41+50	Mar-99	1*	6	5.25	112.5	98.2	14.3	795+	795+
	42+00	Mar-99	1*	[a]						
	42+02	Mar-99	1	5.5	4.65	111.9	97.8	14.2	725	728
	42+50	Mar-99	1*	5.75 [a]	5.30	112.6	98.3	14.0	779	818
	43+00	Mar-99	1*	6	4.92	107.4	93.8	15.9	259	265
	752+00	Mar-99	1*	5.75	5.00	105.5	na	18.1	196	202
	755+00	Mar-99	1*	5	4.36	104.4	na	18.6	408	403
	758+00	Nov-98	1*	6	5.10	104.4	na	18.4	275	285
	761+00	Nov-98	1*	6.5	5.37	102.9	na	19.0	172	182
	764+00	Nov-98	1*	5.5	4.93	99.0	na	21.2	372	382
	767+00	Nov-98	1*	[a]						
	767+02	Nov-98	1	[a]	4.05	104.6	na	18.1	432	417
	767+04	Nov-98	1	[a]						
	770+00	Nov-98	1*	6.5	5.28	103.2	na	19.4	451	474
	773+00	Nov-98	1*	7	5.94	105.8	na	17.7	122	135

* FWD test location

[a] HMA and LFA cores were extracted from the bore hole as one unit due to good bonding between layers

Table 7 Continued. LFA Visual Information and Tested Core Data for the Newer Projects

County	Coring Location Station	Month/Year LFA Placed	Field LFA Core Rating	LFA In-situ Thickness (in.)	Tested Core Height (in.)	Core Density (lbs./ft^3)	Percent Proctor Density (%)	Moisture Content after 48 hr. Soak, (%)	Core UCS (psi)	Proctor UCS (psi)
Smith	493+00	Nov-98	1*	4.75	5.26**				795+	795+
	493+01	Nov-98	1		4.06	112.7	96.3	15.4	795+	795+
	498+00	Nov-98	1*	4.5	4.65**				718	723
	498+01	Nov-98	1		4.07	114.0	97.5	14.6	795+	795+
	503+00	Nov-98	1*	6	5.67**				684	741
	503+01	Nov-98	1		4.69	107.0	91.4	17.5	781	788
	508+00	Nov-98	1*	6.75	6.61**				289	337
	508+01	Nov-98	1		4.93	110.4	95.4	15.5	312	323
	518+00	Nov-98	1*		5.47**				728	776
	517+99	Nov-98	1		4.56	108.9	94.1	17.5	795+	795+
	522+00	Nov-98	1*	6.5	5.85**				358	392
	521+99	Nov-98	1		4.72	111.3	96.2	15.9	575	585
	528+00	Nov-98	1*	6	5.94**				298	330
	528+01	Nov-98	1		5.27	106.2	91.8	19.2	385	408
	609+96	Jun-99	1	6.25	5.24**				456	479
	609+99	Jun-99	4							
	610+00	Jun-99	4*							
	610+01	Jun-99	1		5.16	97.8	83.4	20.9	294	308
	613+00	Jun-99	1*	6.25	6.17**				209	238
	613+01	Jun-99	1		5.06	106.1	90.5	16.7	231	242
	615+98	Jun-99	3	6	4.74**				85	87
	616+00	Jun-99	5*							
	616+01	Jun-99	3		4.57	105.2	89.7	17.9	130	131
	619+00	Jun-99	1*	7.75	6.74**				630	777
	619+01	Jun-99	1		4.37	104.7	89.2	18.0	625	620
	622+00	Jul-99	2*	5.5	5.12**				254	264
	622+01	Jul-99	1		4.69	115.1	98.1	13.9	471	477
	625+00	Jul-99	5*							
	625+01	Jul-99	3		4.08	106.4	90.7	18.4	357	347
	628+00	Jul-99	6*							
	628+01	Jul-99	6		4.07	100.5	85.7	21.8	103	100
	630+99	Jul-99	6							
	631+00	Jul-99	6*							

* FWD test location
** Relatively thick caps used on these cores (approximately 1/4" thick)

Table 7 Continued. LFA Visual Information and Tested Core Data for the Newer Projects

County	Coring Location Station	Month/Year LFA Placed	Field LFA Core Rating	LFA In-situ Thickness (in.)	Tested Core Height (in.)	Core Density (lbs./ft^3)	Percent Proctor Density (%)	Moisture Content after 48 hr. Soak, (%)	Core UCS (psi)	Proctor UCS (psi)
Tippah	162+98	Jul-98	5							
	163+00	Jul-98	3*	6.5	4.07	109.4	na	17.0	427	413
	166+98	Jul-98	4							
	167+00	Jul-98	4*							
	170+98	Jul-98	3	8.5	4.09	108.3	na	16.4	390	378
	171+00	Jul-98	5*							
	175+00	Jul-98	1*	7.25	5.87	108.7	na	15.9	302	333
	179+00	Jul-98	1*	7	4.76	107.6	na	15.3	166	169
	182+98	Jul-98	1	8	5.45	103.6	na	18.6	249	266
	183+00	Jul-98	4*							
	186+98	Jul-98	4							
	187+00	Jul-98	5*							
	191+00	Jul-98	1*	7	5.43	11.9	na	15.2	795+	795+
Wilkinson	164+00	Oct-98	1*	5.5	4.08	113.5	94.3	15.1	759	735
	168+98	Oct-98	3	7.3	4.21	108.1	89.8	17.2	421	412
	169+00	Oct-98	5*							
	174+00	Oct-98	3*	5.3	4.08	109.4	91.0	17.3	629	609
	179+00	Oct-98	3*	9	4.22	111.3	92.6	16.0	795+	795+
	184+00	Oct-98	1*	4.3	4.01	112.4	93.5	15.2	478	462
	189+00	Oct-98	1*	4.3	4.08	104.6	87.0	17.6	329	319
	195+00	Oct-98	1*	5.5	4.37	119.9	99.7	12.1	795+	795+
	199+98	Oct-98	2	7.3	4.06	104.7	87.0	19.7	427	413
	200+00	Oct-98	5*							

* FWD test location

Table 8. LFA Visual Information and Tested Core Data for the Older Projects

County	Coring Location Station	Month/Year LFA Placed	Field LFA Core Rating	LFA In-situ Thickness (in.)	Tested Core Height (in.)	Core Density (lbs./ft^3)	Percent Proctor Density (%)	Moisture Content after 48 hr. Soak, (%)	Core UCS (psi)	Proctor UCS (psi)
Forrest/Perry	288+00	Dec-89	1*	5	4.23	111.1	N/A	16.2	795+	795+
	293+00	Dec-89	1*	5.5	4.92	114.4	N/A	14.7	795+	795+
	298+00	Dec-89	1*	5	4.21	119.2	N/A	13.1	795+	795+
	303+00	Dec-89	1*	5	4.23	114.1	N/A	15.1	795+	795+
	330+00	Dec-89	5*							
	330+01	Dec-89	5							
	330+02	Dec-89	5							
	330+03	Dec-89	5	[a]						
	335+00	Dec-89	2*	6	4.28	110.3	N/A	15.8	511	502
	340+00	Dec-89	5*							
	340+01	Dec-89	5							
	345+00	Dec-89	3*	6	4.45	111.4	N/A	16.2	795+	795+
	516+00	Apr-90	4							
	516+01	Apr-90	1	6	4.83	108.5	N/A	17.3	795+	795+
	519+00	Apr-90	4							
	519+01	Apr-90	1	5.5	4.74	109.7	N/A	17.4	795+	795+
	524+00	Apr-90	4							
	524+01	Apr-90	2	7	4.27	111.2	N/A	16.1	779	763
	528+00	Apr-90	1	6.5	4.73	109.6	N/A	16.9	795+	795+
	530+00	Apr-90	1	6	4.42	109.4	N/A	16.3	438	435
	535+00	Apr-90	1	5	4.06	104.4	N/A	18.8	646	624
	540+00	Apr-90	4							
	540+01	Apr-90	4							
	540+02	Apr-90	4							
	540+03	Apr-90	1	5.25 [b]	4.28	107.7	N/A	18.4	510	500
	545+00	Apr-90	4							
	545+01	Apr-90	1	5.25 [b]	4.49	106.9	N/A	18.5	806	804

[a] Coring attempt made between wheel paths instead of in outside wheel path
[b] Core obtained from outside wheel path instead of between wheel paths
* FWD test location

Table 8 Continued. LFA Visual Information and Tested Core Data for the Older Projects

County	Coring Location Station	Month/Year LFA Placed	Field LFA Core Rating	LFA In-situ Thickness (in.)	Tested Core Height (in.)	Core Density (lbs./ft^3)	Percent Proctor Density (%)	Moisture Content after 48 hr. Soak, (%)	Core UCS (psi)	Proctor UCS (psi)
George	84+96	May-92	4*							
	84+98	May-92	1	6	4.42	106.6	N/A	17.3	736	728
	124+04	Jun-92	1*	5.5	4.43	109.0	N/A	17.1	795+	795+
	242+45	Jun-92	5*							
	242+47	Jun-92	5							
	272+44	Jun-92	1*	5.5	4.47	119.2	N/A	11.2	625	620
	315+00	Apr-92	1*	4.25	4.04	96.3	N/A	23.4	489	472
	351+36	Apr-92	4*							
	351+38	Apr-92	2	4	4.12	111.5	N/A	15.1	312	302
	390+21	May-92	5*							
	390+23	May-92	5							
	424+57	May-92	4*							
	424+59	May-92	5							
Jones/Wayne	102+00	Sep-92	1*	8.25	4.94	112.4	N/A	14.6	795+	795+
	107+00	Sep-92	1*	7.5	5.69	114.8	N/A	14.5	795+	795+
	111+98	Sep-92	1	8	5.60	117.9	N/A	12.2	215	231
	112+00	Sep-92	4*							
	112+02	Sep-92	4							
	169+00	Sep-92	2*	6.25	3.97	115.2	N/A	13.1	354	340
	170+00	Sep-92	4*							
	170+02	Sep-92	2	6	4.11	117.7	N/A	12.2	306	297
	171+00	Sep-92	1*	[c]						
	172+00	Sep-92	1*	6.5	4.75	114.7	N/A	15.1	795+	795+
	376+00	Jul-92	1*	6	4.93	115.1	N/A	14.2	795+	795+
	381+00	Jul-92	4*							
	381+02	Jul-92	1	6	4.80	115.2	N/A	14.4	630	638
	386+00	Jul-92	1*	6	4.89	120.1	N/A	12.6	795+	795+
	411+00	Jul-92	1*	6	4.63	117.6	N/A	13.9	795+	795+
	416+00	Jul-92	2*	6.75	4.11	111.6	N/A	15.8	451	439
	421+00	Jul-92	4*							
	421+02	Jul-92	1	6.25	5.02	116.2	N/A	14.5	795+	795+
	456+00	Jul-92	1*	7	5.93	116.5	N/A	13.6	675	746
	461+00	Jul-92	4*							
	461+02	Jul-92	4							
	461+04	Jul-92	4							
	466+00	Jul-92	1*	7.25	4.95	117.3	N/A	13.9	602	619

[c] Core not tested due to omission during UCS testing
* FWD test location

Table 8 Continued. LFA Visual Information and Tested Core Data for the Older Projects

County	Coring Location Station	Month/Year LFA Placed	Field LFA Core Rating	LFA In-situ Thickness (in.)	Tested Core Height (in.)	Core Density (lbs./ft^3)	Percent Proctor Density (%)	Moisture Content after 48 hr. Soak, (%)	Core UCS (psi)	Proctor UCS (psi)
Yalobusha	340+04	Sep-89	1*	6	5.05	108.2	94.4	16.1	467	484
	341+94	Sep-89	4							
	341+96	Sep-89	4*							
	360+04	Sep-89	1*	5.8	4.56	112.8	98.4	14.9	795+	795+
	361+96	Sep-89	1*	7.5	5.45	110.3	96.2	15.0	606	646
	402+04	Sep-89	1*	7.5	6.11	105.3	90.6	18.4	311	348
	403+96	Sep-89	1*	6	4.88	111.5	95.9	14.8	727	744
	420+04	Sep-89	1*	6	4.42	108.2	95.1	17.1	670	664
	421+96	Sep-89	1*	6.8	5.35	107.6	94.5	15.9	609	644
	455+04	Oct-89	1*	8	6.53	112.8	96.8	15.0	795+	795+
	456+96	Oct-89	1*	9	7.74	110.5	94.9	14.8	527	675
	488+04	Oct-89	4*							
	488+06	Oct-89	1	8	6.80	105.4	90.5	17.1	472	559
	488+96	Oct-89	1*	7	5.85	107.6	92.4	16.4	396	435
	492+04	Oct-89	1*	8	6.03	103.1	91.1	18.1	408	453
	493+96	Oct-89	1*	7	5.95	108.6	95.9	16.2	380	421
	507+04	Oct-89	4*							
	507+06	Oct-89	1	7.8	5.47	102.6	90.7	18.8	375	401
	508+96	Oct-89	1*	8	6.49	110.7	97.8	14.6	392	453

* FWD test location

Table 9. Summary of Visual Examination of LFA Cores

	Number of Test Locations	Visual Classification					
		1	2	3	4	5	6
				Newer Projects			
Bolivar	16	8	3	1	2	2	
Clarke	16	15				1	
Smith	15	9	1		1	2	2
Tippah	8	3		1	2	2	
Wilkinson	8	4		2		2	
Summary	63	39	4	4	5	9	2
% of Cores	100	**62**	**6**	**6**	**8**	**14**	**3**
				Older Projects			
Forrest/Perry	7	4		1		2	
George	8	3			3	2	
Jones/Wayne	16	9	2		5		
Yalobusha	16	13			3		
Summary	47	29	2	1	11	4	
% of Cores	100	**62**	**4**	**2**	**23**	**9**	**0**

1. Intact core. Cracks may be present, but core is intact and UCS testing performed on core with the cracks.

2. Cracks on ends or sides of core, but 4.5" sample can be saw cut from core for UCS testing. Broken pieces and fracture faces appear to have experienced minimal degradation during the coring operation, suggesting that the LFA material has a relatively high degree of cementation.

3. Core is missing significant portions from its original LFA layer thickness, but a 4.5 " tall specimen can be saw cut from it for laboratory testing. The missing portions could be due to either grooves being cut from the sides of the core during the coring operation or pieces of the core that disintegrated during the coring operation. Broken pieces and fracture faces appear to have experienced degradation during the coring operation, suggesting a relatively low degree of cementation of the LFA material.

4. Badly cracked or damaged core from which a 4.5" test specimen cannot be saw cut for UCS testing. Broken pieces and fracture faces appeared to have experienced minimal degradation during the coring operation, suggesting that the LFA material has a relatively high degree of cementation.

5. Core is missing significant portions, and a 4.5" sample cannot be saw cut for UCS testing. The missing portions could be due to either grooves being cut from the sides of the core during the coring operation or pieces of the core that disintegrated during the coring operation. Broken pieces and fracture faces appear to have experienced degradation during the coring operation, suggesting a relatively low degree of cementing of the LFA material.

6. LFA material extracted from core hole is soft and can be crumbled between thumb and fingers.

Table 10. Summary Statistics from LFA Coring Operation and UCS Testing

County	Number of Stations Coring Attempted	Number of Stations Core Obtained	Percent Retrieval	Number of Cores >/=500psi	% Core Locations >/=500psi	Number of Cores >795+ psi	% Core Locations >795+ psi	Number of Construction Seasons[a] LFA Cured at Time of Coring
				Newer pavements				
Bolivar	16	15	94	8	50	3	19	2
Clarke	16	16	100	7	44	4	25	2
Smith	15	14	93	6	40	3	20	1.5 - 2
Tippah	8	6	75	1	13	1	13	2.5
Wilkinson	8	8	100	4	50	2	25	2
Summary	63	59	94	26	41	13	21	
				Older Pavements				
Forrest & Perry	8	6	75	6	75	5	63	11
George	8	5	63	3	38	1	13	8.5
Jones & Wayne	16	13	81	10	63	7	44	8 - 8.5
Yalobusha	16	15	94	8	50	2	13	11
Summary	48	39	81	27	56	15	31	

[a] LFA construction season is from March 1st to November 30th

Chapter 4 -- FWD Field Testing and Backcalculation of Pavement Layer Moduli

A total of 119 falling weight deflectometer (FWD) tests were conducted at the nine project sites. The resulting deflection bowls were analyzed by two different computational procedures. One of these procedures, addressed in Chapter 5, is included in the 1993 edition of the AASHTO Guide for Design of Pavement Structures to determine the effective structure number of an existing flexible pavement. The other procedure, included in the current chapter, is the backcalculation of pavement layer moduli.

A Dynatest 8000 series trailer-mounted FWD was used to perform the FWD testing (Figure 15). Two approximately 9000-lb. drops and one approximately 12,000-lb. drop were applied to the pavement at each FWD test location. The loading magnitude of the second drop was used in the backcalculation routine. Seven deflection sensors spaced at 12-inch intervals were used to define the deflection basin of each test location.

The modulus of HMA is temperature dependent. For each project one mid-depth HMA pavement layer temperature was obtained at the time of FWD testing. A hole was drilled, filled with oil and a temperature probe inserted into it to record the temperature (Figure 16). These temperatures were used to evaluate the reasonableness of the backcalculated HMA moduli values and for correction of these values to 68 ^{0}F. Chapter 5 includes a discussion of the requirement, and method, for correcting HMA moduli values for temperature. Appendices C and D include the date FWD testing was performed and the corresponding temperature of the HMA layers for the newer and older projects respectively. Note that, except for the Forrest/Perry project, all of the projects were tested in January and the mid-depth HMA temperatures were between 46 and 49

^{0}F. The mid-depth HMA layer temperature of the Forrest/Perry project was 65.6 ^{0}F with testing performed at the end of February.

The Modulus 5.1 computer program was used to backcalculate the pavement layer moduli at the test locations. While running each backcalculation routine, all seven sensors were weighted the same value for each test location. Poisson's ratios were set at 0.35, 0.30, and 0.40 respectively for the HMA, LFA stabilized material and unstabilized subgrade.

The Modulus program uses a ratio, E4/E5, to calculate the "Depth to Stiff Layer". This program was developed in Texas where the depth to bedrock is often at a shallow depth; therefore, the program uses a default value of 1/100 for this ratio. The depth to a stiff layer is typically much greater in Mississippi than in Texas; therefore, this ratio is changed to 1/5 each time the routine is run for a Mississippi location (Johnson, 2000).

One of the criteria used in the selection of the nine projects was that the pavement structure at each of them can be characterized as a three-layer system; e.g., HMA, LFA stabilized soil base and unstabilized subgrade for the purpose of backcalculation of pavement layer moduli. None of these projects included a chemically stabilized subgrade layer. Three of the four older pavements included an untreated granular subbase layer, which was considered as part of the untreated subgrade in the backcalculation routines. This project selection criterion was used because the focus of the study is the evaluation of the LFA base course, and a minimum of pavement layers increases the accuracy of any assessments made for this pavement layer.

The thickness of the HMA and LFA stabilized layers varied from one FWD test location to another at each of the nine project sites. This fact is not surprising given the nature of the construction practices employed to build roads, but does cause problems when performing backcalculation routines since the output is sensitive to pavement layer thickness. To rectify this situation, the thickness of both the HMA and the LFA stabilized layers, the load, and the deflection of all seven sensors defining the deflection basin, were manually entered into a separate file for each test location. This enabled the determination of pavement layer moduli for each FWD test location from corresponding unique location pavement layer input data. The HMA and LFA stabilized layer thicknesses are recorded in Chapter 3, Tables 7 and 8. Appendices C and D include load and deflection input data for each of the FWD test locations.

Tables 11 and 12 include the results of the backcalculation computations for each FWD test location of the newer and older projects respectively. The HMA moduli values have not been corrected for temperature in these tables. In the "Notes" column are the criteria used for rejecting the backcalculated moduli data of a given location. The data was first reviewed for HMA moduli values exceeding 2500 ksi or the LFA modulus value exceeding the HMA modulus value at a given test location. After removing the data corresponding to these two criteria, the remaining data for each project was evaluated using Chauvenet's criterion for rejecting a data point (Coleman and Steele, 1989).

For the five newer projects the average backcalculated modulus was 423.6 ksi with a standard deviation of 306.1 ksi and corresponding coefficient of variation of 72.3 percent. For the four older projects the average backcalculated modulus was 169.5 ksi with a standard deviation of 114.8 ksi and corresponding coefficient of variation of 67.7 percent. The decrease in the average modulus value between the newer and older pavements is not surprising considering that the FWD test procedure actually measures an effective modulus of a given pavement layer. With time the pavement layer in question cracks due to traffic and environmental affects which reduce the stiffness or modulus of the layer. The coefficient of variation for both the newer and older pavements is similar in magnitude and indicates significant variation in this material property. Some of this excessive variation can be attributed to the use of a backcalculation routine for determining moduli values; however, even with this consideration, the in-situ LFA moduli values are extremely variable.

The LFA Young's modulus value is shown adjacent to the LFA backcalculated modulus value for each FWD station from which a testable LFA core was extracted for testing and a slope determined from the corresponding stress/strain curve. These Young's moduli values have not been normalized for any core height-to-diameter ratio as was done for the UCS values. At many of the test locations a significant difference exists between the backcalculated and Young's moduli values. Possible explanations include the following: (1) the actual location from which the core was retrieved may be 2 feet from the impact point of the FWD, (2) the volume of material tested is different between the two test procedures, and (3) possible degradation of the LFA material due to the coring operation.

The average Young's modulus was 161.8 ksi and 235.6 ksi for the newer and older projects respectively. This increase in average moduli values with time is not surprising considering that relatively intact cores were tested for both time periods and that the strength and stiffness of this stabilized material typically increases with time.

Young's moduli values determined from the slope of stress/strain curves were included in the current study since this data could be readily obtained from the UCS testing of LFA cores. However, subsequent computations and conclusions/recommendations in this report are based upon the LFA backcalculated moduli values since the FWD tests the response of the LFA layer, as opposed to the small sample used for UCS testing.

Tables 11 and 12 include the subgrade backcalculated modulus value for each FWD test location. The corrected subgrade modulus and calculated CBR values for the FWD test locations, excluding those removed due to one of the three criteria, are also included in these tables. The method used to correct the subgrade modulus and calculate a subgrade CBR from a modulus value is discussed in Chapter 2. Note that the three older projects having an untreated granular subbase are excluded from these calculations because the subbase material was included with the subgrade during the backcalculation routines.

During the course of this study a relationship was required between backcalculated modulus and compressive strength of LFA stabilized soil to facilitate flexural stress/flexural strength ratio computations. While the use of this relationship is deferred until Chapter 11, it is included in the current chapter because the data used to develop it is derived from the previous and current chapters. The Proctor UCS and backcalculated moduli data for the five newer projects were used to develop this relationship. Given the omitted backcalculated HMA or LFA moduli values due to one of the three exclusion criteria; i.e., HMA modulus exceeding 2500 ksi, LFA modulus exceeding the HMA modulus and Chauvenet's criteria, and the lack of a testable UCS core from all of the 63 original FWD test locations among the five newer projects, only 44 data points remained to develop the desired relationship.

The remaining data points were plotted, a best-fit curve defined and corresponding equation developed using Excel's curve fitting functions. In Figure 17 the middle curve is the best-fit curve of the 44 data points and is defined by a power equation. The R^2 value of 0.17 numerically indicates what the plot shows, that there is significant spread in the data and the equation can only indicate a rough trend in the plotted data. A maximum of 15% of the 44 data points, or 7 data points, were excluded in two iterations to try to reduce the spread in the data and better define the relationship between LFA modulus and strength.

In addition to the curve providing the best-fit of the 44 data points, Figure 17 also illustrates the process of the first iteration. In this iteration two curves, one above and one below the curve defined by the power equation, were developed so that some of the data points fell outside of the band defined by these two additional curves. The upper curve corresponds to computed moduli values that are 175% greater than the values calculated by the power equation, and the lower curve corresponds to computed moduli values that are 175% less than the values calculated by the power equation. For example, in Figure 17 the end point of the power equation curve closest to the y-axis has the coordinates (100,145.85). The end point of the upper curve closest to the y-axis is (100,401.1). The value 401.1 is the sum of 145.85 and 1.75 times 145.85. The end point of the lower curve closest to the y-axis is (100,-109.4). The value -109.4 is the sum of 145.85 and -1.75 times 145.85.

The width of the band can be adjusted by varying the percentage used in the calculations so that any desired number of the 44 points can be located outside of this band. The value of 175% was selected to cause three points to fall outside of the band

and thus be considered outliers. Note that all three of these points are located above the upper curve.

These three outliers were removed, and a new best-fit curve was defined with the remaining 41 data points as part of the second iteration of this process (Figure 18). A second power equation provided a curve with the best fit of these 41 data points. Note that the R^2 associated with this second equation is 0.34. This is a significant improvement over that of the first power equation and indicates that this equation better defines the relationship between the two parameters of modulus and strength.

Figure 18 illustrates two additional curves, one above and one below the curve defined by the second power equation. A plus/minus 120% of the computed moduli values were used in developing these curves so that four additional data points fell outside of the band defined by these curves. Note that this is a significant reduction in percentage used as compared to the first iteration. These four points are located above the upper curve as shown in Figure 18.

These four outliers were removed and a final best-fit curve defined with the remaining 37 data points (Figure 19). The following second order equation defining this curve is:

$$\text{LFA } E_{back} = 0.00030523*(\text{LFA UCS})^2 + 0.56723519*(\text{LFA UCS}) \quad \text{Equation 3}$$

E_{back} is the LFA backcalculated modulus in ksi units and the LFA UCS is in psi units. Note that significant scatter still exists as indicated by an R^2 of 0.44, but this is a significant improvement compared to the R^2 of the equation in the second iteration.

This relationship is used in Chapter 11 of the current study, but it is recommended that it be further refined by subsequent research.

The selection of two iterations such that three outliers were selected in the first iteration and four in the second iteration was made by trial and error. A total of seven iterations could have been selected where only one data point was considered an outlier in each of these iterations, or one iteration could have been selected where all seven data points were removed at one time. The premise of this approach is to remove data points in each iteration that results in a new best-fit curve having a higher R^2 value for the remaining data than the best-fit curve of the previous iteration. Any number of points can be omitted in a given iteration so long as the total number of points cumulatively removed in all of the iterations does not exceed the maximum of 15% of the original number of data points. The percentage used in each subsequent iteration should be less than the previous iteration due to a tighter clustering of the remaining data points about the best-fit curve.

Figure 15. Dynatest 8000 Series Trailer-Mounted FWD Used to Perform FWD Testing

Figure 16. Obtaining Mid-Depth HMA Pavement Layer Temperature

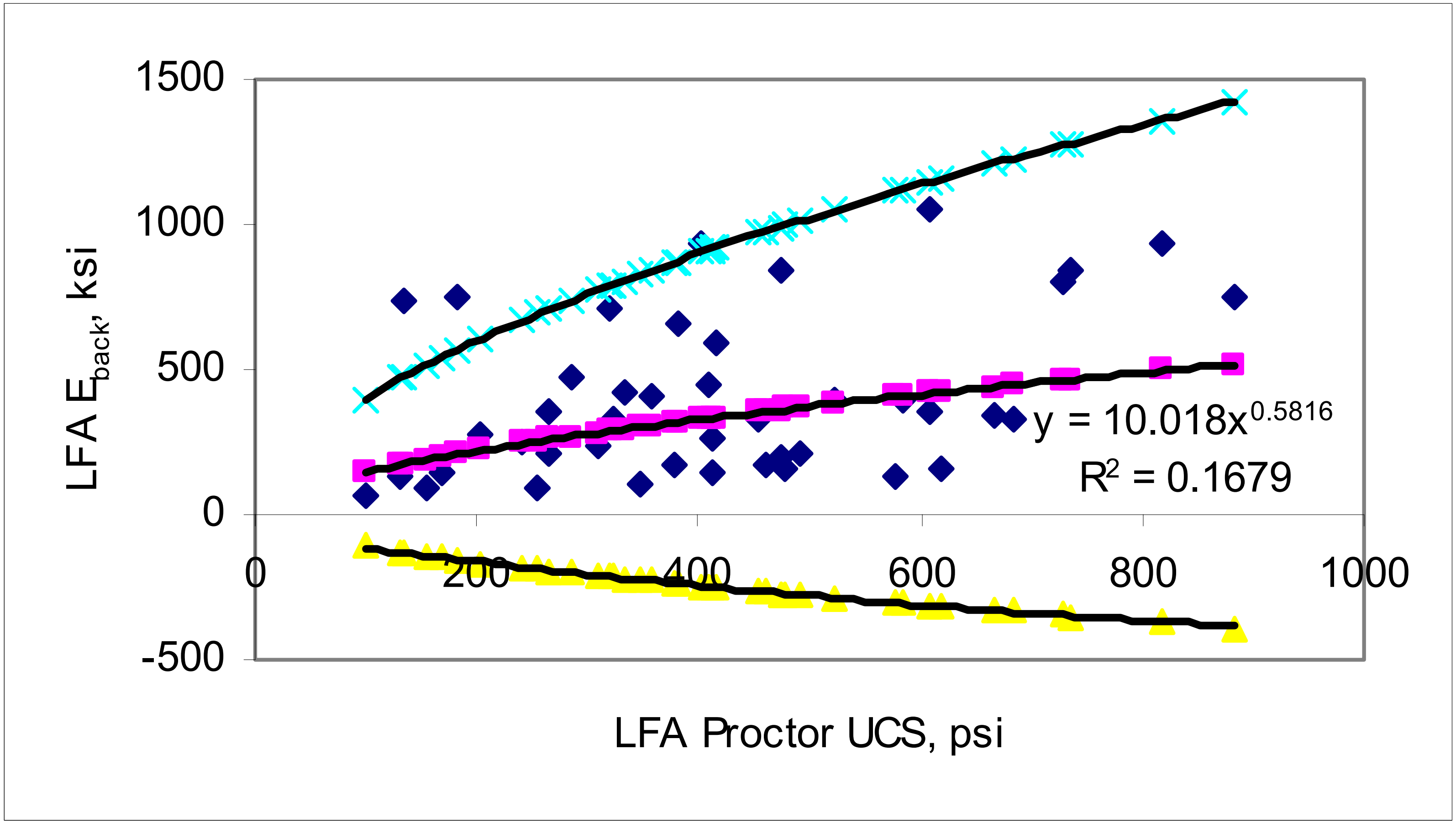


Figure 17. First Iteration for Identification of Data Outliers

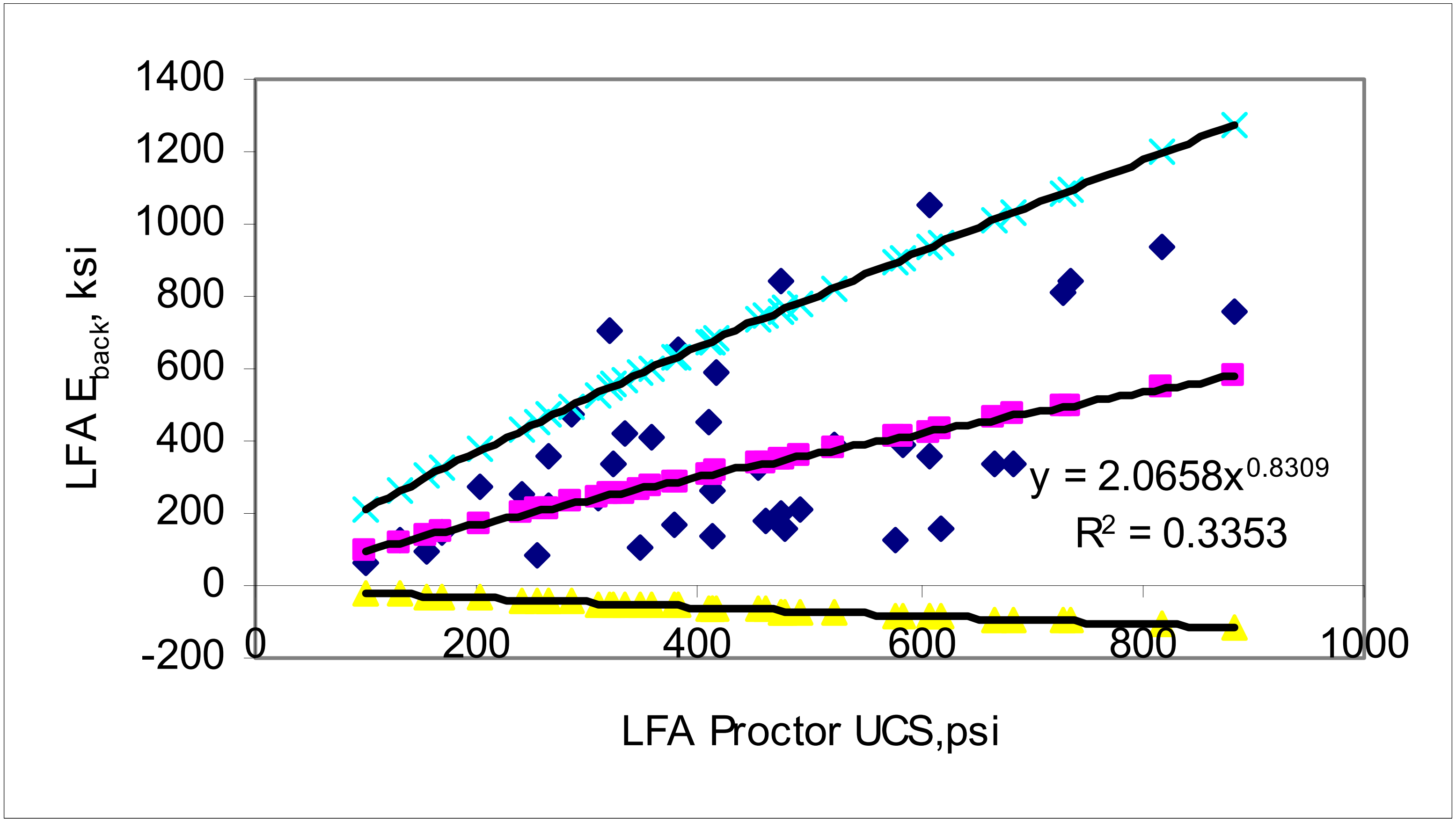


Figure 18. Second Iteration for Identifcation of Data Outliers

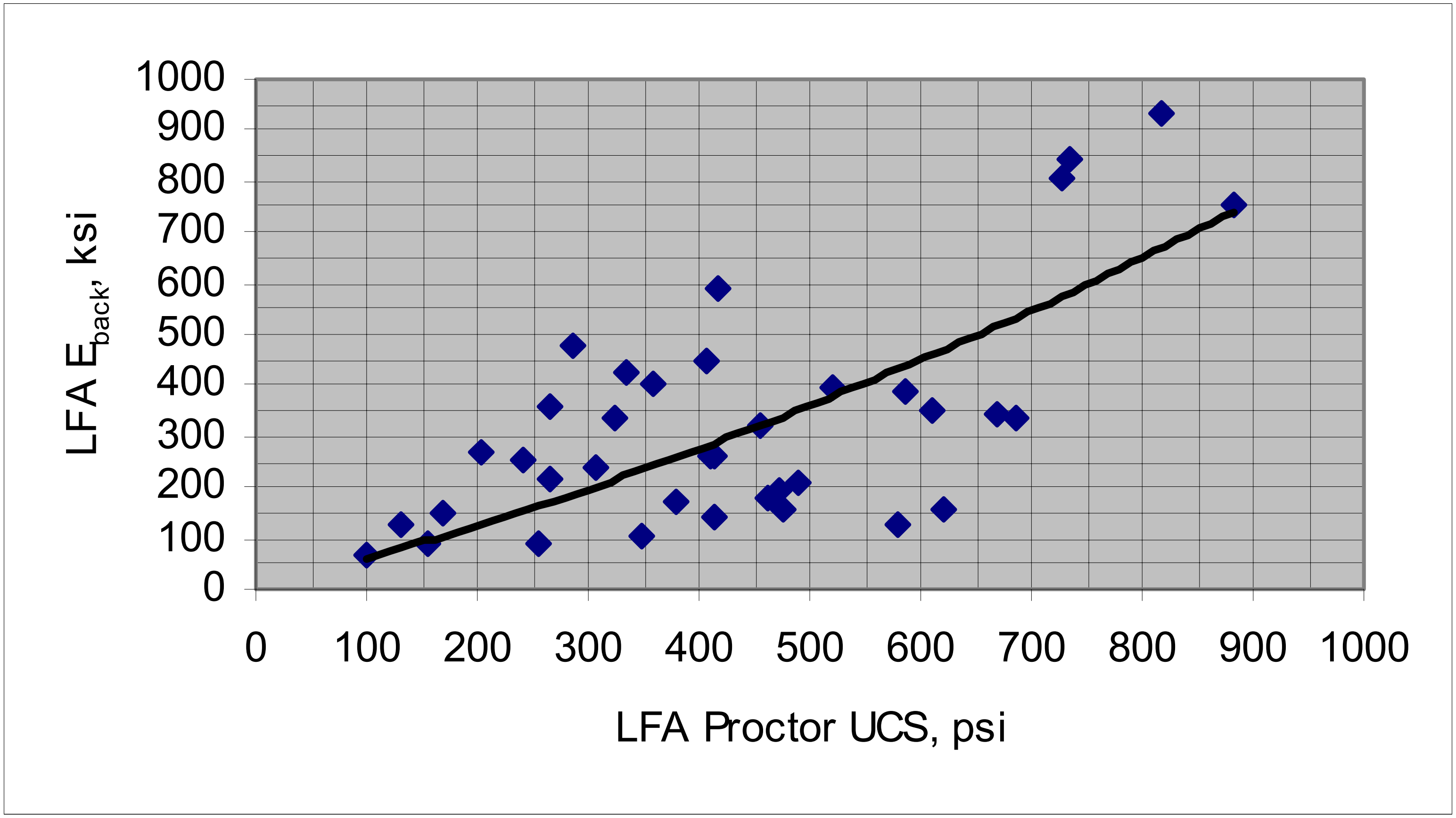


Figure 19. LFA E_{back} vs. LFA Proctor UCS

Table 11. Pavement Layer Moduli and Calculated Subgrade CBR Values for Newer Projects

County	FWD Test Location	HMA E_{back} (ksi)	LFA E_{back} (ksi)	LFA Young's Modulus (ksi)	Subgrade E_{back} (ksi)	Corrected Subgrade Modulus (ksi)	Calculated Subgrade CBR	Notes
Bolivar	290+00	1626	88.4		17.0	8.8	7	
	295+00	1222	353	153.8	16.0	8.3	6	
	300+00	1517	213		15.5	8.1	6	
	305+00	1667	165.1	200.0	16.2	8.4	6	
	335+00	1502	714.8 *		15.5			Outlier per Chauvenet
	340+00	1922	90	23.2	14.5	7.5	5	
	345+00	1631	573.8	252.2	20.6	10.7	9	
	350+00	1486	411	263.2	21.6	11.2	10	
	713+00	1803	211.7		14.6	7.6	5	
	718+00	1442	393.3	141.7	17.0	8.8	7	
	723+00	2010	126.9	190.0	12.0	6.2	4	
	728+00	1880	333.2	89.3	13.4	7.0	5	
	733+00	1767	323.8		13.4	7.0	5	
	738+00	2298	339.6		13.6	7.1	5	
	743+00	1773	406.1	43.3	14.5	7.5	5	
	748+00	2117	197.3	77.8	13.1	6.8	5	
Clarke	39+50	656	1094.7	221.4	16.5			LFA E > HMA E
	40+00	1328	754.1	218.8	16.9	8.8	7	
	40+50	5500	176	251.9	17.4			HMA E too high
	41+00	2390	326.7		17.5	9.1	7	
	41+50	1847	950.1	233.3	19.0	9.9	8	
	42+00	2498	808.9		16.8	8.7	7	
	42+50	2042	932.7	166.7	19.5	10.1	9	
	43+00	1477	358.5		21.8	11.3	10	
	752+00	2032	270.6		20.3	10.6	9	
	755+00	1929	928.3	71.4	21.4	11.1	10	
	758+00	1168	474.5		17.7	9.2	7	
	761+00	1130	746.6		21.6	11.2	10	
	764+00	2412	652.3	70.5	18.3	9.5	8	
	767+00	1863	592		17.1	8.9	7	
	770+00	1868	837.6		14.4	7.5	5	
	773+00	1067	732.6		16.3	8.5	7	

Table 11 Continued. Pavement Layer Moduli and Calculated Subgrade CBR Values for Newer Projects

County	FWD Test Location	HMA E_{back} (ksi)	LFA E_{back} (ksi)	LFA Young's Modulus (ksi)	Subgrade E_{back} (ksi)	Corrected Subgrade Modulus (ksi)	Calculated Subgrade CBR	Notes
Smith	493+00	665	1348.2	220.0	22.8			LFA E > HMA E
	498+00	1133	1663.5		19.2			LFA E > HMA E
	503+00	1103	753.7 *		23.6			Outlier per Chauvenet
	508+00	1651	335.4		18	9.4	8	
	518+00	884	2359	205.6	18.8			LFA E > HMA E
	522+00	1283	389		19	9.9	8	
	528+00	1430	448.4	181.2	29.3	15.2	16	
	610+00	1216	240.5		26	13.5	14	
	613+00	1427	253.4		31.7	16.5	18	
	616+00	1140	129.5	37.5	28.6	14.9	16	
	619+00	1521	156.4	137.1	23.2	12.1	11	
	622+00	1724	160.2		28.5	14.8	16	
	625+00	1134	104		21.5	11.2	10	
	628+00	968	65		21.4	11.1	10	
	631+00	533 *	107.5		17.8			Outlier per Chauvenet
Tippah	163+00	1288	262.8	173.7	34	17.7	21	
	167+00	1074	341.6		24	12.5	12	
	171+00	1896	168.7		16.2	8.4	6	
	175+00	1962	424.8		14.6	7.6	5	
	179+00	1673	145.9		15.6	8.1	6	
	183+00	1205	216.9	133.3	26.7	13.9	14	
	187+00	1636	19.4		8.5	4.4	2	
	191+00	1306	699.4 *		33.4			Outlier per Chauvenet
Wilkinson	164+00	1385	846.8	205.6	17	8.8	7	
	169+00	1259	260.7		18.9	9.8	8	
	174+00	1435	1057	112.5	16.2	8.4	6	
	179+00	1156	836	259.3	17.2	8.9	7	
	184+00	1865	176.6	76.3	15.8	8.2	6	
	189+00	2229	708.1		22.3	11.6	11	
	195+00	2044	1394.7	281.8	19	9.9	8	
	200+00	1419	141.9		14.2	7.4	5	

Table 12. Pavement Layer Moduli for Older Projects

County	FWD Test Location	HMA E_{back} (ksi)	LFA E_{back} (ksi)	LFA Young's Modulus (ksi)	Combined Subbase and Subgrade E_{back} (ksi)	Corrected Subgrade Modulus (ksi)	Calculated Subgrade CBR	Notes
Forrest/ Perry	288+00	355	1004.8	235.7	15.6			LFA E > HMA E
	293+00	279	4114.4		17.1			LFA E > HMA E
	298+00	2032 *	166		18.6			Outlier per Chauvenet
	303+00	835	216.4		17.5			
	330+00	297	88.5		12.5			
	335+00	476	52.8	86.4	15.9			
	340+00	819	311.8		18.1			
	345+00	1,630	49		16.3			
	516+00	299	213.5	309.1	16.0			
	519+00	763	949.5	581.8	23.7			LFA E > HMA E
	524+00	496	196.9		19.5			
	528+00	465	197.7		19.8			
	530+00	812	312	86.5	19.4			
	535+00	1,082	301.3		14.4			
	540+00	559	246.8		14.5			
	545+00	621	314.9		22.3			
George	84+96	779	138.1	241.4	14.8			
	124+04	1389 *	388.1		16.2			Outlier per Chauvenet
	242+45	120	34.6		13.9			
	272+44	788	519.9		21.0			
	315+00	526	357.9	58.2	12.1			
	351+36	251	236.2		18.0			
	390+21	383	46.9		15.6			
	424+57	558	527.8		15.3			
Jones/ Wayne	102+00	3086	115.6	260.9	12.4			HMA E too high
	107+00	579	368.2 *	227.8	20.0			Outlier per Chauvenet
	112+00	984	105.4		14.0			
	169+00	1072	168.4		11.3			
	170+00	1001	143.8		12.6			
	171+00	1438	179.6		14.4			
	172+00	2354	32.3	548.1	17.5			
	376+00	1452	163.6		19.6			
	381+00	1177	161.8	190.9	18.5			
	386+00	1918	130.1	188.9	16.8			
	411+00	1655	159.2	400.0	16.6			
	416+00	1090	84		15.8			
	421+00	1821	96.2	460.6	12.8			
	456+00	1509	181.4	100.0	19.0			
	461+00	1169	82.8		14.0			
	466+00	1789	86.7	122.2	18.8			

Table 12 Continued. Pavement Layer Moduli for Older Projects

County	FWD Test Location	HMA E_{back} (ksi)	LFA E_{back} (ksi)	LFA Young's Modulus (ksi)	Subgrade E_{back} (ksi)	Corrected Subgrade Modulus (ksi)	Calculated Subgrade CBR	Notes
Yalobusha	340+04	348	295.7	212.5	13.8	7.2	5	
	341+96	1,724	13.9		15.3	8.0	6	
	360+04	2,838	56		8.9			HMA E too high
	361+96	1,980	53.1		23.4	12.2	11	
	402+04	1,596	140.7	59.4	11.7	6.1	4	
	403+96	1,446	124.7		12.5	6.5	4	
	420+04	1,752	163.5		11.3	5.9	4	
	421+96	1,426	171	118.8	14.3	7.4	5	
	455+04	2,406	74.5	295.2	7.2	3.7	2	
	456+96	1,645	117.4	263.2	9.9	5.1	3	
	488+04	2,496	54.7		14.5	7.5	5	
	488+96	1,028	124.7	134.8	12.5	6.5	4	
	492+04	1,132	690 *		12.7			Outlier per Chauvenet
	493+96	260	216.6		18.1	9.4	8	
	507+04	1,468	109.3		13.9	7.2	5	
	508+96	4,504	150.1		15.0			HMA E too high

Chapter 5 -- LFA In-Situ Structural Layer Coefficients

MDOT determines the percentages of lime and fly ash to be incorporated into a soil for base course construction based on laboratory Proctor UCS test results. The current MDOT pavement design procedure to determine the layer thicknesses of a new pavement are based on the AASHO Interim Guide for the Design of Pavement Structures – 1972, hereafter referred to as the 1972 Interim Guide. This guide uses structural layer coefficients to characterize material property inputs. Since the design procedure employed by MDOT uses structural layer coefficients to characterize the pavement layer materials, the basis of evaluation used in the current study is the LFA structural layer coefficient.

A structural layer coefficient is basically an equivalency factor. MDOT currently uses a structural layer coefficient of 0.44 for HMA and 0.20 for LFA stabilized soil base course material. One inch of HMA is replaced by 2.2 inches of LFA stabilized soil in a pavement structure. The structural layer coefficient is not a unique value for a given material (Gomez and Thompson, 1983), though it is assumed as such by MDOT for routine pavement design. Structural layer coefficient values vary with layer thickness, material type, material quality, layer location (base, subbase), traffic level, and limiting criterion (stress, strain, deflection, etc.). Two approaches were used in the current study to evaluate the structural layer coefficient of LFA stabilized soil.

LFA Structural Layer Coefficients Based on LFA UCS

Figure 20 illustrates the first approach in the form of a relationship between the structural layer coefficient of a lime stabilized soil base course and the UCS of the lime stabilized soil comprising the base course (Little, 1995). This particular relationship was developed by Thompson based upon a relationship between structural layer coefficient and the 7-day compressive strength of cement stabilized base courses. Little used the rationale that lime stabilized and cement stabilized base courses are similar, with the major difference being that lime stabilized soil experiences strength gain at much slower rates and over longer periods of time relative to the cement stabilized material. The same rationale is applied in the current study since the same similarity and difference between lime and cement stabilized soils are also applicable between LFA and cement stabilized soils. Therefore, the relationship depicted in Figure 20 is considered applicable to LFA stabilized soil base courses. This particular relationship also appears to correspond well with MDOT's 500-psi LFA mix design requirement for base course construction and assigned LFA structural layer coefficient of 0.20, since these two values, when plotted, represent a point on the curve.

The use of UCS to obtain structural layer coefficients provides a relatively easy method to obtain the desired information, but this method only accounts for three of the six variables listed that affect the value of a structural layer coefficient; e.g., material type, material quality as expressed by UCS, and layer location. An additional consideration is that a relatively small quantity of material is tested in the UCS test. When testing LFA cores, a third consideration is that only relatively uncracked and well cemented LFA material can be tested for UCS. Recall that the UCS testing performed in conjunction with this study required the extraction of an intact core from the pavement, which resulted in multiple coring attempts at a given FWD test location. The “best” quality material was therefore sampled from a given FWD test location. A fourth consideration is that the coring operation may have resulted in some degradation of the LFA material, thus reducing the quality of this material prior to UCS testing. Given these considerations, this method was not used to evaluate in-situ LFA structural layer coefficients. However, this method is used in Chapter 7 to support the case for increasing the required level of field compaction of LFA and soil mixtures since a significant relationship exists between level of compaction and UCS. Laboratory compacted samples were used for that evaluation.

LFA Structural Layer Coefficients Based on FWD Test Data

The second approach is based on the use of FWD deflection data and was selected to obtain the in-situ LFA structural layer coefficients of the nine projects. FWD testing encompasses the response of the entire pavement to load application. The results of this testing better reflect the response of a given pavement material within the pavement system as opposed to removing the material from the pavement and testing it as is done with the UCS test. The effect of this is to incorporate more of the variables that impact the value of a LFA structural layer coefficient. More of the LFA material is tested when using the FWD, and the effect of cracks and other imperfections is allowed to impact the measured values. FWD testing is non-destructive testing, and the tested material is not subjected to potential degradation due to the extraction process that is required for UCS testing.

For new pavement design the 1972 Interim Guide procedure determines the thickness of each layer by an equation that relates the summation of the products of each material type structural layer coefficient and corresponding material layer thickness to a pavement structure number, SN. The same basic equation is used in the 1993 Design Guide with additional factors included that account for drainage in untreated base and subbase material layers. SN is referred to as an "abstract number" in the 1993 Design Guide that expresses the structural strength of a pavement required for a given combination of soil support, total traffic, terminal serviceability, and environment.

The 1993 Design Guide includes a procedure for determining the pavement structure number of an existing pavement, referred to as the effective structural number, SN_{eff}, from deflection bowl data obtained with the FWD. The purpose of determining the SN_{eff} of an existing pavement in the current study is to facilitate the computation of the LFA stabilized material structural layer coefficient, a_2, at given test locations. The general equation relating SN_{eff} and a_2 for the three and four-layer pavement systems considered in this study is:

$$SN_{eff} = a_1D_1 + a_2D_2 + a_3D_3m_3 \quad \text{Equation 4}$$

Where: a_1 = HMA structural layer coefficient

D_1 = total thickness of the asphalt in the pavement structure

a_2 = LFA stabilized material structural layer coefficient

D_2 = thickness of LFA stabilized material

a_3 = unstabilized granular material structural layer coefficient

D_3 = thickness of the unstabilized granular material

m_3 = drainage coefficient for untreated granular material subbase

SN_{eff} is determined as outlined in the 1993 Design Guide between pages III-96 and III-102. The values of D_1, and D_2 are determined from coring data for all nine projects and D_3 is determined as discussed in Chapter 3 for the three older projects having an untreated granular material subbase layer. For this third layer, MDOT's design structural layer coefficient of 0.09 is used, and a value of 1.0 assumed for the drainage coefficient. For the five newer pavements the HMA structural layer coefficient was first determined for each test location and then Equation 4 solved for the LFA structural layer coefficient. A different approach was required to determine the LFA layer coefficient for the four older projects. Details related to determining these coefficients are included in subsequent discussion in the current chapter.

A total of 119 FWD tests were conducted at the nine project sites. Chapter 4 includes a discussion regarding the omission of backcalculated HMA or LFA moduli values due to one of three exclusion criteria; e.g., HMA modulus exceeding 2500 ksi, LFA modulus exceeding the HMA modulus, and Chauvenet's criteria. These same locations were also omitted from the summary statistics for the current chapter. The questionable backcalculation results might be due to errors in the measurement or recording of the corresponding deflection values defining the deflection bowl, or applied load. Erroneous deflection bowl data results in the calculation of misleading SN_{eff} and backcalculated HMA pavement layer moduli values.

LFA Structural Layer Coefficients for the Five Newer Projects

The determination of the in-situ HMA structural layer coefficient for each test location in the five newer projects utilized the relationship shown in Figure 21. This figure is a reproduction of Figure 2.5 found on page II-18 of the 1993 Design Guide. Note that an elastic modulus measured at 68 ^{0}F is required to find the corresponding a_1 value. In this study the backcalculated HMA moduli values were corrected to equivalent laboratory elastic moduli values at 68 ^{0}F. Given that HMA is a viscoelastic material, the properties of which vary with rate of loading and temperature at the time of loading, two steps were required to make the necessary correction.

The first step accounts for the difference in the rate of loading between the FWD and laboratory modulus testing procedures. The correction applied to account for this difference is not a unique value. The divergence between test results increases with increasing test temperature. Figure 22 is a graph of data which provides the correction factor based upon the temperature at which the comparison is made (Von Quintus and Killingsworth, 1998). This data is found on page 96 of the given reference. Note that at a test temperature of 41 ^{0}F, the correction factor is one and increases to 4 at a test temperature of 104 ^{0}F. In the current study, the HMA backcalculated moduli values were corrected to equivalent laboratory moduli values using the mid-depth HMA pavement layer temperatures recorded at the time of FWD testing. This resulted in relatively minor corrections to these backcalculated values since all of the pavements were tested in mid-winter when the pavement mid-depth temperatures were in the mid to upper 40s. The exception to this was the Forrest/Perry project, which had a greater correction applied due to the greater temperature at the time of testing of this particular pavement.

The second step accounts for the difference in the equivalent laboratory moduli values at the field testing temperatures and 68 ^{0}F. The equation used for this correction was developed from FWD test data obtained from several Texas pavements and the backcalculated moduli values were obtained using the Modulus backcalculation computer program (Chen, et al., 2000). MDOT uses the Modulus 5.1 backcalculation program for routine pavement analyses. Given that the referenced study and the current study both used Modulus for backcalculation, and the fact that the state of Mississippi is located longitudinally similar to the state of Texas, the following equation from the given reference was used for this second step of the moduli correction process:

$$E_{Tw} = E_{Tc}/(((1.8*T_w + 32)\text{^}2.4462)*(1.8*T_c + 32)\text{^}{-2.4462}) \quad \text{Equation 5}$$

Where: E_{Tw} = adjusted modulus of elasticity at T_w (MPa)

E_{Tc} = measured modulus of elasticity at T_c (MPa)

T_w = temperature to which the modulus of elasticity is adjusted (^{0}C)

T_c = the mid-depth temperature at the time of FWD data collection (^{0}C)

This equation was developed using uncracked pavement sections and is not applicable to cracked pavement sections. Therefore, this equation is only applicable for the five newer pavements of the current study since they were intact at the time of FWD testing. All four of the older pavements had at least low severity level cracking at the majority of the FWD test locations.

Table 13 includes a summary of the HMA and LFA structural layer coefficients calculated for the five newer pavements. Note the average HMA in-situ structural layer coefficient determined for each of these pavements. The low average of 0.423 for Smith County and the high average of 0.465 for Bolivar County closely bracket the design HMA coefficient of 0.44. Credence for the two-step HMA moduli correction process adopted for this study is provided by these average in-situ values being in such close proximity to the design value. Note that the average for Tippah County is equal to the design value.

Appendix E provides supporting data and the results of computations for determining the in-situ LFA structural layer coefficient for each FWD test location in each of the five newer projects. A normalized LFA a_2 value is calculated by multiplying the in-situ LFA a_2 by the in-situ LFA layer thickness and then dividing the product by the LFA design thickness. Normalization of the data allows for direct comparison of average LFA a_2 values for each project relative to the other projects and with the design value of 0.2. These comparisons are listed in Table 13.

The average normalized LFA structural layer coefficient for the five newer pavements is 0.232 with 67 percent of the tested locations exceeding the design value. The average exceeds the design value, and taken on this merit alone, indicates excellent early performance of the LFA stabilized soil base courses. However, note that the coefficient of variation for these pavements is 32 percent, indicating a huge variation in the in-situ properties of this stabilized material. Individually, Clarke County had the least variation with an 18.9 percent coefficient of variation, and Wilkinson County had the most variation with a 50.3 percent coefficient of variation. The large variation in the quality of the in-situ material suggests a significantly lower level of performance than the average values indicate when the concept of reliability is introduced into the evaluation scheme.

Figure 23 illustrates the relationship between a design LFA structural layer coefficient at 90 percent reliability and the average LFA structural layer coefficient required to obtain the design value for three different levels of variability. Table 2.2 in the 1993 Design Guide provides suggested levels of reliability for various functional classifications of roads. A value of 90 percent was selected based on this table.

The current variability line in Figure 23 corresponds to the amount of variability observed in the in-situ LFA structural layer coefficients for the five newer projects. Given an average of 0.232 at the current level of variability, the corresponding design value should be 0.14. This value is 30 percent less than the design value currently used by MDOT. Figure 23 illustrates three approaches to achieve the current MDOT design value of 0.20. One approach is to hold the variability constant, but increase the average. Assuming no change in variability, if the average is increased to 0.295, then the design level of 0.2 can be achieved with 90 percent reliability.

The second approach is to hold the average constant, but reduce the variability. Observe that by reducing the variability by 25 percent, for the same average of 0.232, the design value could be increased from 0.14 to 0.16, and for a total reduction of 50 percent in variability, the design value could be 0.18. A further reduction in variability would be required to achieve the 0.2 design value.

The third approach is a combination of both the first and second approaches and provides the basis for the recommendations made in this study to improve the performance of LFA stabilized soil materials. Chapter 7 discusses a method to increase the average and Chapter 9 discusses methods to decrease the variability of LFA stabilized soils.

Determination of LFA Structural Layer Coefficients for the Four Older Projects

It is difficult to assign a structural layer coefficient to materials that have experienced degradation due to the effects of traffic loading and the environment. The 1993 Design Guide includes Table 5.2 which provides suggested layer coefficients for existing HMA pavement layer materials based upon the type and amount of cracking noted in the surface of the pavement. This table is reproduced as Table 14 in the current study. The relatively wide ranges indicated for the HMA layer preclude the use of this table for calculating the in-situ LFA structural layer coefficient from Equation 4, since values selected for the HMA layer directly impact the calculated value for the LFA layer. However, the ranges shown in Table 14 can be used to check the reasonableness of calculated values.

Figure 24 illustrates the approach used to evaluate the LFA structural layer coefficients for the four older projects. The data from the 54 FWD test locations used to determine the average LFA structural layer coefficient of 0.232 for the five newer projects was used to develop a relationship between in-situ LFA structural layer coefficient and backcalculated LFA modulus. This relationship is expressed as:

$$a_2 = 0.03184616 * (E_{back} \wedge 0.33057336) \quad \text{Equation 6}$$

Where: a_2 = in-situ LFA structural layer coefficient

E_{back} = LFA backcalculated modulus, (ksi)

None of the 54 data points were considered an outlier. An R^2 of 0.84 indicates an excellent relationship when considering that this is field derived data, but not surprising given that both corresponding values of layer coefficient and modulus for each point were derived from the same deflection bowl of a given FWD test location.

The LFA backcalculated modulus value from each of the FWD test locations in the four older projects was entered into Equation 6 to obtain the corresponding in-situ LFA structural layer coefficient. Appendix F includes these calculated layer coefficient values. Table 15 provides a summary of the average in-situ LFA structural layer coefficient for each of these older pavements as well as a combined average for all of these pavements.

The type and level of severity of pavement cracking noted at the FWD test locations is included in Table 6 of Chapter 3. Using this crack type/severity level information, it is noted that the averages shown in Table 15 compare favorably within the ranges shown in Table 14 for a stabilized base course layer.

The average for all four older pavements was 0.165 with a coefficient of variation of 23.3 percent. This average is less than the design of 0.2 and is expected due to traffic loading and environmental affects on these older pavements. The variability calculated for the LFA material in the older pavements is less than the variability calculated for the newer pavements. This reduction in variability can probably be attributed to the use of Equation 6 to calculate the LFA layer coefficients rather than an actual reduction in variability, since both the older and newer pavements were constructed using similar field-mixed-in-place methods.

Verification of Equation 6

While the average LFA structural layer coefficients determined using Equation 6 for the four older projects appear reasonable based on Table 14, another check was utilized to try to verify these results. This additional verification was made since Equation 6 was developed from data obtained from uncracked newer pavements and then applied to cracked older pavements.

To accomplish this additional verification, the deflection basin data of the four older projects was used to determine SN_{eff} of each of the FWD test locations in these older projects using the same procedure as that used for the five newer projects; i.e., the 1993 Design Guide pages III-96 to III-102. The difference in the use of SN_{eff} between the newer and older pavements is that in the newer pavements, the HMA structural layer coefficient was first determined from HMA backcalculated moduli values and then the LFA structural layer coefficient calculated using Equation 4, whereas for the older pavements the LFA structural layer coefficient was first determined using Equation 6 and then the HMA structural layer coefficient calculated from Equation 4. The reasonableness of the calculated HMA structural layer coefficients is the basis of verification for using Equation 6.

Appendix F provides the supporting data and results of the computations for determining the in-situ HMA structural layer coefficient for each FWD test location in each of the four older pavements. Table 15 includes the average HMA structural layer coefficient for each of these pavements. Note that these individual project averages are very high considering that they are aged pavements, with two of these averages exceeding the HMA design value of 0.44. The averages shown for the Forrest/Perry and Yalobusha projects are relatively more reasonable, with respective values of 0.42 and 0.43, than the averages shown for the Jones/Wayne and George projects with respective values of 0.54 and 0.49.

All four averages are above the ranges suggested in Table 14. The generally low severity level of surface cracking common to all of the older pavements suggests, based on this table, similar HMA structural layer coefficient values for these pavements. The range in averages from 0.42 to 0.54 does not substantiate this observation.

Chapter 3, Table 6 includes the layer thicknesses of HMA, LFA stabilized soil and unstabilized granular soil pavement layers for the four older pavements. Recall that the thickness of the granular material layer was not directly measured, but was a derived value based on the summation of design thickness of both the stabilized base and unstabilized subbase layers minus the measured stabilized base layer. The Yalobusha project did not have any subbase layer, and the Forrest/Perry project had a calculated subbase layer thickness ranging from 2.75 to 4 inches. The Jones/Wayne project calculated subbase layer thickness varied from 6.25 to 10 inches, and the George project calculated values varied from 12.5 to 14 inches. The thickness of untreated subbase is represented as D_3 in Equation 4. Suppose the actual in-situ layer thickness was significantly different than the corresponding calculated value. Entering this actual in-situ thickness value into Equation 4, in lieu of the calculated value, would result in a significantly different value for the HMA structural layer coefficient. If there is a significant difference between the actual in-situ thickness and calculated thickness in the Jones/Wayne and George projects, and no significant difference between these values for the Forrest/Perry project, then variance between these thicknesses could account for some of the excessive HMA structural layer coefficient determined for the Jones/Wayne and George projects.

Any significant in-situ thickness variation from the calculated value would also impact the result of another series of calculations. Recall that SN_{eff} is determined as outlined in the 1993 Design Guide between pages III-96 and III-102. SN_{eff} is a function of both the total thickness of all pavement layers above the subgrade, D, and the effective modulus of pavement layers above the subgrade, E_p. Any significant in-situ variation in thickness of the untreated subbase from the calculated value impacts the value of D, and therefore SN_{eff}, which in turn impacts the value of the calculated HMA structural layer coefficient.

Revised SN_{eff} Based on Combined Similar Layers

Recall from Chapter 2, Table 4 that a sandy topping material comprised the subbase layers of the George and Jones/Wayne projects and that a slightly better material was used for the Forrest/Perry project. While these materials were used in a subbase course application, their modulus values probably do not exceed that of a very good subgrade and probably do not represent an improvement over the existing subgrade by more than about 10 ksi. In contrast, the modulus values of these subbase layers are significantly less than those corresponding to the overlying LFA base layers. In Chapter 4 it was noted that the subgrade and unstabilized subbase layer of these three projects were combined into one layer for the purpose of backcalculating the HMA and LFA stabilized soil base moduli. Combining materials having similar modulus values is common practice when performing backcalculation routines. This approach is considered to improve the accuracy of the backcalculated moduli values of the remaining layers.

Assuming that the subgrade and untreated subbase material moduli values of the three older projects are similar in relative magnitude, the same approach of combining similar layers into one layer was used to determine a revised SN_{eff}. The calculated thickness of the subbase layer was omitted, and only the summation of HMA and LFA base course layer thicknesses used for the total depth of pavement, D, in the calculations to determine this revised SN_{eff}.

Appendix F provides the revised SN_{eff} and the corresponding revised in-situ HMA structural layer coefficient for each FWD test location in each of the three older pavements that included the untreated subbase layer. Table 15 includes the average revised HMA structural layer coefficient for each of these projects. Note the significant reduction in these averages compared to those calculated from SN_{eff}. The averages for Forrest/Perry and George projects now compare favorably within the ranges shown in Table 14. The averages for Jones/Wayne and Yalobusha projects are somewhat high relative to these ranges, but are not unreasonable when considering that these pavements may have experienced some stiffening due to oxidation of the HMA layer. The overall average for the four older pavements is 0.401, which is at the upper end of the range for existing pavements having little or no alligator cracking and/or only low-severity transverse cracking.

Based on the use of a revised SN_{eff} and the corresponding revised HMA structural layer coefficient, the use of Equation 6 to predict the in-situ LFA structural layer coefficient from the backcalculated LFA modulus value appears to have merit. Since the untreated subbase soil was relatively similar to the underlying untreated subgrade, it was combined with the subgrade for both the backcalculation procedure and the determination of the revised SN_{eff} in order to achieve the reasonable results shown in Table 15; therefore, use of Equation 6 should be restricted to a similar pavement structure; i.e., a three-layer system, as that reviewed in the current study until its applicability is verified for differing pavement structures.

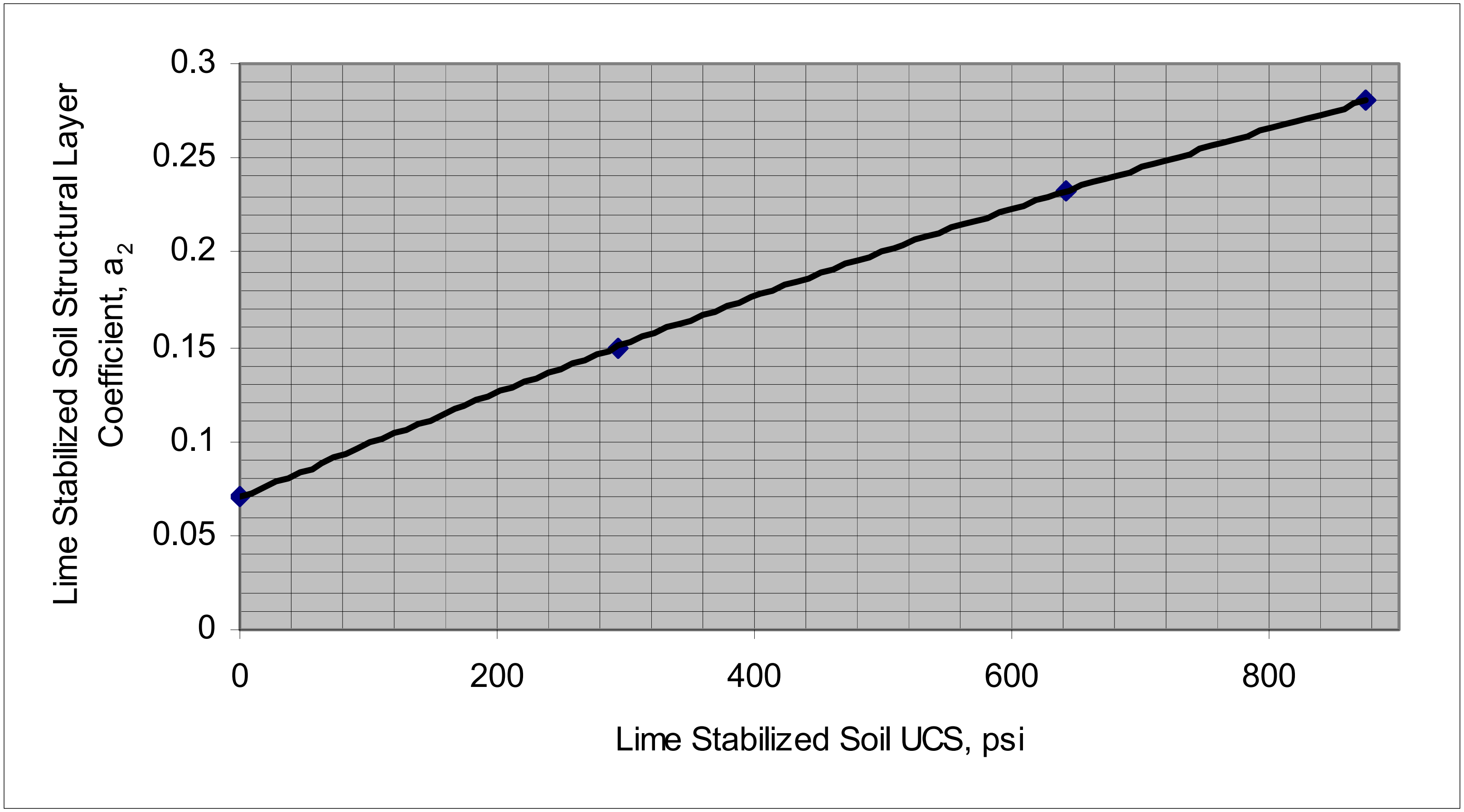


Figure 20: Lime Stabilized Soil Structural Layer Coefficient vs. UCS

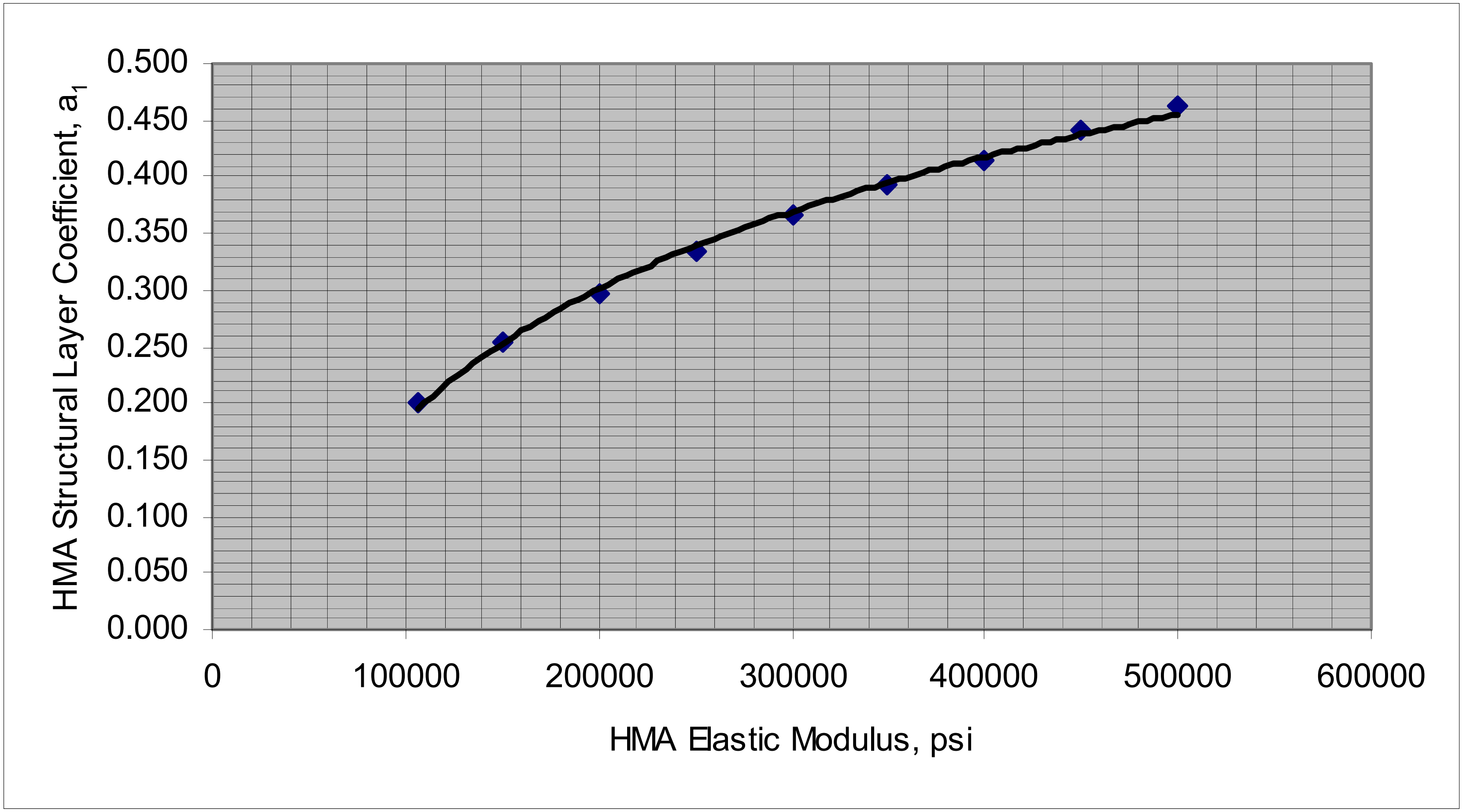


Figure 21. HMA Structural Layer Coefficient vs. HMA Elastic Modulus

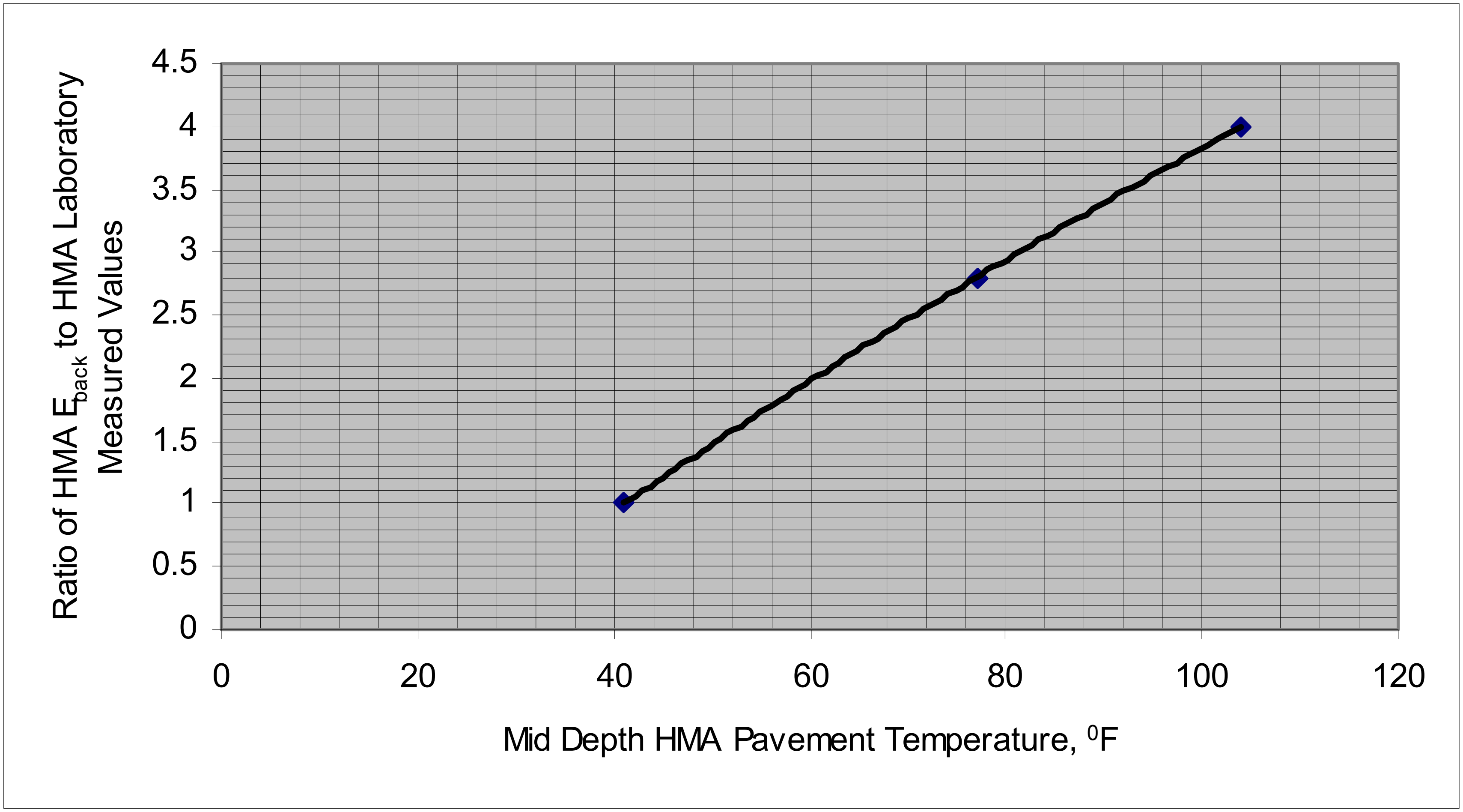


Figure 22. Ratio of HMA E_{back} to HMA Laboratory Measured Values vs. Mid-Depth HMA Pavement Temperature

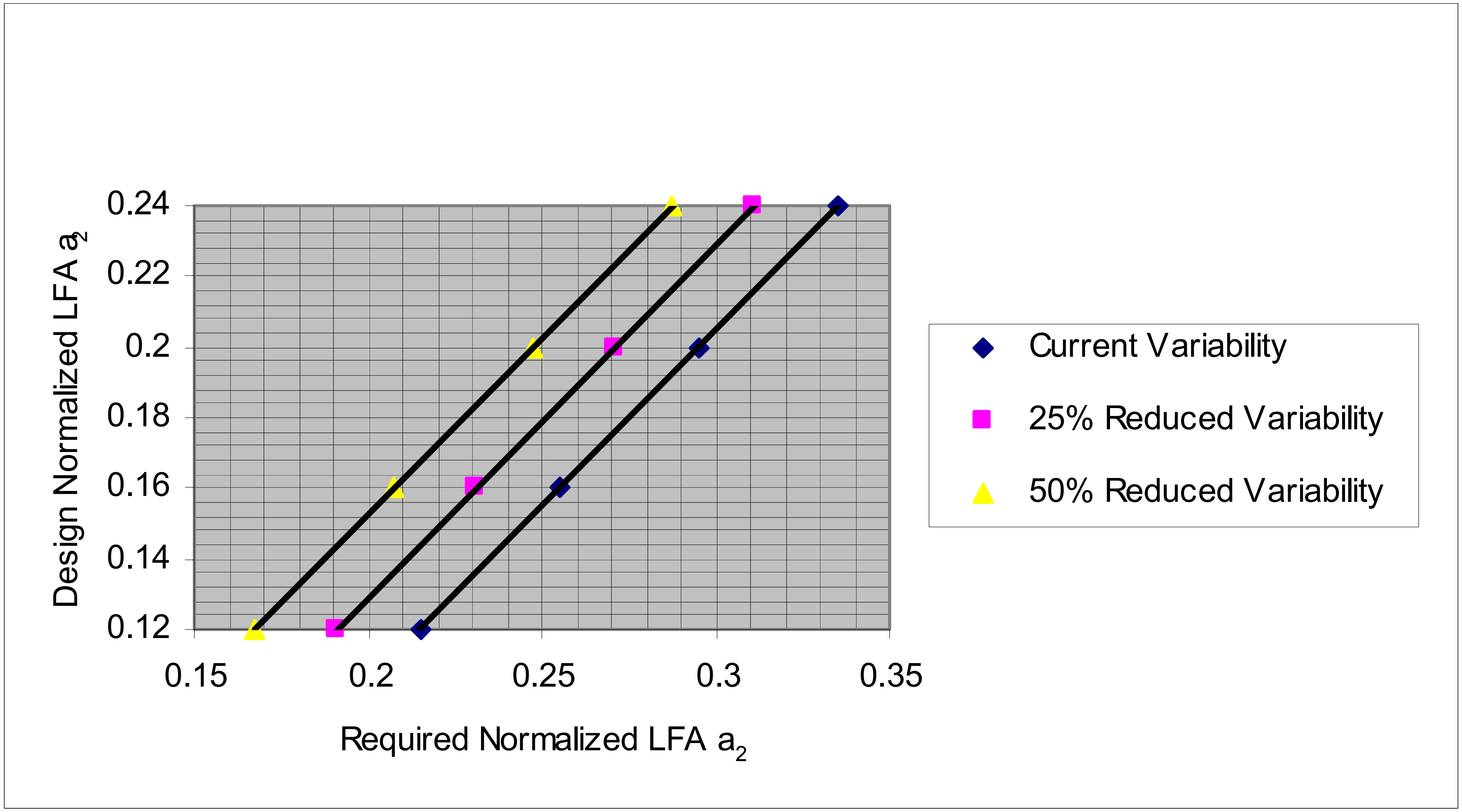


Figure 23. Design Normalized LFA a_2 vs. Required Normalized LFA a_2 at 90% Reliability for 3 Different Levels of Variability

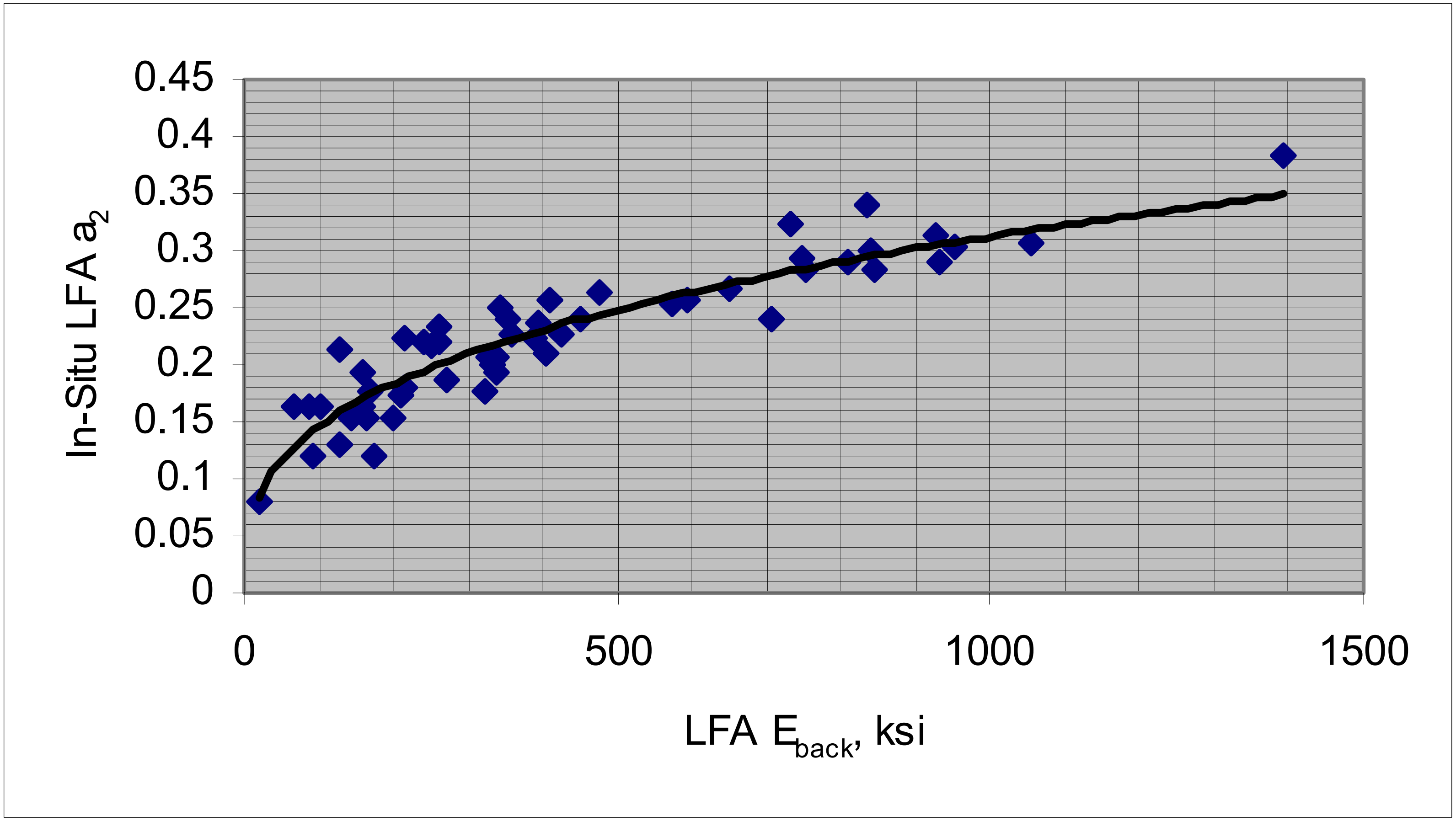


Figure 24. In-Situ LFA Structural Layer Coefficient vs. LFA Backcalculated Modulus for 5 Newer Projects

Table 13. Summary of Structural Layer Coefficients For 5 Newer Projects

County	Number of Tested Locations	Number of Omitted Locations	Normalized LFA a_2 Average	% of Design	Normalized LFA a_2 Coef. of Var.	Normalized LFA a_2 Maximum	Normalized LFA a_2 Minimum	Locations # < 0.2	Locations % >= 0.2	HMA a_1 Average
Bolivar	16	1	0.216	8.0	25.4	0.32	0.13	7	53	0.465
Clarke	16	2	0.273	36.5	18.9	0.38	0.18	1	93	0.462
Smith	15	5	0.214	7.0	22.7	0.26	0.13	3	70	0.423
Tippah	8	1	0.177	-11.5	30.7	0.23	0.07	4	43	0.44
Wilkinson	8	0	0.259	29.5	50.3	0.51	0.09	3	63	0.448
Summary	**63**	**9**	**0.232**	**16.0**	**32.0**	**0.51**	**0.07**	**18**	**67**	**0.451**

Table 14. Suggested Layer Coefficients for Existing HMA Pavement Layer Materials

Material	Surface Condition	Coefficient
HMA Surface	Little or no alligator cracking and/or only low-severity transverse cracking	0.35 to 0.40
	<10 percent low-severity alligator cracking and/or <5 percent medium- and high-severity transverse cracking	0.25 to 0.35
	>10 percent low-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >5-10 percent medium- and high-severity transverse cracking	0.20 to 0.30
	>10 percent medium-severity alligator cracking and/or <10 percent high-severity alligator cracking and/or >10 percent medium- and high-severity transverse cracking	0.14 to 0.20
	>10 percent high-severity alligator cracking and/or >10 percent high-severity transverse cracking	0.08 to 0.15
Stabilized Base	Little or no alligator cracking and/or only low-severity transverse cracking	0.20 to 0.35
	<10 percent low-severity alligator cracking and/or <5 percent medium- and high-severity transverse cracking	0.15 to 0.25
	>10 percent low-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >5-10 percent medium- and high-severity transverse cracking	0.15 to 0.20
	>10 percent medium-severity alligator cracking and/or <10 percent high-severity alligator cracking and/or >10 percent medium- and high-severity transverse cracking	0.10 to 0.20
	>10 percent high-severity alligator cracking and/or >10 percent high-severity transverse cracking	0.08 to 0.15
Granular Base or Subbase	No evidence of pumping, degradation, or contamination by fines	0.10 to 0.14
	Some evidence of pumping, degradation, or contamination by fines	0.00 to 0.10

Table 15. Summary of Structural Layer Coefficients For 4 Older Projects

County	Number of Tested Locations	Number of Omitted Locations	LFA a_2 Based on Equation 6 Average	% of Design	LFA a_2 Based on Equation 6 Coef. of Var.	LFA a_2 Based on Equation 6 Maximum	LFA a_2 Based on Equation 6 Minimum	HMA a_1 Calculated from SN_{eff}	Revised HMA a Calculated from Revised SN_{eff}
Forrest/Perry	16	4	0.18	-10.0	22.3	0.21	0.12	0.416	0.38
George	8	1	0.186	-7.0	25.1	0.25	0.10	0.485	0.312
Jones/Wayne	16	2	0.155	-22.5	13.7	0.18	0.10	0.542	0.434
Yalobusha	16	3	0.152	-24.0	36.6	0.21	0.08	0.434	0.434
Summary	**56**	**10**	**0.165**	**-17.5**	**23.3**	**0.25**	**0.08**	**0.47**	**0.401**

Chapter 6 Fly Ash

MDOT allows the use of both Class F and C fly ashes for LFA stabilized soil base course construction. Due to the difference in the chemical composition between these two ashes, the use of a Class C ash in a given soil and LFA blend typically results in a greater rate of initial increase in strength than that of a similar blend utilizing a Class F ash. The difference in relative rate of increase in strength can have a significant impact on the compaction characteristics of a given LFA mixture, which should to be accounted for both in the laboratory design and field construction phases of a LFA stabilization project. This difference may also be a consideration for use of Class C ash in lieu of Class F ash for late season LFA stabilized soil base course construction.

In addition to the differing relative increases in strength of these two classes of fly ash, the quality of fly ash supplied to MDOT soil stabilization projects has been a concern; therefore, a review of relevant fly ash topics is included in this chapter. Fly ash is a by-product of pulverized coal combustion used in electric power generating plants. Three characteristics of fly ash considered significant for the production of good quality LFA stabilized soil material include fineness, chemical composition, and uniformity.

Fineness of Fly Ash

The fineness of a particular ash is a measure of the percent retained on the No. 325 sieve (Fly Ash Facts for Highway Engineers, 1995). A coarser gradation can result in a less reactive ash, which in turn can cause a reduction in the ultimate strength gain of the LFA mix.   This was illustrated during the LFA mix design phase for the construction of U.S. 98 in Forrest and Perry Counties from Ralston to New Augusta. A LFA mix design utilizing conditioned ash from the Mississippi Power Plant at Escatawpa, Mississippi, resulted in 28-day strengths that did not meet the required 500 psi design criteria. This ash had 41 percent retained on the No. 325 sieve, which was cited as the primary reason for the lack of strength development (Jordan, 1989). AASHTO M 295 limits the maximum percent retained on this sieve to 34 percent.

Chemical Composition of Fly Ash

The chemical composition of a given fly ash dictates its classification into either a Class F or Class C ash. Other considerations relevant to fly ash chemical composition are the sulfate content and loss on ignition.

The source of coal burned directly impacts the chemical composition of the resulting fly ash. Class F fly ash is normally produced from burning anthracite or bituminous coal, and Class C fly ash is normally produced from burning lignite or subbituminous coal. These coal classifications are based upon differences in the kind of plant materials originally deposited, the degree of metamorphism that these plant materials experienced subsequent to deposition, and the range of impurity existing within the deposit (Bates, 1984). As with any natural deposit of material, variations occur within the ranges of the defining classification parameters from one coal deposit to another. This means that the elemental composition of Class F ashes from different sources of anthracite or bituminous coal is not the same, and they will not react with lime to the same extent. Similarly, the elemental composition of Class C ashes is not the same from different sources of lignite or subbituminous coal and will not experience pozzolanic and hydration reactions to the same degree. These considerations form the basis for MDOT’s current specification dictating that different classes of fly ash or different sources of the same class shall not be mixed or used in the construction without written permission from the engineer.

Fly ash is designated as either Class F or Class C based upon its chemical composition as defined in AASHTO M 295. Class F ashes generally contain less than 10 percent calcium oxide, or lime, whereas Class C ashes may contain more than 20 percent lime (Fly Ash Facts for Highway Engineers, 1995). This difference in the lime content can significantly impact the strength gain characteristics of soils stabilized with a Class F ash as opposed to a Class C ash.

Class F Fly Ash

Class F fly ash is a pozzolanic material, which means that it requires lime and water to affect strength gaining chemical reactions in the stabilized material. These pozzolanic chemical reactions are temperature dependent with higher curing temperatures affecting an increased rate of strength gain. The relatively high Mississippi late spring, summer, and early fall temperatures can affect significant pozzolanic chemical reactions, which result in the development of acceptable levels of stabilized material strength to facilitate subsequent construction operations. However, with the typically lower temperatures associated with late fall, there is a reduction in the rate of pozzolanic chemical reactions. This impacts late season construction considerations since little strength is developed in the stabilized material before the onset of the cool and wet winter months. Below about 40 ^{o}F these pozzolanic reactions stop, which results in no strength gain during periods of time when temperatures fall below this level. As a consequence, LFA stabilized material, using Class F fly ash and placed in late fall, will experience little strength gain until the following late spring elevated temperatures initiate further pozzolanic chemical reactions.

The effect of differing curing temperatures on the strength development of a LFA stabilized soil using Class F fly ash was considered by a limited laboratory investigation. A sample was obtained from the red sand topping material placed on the roadbed of the west bound lanes of U.S. 82 near Eupora, Mississippi, for a LFA stabilized base course. The non-plastic, Class 9, Group C topping material is typical of soils utilized for this purpose, with 100 percent passing the No. 10 sieve and 23 percent passing the No. 200 sieve.

Thirty 4-inch diameter Proctor size cylinders were fabricated with 3 percent lime, 12 percent fly ash and soil blend using Standard Proctor compaction effort. The 30 cylinders were subdivided into five sets with six cylinders per set. Each of these sets was subjected to a unique temperature, or sequence of temperatures, for curing and then tested for UCS strength. Figure 25 shows these five curing temperature/temperature sequences. Note that the soak time referenced in this figure is five hours. For LFA design, laboratory compacted Proctor-size cylinders are soaked for five hours prior to UCS testing, as opposed to 48 hours for LFA cores.

The no-cure set of cylinders was tested to see what strength the compacted LFA mix possessed before any additional strength developed due to the onset of pozzolanic reactions, but these cylinders were not be soaked because the non-plastic material would have disintegrated upon placement in water. An average of 39 psi UCS was obtained for this set of six cylinders.

The 100 ^{0}F curing temperature for 28 days corresponds to that used by MDOT for an LFA mix design. An average UCS of 590 psi was obtained, which exceeded the design requirement of 500 psi.

The 73 ^{0}F curing temperature for 28 days corresponds to that used by MDOT for samples of field-mixed LFA and soil blends. The average UCS was 115 psi, which is only 19 percent of the strength developed using 100 ^{0}F curing temperature for the same length of time.

The 50 ^{0}F curing temperature for 90 days was used to try to simulate the effect of the cool winter temperatures that typically occur during the months of December, January, and February. This 50 ^{0}F temperature was estimated based on the HMA mid-depth temperatures measured during FWD testing and was assumed representative for base course material underlying some HMA cover. The premise that the LFA stabilized material is covered during this time period is that Mississippi specifications do not allow the construction of LFA stabilized soil base courses during these months, and the contractors are encouraged to have this material covered with the next course of the pavement during this period of time.

As will be discussed in Chapter 8, the six cylinders included in this 50 ^{0}F curing set were further divided into two sets of three cylinders each to observe the effects of two different moisture conditioning methods. There was no significant difference in the average UCS between these two subsets; therefore, the average of the six cylinders is included in Figure 25. This average was 66 psi, little more than the strength of the no-cure, no five-hour soak average. Note that the cylinders cured at 50 ^{0}F were soaked for five hours prior to UCS testing, so a direct comparison in UCS cannot be made between these two sets of six cylinders. The point to be made here is that little strength development can be expected during the cool winter months, which is an important consideration for late season LFA base course construction utilizing Class F fly ash.

The 50 ^{0}F curing for 90 days followed by 28 days of curing at 100 ^{0}F temperature sequence was used to see if the LFA stabilized soil, placed at the end of one construction season, and experiencing little increase in strength over the subsequent winter months while subjected to saturating moisture conditions, would gain strength with increase in temperature during the following construction season. The average UCS of these cylinders was 441 psi, with 75 percent of the strength obtained with curing corresponding to that for an LFA mix design, and 88 percent of the design strength of 500 psi. This is a significant improvement over the 66 psi recorded for the 90-day curing at 50 ^{0}F and illustrates that the pozzolanic reactions responsible for increases in strength of an LFA mix do activate given a sufficient increase in temperature.

The difference between 590 psi and 441 psi may be attributed to the moisture conditioning of these samples during the 90-day curing at 50 ^{0}F. It is possible that continued curing of the cylinders past 28 days at 100 ^{0}F prior to UCS testing would have resulted in UCS strengths exceeding the 500 psi design strength, but this was not investigated in this laboratory evaluation.

Class C Fly Ash

Class C fly ash is also a pozzolanic material and gains strength through pozzolanic chemical reactions, but since it contains more inherent lime than the Class F ash, it possesses a self-cementing component of strength gain when combined with water.

The reactions associated with this self-cementing aspect are similar to, but faster than, the hydration of Portland cement. The hydration of Portland cement is retarded by the addition of gypsum, which enables time for concrete finishers to complete concrete placement work. Class C fly ash does not contain a retarder, thus the initial gain in strength associated with the hydration of this ash occurs at a greater rate than that of Portland cement. This aspect of Class C ash needs to be accounted for in both the design and construction of LFA stabilized materials as it can be either an asset or a detriment to the final quality of this material. This rapid initial gain in strength may be utilized as an asset for use in late season construction, but becomes a hindrance to attaining high levels of field compaction if the compaction is not completed in an expedient manner following placement and mixing of the fly ash. Chapter 7 addresses the importance of, and issues related to, field compaction of LFA stabilized materials.

As previously discussed, the source of coal is a factor affecting the extent of pozzolanic and hydration reactions observed among different Class C ashes. Another factor affecting the extent of the hydration reactions is the variation in the mineralogy of the Class C fly ash due to the process of coal combustion used at the various power producing plants. For example, if the coal is burned at temperatures exceeding about 1200 ^{0}C, and then the combustion products cooled relatively quickly, the ash produced will be a predominantly glassy or amorphous phase material. If the boiler design or operation allows for a slower rate of cooling of the fly ash, the formation of crystalline phase calcium compounds occurs in addition to the glassy phase material. The glassy phase materials usually comprise from between 60 to 90 percent of the Class C ash. The crystalline phase includes compounds of tricalcium aluminate, calcium oxide, and calcium sulfate. The significant point is that if the same source of coal is burned at two plants which use a different process for coal combustion, the resulting Class C fly ash from each plant will not have the same hydration properties due to the variation in the presence and relative proportions of the amorphous phase material and the crystalline compounds (Soil and Pavement Base Stabilization with Self-Cementing Coal Fly Ash, 1999).

Sulfur Content in Fly Ash

The sulfur content in fly ash is another important chemical consideration for LFA soil stabilization. AASHTO M 295 limits the amount of sulfur, in the form of sulfur trioxide, (SO_3), to 5 percent for both Class C and F ashes. This chemical requirement is derived from the fact that deterioration, and in some cases ultimate failure, by expansion from sulfate reactions with lime-stabilized soils has occurred in Nevada, Kansas, Texas, and Mississippi (Rollings and Rollings, 1996). In one of MDOT's early LFA stabilization projects, conditioned fly ash was used in a 9.58-mile segment of U.S. 84 in Wayne County between the Jones-Wayne County line and Waynesboro. Conditioned fly ash is fly ash with about 20 percent water added to it (Fly Ash Facts for Highway Engineers, 1995). A 6-inch LFA stabilized soil base course was constructed using 3 percent lime and 12 percent conditioned fly ash. About four weeks after construction the base course began to experience blow-ups with a total of 28 such occurrences during the second month subsequent to base construction. Figure 26 illustrates the surface deformation associated with a blow-up that occurred on this project.

In an effort to determine the cause of these blowups, x-ray diffraction testing was conducted on material obtained from within the areas of these blow-ups. The mineral ettringite was identified in this material. Ettringite can form when soluble sulfates, calcium and alumina are present in the stabilized soil system. This mineral occupies more than double the volume of its constitutive components, thus expansion occurs with its formation (Petry and Little, 1991). Subsequent hydration of the ettringite crystal effects an additional increase in the mass of this mineral (Mitchell and Dermatos, 1992). This expansion with formation and subsequent hydration of ettringite constitutes the mechanism of heave observed in pavement layers experiencing this phenomenon. In LFA stabilized soils, the lime supplies the calcium, and the fly ash supplies the alumina. In the case of the Wayne County project the conditioned ash had been obtained from a land fill. High sulfate-content scrubber sludge had also been placed in this land fill, and it is believed that the fly ash became contaminated with sulfates due to its close proximity to the scrubber sludge (Crawley, 1990).

The occurrence of blow-ups continued for about six months subsequent to the base course construction after which the stabilized soil system appeared to stabilize. The base course material in the areas where the blow ups occurred was removed and replaced with HMA. Testing for the presence of sulfates in potential sources of fly ash since the time of construction of the Wayne County project has resulted in avoiding further cases of this problem in MDOT LFA base course construction for both dry and conditioned fly ashes.

Loss on Ignition

Loss on ignition (LOI) is another chemical parameter associated with fly ash. LOI is a measure of the unburned carbon or coal remaining in the ash. Fly ash is used in both LFA soil stabilization and Portland cement concrete. When fly ash containing relatively high carbon contents is used in Portland cement concrete, significant air-entrainment problems can occur which may adversely affect the performance of that concrete (Fly

Ash Facts for Highway Engineers, 1995). AASHTO M 295 limits the LOI to a maximum of 5 percent for both Class F and C ashes. Since air entrainment is not a consideration with LFA stabilized soil, such a stringent limitation may not be required, and Section 714.05 of The Mississippi Standard Specifications for Road and Bridge Construction allows a maximum of 10 percent LOI for soil stabilization. Class F fly ash with an LOI of 16 percent was successfully utilized in a stabilized base course of a ramp in Delaware, and a 12 percent LOI fly ash was successfully used in Michigan for a base course (Golden, 2002). This limited data supports the current MDOT requirement for LOI when the fly ash is used for soil stabilization; however, it is recommended that research be conducted to quantitatively evaluate the impact of LOI on the reactivity of the fly ash.

Uniformity of Fly Ash

A third characteristic of fly ash affecting the quality of a LFA stabilized material is the uniformity of that ash from load to load as it is delivered to a given project site. For example, sources of coal are often blended at the production facility to achieve maximum efficiency from the available fuel, and even where sources are not changed, variations in blending can affect ash chemistry (Fly Ash Facts for Highway Engineers, 1995). Both the physical and the chemical properties of the fly ash used in the LFA mix design for a given project should be maintained in all of the shipments of ash to that project during field construction. This will aid in producing a consistent product along the length of that project with a quality corresponding to its design.

This topic of fly ash uniformity is a current concern to MDOT. Section 714.05.1 of The Mississippi Standard Specifications for Road and Bridge Construction specifies that the acceptance of fly ash shall be based on certified test reports, certification of shipment from the supplier and tests performed on samples obtained after delivery in accordance with the Department SOP. Current sampling frequency of the fly ash is one gallon for each 200 tons delivered to the project site. The fly ash specifications and associated quality conformance testing necessitates the development of an effective Quality Control/Quality Assurance (QC/QA) program to control the quality of fly ash shipped to MDOT projects.

The tests conducted by the MDOT Central Laboratory account for an elemental analysis of the fly ash; i.e., what elements are contained in the ash, which corresponds to the general basis for most specification requirements, but these tests do not provide a mineralogical assessment of that ash. For example, an elemental analysis will provide the amount of calcium in a sample of fly ash, expressed in the form of calcium oxide, but does not distinguish how much of the calcium is included in the amorphous form and how much is in each of the potential crystalline forms constituting that sample of fly ash (Soil and Pavement Base Stabilization with Self-Cementing Coal Fly Ash, 1999).

As previously discussed, the mineralogical composition is of particular importance when dealing with a Class C ash due to its impact on the hydration properties of that ash. This means that, in addition to consideration of consistency of the elemental makeup of a Class C ash, the consistency of the mineralogy of that ash should also be maintained between the design and all shipments to a given project. This becomes even more significant from a stabilization perspective when the self-cementing component of strength gain of this ash is considered in the performance of the stabilized base course layer, such as for late fall construction.

Performing an X-ray diffraction test on a sample of fly ash is one method of determining the mineralogical composition of that ash, but this test requires the requisite test equipment and trained technician to perform the test procedure. A simpler, but still relevant, test procedure is needed to fulfill a QC/QA test function.

Pozzolanic Reaction Test

The development of a pozzolanic reaction test has been suggested to fulfill the requirements of a QC/QA fly ash test (Little, 2002). A blend of lime and fly ash, with the same proportions as that required in the corresponding LFA mix design, is made into cubes, subjected to an accelerated rate of curing for two days, and then tested for UCS. The MDOT LFA mix design process requires up to 28 days before a proposed mix design is found acceptable for use. A pozzolanic reaction test could be used in screening potential combinations of lime and fly ash that do not sufficiently react before their use in the more time consuming LFA mix design process. During the course of field construction samples of the lime and fly ash being delivered to the project site could be obtained and tested using this procedure to ensure the same reactivity as that observed during the design process. It is recommended that a research study be initiated to develop a pozzolanic reaction test to establish acceptance/rejection criteria of a given LFA blend.

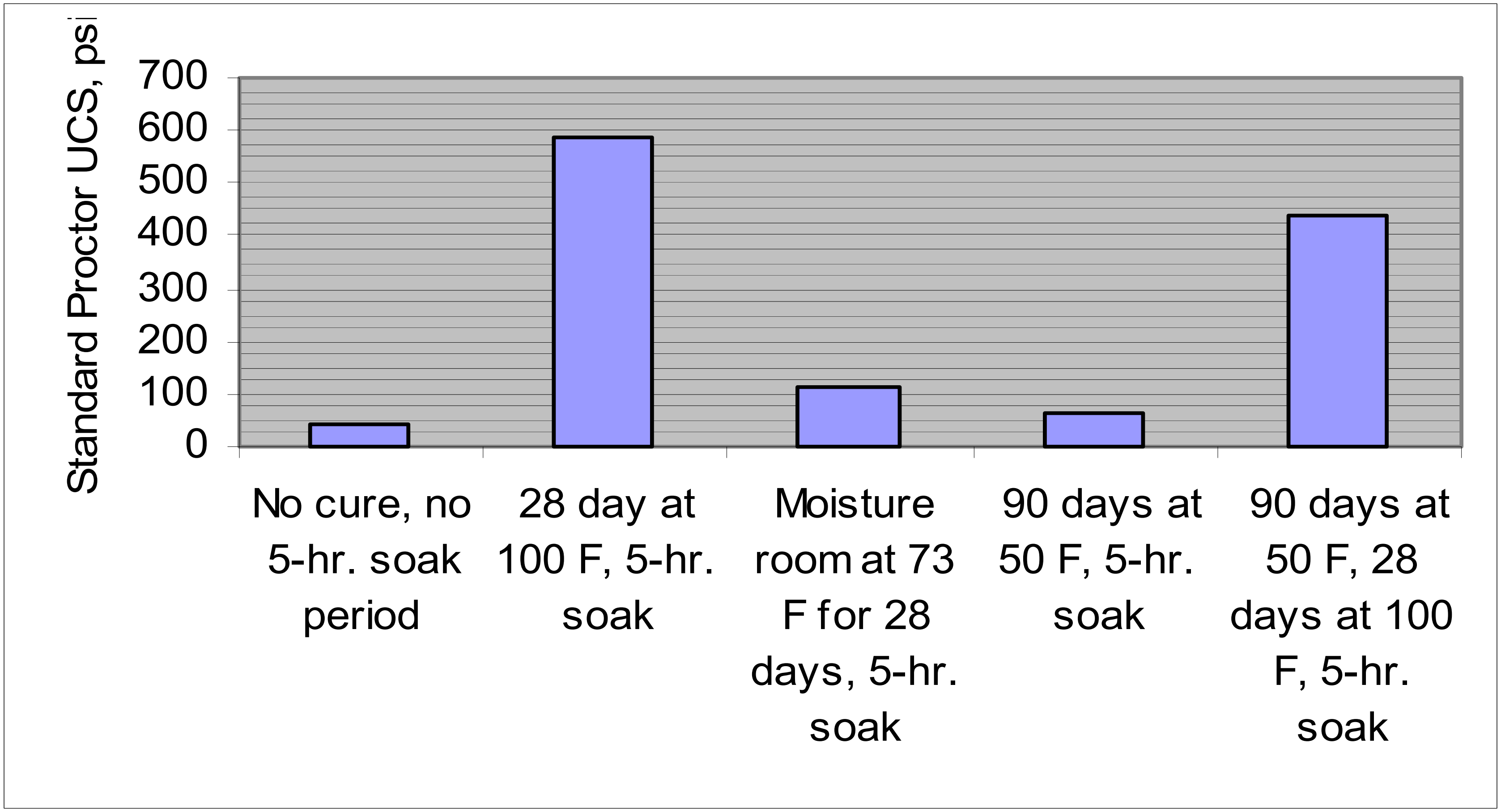


Figure 25. Variation in Strength of Laboratory Mixed and Compacted LFA Stabilized Soil for Various Curing Conditions

Figure 26. Surface Deformation Associated with a Blow-Up On U.S. 84 in Wayne County

Chapter 7 -- LFA Stabilized Soil Base Course Field Compaction Requirements

Chapter 5 noted the large variability in the in-situ LFA structural layer coefficient for the pavements tested in this study. While the average LFA layer coefficient for the five newer pavements exceeded the design value of 0.20, the large variability requires a significant reduction in the design value in order to design this pavement layer with 90 percent confidence. Figure 23 from Chapter 5 illustrated three approaches to achieve the current MDOT design value of 0.20. One of these approaches is to hold the variability constant, but increase the average value for this layer coefficient. As will be demonstrated in the current chapter, increasing the required level of field compaction is one way to increase this value. Increased levels of field compaction also reduce the amount of water that can be absorbed by the base course layer, which is a consideration for the durability of this pavement construction material. In this report standard and modified Proctor compaction efforts will be referred to as standard effort and modified effort respectively. Standard and modified proctor compacted densities will be referred to as standard density and modified density respectively.

Level of Compaction and LFA Stabilized Soil UCS

The effect of level of compaction, or density, on the strength of a LFA stabilized soil has been documented for over 45 years. In one laboratory study the strength of a LFA stabilized sand, classified as an A-2-4-(0), was increased 78 percent for a given lime, fly ash and soil blend simply by increasing the compaction effort from standard to modified effort (Viskochil, Handy, and Davidson, 1958). In a subsequent laboratory study three different fly ashes were evaluated for a given lime and dune sand blend. Depending on the particular fly ash used, the strength of the LFA stabilized dune sand was increased from 84 to 140 percent due to the same increase in compaction effort (Mateos and Davidson, 1963).

The two referenced laboratory studies address levels of compaction equal to and greater than that obtained with the standard effort. Figure 27 illustrates the variation in strength for levels of compaction less than 100 percent standard effort for four different materials stabilized with LFA. The data for this figure was obtained from Figure 27 of NCHRP Synthesis of Highway Practice 37, Lime-Fly Ash-Stabilized Bases and Subbases, hereafter referred to as NCHRP 37.

The data provided in the literature illustrates that increasing the level of compaction of a LFA stabilized soil affects increases in the strength of that material. From Chapter 5, Figure 20 demonstrates that as the UCS of the stabilized material increases, the structural layer coefficient also increases for that material. The red sand topping referred to in Chapter 6 was used in a limited laboratory investigation to demonstrate the effect of compaction level on the structural layer coefficient of a LFA stabilized soil. This topping material was selected for the investigation because it represents the type of soils frequently encountered in Mississippi LFA stabilization projects.

The range of percent standard densities evaluated in the investigation was based upon the field density requirements for this material at the onset of the current study. Initially, the density requirement for a completed LFA stabilized soil base course was determined by first referencing Special Provision No. 907-311-6, “Lime-Fly Ash Treated Courses” dated October 9, 2000. This Special Provision referenced paragraph 308.03.9 of The Mississippi Standard Specifications for Road and Bridge Construction which included the density requirement for a soil-cement stabilized pavement layer. The required density was based on the pavement layer under consideration and the type of soil included in the stabilized blend. For the Class 9 red sand topping material used in a base course application, the specifications required a 94 percent standard density lot average when stabilized with either cement or LFA. When this same material blend was used for a chemically stabilized subgrade layer, the compaction requirement was 93 percent. For the current study, the laboratory endeavor considered a range from 93 to 100 percent standard density.

The standard effort requires 25 blows per each of three layers in a 4-inch diameter mold to obtain 100 percent standard density. A curve of number of blows per layer versus percent standard density was developed in this laboratory investigation for the blend of LFA and red sand topping material. From this curve it was determined that compacting the blended material in three layers with 11 blows per layer resulted in a compacted density of approximately 93 percent standard density, with 15 blows per layer required for 96 percent standard density. Note that compacting this material to 100 percent standard density required over twice the number of blows per layer as required for 93 percent standard density.

Three sets of six cylinders were compacted for this laboratory investigation. Each of these cylinders was compacted in three layers. One set of six cylinders was compacted with 25 blows per layer, the second set with 15 blows per layer, and the third set with 11 blows per layer. These cylinders were cured for 28 days at 100 ^{0}F, soaked for five hours and then tested for UCS. Table 16 includes the dry density and UCS after soaking for each of these 18 cylinders as well as the corresponding average values for each of the three sets of cylinders. Figure 28 illustrates the change in LFA UCS with variation in the percent standard Proctor density. Note that a 50 percent increase in UCS can be realized by increasing the compaction level of this material from 94 to 100 percent standard density.

Figure 28 illustrates an important point. The relationship between material design and construction requirements need to be clearly understood. For a LFA mix design cylinders are compacted in the laboratory to 100 percent standard density. Acceptance of the mix is based upon the achievement of a 500 psi Proctor UCS. Construction specifications at the initiation of this study required that the LFA base course would be compacted to a minimum of 94 percent standard density. These specifications were in affect requiring a field Proctor UCS of 392 psi, or about 22 percent less than the laboratory mix design required strength. For the particular LFA and soil blend considered in this example, the material would need to be compacted to 97.3 percent standard density in order to achieve the laboratory design strength of 500 psi.

As noted in Chapter 5, the use of UCS to determine a structural layer coefficient is not a preferred method; however, it allows the use of an easily obtained laboratory test value that is amenable to evaluating the change in layer coefficient given a change in compacted density. The UCS of each cylinder was used to determine a structural layer coefficient for the LFA stabilized material using the relationship shown in Figure 20. The equation for this relationship is:

$$a_2 = (-0.0000000554*(UCS^2)) + (0.000289*UCS) + 0.07 \quad \text{Equation 7}$$

Table 16 includes the LFA a_2 for each of the 18 cylinders as well as the corresponding average values for each of the three sets of cylinders.

Figure 29 illustrates the change in LFA a_2 with variation in the percent standard density. For the particular LFA and soil blend considered in this example, the material would need to be compacted to 97.3 percent standard density in order to achieve the design LFA structural layer coefficient of 0.20. Increasing the level of compaction from 94 to 100 percent standard density increases the layer coefficient from 0.175 to 0.221, which is a 26 percent increase in this value. While 0.221 exceeds 0.20, Figure 23 from Chapter 5 requires an average LFA structural layer coefficient of 0.295 for the current level of field variability to achieve 0.20 with a 90 percent level of confidence. Substituting 0.295 into Equation 7 and solving for UCS, an UCS of 954 psi is required to obtain this average value for the layer coefficient.

Suggested Upper Limit for Level of Compaction

The current study did not evaluate the affect of compaction levels exceeding 100 percent standard density on the UCS. Assuming the minimum value of 78 percent improvement in strength from the reference by Viskochil, et. al., and applying it to the blend used in the current study, a UCS of 1,050 psi could possibly be obtained by increasing the compaction level from 100 percent standard effort to 100 percent modified effort. This illustrates that in order to achieve 954 psi UCS and the corresponding desired structural layer coefficient of 0.20 with 90 percent reliability, a relatively high percentage of modified density would need to be obtained in the field if the current level of variability is retained in the constructed base layer. Consistently achieving such a high level of density would be an unrealistic expectation given the extent of low strength fine grained soils that constitute the pavement foundation across much of the State.

Evidence of the prevalence of low strength fine grained subgrade soils in Mississippi is provided by a review of recommended pavement designs. The soaked CBR value of a given soil provides an indication of the strength of that soil for supporting an overlying pavement structure. A review of recommended pavement designs for each of the six Districts comprising Mississippi revealed a prevalent design CBR, which is a soaked CBR value, of 5 for the design soil in all but the 6th District. A CBR design value of 5 is considered a minimum value in MDOT pavement design practice. Existing soil within the top 3 feet of the subgrade that does not have a value of 5 or greater is typically removed and replaced with a better quality material. In a few cases the lack of locally available better quality material has required the use of the on-site soil with a corresponding lower design CBR value.

The prevalence of low strength fine grained foundation soils in effect places an upper limit on the level of compaction that can be achieved in the overlying pavement layers. Given this limitation, it is not sufficient to use an increase in the required level of field compaction as the only remedial action to obtain the desired layer coefficient with the desired level of reliability. Reducing the variability of the in-situ layer coefficient in conjunction with increasing its average value must be accomplished in order to obtain these two objectives. Chapter 8 addresses the issue of reducing the variability of the layer coefficient in the constructed base course layer.

An upper limit must be determined for a level of compaction which can be consistently achieved with reasonable effort. This upper limit will facilitate the evaluation of the maximum increase in average LFA layer coefficient that can be realized from this remedial action. Since MDOT has not been requiring densities in excess of 100 percent standard density, there is no road construction experience within Mississippi upon which to base this determination. However, the construction of the soil foundation for the Nissan plant near Canton, Mississippi, provides an idea for the selection of this upper limit.

A considerable amount of soil foundation construction for the Nissan plant was conducted during February and March of 2001. A tan silty clay with traces of fine sand and a plasticity index varying from 16 to 21 percent was treated with between 2 and 3 percent lime to facilitate its use for fill construction during these typically wet winter months. The required level of field compaction was 98 percent standard density. Table 17 includes compaction statistics for four days of earth work on this project. Note that 49 percent of the test results were equal to or exceeded 100 percent standard density and that 5 percent were equal to 103 percent standard density for the four days considered in this example.

A lime treated clay soil and a LFA stabilized granular soil are not the same materials; however, the experience with the treated clay suggests that 103 percent standard density may be a reasonable value for consideration as an upper limit for a LFA stabilized soil base course. Only 5 percent of the test results were at this level; however, 98 percent standard density was required, and it is reasonable to assume that a greater percentage of the test results would have equaled or exceeded 103 percent had the required density been greater than 98 percent.

In response to a recommendation from the MDOT Materials Division, three LFA and soil blends were evaluated in the laboratory to estimate what percentage of modified density corresponds to 103 percent standard density for a typical Mississippi LFA and soil blend. Table 18 lists the maximum dry density and optimum moisture content for these three blends from both the standard and modified Proctor compaction tests. From this data a value of 96 percent modified density was selected to correspond to the 103 percent standard density.

The 96 percent modified density is in general agreement with some of the suggested levels of required compaction found in the literature for both lime and LFA stabilized soils. A Chemical Lime Group publication suggests 95 percent of modified density for the compaction of lime treated material (Lime Uses in Transportation Construction, 1992). For lime-treated fine-grained soils this degree of compaction is difficult to achieve; however, it is possible for more granular soil-lime mixtures (Lime Stabilization, 1987, Little, 1995). Table 4 in Chapter 2 shows that the LFA stabilized soils used for base course construction fall under the latter category as a granular material.

An American Coal Ash Association (ACAA) publication recommends two values for the level of required compaction. On page 46 it suggests 97 percent of standard density, and on page 12 of Appendix A it suggests 97 percent of modified density, method C, with the exception that the requirement for five layers is changed to three layers in the compaction procedure (Flexible Pavement Manual, 1991). Using three layers instead of five layers provides a compaction effort intermediate between the standard and modified efforts. This intermediate level of compaction is also specified in paragraph 10.3 of ASTM C593, Standard Specification for Fly Ash and Other Pozzolans for Use With Lime. An inquiry was submitted to the ACAA as to which level of compaction should be required for field construction.

The response to this inquiry acknowledged that increasing the compacted densities increases the strength of the stabilized material. Preference was given to the higher required density with qualifications that included economics and the quality of the pavement foundation (Boggs, 2002).

The first qualification is based upon economics since the greater effort required for the increased compacted densities translates into greater construction costs to the State. MDOT's State Estimator suggested that increasing the required level of field compaction to modified density would cost approximately 10 percent more for item 907-311-A: Processing Lime and Fly Ash Treated Course. For a 6-inch base course this would translate into about $0.13 more per square yard.

The second qualification, in which economics also have an impact, is the quality of the pavement foundation. The ACAA response addressed the issue of poor sub-soil conditions resulting from either poor soil horizons or high groundwater levels. In these areas it is very difficult to obtain the higher compaction targets.

A similar concern was also expressed by the Blain Companies in a letter dated May 28, 2002, to Mr. Owen Richards and Mr. David Trevathan, both Mississippi Road Builders Association Committee members. The Blain Companies has extensive experience in chemical base and subbase soil stabilization projects in Mississippi, as well as other southeastern states. This letter stressed the need for a good pavement foundation in order to obtain the higher compacted base course density being proposed in the current study, and suggested that such a foundation could be obtained by increasing density requirements from bottom to top. The need for a firm foundation upon which to adequately compact overlying materials is also stressed in other references (Rollings and Rollings, 1996, NCHRP 37, 1976).

Required Improvements in the Pavement Foundation

The pavement foundation includes the basement and design soils and the chemically stabilized top 6 or 8 inches of the design soil prism.

Basement and Design Soils

At the onset of this study the required density for basement and design soils was 94 and 96 percent standard density respectively. In response to the bottom to top approach for pavement foundation improvement it is recommended to increase the basement and design soil requirements to 96 and 98 percent standard density respectively.

Figure 30 is an illustration of the potential improvement in the strength of a fine-grained soil with increasing compacted density. The data for the three points defining the curve from 100 to 109.4 percent standard density was obtained from the reference by Rollings and Rollings, 1996. Specimens of a sample of Mississippi lean clay, with a liquid limit of 41 and a plasticity index of 20, were compacted at varying molding moisture contents with three different levels of compaction effort. These compaction efforts included both the standard and modified efforts and one intermediate level of effort. The soaked CBR was determined for each of these specimens. For Figure 30 the three soaked CBR values are from the specimens molded at the optimum moisture content corresponding to each of the three compaction efforts. The three data points were plotted and a best-fit curve developed with Excel's curve-fitting function. The points below 100 percent standard density are extrapolated values from the developed curve since the referenced laboratory test results did not include data within this range of interest. Figure 30 illustrates that if this lean clay were placed as design soil the soaked CBR would increase from 7.9 to 9.7 by increasing the density from 96 to 98 percent standard density. This represents a 23 percent increase in the strength of this layer.

Increased levels of required compaction do not guarantee a stronger pavement foundation in all cases encountered in the field. Laboratory investigations have demonstrated that fine-grained soils compacted using a modified Proctor effort and wet of optimum can experience a decrease in strength with increasing compacted density (Rollings and Rollings, 1996). The occurrence of this phenomenon in Mississippi road construction is highly unlikely since the range of compacted densities considered for subgrade soils is below 100 percent standard density.

High Volume Change Soils in the Design Soil Prism

Special consideration should be made for high volume change soils when they are encountered in the design soil prism. When high volume change soils are compacted to relatively high levels of density these soils are subject to changes in volume with changes in moisture content. MDOT SOP No. TMD-20-14-00-000 entitled “Standard Design Procedures for Construction of Roadways Through High Volume Change Soils” lists three methods for contending with high volume change soils. Method 1 is the preferred method since it requires the removal and replacement of these type soils from the design soil prism. In some cases there are no locally available better quality borrow materials within an economical haul distance and the pavement designer is required to contend with these soils within the design soil prism. Method 2 allows up to 18 inches, and Method 3 allows up to 28 inches, of untreated high volume change soil to remain within the bottom of this soil prism. In these cases the 98 percent standard density requirement may be too high, and consideration should be given to possibly lowering this recommended density requirement for the untreated materials remaining in the soil prism. This evaluation should be performed on a case-by-case basis rather than automatically reducing the required level of compaction for every situation encountered in the field.

Chemically Stabilized Subgrade Layer

A major improvement in Mississippi pavement foundations was implemented in 1999 with the requirement for a chemically stabilized subgrade layer located immediately beneath the pavement structure. This is typically a 6- or 8-inch layer of subgrade soil mixed with lime, LFA or cement, with the selection of stabilizer depending on the characteristics of the soil. The chemically stabilized subgrade layer provides a construction platform that allows the attainment of higher compacted densities in the overlying pavement materials due to its stiffening effect on the pavement foundation. The presence of this layer also serves to reduce the flexural stress/flexural strength ratios that develop in the overlying base course due to traffic loading. This topic is addressed in detail in Chapter 11. As with a LFA stabilized soil base course, the quality of the chemically stabilized subgrade layer is improved with compaction to relatively high levels of density.

Recommended Compaction Level for Lime-Stabilized Fine-Grained Subgrade Soils

At the onset of this study the required level of compaction for a lime stabilized fine-grained soil was 95 percent standard density. Given the prevalence of weak subgrade soils throughout Mississippi, especially in the northern and central regions of the State, and the low levels of required compaction in the basement and design soils, 95 percent was a reasonable value. However, increasing the level of compaction in these foundation soils improves the strength of these soils, thereby allowing an increase in the level of required compaction for the overlying chemically stabilized subgrade layer. It is recommended to increase the required compacted density of the lime-treated subgrade layer to 100 percent standard density for all new pavement construction that includes a design soil CBR equal to or in excess of 5.

In cases where the lack of locally available better quality material has required the use of on site materials with a design CBR of less than 5, or the use of high volume change soils requiring a reduction in recommended density, in the design soil prism, a sufficiently stiff soil foundation may not be available to support the recommended increase in level of compaction for the overlying lime stabilized subgrade layer. In these cases it may be necessary to maintain the current 95 percent standard density requirement for this stabilized layer. However, an evaluation should be performed on a case by case basis rather than automatically reducing the required level of compaction for every weak foundation condition encountered in the field. The contractor should employ every reasonable means available to achieve 100 percent standard density. If it is demonstrated to be impossible to consistently achieve this level of compacted density, then a reduced level of required compaction should be allowed for the lime stabilized subgrade layer of that particular project.

In those cases where the lime stabilized subgrade layer cannot be compacted to 100 percent standard density, the resulting pavement foundation may not be stiff enough to support the recommended 96 percent modified density in the overlying LFA stabilized soil base course. A corresponding reduction in the recommended base course density may be required; however, as with situations involving the lime stabilized subgrade layer, this should also be decided on a case by case basis. Reducing the required base course density will reduce the quality of the base course material, which should be reflected in the pavement design process by the use of a lower structural layer coefficient for this material.

Recommended Compaction Level for LFA or Cement-Stabilized Coarse-Grained Subgrade Soils

At the onset of this study the required levels of compaction for a LFA or cement stabilized soil was based on the pavement layer under consideration and the type of soil included in the stabilized blend. For example, recall that for the Class 9 red sand topping material used in a base course application, the specifications required a 94 percent standard density lot average. When this same material blend was used for a chemically stabilized subgrade layer, the compaction requirement was 93 percent. Subgrade soils to be stabilized with either cement or LFA typically possess greater inherent strength than fine-grained soils requiring stabilization with lime; therefore, these foundation soils will typically support greater levels of compaction in overlying layers. It is recommended to compact cement or LFA stabilized subgrade layers to 100 percent standard density.

Current Required Compaction Levels

The required level of field compaction was increased for a LFA stabilized soil base layer during the course of this study. Special Provision No. 907-311-7 “Lime-Fly Ash Treated Courses,” dated November 26, 2002, refers to Special Provision No. 907-308-1

“Portland Cement Treated Courses,” dated November 26, 2002, which dictates that the average of five density tests for a given lot are required to equal or exceed 98 percent standard density, with no single density test below 94 percent.

The required level of field compaction was increased for all but one of the materials reviewed for the pavement foundation during the course of this study. The current specification requires 95 and 98 percent standard density respectively for the basement and design soils. The requirement for lime stabilized fine-grained subgrade soils remains unchanged at 95 percent standard density, and the requirement for LFA and cement stabilized subgrade soils has been increased to 98 percent standard density.

Summary of Compaction Levels

Table 19 provides a summary of the required compaction levels at the onset of this study, the current required levels and the recommended levels proposed in the current study for the basement and design soils, the chemically treated subgrade layer, and the LFA base course. For the required compaction levels at the onset of this study, the red sand topping is used as the reference material in those instances where a variable requirement for compaction, based on the type of soil stabilized, is made in the specifications. The densities listed under the “Recommended” column provide the bottom to top increase in compacted densities as suggested in the letter from the Blain Companies.

A significant increase in recommended levels of compaction is proposed in the current study. A review of these recommendations was performed by Dr. Dallas Little (Little, 2002). Dr. Little indicated that the 96 percent modified density requirement for the LFA stabilized soil base course was feasible given that a chemically treated subgrade layer is now included in the pavement foundation and assuming MDOT pursues the recommended bottom-to-top improvement in compacted densities of the layers comprising the pavement foundation.

Tentative LFA Stabilized Soil Design Structural Layer Coefficients

Table 20 provides a list of tentative LFA stabilized soil design structural layer coefficients for varying levels of compacted density and levels of variability. To develop this table the average in-situ LFA layer coefficient of 0.232 corresponding to the average in-situ compacted density of 95.7 percent standard density of the five newer projects is first adjusted for various levels of compacted density. Then the variability of the in-situ material is considered for the various levels of compacted density to provide the tentative design layer coefficient values for 90 percent confidence.

The approach discussed in Chapter 5 for obtaining an LFA structural layer coefficient based on the UCS of the LFA stabilized material was used to make the adjustments for variation on compacted density. As previously discussed, the use of UCS to determine a structural layer coefficient is not a preferred method; however, it allows the use of an easily obtained laboratory test value that is amenable to evaluating the change in layer coefficient given a change in compacted density. The UCS laboratory test results of the red sand topping material were used to obtain structural layer coefficient values for various levels of compacted density via the successive use of Figure 28 followed by Figure 29. These values are entered under the column “Average LFA a_2 LFA UCS” in Table 20.

The value for 96 percent modified density is approximately equivalent to 103 percent standard density and the LFA layer coefficient for this level of compaction was obtained by extrapolation of the line in Figure 29. This is probably a conservative estimate for this particular value since a greater rate of strength increase with compacted density typically occurs for compaction levels exceeding 100 percent standard density.

The average field density was 95.7 percent standard density for the five newer projects. Based on the UCS method the blend of topping and LFA has a layer coefficient of 0.188 corresponding to this level of compaction. The actual average LFA layer coefficient based on the AASHTO method for the five newer projects was 0.232, or 0.044 greater than that for the topping blend using the UCS method. Each value under the column "Average LFA a_2 AASHTO" is 0.044 greater than the adjacent value shown under the column "Average LFA a_2 LFA UCS." The "average" values based on the AASHTO method were then corrected to design values for three levels of field variability using Figure 23.

These tentative design values need to be verified in the field on several projects using the preferred AASHTO procedure as discussed in Chapter 5 before assignment of these values for routine MDOT pavement design. Recall from Chapter 5 that one of the variables affecting the value of a structural layer coefficient is the location of the layer of interest within the pavement structure. Field verification with new pavements will also include the chemically stabilized subgrade layer; therefore, the revised layer coefficients will account for this added layer in the pavement structure.

The values in Table 20 do constitute the basis for the recommendations being made in both the current and the following chapters of this report. The design value can be increased by increasing the required level of field compaction, but this remedial measure alone does not provide a sufficient improvement in the quality of the material to maintain the current MDOT design level of 0.20. The variability of the in-situ material must be reduced in order to use this design value with a relatively high level of confidence. Note that reducing the level of variability in conjunction with increasing the level of compaction may allow the use of a design layer coefficient in excess of 0.20 thus potentially reducing the required thickness of overlying pavement layers.

Durability and Compacted Density

As discussed in Chapter 6, LFA stabilized material requires time and temperatures exceeding 40 ^{0}F for effective strength gain to occur, especially when a Class F fly ash is used in the blend. This is an important consideration for late season LFA construction given the relatively cool temperatures of late fall and winter. The saturation of compacted LFA and soil mixtures, before the occurrence of significant strength gain, was identified as one of the reasons for several premature pavement failures in Mississippi (Crawley, 1998).

NCHRP No. 37 includes a discussion of distress observed in three pavements that included a LFA stabilized soil layer.

> "The types of distress observed during this investigation suggest that three factors are involved in the pavement distress. The distress is due primarily to deterioration of the LFA material. This deterioration is, in turn, the direct result of excess moisture in the base material and inadequate density of the LFA, especially along the pavement edge. As illustrated by

> the data from Winchester Road, when adequate density is achieved, LFA materials develop and maintain a high level of strength. Conversely, it can be shown that reductions in the compacted density result in significantly lower strength and sharply reduced durability for these materials."

Table 16 includes the moisture content, after soaking for five hours, for each of the 18 cylinders as well as the corresponding average values for each of the three sets of six cylinders. Figure 31 illustrates the reduction in moisture content with increasing compacted density for the LFA stabilized red sand topping. Note that increasing the density from 94 to 100 percent standard density resulted in an 18 percent reduction in the amount of water absorbed in this stabilized material. Compaction above 100 percent standard density would result in an even greater reduction in the amount of absorbed water.

The red sand topping had 23 percent non-plastic fines passing the No. 200 sieve. When the 3 percent lime and 12 percent Class F fly ash were added for stabilization, an additional 15 percent "fines" were mixed into this soil. Initially, before any pozzolanic reactions occur, the strength and behavior of this material in a pavement layer corresponds to essentially that of a silty sand soil or an unbound granular material. Given sufficient time and curing temperatures the blend experiences pozzolanic reactions and becomes more like a cement-bound material. If this type of blend is placed in late fall and little strength gain occurs during the following winter months, the response of this material to increases in moisture content will be more like that of an unbound granular material, not a cemented material.

An indication of the difference in behavior between the uncured and cured blend can be obtained by looking at the possible increase in the design structural layer coefficient as the material is transformed from an unbound granular material to that of a cemented material. The red sand topping including the lime and fly ash “fines” would classify as a borderline Class C Group 9 material since the total “fines” are just under 40 percent and 100 percent of the blend is allowed to pass the No. 10 sieve for this classification. Class C Group 9 materials can be used as an unbound granular subbase material with an assigned design structural layer coefficient of 0.09. Thus, right after placement this LFA and soil blend would behave as an unbound granular material with a design layer coefficient of 0.09, and with sufficient curing would become a cemented material with a design layer coefficient of 0.20.

For cement bound materials the presence or absence of moisture has no effect on the direct response of this material during deflection testing. However, for unbound materials, at a given density and stress level, moisture content is probably the most significant factor affecting the modulus of this material. The modulus of an unbound material can decrease by several factors with increasing moisture content (Pavement Deflection Analysis, 1994).

The following quote succinctly addresses the issue of density in relationship to the strength of an unbound subgrade or base course material (Yoder and Witczak, 1975):

> “Proper compaction of subgrades and base courses for highways and airports is essential. Compaction increases density with a consequent lower potential of moisture content, even in the event of subsequent saturation. Both of these factors result in an increase in strength.”

Based on the forgoing discussion, increasing the level of required density of the LFA and soil blend will result in an increase in the unbound strength of the base course layer at the time of placement. This will aid in reducing the incidence of premature pavement failures due to saturation of this layer prior to significant pozzolanic-induced strength gain.

Compaction Considerations When Using Class C Fly Ash

Chapter 6 included a discussion on the fundamental difference between the strength gain characteristics of a LFA stabilized soil when using a Class C ash as opposed to using a Class F ash. In addition to the pozzolanic reactions that both classes of ash experience, Class C ash has a hydration component that can potentially increase the early strength gain of the LFA stabilized soil. Recall that the initial gain in strength associated with the hydration of this ash occurs at a greater rate than that of Portland cement. In order to derive the benefit of the hydration component, the compaction of the blend must be completed in an expedient manner.

Some areas of the United States have access to very reactive Class C fly ashes. These ashes are referred to as self-cementing fly ashes since the strengths developed with hydration allow them to be used for soil stabilization without the addition of lime. Both fine and coarse grained soils have been stabilized with these self-cementing ashes. MDOT does not use a Class C fly ash in a soil stabilization application without the addition of lime, and the practice of LFA stabilization is limited to soils having a plasticity index of 10 or less. The following example uses a self-cementing ash in a clay soil without the addition of lime, but is included in the current discussion since it provides an excellent illustration of the potential affect of delayed compaction when using this class of ash.

The curves in Figure 32 were developed for a Class C fly ash and clay soil blend and illustrate the significant impact of a two-hour delay in compaction on the density of the blend (Ferguson and Levorson, 1999). One factor for this reduction is the free lime from the ash reacting with the clay minerals and producing flocculation and agglomeration of particles within the blend. However, a second and more significant factor is the cementitious products formed during hydration of the ash. This second factor is relevant to MDOT LFA stabilization projects using a more granular material.

> "The primary influence on the compaction characteristics; however, is the cementitious products formed during hydration of the ash. Cementitious bonds formed between soil grains must be disrupted in order to relocate the grains into a more dense state. The effective compactive energy is thus reduced by the amount of energy required to disrupt the bonds. Also, the compactive energy applied may not be sufficient to disrupt all bonds and the soil grains may remain in a relatively loose state. The reduction in maximum density with delayed compaction is dependent primarily on the hydration characteristics of the ash. Clay soils stabilized with a 'hot ash' can have a .....10 to 15 pcf reduction in maximum density with a 2 hour compaction delay."

The 10 to 15 pcf reduction in maximum density is associated with a stabilized clay soil; therefore, such a large reduction would not be expected for the granular materials used in MDOT stabilization work. The point being made here is that a reduction does occur with delayed compaction. This can have implications for the quality control associated with construction of a LFA stabilized soil base course when using a Class C ash. The maximum dry density and optimum moisture content of a blend of LFA and soil is determined for material that is compacted immediately after mixing; however, a two-hour delay between mixing and compacting is allowed during field construction. The resulting Proctor curve controlling the quality control may not be representative of the material in the field. When using Class C fly ash for a LFA stabilization project, it is recommended to maintain the same delay in compaction when developing the daily Proctor curve for controlling field densities as the delay in compaction during construction. This recommendation is particularly important when applied to materials being compacted to 96 percent modified density.

The curve in Figure 33 was developed using the same material and reference as for Figure 32. Neither of the maximum UCS values shown in this figure satisfies the 500 psi design strength for an MDOT LFA mix design. The point being made here is that delayed compaction can cause a reduction in the strength achieved in the field relative to the design strength since LFA mix design cylinders are fabricated immediately after mixing of the materials in the laboratory. It is recommended to maintain the same delay in compaction during the laboratory design phase as the delay in compaction during construction. This requirement may lead to the incorporation of a greater percentage of a given Class C ash, or possibly the exclusion of the particular ash; however, the laboratory derived strength will more closely model that being obtained in the field.

The discussion regarding compaction of a LFA stabilized soil using Class C ash has focused on the effect of delayed compaction with corresponding reductions in both density and UCS. However, if minimal delay between mixing and compaction is promoted during field construction, the gain in strength due to the hydration of the Class C ash could be a benefit for late fall construction. Providing sufficient supplies of Class C ash are available, the exclusive use of this type fly ash with lime during late fall construction may help to alleviate some of the negative affects observed due to the saturation of this layer. This would necessitate a change in the methodology currently employed in field construction in that only relatively short sections could be mixed at a time in order to facilitate timely compaction of the blend (Ferguson and Levorson, 1999). It is recommended that further research be pursued to investigate this possibility.

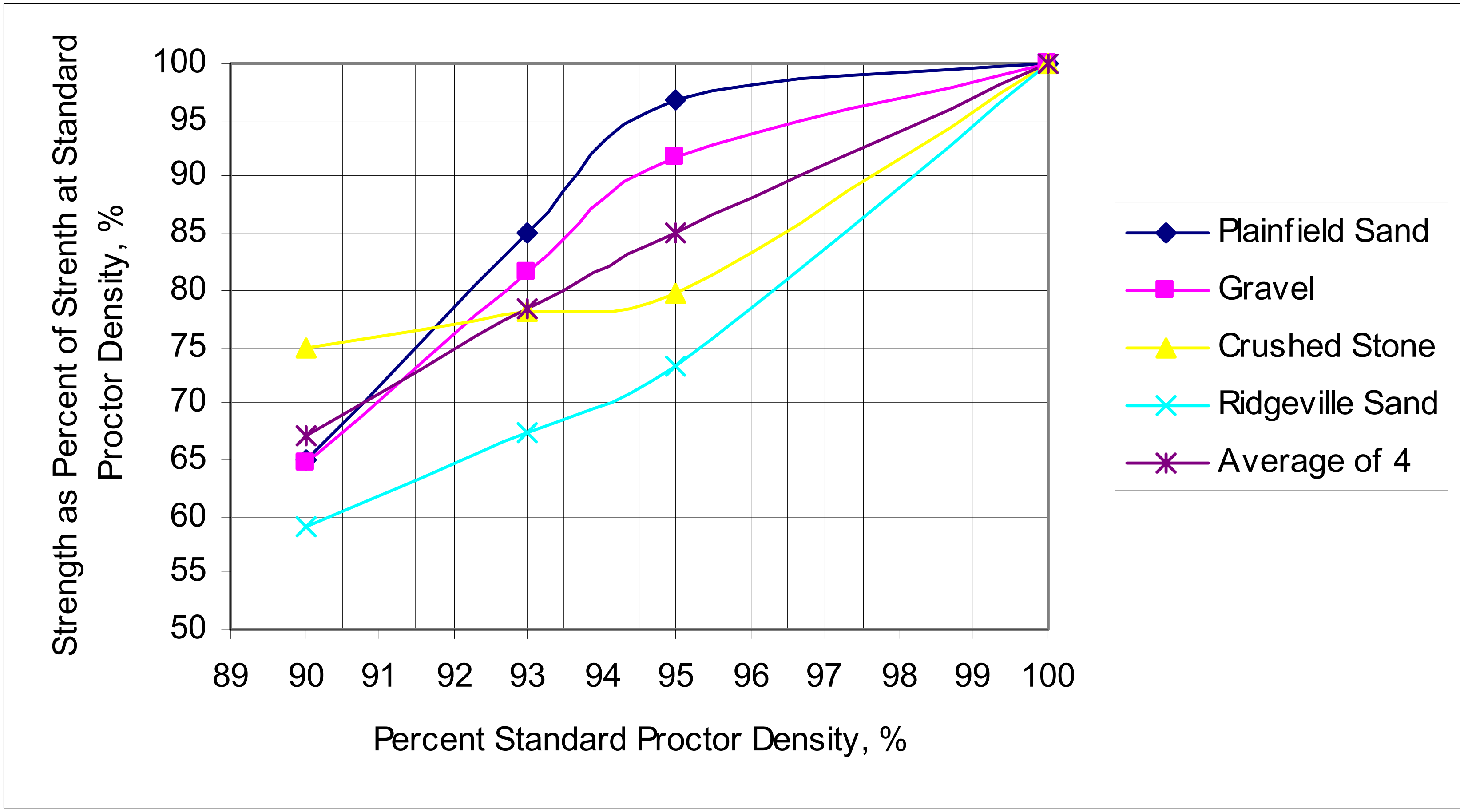


Figure 27. Strength as Percent of Strength at Standard Proctor Density vs. Percent Standard Proctor Density

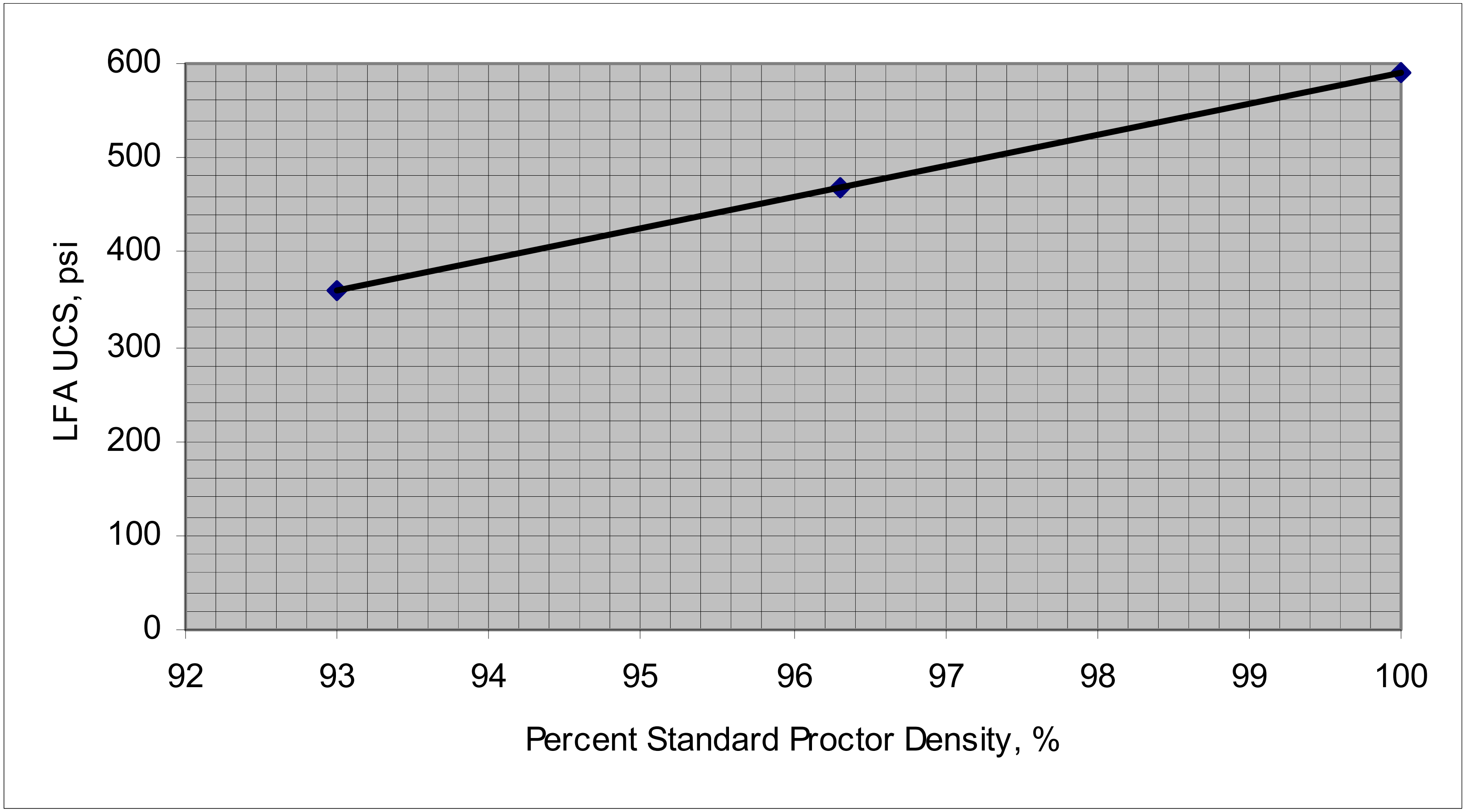


Figure 28. LFA UCS vs. Percent Standard Proctor Density for Red Sand Topping Material

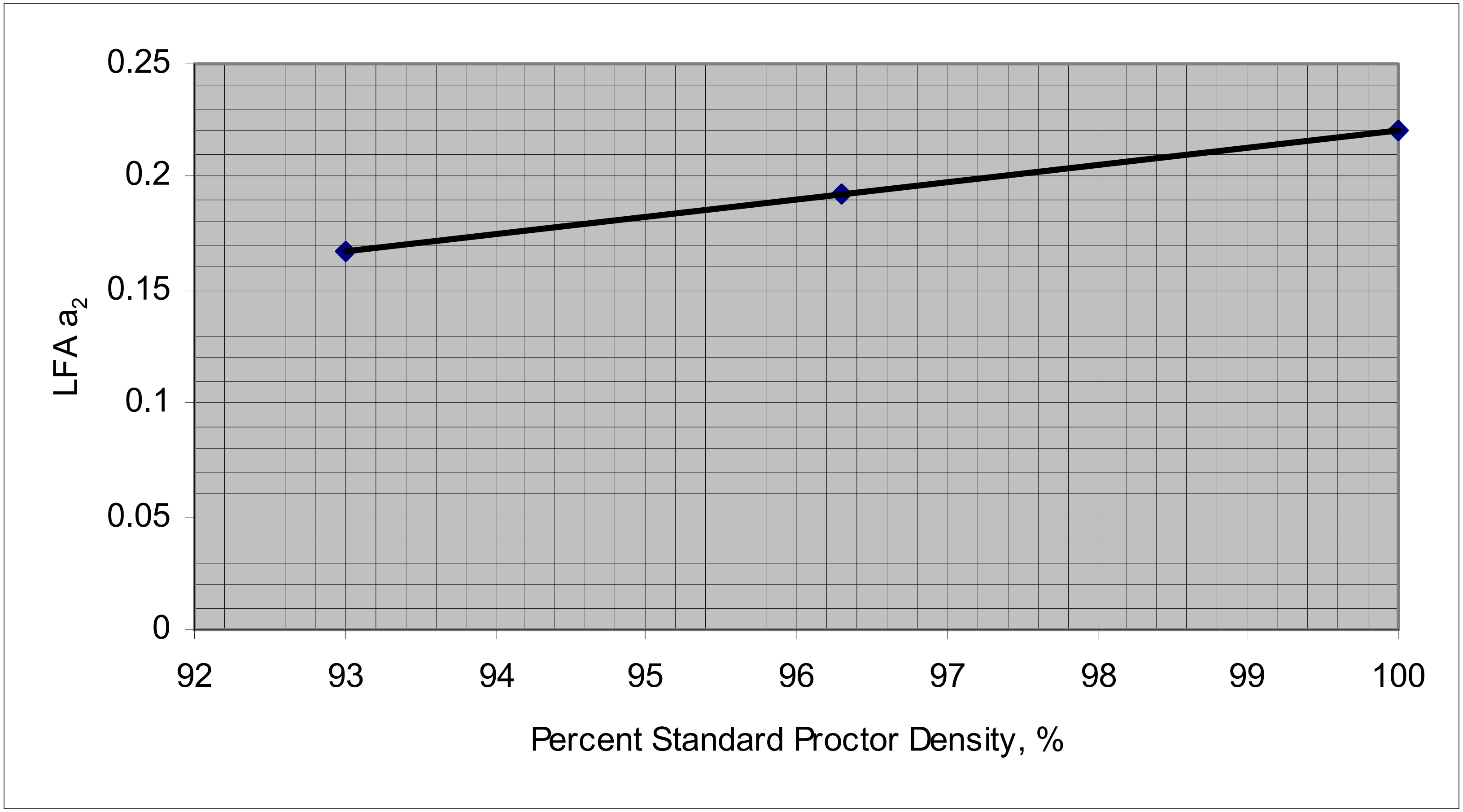


Figure 29. LFA a_2 vs. Percent Standard Proctor Density for LFA Stabilized Red Sand Topping Material

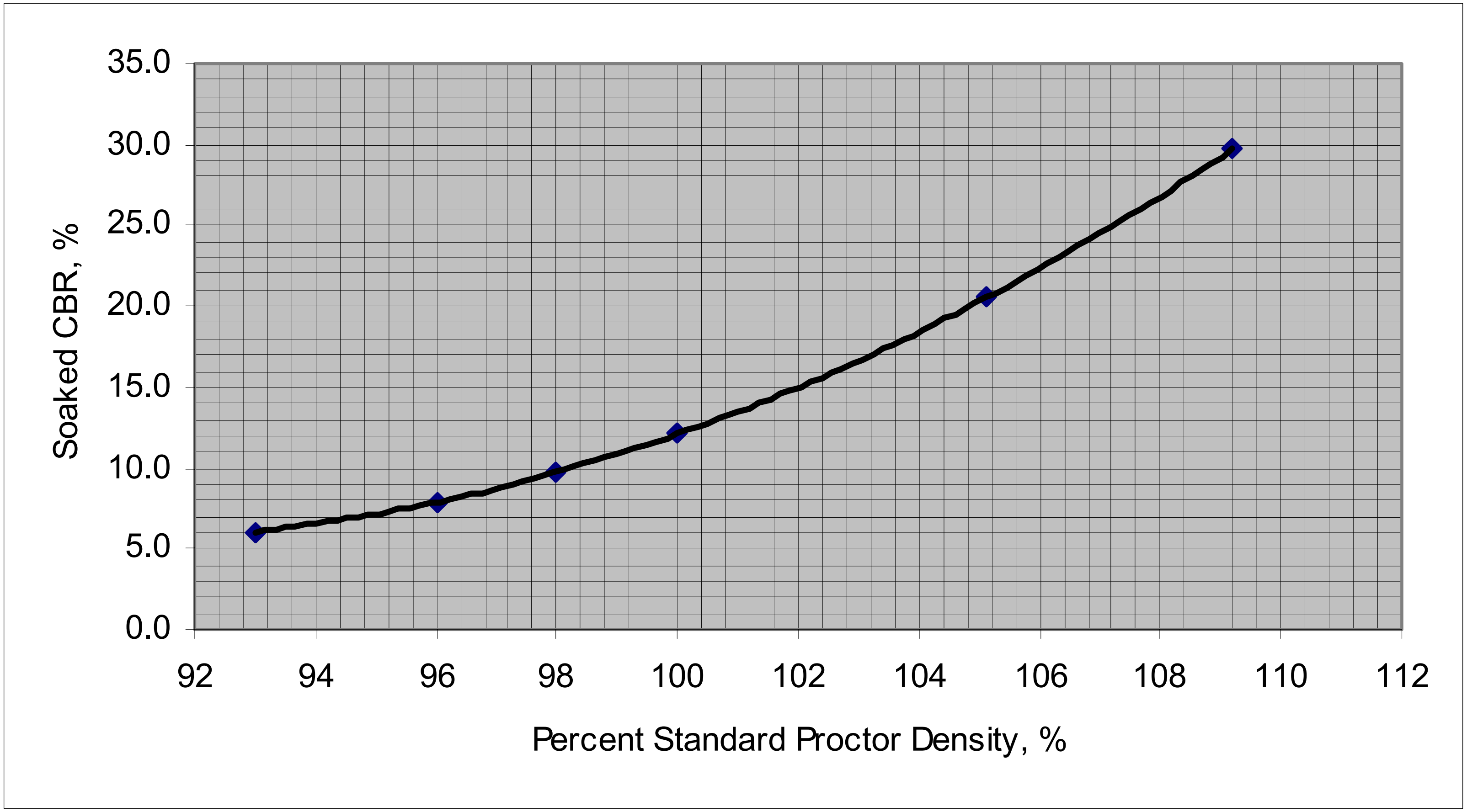


Figure 30. Soaked CBR vs. Percent Standard Proctor Density

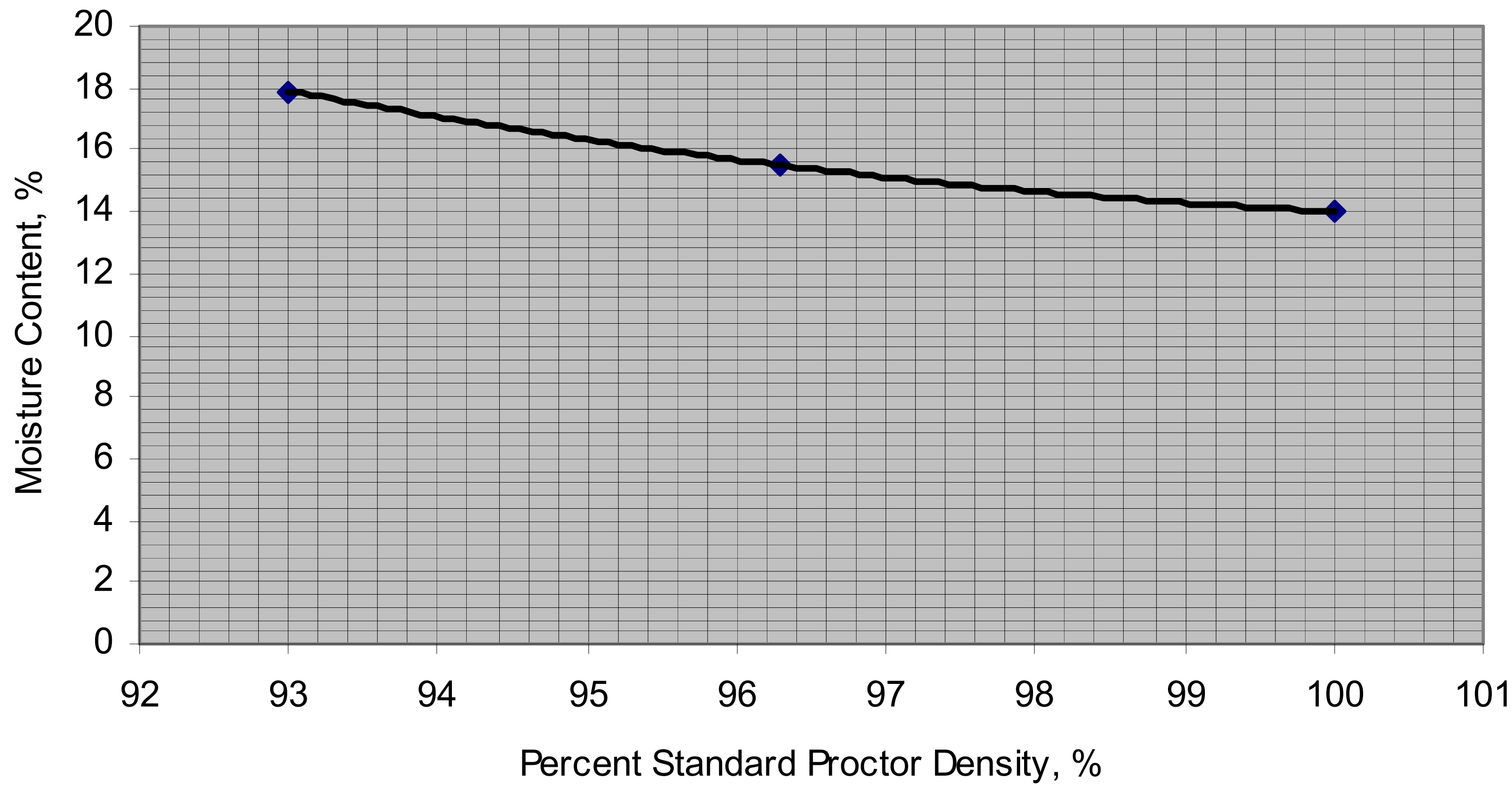


Figure 31. Moisture Content After 5 Hours Soaking vs. Percent Standard Proctor Density for LFA Stabilized Red Sand Topping Material

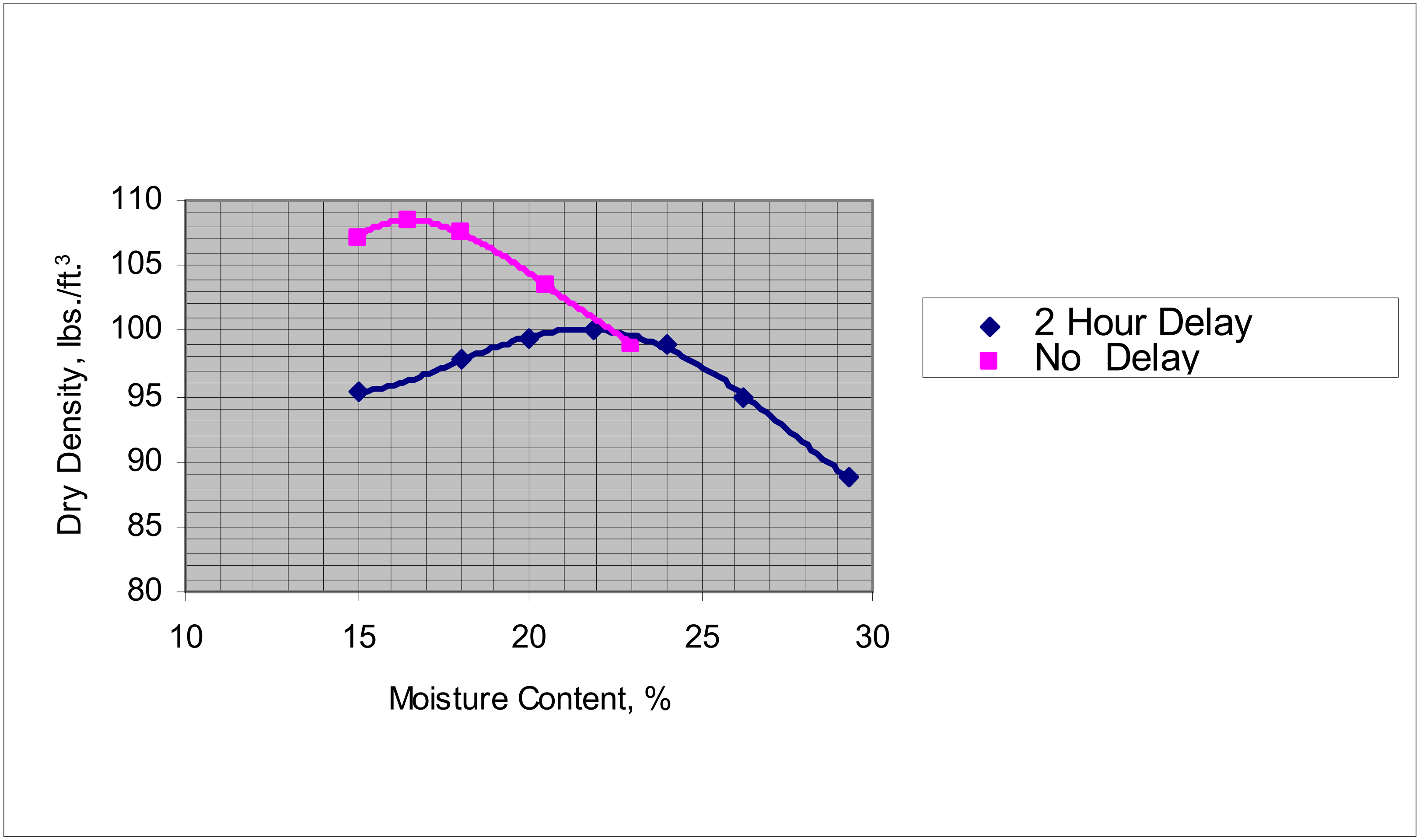


Figure 32. Dry Density vs. Moisture Content for a Self-Cementing Fly Ash Stabilized Clay Soil

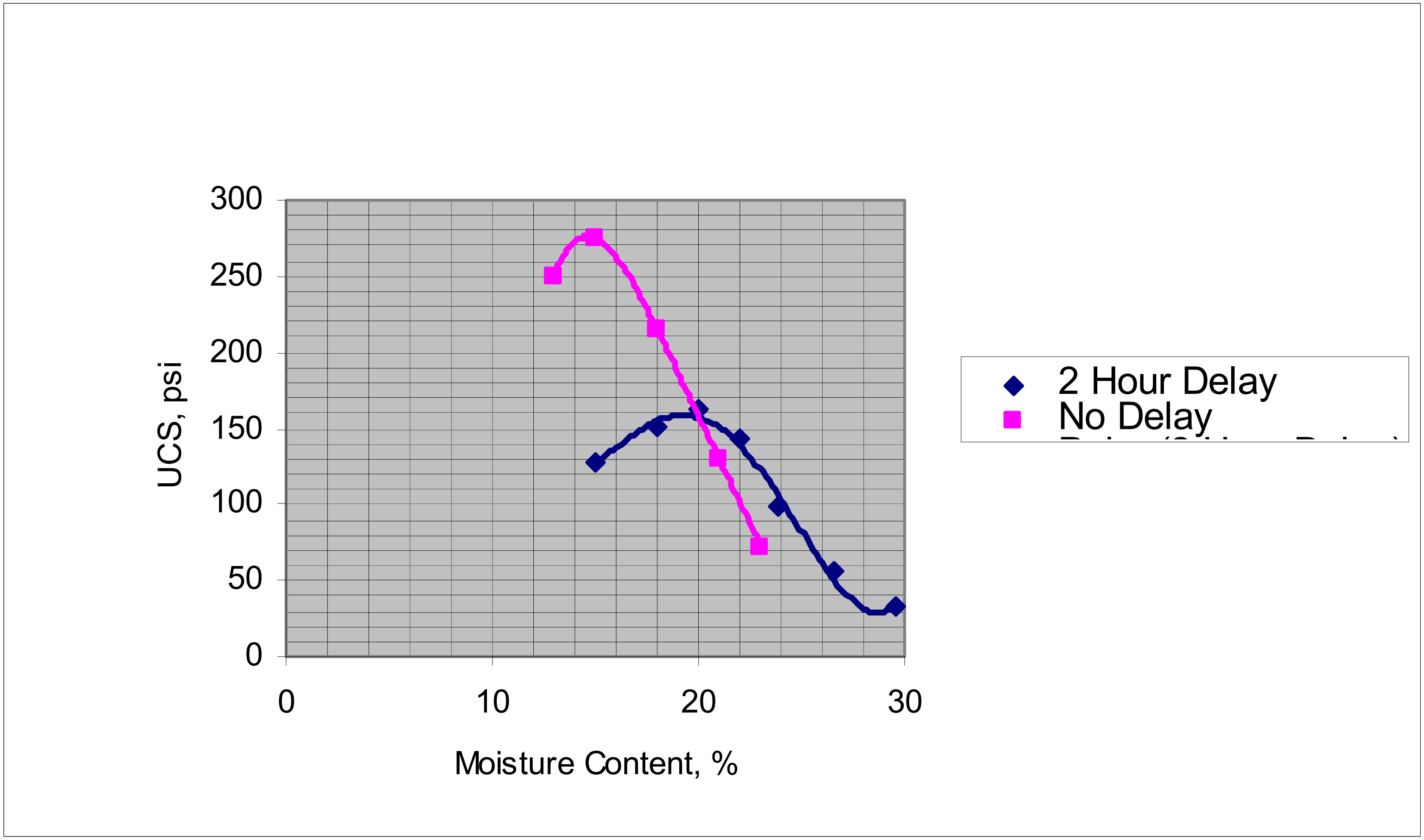


Figure 33. UCS vs. Moisture Content for a Self-Cementing Fly Ash Stabilized Clay Soil

Table 16. Dry Unit Weight and Moisture Content for 3 Different Compaction Levels

Blows per layer = 25

Laboratory ID Number	Dry Unit Weight (lbs/ft^3)	UCS (psi)	Moisture Content (%)	LFA a_2
1	111.51	490	14.7	0.198
2	114.66	695	13.6	0.244
3	113.6	498	13.7	0.200
4	113.36	582	14.1	0.219
5	112.76	630	14.1	0.230
6	113.48	647	14	0.234
Average:	113.2	590	14.0	0.221

Percent Standard Proctor Compaction = 100% for 25 blows/layer

Blows per layer = 15

Laboratory ID Number	Dry Unit Weight (lbs/ft^3)	UCS (psi)	Moisture Content (%)	LFA a_2
7	109.74	505	15.2	0.202
8	109.56	439	15.3	0.186
9	109.39	533	15.4	0.208
10	106.86	379	16.2	0.171
11	109.42	490	15.3	0.198
12	109.28	462	15.5	0.192
Average:	109.0	468	15.5	0.193

Percent Standard Proctor Compaction = 96.3% for 15 blows/layer

Blows per layer = 11

Laboratory ID Number	Dry Unit Weight (lbs/ft^3)	UCS (psi)	Moisture Content (%)	LFA a_2
13	105.93	395	22.8	0.175
14	104.54	348	17.3	0.164
15	106.14	376	16.5	0.171
16	104.96	343	16.9	0.163
17	105.3	367	16.9	0.169
18	104.82	327	17	0.159
Average:	105.3	359	17.9	0.167

Percent Standard Proctor Compaction = 93.0% for 11 blows/layer

Material Tested: Red Sand Topping - 100 % passing the #10 sieve, 23 % passing the # 200 sieve, Non-Plastic

Note: All cylinders cured for 28 days at 100 ^{0}F and then soaked for 5 hours before UCS

Table 17. Nissan Plant Compaction Statistics

Date	Number of Compaction Tests Performed	Average Standard Proctor Density, %	Number of Test Results Equal to or Exceeding 100 % Standard Proctor Density	Number of Test Results Equal to 103 % Standard Proctor Density
26-Feb-01	31	99.7	14	2
27-Feb-01	132	99.3	50	0
5-Mar-01	75	100.5	50	11
16-Mar-01	82	99.8	44	4

Table 18. Comparison Between Standard and Modified Proctor Densities for LFA Stabilized Soil

		Standard Proctor		Modified Proctor	
Layer Stabilized	Project Number/ County	Maximum Dry Density (lbs./ft.^{3})	Optimum Moisture Content (%)	Maximum Dry Density (lbs./ft.^{3})	Optimum Moisture Content (%)
Base Course	102673-901000 Attala	113.1	13.1	118.6	10.8
Subgrade	102068-301000 Coahoma	111.3	14.5	121.9	11.8
Subgrade	102662-301000 Montgomery	118.3	12.2	127	9.8

Table 19. Summary of Compaction Requirements for Basement and Design Soils, Chemically Treated Subgrade, and LFA Stabilized Soil Base Course

Material	Onset of Study	Current	Recommended
LFA Stabilized Soil Base	94% Standard [a]	98% Standard	96% Modified
Lime-Treated Subgrade	95% Standard	95% Standard	100% Standard
LFA-Treated Subgrade	93% Standard	98% Standard	100% Standard
Cement-Treated Subgrade	93% Standard [a]	98% Standard	100% Standard
Design Soil	96% Standard	98% Standard	98% Standard
Basement Soil	94% Standard	95% Standard	96% Standard

[a] Assuming the use of the red sand topping material

Table 20. Tentative Design LFA Structural Layer Coefficients for Different Levels of Required Field Compaction and Different Levels of Field Variability Assuming 90 Percent Confidence

Required Compaction Level	Average LFA a_2 LFA UCS	Average LFA a_2 AASHTO	Tentative Design LFA Structural Layer Coefficients: Current Level of Field Variability	75% of Current Level of Field Variability	50% of Current Level of Field Variability
94% Standard	0.175	0.219	0.12	0.15	0.17
95.7% Standard	0.188	0.232	0.14	0.16	0.18
96% Standard	0.19	0.234	0.14	0.16	0.19
98% Standard	0.206	0.25	0.16	0.18	0.20
100% Standard	0.221	0.265	0.17	0.19	0.22
96% Modified	0.244	0.288	0.19	0.22	0.24

Chapter 8 -- LFA Mix Design Procedure

The MDOT Central Laboratory has performed over 200 LFA mix designs since the inception of this type soil stabilization in Mississippi. The current LFA mix design procedure requires the determination of the percent lime and fly ash to be added to a soil so that the mixture obtains a minimum UCS after curing for a prescribed time and temperature.

The soils typically stabilized in Mississippi with LFA are granular with the plasticity index (PI) of these soils limited to 10 or less. A review of 182 designs was conducted which included designs for both base course construction and subgrade stabilization. This review indicated that 5 percent of the soils were classified as A-1-a, 3 percent, A-1-b, 82 percent, A-2-4, and 6 percent classified as the fine grain soil type, A-4.

The Mississippi Standard Specifications for Road and Bridge Construction dictate the material quality requirements for the lime and fly ash used in LFA soil stabilization. Section 714.03 addresses the lime, and section 714.05 addresses the fly ash. The quality of the fly ash supplied to MDOT stabilization projects has been a topic of concern; therefore, a discussion of fly ash is included in Chapter 6.

The MDOT Central Laboratory performs the LFA mix designs. Depending on the class of soil, the contractor submits from 200 to 300 pounds of soil and four gallons of fly ash to the Central Laboratory for the design. Stock lime is supplied by the Central Laboratory. There is very little variability between lime sources, which is why stock lime can be kept at the Central Laboratory for LFA designs. However, there is significant variability between fly ash sources; therefore, the contractor must submit a sample of the fly ash that is proposed for use in the pavement construction.

The raw soil is air-dried and then pulverized, excluding stone, to pass through the No. 4 sieve to prepare it for the mix design. In this text raw soil refers to the soil before the addition of any lime or fly ash. The grain size distribution, Atterberg limits and standard Proctor density (AASHTO T-99) of the raw soil are determined as part of the design procedure.

The standard Proctor density is then determined for normally two different proportioned mixtures of soil, lime and fly ash. Computed on a dry weight basis, these include a 3 percent lime, 12 percent fly ash (3/12) and a 4 percent lime, 12 percent fly ash (4/12) with soil blend. Four proctor size cylinders (4-inch diameter by 4.56-inch high) of each blend are fabricated by compaction to standard proctor density at the optimum moisture content corresponding to the required blend. Each of the eight cylinders is then placed in an individual plastic bag. Each bag is then placed in a one-gallon metal can, then all eight cans are placed in the curing room. The curing room is a dry-heat room with the temperature set at 100 ^{0}F.

MDOT LFA design requires that the percentages of lime and fly ash selected for field construction result in the cylinders achieving an UCS of 500 psi after 28 days of curing. UCS testing is conducted in accordance with MT-26, Compressive Strength of Soil Cement Cylinder and Cores, which includes presoaking the cylinders in water for five hours prior to performing the UCS test. To expedite the LFA design to the contractor, two of the four cylinders of each blend are tested after 14 days of curing. The average UCS of each pair of cylinders is compared to the required design value. The blend with the lowest lime content achieving the design value is then selected for field use. Past experience indicates that the use of Class C fly ash will sometimes facilitate the achievement of the design value requirement after 14 days of laboratory curing. This can be attributed to the self-cementing characteristic of this ash when blended with water. If neither blend results in the design value at 14 days, the remaining two pairs of cylinders are tested for UCS after a total of 28 days of laboratory curing. Again, the blend with the lowest lime content achieving the design value is selected for field use.

Should neither blend result in the attainment of the design value after 28 days of laboratory curing, several options are available to the contractor to try to obtain an acceptable design for the given project. These include (1) the evaluation of a 5/15 blend using the same fly ash, or (2) the contractor submitting a fly ash sample of a different class or from a different source, and the procedure repeated using the 3/12 and 4/12 blends for the second fly ash sample. Past experience indicates that if the 4/12 blend does not work for a given fly ash, usually a 5/15 blend using the same ash does not work either, and a different fly ash source must be selected for use in the design. A third option for base course construction is the use of a different source of granular material. Regardless of the option selected, the contractor is responsible for furnishing all materials to achieve the required strength design.

This design procedure is effective for eliminating, from a strength perspective, unacceptable sources of fly ash from use in field construction. It is imperative, however, that for this design procedure to continue to be effective throughout the duration of the field construction, that the fly ash used in the field has the same physical and chemical properties as the fly ash used in the design.

The review of the 182 LFA mix designs indicated that 76 percent of these designs had a 3/12 blend and 20 percent had a 4/12 blend. A review of the mix designs accepted for construction from between April 15, 1999, to November 10, 2000, indicated that 77 percent included Class F fly ash and 23 percent included Class C fly ash.

Class C Fly Ash

As discussed in Chapter 7, when using Class C fly ash in the blend, delayed compaction can cause a reduction in the achieved strength relative to that obtained by compaction immediately following blending of the materials. LFA mix design cylinders are typically fabricated immediately after mixing of the materials in the laboratory. Field construction specifications require that the blended material be compacted within two hours from the time of mixing; therefore, it is recommended to allow a two-hour delay between material blending and the fabrication of LFA mix design cylinders when using Class C ash in the blend. This requirement may lead to the incorporation of a greater percentage of a given Class C ash, or possibly the exclusion of the particular ash; however, the laboratory derived strength will more closely model that to be potentially obtained in the field.

Correspondence of Materials Used in LFA Mix Design and In Field Construction

In order for a given LFA mix design to be applicable to a given project, the lime, fly ash and soil used in the field construction must have similar properties to those used in the mix design. The contractor is required to submit to the Central Laboratory a sample of soil and fly ash that is representative of the these materials proposed for use in subsequent construction. Obtaining a representative sample of each of these two inherently variable materials is very difficult. Attention to detail in the sampling stage of a

LFA mix design process must be coupled with periodic testing of the raw materials being utilized during field construction to ensure this correspondence in raw material properties.

Deviations in raw material properties constitute one of the causes of the variation noted in the properties of a completed base course. One source of variance between the LFA material properties determined for the mix design blend and those of the corresponding field-mixed material is the technique of mining the sample of soil submitted for the mix design and the mining of the soil for field construction. If the borrow pit soil is stratified and the sample of soil submitted for the mix design is a blend of several strata, then the soil used for the base course must be a mixed blend of the same combination of strata.

Correlate Laboratory LFA Mix Design Compaction Effort With Specified Field Compaction Level

Chapter 7 discussed the relationship between level of compaction and UCS for a chemically stabilized soil. Based on this relationship, the level of compaction used in the laboratory mix design procedure should be the same as the level of specified compaction for the in-situ stabilized material since the mix design is dictating the percentages of lime and fly ash to be incorporated in the field. In Chapter 7 a recommendation was made to increase the level of compaction for a LFA stabilized base layer to 96 percent modified Proctor effort. In Chapter 11 an in-situ LFA Proctor UCS value of 400 psi is recommended for the base layer.

In recognition of the fact that a laboratory mixed sample of lime, fly ash, and soil will be better proportioned and blended than the corresponding field mixed sample, it is recommended that the current base layer design UCS of 500 psi be maintained for the selection of the percentages of lime and fly ash. However, the laboratory-mixed material cylinders should be compacted with a modified Proctor compactive effort in accordance with AASHTO T-180, with the exception that the blows per layer will be adjusted so that the compacted density is approximately 96 percent modified density. The blows per layer will be a fixed value for every LFA mix design performed at the MDOT Central Laboratory. This number will be determined for the most prevalent type of soil stabilized with LFA in Mississippi; i.e., an A-2-4 soil type.

To facilitate a comparison of UCS test results of laboratory mixed material cylinders for an LFA mix design to corresponding field mixed material cylinders for QC/QA, the adjusted number of blows per layer will also need to be applied to the field mixed soil cylinders. Using this approach the density of the laboratory mixed material design cylinders and the field mixed material QC/QA cylinders will be correlated to the specified field construction density of 96 percent modified Proctor density.

Chapter 9 -- LFA Material and Layer Thickness Variability

Chapter 5 noted the large variability in the in-situ LFA structural layer coefficient for the pavements tested in this study. While the average LFA layer coefficient for the five newer pavements exceeded the design value of 0.20, the large variability requires a significant reduction in the design value in order to design this pavement layer with 90 percent confidence. Figure 23 from Chapter 5 illustrated three approaches to achieve the current MDOT design value of 0.20. Chapter 7 addressed one of these approaches, which was to hold the variability constant but increase the average value by increasing required levels of field compaction. The current chapter addresses a second approach, which is to hold the average constant but reduce the variability. As discussed in Chapter 7, a combination of both of these approaches, or the third approach, will be required to achieve the 0.2 design value in the field with 90 percent confidence.

LFA Stabilized Soil Material Property Variability

Significant variation exists in the quality and properties of a given LFA stabilized soil base course. Evidence of this variation has been documented both visually and numerically. The LFA core ratings provide visual evidence of this variation. Table 7 in Chapter 3 illustrates that all five of the newer projects include LFA stabilized soil that ranges from well cemented material providing excellent testable cores to relatively poorly cemented material from which no core could be obtained for testing.

LFA backcalculated moduli and in-situ structural layer coefficient values provide numerical evidence of LFA material variability. The average backcalculated modulus for the five newer projects was 423.6 ksi with a standard deviation of 306.09 ksi and a corresponding coefficient of variation of 72 percent. The average LFA structural layer coefficient of the five newer projects was 0.232 with a standard deviation of 0.074 and a corresponding coefficient of variation of 32 percent.

Based upon visual observation, and backcalculated moduli and in-situ structural layer coefficient values, it is concluded that MDOT LFA stabilized soil base courses possess highly variable material properties. It can be numerically demonstrated that variations in the LFA material modulus from one location to another within a given pavement results in differential performance throughout the length of that pavement. An example of a documented project that experienced premature pavement failure due to highly variable LFA and HMA material properties is the phase two project constructed in 1985-1986 on US 84/98 in Adams County. Details of this project are included in Chapter 1.

Figure 34 is a duplication of Figure 19 from the referenced source that illustrates the multitude of potential sources affecting variation in the properties of a LFA base course (NCHRP No. 37, 1976). The primary focus of this chapter is field construction procedures to reduce this variability. Two potential methods to reduce variability are (1) improving the current method of field-mixed-in-place, and (2) plant mix with placement of the blended material via a paver. The former method constitutes the predominant discussion included in this chapter because the in-state soil stabilization contractors have made substantial investments in pulvamixers for field-mixed-in-place construction.

Excessive variability in LFA stabilized soil pavement layer properties is not a problem unique to LFA stabilized soil base course construction. Currently there is significant interest within MDOT to construct chemically stabilized soil base courses using Portland cement as the stabilizing agent due to problems associated with the use of LFA as the stabilizing agents. Portland cement is an excellent alternate for LFA, but its use does not automatically eliminate problems with variability because both materials are spread and incorporated into the soil using the field-mixed-in-place method. Similar problems are encountered with the use of soil cement as attested to by the following quote (Hadley, 1991):

> "On the bases of the low recovery rate in good field cores for job verification; observation of various cracks, lamination, compaction planes, and layer separations in field cores obtained during the second major coring operation to cores for fatigue-resilient testing; and the subsequent low number of clear core specimens found to exist during the material characterization study, the mixed-in-place soil-cement construction procedure presently used apparently does not provide the quality and uniformity expected in a cement-stabilized base layer."

A primary source of variability is the current method of field-mixed-in-place construction.

Field Construction Procedures - Introduction

The Central Laboratory submits the completed LFA design to the contractor, and the contractor in turn uses the blend recommendation for construction of the LFA stabilized base course. At the onset of this study the specifications for the construction of such a base layer were included in Section 311, "Lime-Fly Ash Treated Courses", of The Mississippi Standard Specifications for Road and Bridge Construction, except as modified by Special Provision No. 907-311-6, "Lime-Fly Ash Treated Courses", dated

October 9, 2000. This Special Provision was revised during the course of the study on November 26, 2002, and assigned No. 907-311-7.

The specifications allow the contractor the option to use either the mix-in-place or a central plant to blend the lime, fly ash, soil, and water. With the exception of the one project discussed in Chapter 1 on U.S. 84/98 in Adams County, all LFA stabilization work in Mississippi is accomplished using the mix-in-place method for blending the these materials.

Borrow Material

The soil used for the base course is usually obtained from a local borrow pit from which the 200 - 300 pound soil sample was obtained for the LFA mix design. Multiple borrow pits, each with a unique corresponding LFA mix design, are often utilized on relatively long projects to reduce soil haul distance. The soil is transported to the roadbed, placed, and compacted in accordance with paragraph 304.03.6 of The Mississippi Standard Specifications for Road and Bridge Construction. This specification requires that when a course or layer is to be subsequently chemically treated, the required lot density for the portion to be treated shall equal or exceed 93 percent with no single density test in the lot below 89 percent.

The raw soil course is then typically blue topped and clipped with a motor grader to establish the proper thickness, grade, and surface tolerance. Paragraph 907-321.03.7.2.4, subparagraph “a” of Special Provision No. 907-321-2, In-Grade Preparation dated January 3, 2002, dictates that where a course is to be treated and the next course is a drainage layer or bituminous pavement, the in-place surface tolerance is

+/- ½ inch. These requirements for compaction and surface tolerance are necessary to ensure that a relatively uniform density and thickness of soil is in place across the road bed. Therefore, when the theoretically uniform layers of fly ash and lime are spread and then mixed into the soil, a blend having the correct percentages of lime and fly ash is obtained throughout the areal extent and depth of the base layer.

The +/- ½ inch requirement for surface tolerance is a source of variation in the actual percentage of fly ash and lime blended into the soil. Assuming the pulvamixers maintain an elevation consistent with the plan grade and cross slope during mixing, with the top surface of the raw soil varying from design grade by either ½ inch high or low, the total thickness of the raw soil being blended can vary by one inch from location to location across the roadbed. For instance, assuming a 6-inch LFA design layer thickness, the allowed variation in raw soil thickness being mixed by the pulvamixer is from 5.5 to 6.5 inches. For a LFA design of 12 percent fly ash, this variation in raw soil layer thickness could cause a variation in actual fly ash application rate from 13 percent for the 5.5-inch thick areas to 11 percent for the 6.5-inch thick areas, or a total 2 percent range in the actual applied fly ash rate. This variation would occur even if the exact computed dosage of 12 percent fly ash for a 6-inch thick layer were spread and blended into the soil. The other assumption made here is that the density of the raw soil is a constant across the roadbed. Variations in the density of the raw soil will occur as an inherent aspect of field compaction operations even with good moisture control. Since this is not the case, an even greater discrepancy between the theoretical and actual applied percentage of fly ash can occur in the final LFA blended material.

Fly Ash Spreading Operations

A given tanker truck moves up and down the road between two set stations, so the amount of fly ash placed between these stations can be reasonably controlled; however, this does not ensure that the fly ash is uniformly distributed across the road bed between these two stations. The uniformity of fly ash spread can be ascertained by the use of 3-feet square mats placed in the road bed prior to placement of the fly ash. After the fly ash is spread, the amount of ash on each mat is weighed, and the variability of the weight retained on each of these mats is used to determine the uniformity of the spread. District 1 used this procedure and found that there is a large variation in the spread of fly ash across a given road bed.

The LFA mix design dictates the percentages of lime and fly ash to be added to a given soil to obtain the desired strength in the stabilized material. As discussed in Chapter 8, 12 percent fly ash is the amount typically specified for LFA soil stabilization to obtain the target UCS. A nonuniform spread of fly ash contributes to percentages other than 12 percent actually being incorporated into the soil during field construction. Variations below the design percentage of fly ash in the field-blended material will cause a reduction in the strength and stiffness properties obtained in the base course, with some areas not achieving the desired strength and stiffness required for long-term base course performance.

Figure 35 illustrates a major problem with the current method of spreading either lime or fly ash. Huge clouds of dust are typically generated which can reduce visibility to zero across adjacent traffic lanes. In addition to causing a traffic hazard, the dust can be problematic to adjacent structures, especially in urban areas. Note the mailbox in the figure, indicating a residence in close proximity to the spreading operation.

A Vane Feeder Spreader developed by Cutrell Trucking of Amarillo, Texas, offers a solution for reducing both the variability in the spread and the magnitude of the dusting problem. Spread uniformity data supplied by this company indicates a very uniform application of material using this spreading device. Figure 36 illustrates the use of a Vane Feeder Spreader being operated in an environment with winds reported at 21 mph and gusting to 25 mph (De Shong, 2002). It is recommended that this method be investigated for use in Mississippi.

Even with improvements in the fly ash distribution technique, variation in actual fly ash distribution is an inherent aspect of using the field-mixed approach for constructing this pavement layer. As previously discussed concerning the borrow material, both the +/- ½ inch tolerance in grading and the variable compacted density also contribute to the total variation in the actual fly ash percentage in place in the field blend. Increasing the target fly ash content applied in the field would provide a measure of confidence for achieving, as a minimum, the design fly ash content in the blend throughout the length and width of this pavement layer. Typical design specifications require the target fly ash contents applied in the field to be one to two percent greater than that of the mix design (Ferguson and Levorson, 1999). It is recommended that MDOT increase the target fly ash content applied in the field by 2 percent over that required in a given LFA design. In the majority of cases, this means placing 14 percent fly ash instead of 12 percent.

Lime Spreading Operations

Similar to the fly ash, variations in the lime spread can be expected, and a large cloud of lime can often be observed in the vicinity of the lime placement operation. It is recommended that the Vane Feeder Spreader be investigated for use in spreading the lime in addition to the fly ash. Based on the same rationale as that used for the fly ash, it is also recommended that the target spread rate of the lime in the field be increased over that called for in the LFA design. An increase of 0.5 percent is recommended to maintain the relative percentages of lime and fly ash applied in the field as that required in the design. This change would generally result in the placement of either 3.5 or 4.5 percent lime.

Water

Water is required to facilitate compaction of the blended material and for the pozzolanic and hydration reactions that are responsible for the required strength and stiffness gains of the stabilized material. Water is typically added to the blended material from the back of a gravity feed distributor truck as illustrated in Figure 37. This method results in a nonuniform placement of water across the road bed, especially in areas having significant grade or superelevation. Given the natural variation of the in-situ moisture of the soil and this nonuniform distribution of water, areas of the blended material have varying moisture contents both above and below the optimum moisture content of the blend. Two effects of this moisture variation include difficulty in achieving the required density during the compaction process and the variation in the compacted density from location to location within the stabilized base course.

The first of these two effects is a well known fact to road construction personnel. Moisture contents significantly different from optimum moisture comprise one of the main factors for inadequate field compaction requiring either the addition or removal of water to rectify the problem. If compaction requirements are not very high, then the required density can be achieved across a wider range of moisture contents. If, however, there is a relatively high density requirement for the stabilized material, such as the 96 percent modified Proctor density proposed in the current study, then maintaining field moisture within a narrower range of optimum becomes an important factor in achieving the required density.

The second of these two effects is not as readily apparent but possesses implications for long-term pavement performance and durability. As discussed in Chapter 7, the compacted density of a stabilized soil blend significantly impacts the level of the strength and stiffness achieved in that material. Thus, all other factors being equal, a variation in moisture content causes a variation in compacted density, which in turn leads to a variation in the strength and stiffness obtained in the completed base course from location to location across the road bed. Based on these two implications of variation in compaction moisture content, it is imperative that a better method be developed to add controlled amounts of water to the material.

One potential method is called nursing, wherein a water truck is attached to a pulvamixer via a hose and the water applied to the LFA and soil blend in the mixing chamber of the pulvamixer (Figure 38). The amount of water entering this chamber is adjusted through a water metering system. A moisture density gauge can be used to obtain an estimate of the moisture content of the blended material right behind the pulvamixer, and then required adjustments made to the water flow rate via the metering system. When dealing with LFA soil stabilization, a major problem with the nursing method is that if the LFA and soil blend is dry and significant dusting occurs during mixing, contractor personnel are exposed to caustic conditions. When dry conditions occur, an initial increment of water can be added to the roadbed using the current method of gravity feed water trucks and the material mixed, thus reducing the amount of dust created during subsequent mixing. Additional controlled amounts of water can then be added to the material through the nursing method to obtain field moisture contents close to optimum. It is recommended that the method of nursing be investigated to add controlled amounts of water to the blend of LFA and soil material.

The Georgia Department of Transportation, GaDOT, uses a construction practice for soil cement construction that could possibly be adopted by MDOT for LFA construction to address the issue of moisture control in conjunction with the safety of construction personnel. Immediately prior to spreading cement, the moisture content of the in-place material to be stabilized is adjusted to within 100 to 120 percent of optimum moisture content (Supplemental Specification Section 301 – Soil-Cement Construction). Applying this to MDOT LFA construction, the moisture content of the raw soil for the base course would be adjusted to within 100 to 120 percent of the optimum moisture for the blend immediately prior to spreading the fly ash and lime. The method of nursing could be used to perform this moisture adjustment. The additional water applied above optimum moisture content would provide allowance for evaporation of some of the moisture prior to the final blending of the mix (Halsted, 2002).

Blending

The soil, lime, fly ash, and water are blended with a pulvamixer (Figure 39). Inadequate blending can leave bodies of unmixed soil within the stabilized pavement layer as illustrated in Figure 40. These unmixed bodies of soil can reduce the overall strength and stiffness of this pavement layer as illustrated by the crack located adjacent to the unmixed clod running through the core in Figure 41. Fortunately, the relatively large unmixed bodies of soil shown in these figures do not represent the norm; however, uniform blending of the constituent materials in the field cannot be overemphasized for attainment of a strong and durable LFA stabilized soil base course.

Section 311.03.6, Fly Ash – Lime and Water Mixing Phase, of The Mississippi Standard Specifications for Road and Bridge Construction, includes the MDOT specifications requiring that all of the blended field-mixed material pass a two-inch sieve with 60 percent passing the No. 4 sieve. In the LFA mix design, 100 percent of the soil, excluding stone, is passed through the No. 4 sieve. This discrepancy in requirement for pulverization between design and field may contribute to the observed difference in strength between laboratory and field-mixed LFA cylinders.

A review of industry recommended practice for the degree of pulverization for lime stabilization indicates that 100 percent of the blended material, excluding any non-slaking fractions, should pass the one-inch sieve and 60 percent pass the No. 4 sieve (Lime Stabilization Construction Manual, Bulletin 326, 1991). For soil cement stabilization 100 percent of the material, exclusive of gravel or stone, should pass the one-inch sieve, and 80 percent should pass the No. 4 sieve (Soil-Cement Inspectors Manual, 1984). Given these industry recommendations for both lime and soil cement stabilization, it is recommended that MDOT increase the pulverization requirement for LFA stabilization to 100 percent of the blended material, excluding gravel or stone, passing the one-inch sieve.

Compaction

Field compaction begins subsequent to the blending operation. Typically the base course material is first compacted with a sheepsfoot roller (Figure 42) and then compacted with a rubber tire roller (Figure 43). A sheepsfoot roller affects compaction from bottom up in the layer, and the rubber tire roller affects compaction from the top down in the pavement layer. The application of these two type rollers in this manner can result in good compaction throughout the depth of the pavement layer. Heavier rollers than those illustrated may be required to obtain the recommended 96 percent modified density.

MDOT specifications require that the final compaction of LFA and soil mixtures be completed within two hours from the time of initiation of the mixing operation. There has been some discussion to relax this time requirement due to situations that occur during construction. For example, if density test results are received at the end of the day that indicate inadequate compaction due to either too little or too much moisture in the base course material, it may be the next day before corrective action can be completed to achieve the required density. If the moisture content of the soil is adjusted prior to the spreading of the fly ash and lime, as previously discussed, there will be a reduction in the incidence of times requiring following day corrective action.

The argument raised in favor of extending this time is based on the typically slow rate of strength gain associated with this type of stabilization. This is a valid argument when Class F fly ash in being utilized in the blend; however, the specifications governing the compaction of blends utilizing Class C fly ash should be the same as that for soil cement. Chapter 6 addressed the similarity and difference in the mechanisms of strength gain between these two classes of fly ash. Chapter 7 addressed field compaction of LFA mixes and considered the topic of delayed compaction when using Class C fly ash.

Surface Tolerance

After final compaction of the base layer, the surface is blue-topped and then clipped with a motor grader to make it conform to the required surface tolerance of this pavement layer. Paragraph 907-321.03.7.2.2, subparagraph “c” of Special Provision No. 907-321-2, In-Grade Preparation, directs that if a drainage layer is the next course, then the surface tolerance of the LFA stabilized soil base course is +/- ½ inch. If the next course is bituminous pavement, the same surface tolerance applies (subparagraph “e”).

An important observation was made during the coring operations. There is significant variation in the in-situ LFA stabilized soil base course layer thickness within the majority of the pavements cored for this study (Table 21). Two of the newer pavements have a difference between minimum and maximum layer thickness of at least 4 inches, which is 67 percent of the design layer thickness. As with variations in material properties, it can be numerically demonstrated that variations in the LFA stabilized soil base course layer thickness from one location to another within a given pavement results in differential performance throughout the length of that pavement.

It is recommended that an autograde trimmer, operated off from a string-line, be used to further control the extent of surface undulations in the base course, thus helping to reduce the overall variability in the in-situ layer thickness (Figure 44). An added benefit from using this trimmer would be to provide a more uniform surface for the placement of subsequent lifts of asphalt, leading to a smoother finished surface of the pavement. Due to the hydration characteristics of Class C fly ash, the use of this type of ash in the blend may require the surface of the compacted base course to be trimmed with the autograde trimmer on the same day that the ash is incorporated into the soil. The use of Class F fly ash would allow more flexibility in the scheduling of the use of this trimmer.

Curing

The final step in the construction of a LFA stabilized soil base course is the proper curing of this course. This is an extremely important step for the production of a quality stabilized material that should be considered on equal par to that of curing Portland cement concrete. The purpose of curing is to maintain moisture in the layer to facilitate the pozzolanic and hydration reactions necessary for obtaining the levels of strength and stiffness required in the stabilized material for long-term pavement performance.

A specific problem noted, due to lack of attention to proper curing, is the formation of a dry crust, or layer, of LFA and soil on top of the base course. This occurs when the LFA material is not kept continually moist until either the curing seal is applied or the next pavement course is placed over the stabilized material. This dry crust does not cure and delaminates from the rest of the base course layer. This dried layer prevents good bonding between the base course and the overlying pavement layer, which may cause shoving to occur within the pavement. In addition to creating potential shoving, this dried layer reduces the effective thickness of the stabilized base course (Crawley, 1990).

Delamination can significantly increase the level of flexural stresses developed at the bottom of the base course when the pavement is subjected to traffic loading. Increased flexural stresses result in a reduction in the number of load repetitions that can be applied before fatigue failure occurs within the pavement structure. It is recommended that if this dry crust forms, it should be removed by blading the base course with a motor grader or auto trimmer prior to sealing the course with a bituminous material.

At the onset of this study the MDOT specifications required that the LFA course be sealed with one of the specified bituminous materials within 48 hours after placement of the course. These bituminous materials are required to be applied with a pressure distributor at the rate of 0.10 to 0.25 gallon per square yard or as directed by the Engineer.

MDOT shortened the maximum allowed time for placement of the curing seal during the duration of this study. Special Provision No. 907-311-7 states that the completed course is to be covered with a bituminous curing seal as soon as possible, but no later than 24 hours after completion. It is recommended that the course be kept continuously moist in the interim period between completion of that course and the application of the bituminous seal. This recommendation was included as a specification requirement in paragraph 311.03.8 – Protection and Curing on page 311-3 of The Mississippi Standard Specifications for Road and Bridge Construction, but omitted in both the previous and current Special Provisions governing construction of LFA treated courses.

Reduction in Variability and Impact on LFA Structural Layer Coefficient

It is estimated that adopting the recommendations included in this chapter for field-mixed-in-place construction would reduce the variability to 75 percent of the current level. Table 20 in Chapter 7 provides an indication of the potential increase in the design LFA structural layer coefficient given this reduction in variability and varying levels of compacted density.

A central mixing plant was used for blending the lime, fly ash, soil and water for the US 84/98 project in Adams County as discussed in Chapter 1. A judgment regarding the veracity of using a plant mix approach for blending these materials should not be made based on this project since an old plant was used that experienced problems with proportioning. Modern mixing plants used in HMA and Portland cement concrete production are fully automated and produce tons of high-quality mix for road construction. Use of a mixing plant is the recommended method of blending the lime, fly ash, soil, and water because it allows greater control in the proportioning of these materials and yields a more uniform product (American Coal Ash Association, 1991, NCHRP No. 37, 1976).

It is estimated that using plant mixed LFA and soil blends would result in a 50 percent reduction from the current levels of variability. Table 20 in Chapter 7 provides an indication of the potential increase in the design LFA structural layer coefficient given this reduction in variability and varying levels of compacted density.

Table 22 includes the average in-situ HMA layer thickness for each of the nine projects. HMA is placed with a paver. By comparing Tables 21 and 22 a reduction in the value of the coefficient of variation for the HMA layer thickness relative to the LFA layer thickness is observed for the majority of the projects. The greatest difference observed between the maximum and minimum LFA layer thickness among the nine projects is 4.7 inches, whereas the greatest difference for the HMA layer thickness is 2.25 inches. These observations indicate that the placement of LFA and soil blends with a paver instead of the current field-mixed-in-place construction method may reduce the variability in LFA layer thickness.

It is recommended that several projects be constructed using the recommendations included in this chapter for modifying the field-mixed-in-place method and that several additional projects constructed with plant mixed material placed with a paver. Evaluation of these projects would enable a determination of the actual reduction in in-situ LFA material property variability by using either of these methods relative to the current method of field construction.

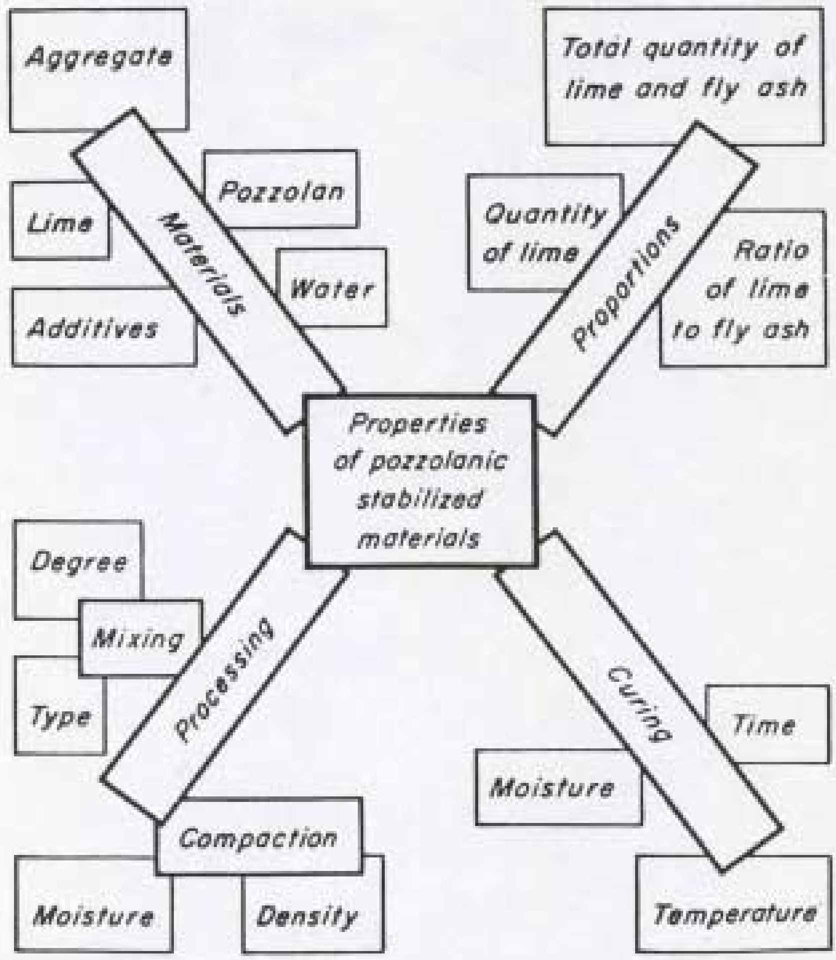


Figure 19. Factors influencing the properties of lime-fly ash-stabilized materials (33).

Figure 34. Sources of Variation in LFA Stabilized Soil

Figure 35. Dust Problem Associated with Current Method of Spreading Lime and Fly Ash

Figure 36. Vane Feeder Spreader (Photo courtesy of De Shong)

Figure 37. Current Gravity Feed Distribution Truck Method Used for Placement of Water Across the Roadbed

Figure 38. Proposed Method of Nursing to Incorporate Water Into LFA and Soil Blend (Photo courtesy of Halsted)

Figure 39. Pulvamixer Used for Blending LFA, Soil, and Water

Figure 40. Clod of Unmixed Soil Within the LFA Base Layer Due to Inadequate Field Mixing

Figure 41. Cracking of LFA Base Layer Adjacent to an Unmixed Clod

Figure 42. Compaction with a Sheepsfoot Roller

Figure 43. Compaction with a Rubber Tire Roller

Figure 44. Use of Autograde Trimmer to Control Extent of Surface Undulations in the Base Layer (Photo courtesy of Halsted)

Table 21. LFA Thickness Data from Core Data

County	# Cored Locations	Design Thickness (in.)	Average Thickness (in.)	Difference from Design (%)	Maximum Thickness (in.)	Minimum Thickness (in.)	Coefficient of Variation [a] (%)
Newer Pavements							
Bolivar	16	6	6.8	13.3	9.5	5.5	15.7
Clarke	16	6	6.02	0.3	7.0	5	8.9
Smith	15	6	6.04	0.7	7.8	4.5	15.7
Tippah	8	8	7.34	-8.3	8.5	6.5	9.3
Wilkinson	8	6	5.91	-1.5	9.0	4.3	26.6
Older Pavements							
Forrest & Perry	16	6	5.56	-7.3	6.5	5	10.2
George	8	6	4.72	-21.3	5.5	3.75	15.6
Jones & Wayne							
Section I	4	6	6.31	5.2	6.5	6	3.8
Section II	12	7	6.94	-0.9	8.3	6	11.6
Yalobusha	16	6	7.09	18.2	9.0	5.5	14.3

[a] Based on average in-situ thickness values.

Table 22. HMA Thickness Data from Core Data

Newer Pavements

County	# Cored Locations	Design Thickness (in.)	Average Thickness (in.)	Difference from Design (%)	Maximum Thickness (in.)	Minimum Thickness (in.)	Coefficient of Variation [a] (%)
Bolivar	16	8	7.85	-1.9	9	7	6.7
Clarke	16	5.75	5.45	-5.2	6.75	4.5	12.5
Smith	15	7.25	7.48	3.2	8.75	6.88	6.6
Tippah	8	7.5	7.5	0.0	8.5	6.75	6.9
Wilkinson	8	5.5 *	5.48	-0.4	6	4.8	9.1

* 2" Surface course not placed yet at time of coring.

Older Pavements

County	# Cored Locations	Design Thickness (in.)	Average Thickness (in.)	Difference from Design (%)	Maximum Thickness (in.)	Minimum Thickness (in.)	Coefficient of Variation [a] (%)
Forrest & Perry	16	6	5.95	-0.8	7.5	5.25	10.5
George	8	4.5	4.34	-3.6	5.25	3.5	13.4
Jones & Wayne							
Section I	4	4.5	4.75	5.6	5.25	4	12.9
Secton II	12	6	5.95	-0.8	6.75	5	8.9
Yalobusha	16	4.5	4.38	-2.7	5	3.5	9.3

[a] Based on average in-situ thickness values

Chapter 10 -- Sampling and Testing Field-Mixed LFA and Soil Mixtures and Late Fall Construction

This chapter addresses issues related to quality control and quality assurance (QC/QA) for LFA stabilized soil base courses and considerations regarding late fall construction given the climatic conditions of Mississippi.

Sampling and Testing of Field-Mixed LFA and Soil Mixtures

Each day that field construction is conducted for a LFA stabilized soil base course or subgrade a sample of the field-mixed material is obtained after blending the soil, lime, fly ash, and water. This sample is used for the pulverization test, a standard Proctor test and the fabrication of UCS cylinders.

Pulverization Test

The degree of blending in the field is controlled by the pulverization test in which samples of field-mixed material are passed through the No. 4 sieve, and the results then compared to the specification requirements. These requirements are included in Chapter 9.

Proctor Compaction Test

The standard Proctor test, or its one-point variant test, is performed at the beginning of each day's LFA field production. This test is performed on a daily basis in recognition of the fact that the soil being delivered from the borrow pit, or the existing subgrade soils, are not uniform, and changes in these soils affect the maximum dry density and optimum moisture content of the blend in the field. The results of the daily Proctor test are used to control the compaction of the base course or stabilized subgrade completed during that day. The practice of the use of either the full compaction test or the one-point test varies with the project office performing the field quality control. In some cases, the original LFA mix design Proctor is used for daily quality control, but this should be strongly discouraged due to the variable nature of the soil used in this type pavement layer construction.

Fabrication and Transport of UCS Test Specimens

A portion of the field-mixed LFA and soil sample is used to fabricate, as a minimum, one 4-inch diameter Proctor-size cylinder to represent a given day's production of LFA stabilized soil material. After compaction of the blended material to 100 percent standard density, the cylinder is either extruded in the field laboratory or transported to a district laboratory while still in the mold, and then extruded at that district location. In either case, the cylinders of blended material are removed from the Proctor mold by means of a hydraulic jack and a circular plate sized to be pushed through the Proctor mold.

LFA stabilized soil cylinders are typically tender and easily damaged right after fabrication due to the minimal cohesion possessed by this material. As a result, the extrusion process from the Proctor mold and transport to the curing room can damage these cylinders. It is recommended that field-mixed material cylinders be fabricated in split-mold Proctor molds. The use of a split mold should facilitate the removal of the tender LFA cylinder from this mold with less effort than that required using a solid mold.

The field-mixed cylinders can be damaged during transit to the district or central laboratories for placement in the curing room. It is recommended that these cylinders be transported back to the laboratory in the split molds and then removed from these molds. This should help to reduce transport-induced damage to the cylinders. Precautions should be taken to avoid moisture loss from these cylinders while in transit.

Upon arrival at the laboratory, the cylinders are placed in individual plastic bags, and the bags then placed in a moisture room for 28 days of curing at 73 ^{o}F. Subsequent to the 28-day curing period, the cylinders are soaked for five hours and then subjected to UCS testing.

Difference in UCS Between Field-Mixed Material Cylinders and Laboratory –Mixed Design Cylinders

A significant difference in the UCS of field-mixed material cylinders, as compared to the strengths obtained from the laboratory-mixed material cylinders associated with a LFA mix design, occur on all LFA soil stabilization projects. The UCS of the field-mixed cylinders are always lower than the corresponding laboratory-mixed design cylinders. As a result, the strengths of the field-mixed cylinders are recorded, but not utilized for quality control unless the strengths are abnormally low when compared to the strengths normally obtained for field-mixed material. A review of the UCS test results from 139 field-mixed cylinders, representing six projects under construction during the 2001 construction season, indicated an average UCS of 70.4 psi with a coefficient of variation of 58 percent. This average value is 86 percent below the design value of 500 psi. This variation in UCS test values is in general agreement with the variability noted in Chapter 4 for the backcalculated moduli values of both the older and newer pavements.

Several factors contribute to the total discrepancy noted between field-mixed and laboratory-mixed cylinder UCS test results. A significant factor, as discussed in Chapter 6, is the difference in curing temperature. As previously noted, the laboratory-mixed cylinders are cured for 28 days at 100 ^{0}F, and the field-mixed cylinders are cured for the same time period but at 73 ^{0}F. Another significant factor is the difference in the degree of proportioning and blending between the laboratory and in the field. Laboratory proportioning of LFA and soil blends result in much more uniform blends than that possible in the field using the current field-mixed-in-place method of blending the materials. A comparison of the UCS of laboratory and field mixed materials cured at these two temperatures provides insight into the combined effects of these two factors.

Recall from Chapter 6 that a limited laboratory investigation was conducted to investigate the affects of curing temperature on the UCS of LFA stabilized soil. The material used to fabricate the cylinders for this laboratory investigation was laboratory proportioned and blended material. Table 23 includes the individual cylinder UCS test results for two sets of six cylinders that were fabricated as a part of this laboratory investigation. One set was cured at 73^{0}F and the other set was cured at 100^{0}F. The set cured at 73^{0}F had an average UCS of 67 psi with a coefficient of variation of 6.8 percent. The set cured at 100^{0}F had an average UCS of 590 psi with a corresponding coefficient of variation of 14.1 percent. The difference in the average UCS due to the difference in curing temperatures is 523 psi.

A total of 32 cylinders were fabricated over a two-day period from a blend of field-mixed-in-place lime, fly ash, and soil. This field blended material was obtained from the same project that the soil was obtained for the limited laboratory investigation referenced in the preceding paragraph. The 16 cylinders that were fabricated on each of these days were subdivided into two groups of eight cylinders. One group was cured for 28 days corresponding to the design temperature of 100 ^{0}F, and the other group was cured for 28 days at 74 ^{0}F, which is slightly higher than that typically used for field-mixed cylinders.

Tables 24 and 25 illustrate the significant difference in UCS due to the difference in curing temperature. For the cylinders fabricated on June 14, 2001 (Table 24), the difference in average UCS between the two curing temperatures is 223 psi, and for June 15, 2001 (Table 25), the difference is 316 psi. The coefficient of variation for each of the four sets of eight cylinders is in very good agreement ranging from 24.8 to 27.5 percent.

Comparison of the UCS test results for the laboratory-mixed cylinders relative to the field-mixed cylinders can be done because the raw soil and fly ash used for all of the cylinders referenced in the current discussion came from similar sources. At 73 ^{0}F, the average UCS of the laboratory-mixed cylinders is somewhat higher than that of the field-mixed cylinders; however, at the higher curing temperature of 100 ^{0}F, there is a significant increase in the average strength of the laboratory-mixed cylinders compared to the field-mixed cylinders. At the 100 ^{0}F curing temperature, the coefficient of variation is 82 percent greater for the field-mixed material compared to the laboratory-mixed material.

Differences in the UCS results are expected between the laboratory-mixed and the field-mixed cylinders. The percentages of lime and fly ash in the laboratory-mixed cylinders are different than that of the field-mixed cylinders, with the laboratory-mixed cylinders possessing percentages close to the design percentages. The soil used in both sets of cylinders is not exactly the same even though the soil came from the same borrow pit. The compaction moisture content in the field may not be close to optimum for the exact blend of materials used to fabricate the field-mixed cylinders, whereas the laboratory-mixed cylinders were fabricated close to optimum moisture content. This would affect the density of the cylinders, even when compacted with the same compaction effort.

Comparison of the coefficient of variation of the UCS test results for the laboratory-mixed cylinders relative to the field-mixed cylinders illustrates the difference in the uniformity of the blended material when proportioning and blending the materials in the laboratory compared to proportioning and blending the materials using the current field-mixed-in-place method of construction. These two methods represent the opposite in extremes for blending the materials. Improving the current method of field-mixed-in-place or going to plant mixed material would provide a blend having a degree of uniformity at some intermediate point between these two extremes.

QC/QA

The field-mixed cylinder UCS test results should be used for QC/QA of LFA base course construction. As discussed in Chapter 8 the field mixed material cylinders should be compacted to 96 percent modified Proctor density to correspond to the recommended in-situ density of the LFA base layer. Compaction of these cylinders to this level of density, transporting them back to the laboratory in split-molds, and then curing at 100 ^{0}F instead of the current 73 ^{0}F, should provide field-mixed cylinder UCS values that can be used for the purpose of QC/QA.

Due to the inherent variations in relative percentages of lime and fly ash, variations in the quality of the fly ash and properties of the borrow material, and the molding moisture content of the field-mixed cylinders, some UCS value less than the 500 psi design value should be set as the required strength for these cylinders. Chapter 11 includes a discussion that recommends a Proctor UCS value of 400 psi for a LFA stabilized soil base course.

Current MDOT policy is to fabricate one Proctor size UCS sample for every 8,000 square yards of LFA stabilized material placed, with a minimum of one per day's production. One QC/QA option is that this number be increased to two samples for every 8000 square yards, with a minimum of two per day's production when that day's production does not exceed 8,000 square yards. The UCS test result reported and compared to the strength requirement should be the average of these two cylinders.

The next issue is to determine what remedial action should be required when the achieved strength does not equal or exceed the required strength for the LFA stabilized soil base course. GaDOT has a specification for cement stabilized soil base courses that contains a "Strength Correction Chart" to address this issue. GaDOT requires 300 psi UCS of cement stabilized soil cores obtained from the roadbed. For UCS values between 200 to 299 psi, the corrective work required for 6- and 8-inch thick base courses is to add 135 lbs. of HMA per square yard. A minimum of 150 feet of HMA is placed when correcting areas of deficient strength. For UCS values less than 200 psi, the affected area is reconstructed. All corrective and reconstructive work requiring HMA is performed at no additional cost to the Department (Supplemental Specification Section 301 – Soil-Cement Construction). This requirement for corrective work at no expense to the State serves as a strong motivation for the contractors to provide a stabilized base course possessing the required strength. It is recommended that MDOT develop a specification that includes corrective work to be performed when adequate strengths are not achieved in LFA stabilized soil base courses.

Another issue that should be addressed in a QC/QA program for LFA stabilized soil base course construction is layer thickness. Figure 45 illustrates the affect of inadequate in-situ LFA base course thickness. This core was obtained from the George County project, which had a six-inch design base course. The in-situ layer thickness at the station from which this core was obtained was four inches. Note the extensive amount of cracking in the core, which is a consequence of the load-induced flexural fatigue experienced by this material. It is recommended that the QC/QA program include measuring the in-situ LFA base course layer thickness the same day that the base course is constructed to ensure that the design layer thickness is achieved in the field. The dynamic cone penetrometer (DCP) could be employed for this purpose.

Remedial action for inadequate base layer thickness depends upon the type of fly ash used in the blend. When Class F ash is used in the blend, the base layer can be remixed with the pulvamixer to a depth sufficient to obtain the desired layer thickness. When Class C fly ash is used in the blend, remixing is not advised, and the inadequate base layer thickness should be compensated with an additional thickness of overlying pavement layer material.

Late Fall Construction

A concern regarding LFA stabilized soil base course construction is the quality of this stabilized material when it is placed near the end of a given construction season. The construction season for this material in Mississippi is March 1 through November 30. As discussed in Chapter 6 both temperature and time are required for strength development in this material. Given the typical cool winter months experienced in Mississippi, little gain in strength is anticipated in base courses stabilized with LFA during this period of time, especially when Class F ash is utilized in the blend.

Degree Days

The concept of a degree day (DD) was developed to aid transportation agencies in selecting an appropriate date to end LFA stabilized soil construction activities prior to the onset of the first freeze/thaw (F/T) event (NCHRP No. 37, 1976). A sufficient amount of time is required to elapse between the construction of the stabilized base course and the first occurrence of F/T in order for the stabilized material to gain sufficient strength to resist degradation due to that F/T event.

A DD is defined as a unit representing one degree of declination from a standard temperature in the average temperature of one day (American Coal Ash Association, 1991). The reactions responsible for strength development in the stabilized material cease below a temperature of 40 ^{0}F; therefore, a temperature of 40 ^{0}F is often selected as the standard temperature. The average temperature is the temperature of the LFA stabilized material, not the air temperature (NCHRP No. 37, 1976). A good example to illustrate the concept of DD is with the computation of the number of DD associated with an MDOT LFA mix design. The temperature of the curing room is 100 ^{0}F, which represents the average temperature of the LFA and soil blend. The declination is 100 minus 40, or 60 ^{0}F and the curing period is 28 days. The product of 60 ^{0}F and 28 days is 1680 DD. Assuming the blend in the field is the same as the blend in the mix design, any combination of products of declination from 40 ^{0}F and number of days that provide a summation equal to 1680 DD should theoretically yield a Proctor UCS of 500 psi for that material in the field.

F/T is not considered a major issue in Mississippi, although the northern counties do experience a greater incidence of these events than the rest of the state. When a freezing incidence does occur, the length of time is typically short enough that minimal frost penetration is realized for a given locale. Materials that are susceptible to F/T damage and are exposed at the surface may experience degradation.

The Second District encompasses portions of the northern and central regions of the State. The results of a field investigation conducted by this district provides guidance for the amount of cover required to protect an LFA stabilized soil material placed near the end of a given construction season.

A project on Hwy. 61 located from U.S. Hwy. 49 near Lulu to Ms. Hwy. 4 near Clayton in Coahoma and Tunica Counties included an LFA stabilized soil subgrade and an LFA stabilized soil base course. The subgrade was stabilized during the fall of 2000. A portion of this subgrade was covered by six inches of topping material for subsequent stabilization during the following construction season while the rest of the stabilized subgrade remained exposed at the surface throughout the winter of 2000 – 2001. In April of 2001 a DCP was used to evaluate the in-situ CBR of the covered and exposed sections of the stabilized subgrade. The target CBR for this subgrade was 20 since this corresponds to the required CBR for lime stabilized subgrades. The in-situ CBR of the exposed subgrade did not meet this requirement; however, the covered subgrade did meet the requirement (Turner, 2001).

Given the results of this field investigation and the fact that F/T is generally not a major issue in Mississippi, no DD requirements are recommended for LFA stabilized soil base course construction in Mississippi. It is recommended that the LFA stabilized soil pavement layer be provided with at least minimal cover to minimize the potential for degradation due to F/T events.

Potential for Leaching of Unreacted Lime During the Winter Months

A problem that has been observed with late fall placement is the saturation of the LFA and soil blend before significant gain in strength. Chapter 7 included a discussion of this problem and included a recommendation for increasing the compacted density of the material to minimize the amount of absorbed water in this pavement layer. One potential consequence of late season construction is that unreacted lime may leach out of the blend of material and therefore not be available for continued pozzolanic reactions when the temperature increases at the onset of the following construction season.

An attempt was made to model this potential loss of lime in the laboratory. Chapter 6 included a discussion of the impact of curing temperature on the level of strength developed in LFA stabilized soils. Twelve of the cylinders utilized in that laboratory investigation were used to evaluate any potential effects due to late fall construction on the stabilized material. Two sets of three cylinders were cured for 90 days at 50 ^{0}F and then tested for UCS. Two additional sets of three cylinders were first cured for 90 days at 50 ^{0}F, followed by 28 days of curing at 100 ^{0}F, and then tested for UCS. All four sets were cured for the first 30 days of the 90 day curing period with no moisture conditioning.

The two sets of three cylinders that were cured for 90 days at 50 ^{0}F were subjected to a different moisture regime during this low-temperature curing period. For one set each LFA cylinder was placed on a circular porous stone and then placed in a plastic container. The plastic container was then placed in the environmental chamber and the bottom of the container filled with water to a level corresponding to the top of the porous stones. In this manner the LFA stabilized cylinders were saturated via capillary rise for 60 days, with this method of saturation modeling the saturation of a base layer from a sub pavement source of water. This technique of subjecting the LFA and soil cylinders to a saturated condition is similar to that used for the capillary soak associated with the tube suction test (Little and Yusuf, 2001).

The second set of LFA and soil cylinders were also placed on porous stones and then into the environmental chamber. These cylinders were subjected to four two-week cycles of moisture conditioning. Each cycle included one week during which the cylinders were completely submerged. The water was then drained, and the drained condition was maintained for the second week. These cycles were an attempt to model rain occurrences that may act to “flush” the stabilized layer. It was postulated that the water would cause some of the lime to go into solution and then leave via drainage from the cylinders.

The UCS test results for these two sets of three cylinders are shown in Table 26. The capillary soaked cylinders show a slightly less UCS average than the capillary soaked cylinders. Figure 25 in Chapter 6 shows the average for all six of these cylinders subjected to this single temperature curing regime.

The two sets of three cylinders that were cured for 90 days at 50 ^{0}F followed by 28 days of curing at 100 ^{0}F were moisture conditioned in the same manner as described for the first two sets of cylinders. The UCS test results for these two sets are also included in Table 26. There is a difference in the results between these two sets, with the capillary soaked cylinders showing a lower UCS test average. Figure 25 in Chapter 6 shows the average for all six of these cylinders subjected to the two temperature curing regimes. A lower average UCS for the condition of capillary soaking relative to cyclic soaking is observed based on a review of all four sets of three cylinders. It is possible that capillary soaking represents a more severe moisture conditioning method than cyclic soaking.

Regardless of the method used for moisture conditioning, it is noted that the LFA and soil blend experienced a significant increase in strength following the 90 days of curing at 50 ^{0}F after being subjected to the higher curing temperature. This indicates the potential for late fall constructed field mixed blends of LFA and soil to experience increases in strength during the following spring and summer months. Note, however, that these strengths were not as high as the cylinders that were not subjected to the moisture conditioning; i.e., the curing associated with an LFA mix design. This can possibly be attributed to the leaching of some of the unreacted lime during the moisture conditioning phase.

Difference Between LFA Stabilized Soil Cured in the Laboratory and In the Field

A significant difference between the LFA stabilized material cured in the laboratory, and that cured over the winter and following spring and summer months in the field, is that the laboratory cured material is not subjected to any loading. The material in the field will likely be subjected to at least minimal construction loading during this time period. As discussed in Chapter 7, during the winter months the relatively uncured LFA and soil blend behaves more like an unbound granular material than a cemented or bound material. Recall that variations in moisture content significantly affect the stiffness of unbound materials. Another consideration is that the long term performance of the LFA stabilized soil base course can be detrimentally affected if it is loaded while still in a weakly cemented stage. Chapter 11 includes a discussion regarding the loading of partially cured LFA stabilized soil material. Thus, while the laboratory results indicate that pozzolanic reactions can be initiated following a relatively dormant period with corresponding gains in strength, a similar increase in the quality of the stabilized material may not be realized in the field.

Three of the five newer pavements included LFA stabilized soil base course material that was placed both in the fall and following spring/summer time periods, which allows a limited evaluation for potential difference due to late fall construction. Table 27 includes data from these three projects. The stations included in the pavement testing represent material placed during the later part of the 1998 construction season and material placed during the following 1999 construction season. Refer to Table 7 in Chapter 3 for the actual months during which this material was placed on the roadway for each of these three projects. The Proctor strengths could not be utilized in the comparison analyses due to the upper limit imposed by the UCS testing machine; however, the backcalculated LFA moduli and normalized LFA layer coefficient values are available for use in these analyses. The Proctor strengths, core density and core percent standard Proctor densities are included as potential explanatory data.

The statistical F and T tests were selected to compare each set of data from each of the three projects using a level of significance, or alpha, of 0.10. A summary comparison is included at the end of Table 27. For both the Bolivar and Clarke County projects there is no statistical difference between the properties of the LFA stabilized soil material placed in the fall of 1998 relative to the material placed the following construction season. For the Smith County project there is a statistical difference in LFA material properties between these two time periods; however, the data indicates that the material placed during the fall of 1998 is a better quality material than that placed the following summer. For these three projects it can be concluded that there are no detrimental effects to the LFA stabilized soil due to late season construction.

Conclusions Regarding Late Fall Construction

Based on the discussions from the current chapter and Chapters 6 and 7, the conclusions regarding late fall construction of LFA stabilized soil base courses are as follows:

1. Use of LFA stabilized soil as a base course material is acceptable for late fall construction provided that the material is compacted to a minimum 100 percent standard density.

2. Construction loading is kept to a minimum, and the pavement will not be open to traffic during the winter months immediately following construction.

3. The exclusive use of Class C ash in the LFA stabilized soil blend may provide sufficient strength for traffic loading during the winter months immediately following construction if:

    a. The Class C fly ash has a self – cementing component of strength gain to provide acceptable performance of the base course under traffic loading until the pozzolanic strength gain reactions are initiated during the following spring and summer months.

b. Field compaction can be performed in an expedient manner to take advantage of this potential self – cementing component of strength gain.

4. If no Class C fly ash is available with sufficient self – cementing characteristics and the pavement must be opened to traffic, a different chemical stabilizing agent, such as cement, should be used for stabilizing the base course.

5. The stabilized base course should not be exposed at the surface throughout the winter months immediately following construction of this course.

6. The stabilized base course should be covered with, as a minimum, the next course within the given pavement structure.

Figure 45. Cracking in LFA Base Layer Due to Inadequate In-Situ Base Layer Thickness

Table 23. Difference in Strength of Laboratory-Mixed Material Cylinders Due to Difference in Curing Temperature

Laboratory-mixed - cured at 73^0F

Data file name	UCS (psi)	Dry unit weight (pcf)	Moisture Content (%)
BB25B-1	73	115	13.9
BB25B-2	60	115	15.3
BB25B-3	68	115	14.1
BB25B-4	69	115	13.9
BB25B-5	64	114	14.1
BB25B-6	69	115	14.1
Average =	67	114.9	14.2
Std. Dev. =	4.5		
Coef. Of Var. =	6.8		
High =	73		
Low =	60		
Difference =	13		

Laboratory-mixed - cured at 100^0F

Data file name	UCS (psi)	Dry unit weight (pcf)	Moisture Content (%)
BB25-1	490	112	14.7
BB25-2	695	115	13.6
BB25-3	498	114	13.7
BB25-4	582	113	14.1
BB25-5	630	113	14.1
BB25-6	647	113	14
Average =	590	113.2	14.0
Std. Dev. =	82.9		
Coef. Of Var. =	14.1		
High =	695		
Low =	490		
Difference =	205		

Material Tested: Red Sand Topping - 100 % passing the #10 sieve, 23 % passing the # 200 sieve, Non plastic, Obtained from U.S. 82 near Eupora, Mississippi

Note: All cylinders soaked for 5 hours prior to UCS testing

Table 24. Difference in Strength of Field-Mixed LFA Material Cylinders Due to Difference in Curing Temperature for Cylinders Fabricated June 14, 2001.

Field-mixed - cured at 74°F

Data file name	UCS (psi)	Dry unit weight (pcf)	Moisture Content (%)
EU144M	44	111	13.7
EU146M	41	106	15.7
EU148M	43	109	14.4
EU150M	44	108	14.9
EU171M	45	109	15.0
EU172M	57	108	15.4
EU173M	57	107	15.6
EU174M	80	107	15.8
Average =	51	108.1	15.1
Std. Dev. =	13.1		
Coef. Of Var. =	25.6		
High Value =	80		
Low Value =	41		
Difference =	39		

Field-mixed - cured at 100°F

Data file name	UCS (psi)	Dry unit weight (pcf)	Moisture Content (%)
EU144H	379	111	14.1
EU146H	253	107	15.5
EU148H	303	109	14.6
EU150H	322	107	15.4
EU171H	226	109	14.8
EU172H	158	107	16.1
EU173H	244	108	15.6
EU174H	304	107	15.9
Average =	274	108.1	15.3
Std. Dev. =	67.8		
Coef. Of Var. =	24.8		
High Value =	379		
Low Value =	158		
Difference =	220		

Note: All cylinders soaked for 5 hours prior to UCS testing

Table 25. Difference in Strength of Field-Mixed LFA Material Cylinders Due to Difference in Curing Temperature for Cylinders Fabricated June 15, 2001.

Field-mixed - cured at 74°F

Data file name	UCS (psi)	Dry unit weight (pcf)	Moisture Content (%)
EU119M	72	112	14.2
EU124M	49	113	14.1
EU127M	29	110	14.5
EU130M	44	110	14.3
EU134M	60	112	14.2
EU136M	41	109	14.8
EU138M	55	111	14.4
EU140M	40	108	15.0
Average =	49	110.5	14.4
Std. Dev. =	13.4		
Coef. Of Var. =	27.5		
High Value =	72		
Low Value =	29		
Difference =	42		

Field-mixed - cured at 100°F

Data file name	UCS (psi)	Dry unit weight (pcf)	Moisture Content (%)
EU119H	300	112	15.0
EU124H	507	113	14.2
EU127H	260	110	14.4
EU130H	422	110	14.8
EU134H	464	112	13.9
EU136H	235	108	15.1
EU138H	387	110	14.8
EU140H	347	108	15.3
Average =	365	110.6	14.7
Std. Dev. =	97.2		
Coef. Of Var. =	26.6		
High Value =	507		
Low Value =	235		
Difference =	272		

Note: All cylinders soaked for 5 hours prior to UCS testing

Table 26. Difference in Strength of Laboratory-Mixed Material Cylinders Due to Difference in Moisture Conditioning

Laboratory-mixed - cured at 50^{0}F for 90 days
Capillary Soaked

Data file name		UCS (psi)	Dry unit weight (pcf)	Moisture Content (%)
Octcapi1		71	111	15.5
Octcapi2		70	111	15.5
Octcapi3		52	110	16.2
	Average =	64.4	110.8	15.7

Laboratory-mixed - cured at 50^{0}F for 90 days
Cyclic Soaked

Oct4cyc1		65	111	15.8
Oct4cyc2		74	112	15.4
Oct4cyc3		65	110	15.9
	Average =	67.9	111.0	15.7

Laboratory-mixed - cured at 50^{0}F for 90 days
Capillary Soaked
Cured for 28 days at 100 ^{0}F

Novcapi4		377	110	15.2
Novcapi5		461	111	14.8
Novcapi6		360	110	14.8
	Average =	399.2	110.4	14.9

Laboratory-mixed - cured at 50^{0}F for 90 days
Cyclic Soaked
Cured for 28 days at 100 ^{0}F

Nov4cyc4		413	114	14.2
Nov4cyc5		401	113	14.2
Nov4cyc6		632	111	15.2
	Average =	482.0	112.5	14.5

Material Tested: Red Sand Topping - 100 % passing the #10 sieve, 23 % passing the # 200 sieve, Non plastic, Obtained from U.S. 82 near Eupora, Mississippi

Note: All cylinders soaked for 5 hours prior to UCS testing

Table 27. Fall Versus Spring Construction

County	Station	LFA $E_{(back)}$ Modulus, ksi	Normalized LFA Layer Coefficient	Proctor Equivalent UCS, psi	Core Density lbs/cu.ft.	Core % Standard Proctor Density
Bolivar		Fall 1998 Construction				
	290+00	88.4	0.26	255	100.8	88.7
	295+00	353	0.24	609	104.3	91.8
	300+00	213	0.17	No Core Tested		
	305+00	165.1	0.15	795+	112.9	99.3
	335+00	Outlier per Chauvenet		390	105.2	89.6
	340+00	90	0.13	155	105	89.4
	345+00	573.8	0.27	795+	112.5	95.8
	350+00	411	0.32	795+	114.1	97.2
		Spring 1999 Construction				
	713+00	211.7	0.2	490	113.7	97.3
	718+00	393.3	0.25	521	104	89.1
	723+00	126.9	0.16	579	105.1	90
	728+00	333.2	0.25	685	109.9	94.1
	733+00	323.8	0.18	455	109.7	93.7
	738+00	339.6	0.28	668	110.5	94.4
	743+00	406.1	0.19	359	99.7	85.1
	748+00	197.3	0.19	473	107.8	92
Clarke		Fall 1998 Construction				
	758+00	474.5	0.26	285	104.4	Record not located
	761+00	746.6	0.32	182	102.9	Record not located
	764+00	652.3	0.24	382	99	Record not located
	767+00	592	0.26	417	104.6	Record not located
	770+00	837.6	0.32	474	103.2	Record not located
	773+00	732.6	0.38	135	105.8	Record not located
		Spring 1999 Construction				
	39+50	LFA E > HMA E		795+	101.3	88.4
	40+00	754.1	0.31	884	103	89.9
	40+50	HMA E too high		795+	106.4	92.9
	41+00	326.7	0.21	795+	108.2	94.5
	41+50	950.1	0.3	795+	112.5	98.2
	42+00	808.9	0.26	728	111.9	97.8
	42+50	932.7	0.28	818	112.6	98.3
	43+00	358.5	0.23	265	107.4	93.8

Table 27 Continued. Fall Versus Spring Construction

County	Station	LFA $E_{(back)}$ Modulus, ksi	Normalized LFA Layer Coefficient	Proctor Equivalent UCS, psi	Core Density lbs/cu.ft.	Core % Standard Proctor Density
Smith		Fall 1998 Construction				
	493+00	LFA E > HMA E		795+	112.7	96.3
	498+00	LFA E > HMA E		795+	114	97.5
	503+00	Outlier per Chauvenet		788	107	91.4
	508+00	335.4	0.22	323	110.4	95.4
	518+00	LFA E > HMA E		795+	108.9	94.1
	522+00	389	0.24	585	111.3	96.2
	528+00	448.4	0.24	408	106.2	91.8
		Summer 1999 Construction				
	610+00	240.5	0.26	308	97.8	83.4
	613+00	253.4	0.22	242	106.1	90.5
	616+00	129.5	0.26	131	105.2	89.7
	619+00	156.4	0.25	620	104.7	89.2
	622+00	160.2	0.15	477	115.1	98.1
	625+00	104	0.13	347	106.4	90.7
	628+00	65	0.16	100	100.5	85.7
	631+00	Outlier per Chauvenet		No Core Tested		

Summary Comparison

County	Variable	alpha	F-test	T-test
Bolivar	LFA $E_{(back)}$	0.1	no	no
	Norm. LFA a_2	0.1	no	no
Clarke	LFA $E_{(back)}$	0.1	no	no
	Norm. LFA a_2	0.1	no	no
Smith	LFA $E_{(back)}$	0.1	no	yes
	Norm. LFA a_2	0.1	yes	no

Chapter 11 -- Short-Term Construction Loading and In-Service Stress/Strength Considerations of LFA Stabilized Soil Base

This chapter considers the timing of placement of construction equipment onto a recently constructed LFA stabilized soil base layer and provides an estimation of the required in-situ Proctor UCS for this material in service under traffic loading. Flexural stress/flexural strength ratios, hereafter referred to as stress/strength ratios, were computed for various levels of material quality and pavement geometry and then compared to relevant criteria to facilitate the conclusions and recommendations developed in this chapter.

Flexural Stresses

The flexural stresses used in determining the stress/strength ratios for the LFA base course were computed using the layered elastic computer programs WESLEA and Bisar. For the short-term construction loading condition the use of such programs allow a determination of the variation in flexural stresses that develop in the LFA base layer as the chemically stabilized soil base and subgrade layers increase in stiffness (modulus) due to the continuing pozzolanic reactions associated with these stabilized materials. For the in-service loading condition variations in LFA base course stiffness were modeled in these programs to provide a basis for the recommendation to increase the design base thickness to eight inches and select an in-situ LFA Proctor UCS.

None of the pavements considered in the current study included a chemically stabilized subgrade layer; however, current MDOT pavement design/construction practice includes such a layer. These points were included in the letter from the Blain Companies dated May 31, 2002, in response to proposed changes in the Mississippi LFA base course construction specifications. The letter suggested that the performance of the LFA base courses may already be sufficiently improved by the inclusion of the stabilized subgrade layer and that more analyses should be performed to verify the need for making these changes. These programs enable the suggested analyses because the chemically stabilized subgrade layer can be modeled as an additional pavement layer.

In another letter from the Blain Companies dated May 28, 2002, a suggestion was made to increase the thickness of the LFA base course from 6 inches to 8.5 inches. It was postulated that this change would result in a significant increase in the performance of this layer relative to the additional cost for its construction. Layer thickness can be varied in these two programs, both of which allow for modeling a suggested increase in the thickness of the base layer.

The thickness of each layer in the pavement structure is entered into the programs in addition to the corresponding layer material properties modulus and Poisson's Ratio. These values vary based on whether construction or in-service loading is considered and the particular facet reviewed for that consideration. Tables 28 and 30 summarize these values for the construction loading and in-service loading conditions respectively. Appendix G provides details regarding the selection of these material properties and the loading inputs into the computer programs.

Several factors affect the magnitude of the calculated flexural stress in the LFA stabilized soil base course. These include the loading, level of subgrade support, the thickness and stiffness of each of the layers in the pavement structure, including the chemically stabilized subgrade layer, and the condition of bonding between the layers. These factors are addressed in greater detail in subsequent subsections of this chapter.

LFA Flexural Strength

LFA flexural strength can be conservatively estimated as 20 percent of the material's UCS (NCHRP No. 37). One of the test condition variables affecting the UCS of a material is the test specimen length/diameter (L/D) ratio, where a reduction in this ratio typically results in an increase in the measured UCS. This reference does not indicate whether a Proctor size sample, having a L/D ratio of 1.15:1, was used when this estimate was developed, or the standard 2:1 ratio typically required for UCS testing. It was assumed that the reference was considering the use of the standard 2:1 ratio size specimens. LFA Proctor UCS values are approximately 30 percent greater than LFA UCS values for samples having a L/D ratio of 2:1; therefore, the LFA flexural strength was estimated as 0.7 times 0.2, or 0.14, times the Proctor size sample LFA UCS. As LFA stabilized soil cures, it increases in strength; therefore, a variable level of flexural strength was considered when calculating stress/strength ratios.

Stress/Strength Ratios

The flexural stress developed in the LFA base layer was compared to the flexural strength of that layer for various combinations of pavement geometry and pavement layer material stiffness. This comparison was made in the form of stress/strength ratios to facilitate data comparison and observation of trends. The flexural stress due to the imposed loading equals the flexural strength of the material when the stress/strength ratio is equal to one. A ratio exceeding 1 means that the stress exceeds the strength and the material would be expected to crack from a single load application. A ratio less than 1 means that the stress is less than the strength, and more than one load application would be required to crack the material.

The lower the stress/strength ratio, the more loads that can be applied to the LFA base layer before effecting fatigue cracking. A transfer function, or fatigue equation, can be used to relate ratios less than one to the number of loads that can be applied before the base layer experiences such cracking. The following equation is used in this study (American Coal Ash Association, 1991):

$$\text{Log } N = (0.972 - SR) / 0.0825 \quad \text{Equation 8}$$

Where: N = Number of load repetitions

SR = stress/strength ratio

The LFA modulus and Proctor UCS values listed in Tables 28 and 30 were obtained by either field or laboratory testing, or are estimated values. A relationship, Equation 3 in Chapter 4, was developed between LFA backcalculated modulus and LFA Proctor UCS to facilitate computations of either flexural stress or flexural strength values where test data was not available. For example, the average strength of the 139 field mixed material cylinders cured at 73 ^{0}F was approximately 70 psi. A modulus corresponding to this strength was not determined via laboratory or field testing, but was required to compute stresses in the LFA material for construction loading following one month of spring or fall field curing conditions. An estimated modulus of 41,200 psi corresponds to a Proctor UCS of 70 psi based on the use of Equation 3. Due to the low R^2 value associated with the data to derive Equation 3 and the estimation of flexural strength from UCS, the stress/strength ratios presented in this report are useful to represent trends and obtain an estimate for the required Proctor UCS for an LFA stabilized soil base course. Additional research needs to be performed to better define this relationship.

Short-Term Construction Loading

Subsequent to placement, traffic can be turned onto a lift of asphalt as soon as it cools. Soil cement strength and stiffness gains occur relatively quickly due to hydration reactions. However, the pozzolanic reactions effecting increasing strength and stiffness in LFA stabilized soil layers require time and temperatures exceeding 40^0F. This facet of LFA strength and stiffness gain creates a significant implication for subsequent construction operations soon after the placement of a LFA stabilized soil layer.

Two Options Regarding Construction Loading

Two schools of thought currently exist regarding construction loading on a partially cured LFA stabilized soil base course. One is that the stabilized material should be allowed to cure until it has attained sufficient strength and stiffness such that construction loading will not overload and crack the material. The second school of thought is that the stabilized material should be loaded as soon as possible after placement to induce microcracks into this pavement layer. The results of two studies indicate that the microcracks satisfy the propensity of the stabilized material to undergo shrinkage cracking, but do not reflect through the overlying HMA layer. MDOT's current policy is that the subsequent course is not placed on the LFA layer for at least 7 calendar days. During this time period the LFA layer is not subjected to any type of traffic or equipment loading.

Option One -- Wait

The first school of thought, or option, is to wait until the material has developed enough strength and stiffness to carry the loads without cracking (American Coal Ash Association, 1991). This reference recommends that the stabilized layer be allowed to cure for seven days before construction loads are applied and suggests a minimum in-place strength of 350 psi if these loads are to be applied prior to the recommended time of cure. Based on experience obtained from previous MDOT State studies, some LFA stabilized granular material may not achieve this level of strength even after one month of summer field curing (George, 2001, George and Uddin, 2000). Due to the slow cure rate of LFA stabilized material, subsequent construction operations cannot be held up until this level of strength is developed in this material.

While the 1991 American Coal Ash Association reference suggests a minimum strength of 350 psi for the LFA stabilized material, it is instructive to determine if this material really becomes overloaded and cracks from application of construction equipment for UCS values less than this recommended level. Stress/strength ratios are amendable for performing this evaluation. For the construction loading condition the WESLEA program was used to perform the requisite stress calculations for determining these ratios.

Figure 46 illustrates how the stress/strength ratio changes as the LFA stabilized base course material cures and increases in stiffness, or modulus. This figure also illustrates the impact of the inclusion of a construction platform; i.e., LTS, as opposed to no platform, as well as the impact of the quality, or stiffness, of this platform, on the value of stress/strength ratios in the overlying base layer.

The left endpoint of each of the curves shown in Figure 46 corresponds to the approximate degree of stiffness of the LFA stabilized material following one month of spring or fall field curing. The right endpoint of each of these curves corresponds to the recommended 350 psi strength. It was assumed that this reference was considering an UCS of samples having a 2:1 L/D ratio; therefore, an equivalent LFA Proctor UCS of 500 psi was used to obtain a modulus value from Equation 3. This modulus value is the x coordinate of each of the right end points in Figure 46. Note that a Proctor UCS of 500 psi corresponds to MDOT's LFA mix design strength for base course construction.

The upper curve in Figure 46 is for an LFA stabilized base with no underlying construction platform. MDOT currently chemically stabilizes the top 6 inches of the design soil and the impact of the inclusion of this layer is illustrated by the middle and bottom curves in this figure. Figure 46 illustrates that as the construction platform stiffness increases; a corresponding reduction in the stress/strength ratio is observed in the overlying base layer for given values of base layer stiffness.

For LFA stabilized material cured for one month under spring or fall curing conditions and supported by a good construction platform; i.e., platform modulus of at least 40,000 psi in this case, and a subgrade with a CBR of 5, the left endpoint of the bottom curve in Figure 46 indicates that minimal tensile stress is developed at the bottom of the LFA stabilized soil layer; hence, the stress/strength ratio is approximately 0. The LFA material has not cured enough to carry a significant amount of the load and is in effect being "cradled" by the underlying construction platform and subgrade. The first lift of HMA can be placed without overstressing the base material.

Summer curing temperatures are significantly higher than spring or fall curing temperatures, thus effecting a greater degree of cure and developed stiffness in the base layer. Evidence of this is provided from test results obtained during the summer of 2000. A 1000-ft. LFA stabilized soil base layer control section was tested after 28 days of field curing from mid August to mid September. The average backcalculated modulus of this section was 101,500 psi (George, 2001). Referring to Figure 46, assuming a subgrade CBR of 5 and good construction platform, the stress/strength ratio would exceed 1.2. This indicates that the additional curing has provided a sufficient strengthening and stiffening of the LFA material to allow it to carry some of the load with resulting tensile stresses developing at the bottom of the LFA layer. Placement of the first lift of HMA would overstress the base material and cause some cracking within this pavement layer. Figure 46 illustrates, for the given levels of subgrade and construction platform stiffness, a significant increase in the stress/strength ratio as curing progresses from the “One Month Spring or Fall” condition to the “One Month Mid-Summer” condition.

In summary, given LFA base modulus values less than about 75,000 psi and a minimum LTS modulus of 40,000 psi, Figure 46 indicates that construction loading will not crack the bottom of the LFA base course for the current typical MDOT pavement design/construction practice of using 6 inches of LFA stabilized base overlying 6 inches of chemically stabilized design soil. For lower quality construction platforms; i.e., LTS modulus values less than 40,000 psi, and LFA modulus values exceeding 75,000 psi, Figure 46 illustrates that the LFA stabilized base layer will crack due to construction loading, even at the recommended equivalent LFA Proctor UCS value of 500 psi.

Since a good construction platform has such a significant effect on the stress/strength ratio of the overlying base course, the attainment of such a platform is of paramount importance to minimize cracking in this layer from construction loading. The same attention to detail should be observed during the construction of the stabilized subgrade layer as that given to the construction of the LFA base course to ensure the quality of this construction platform. In Chapter 7 the significant effect of relatively high levels of density was demonstrated on the resulting strength of an LFA stabilized material. The positive effects on strength due to increased density are also applicable for soil cement stabilized granular soils and lime stabilized clay soils. It is recommended that all construction platforms constructed with any one of these three stabilized materials be compacted to 100 percent standard density. This change will help the attainment of a good construction platform in the field.

The curves in Figure 46 were developed assuming a 6-inch LFA stabilized soil base layer supported by a 6-inch chemically stabilized subgrade layer. Using the WESLEA program, the effect of increased base and stabilized subgrade layer thicknesses was investigated to evaluate any potential reduction in the stress/strength ratio. The curves in Figure 47 were developed assuming a uniform chemically stabilized subgrade modulus of 40,000 psi. This figure indicates that for relatively low levels of base course stiffness, such as those corresponding to the “One Month Spring or Fall” condition, any of the combinations of base and stabilized subgrade thickness shown will result in stress/strength ratios less than one. For relatively higher levels of base course stiffness, however, such as those corresponding to the “One Month Mid-Summer” condition, an 8-inch base supported by a 6-inch stabilized subgrade is required to maintain the stress/strength ratio below 1. Layer thicknesses greater than 8 inches were not considered as thicker layers would require the placement of more than one lift of material in order to achieve acceptable levels of compaction throughout the depth of the layer.

Note that increasing the thickness of the base course instead of the LTS results in a greater reduction in the stress/strength ratio compared to increasing the thickness of the LTS and maintaining the 6-inch base course thickness. This is attributed to the modeled development of significantly higher levels of modulus in the base layer relative to the stabilized subgrade layer, and the thickening of this higher modulus material as opposed to the lower modulus material.

An added benefit of increasing the thickness of the base layer is that of more easily obtaining the higher levels of compaction in this layer compared to that of the stabilized subgrade layer. This is because the base layer has the added foundation support of the underlying stabilized subgrade layer, whereas the stabilized subgrade layer is compacted directly on top of the untreated subgrade.

The attainment of a uniform 40,000 psi stabilized subgrade modulus is not anticipated by the time of HMA placement for many situations encountered during field construction due to both inadequate curing time and temperature; therefore, an increase in the thickness of the base layer is not recommended for the purpose of reducing the potential for construction loading cracking of this layer. However, it will be demonstrated in the in-service loading discussion that such an increase is necessary for the long-term performance of the pavement.

In conclusion, for option one application of construction loading on a LFA stabilized base course should be made as soon as possible after construction of this layer, before significant curing has occurred, to minimize potential overloading and cracking of this stabilized material.

Option Two – Load ASAP

The previous discussion has indicated that in most cases construction loading will overload the LFA stabilized soil base course and cause it to crack with a single load application. Intuitively, overloading and cracking would be considered a detriment to the integrity of this pavement layer; however, some research indicates that construction load-induced cracking may not significantly affect the long-term strength and stiffness of a chemically stabilized soil pavement layer.

The second option regarding construction loading on a partially cured LFA stabilized soil base layer is that the stabilized material should be loaded as soon as possible after placement to induce microcracks into this pavement layer. The supposition of two recent studies with soil cement stabilization is that precracking the stabilized material will help to control subsequent shrinkage cracking in this material by creating many narrow cracks, instead of fewer and more widely spaced, but greater width cracks (George, 2001, Scullion, 2001). The propensity of this material to undergo shrinkage cracking is satisfied via the formation of these microcracks. These narrow cracks either subsequently heal due to additional curing, or do not reflect through the overlying HMA layers. Short-term construction loading can act as a precracking agent to induce these microcracks into the stabilized material.

The use of construction load-induced microcracking in a chemically stabilized material to mitigate subsequent reflective cracking is beyond the scope of this LFA study. However, this may be a consideration for LFA stabilized soil base layers and will be briefly addressed due to the results of map cracking on one of the older projects considered in this study. One of the tenets of LFA stabilized soil pavement layers is that it cures slowly, thus the shrinkage cracking problem is not as significant as it is for soil cement stabilized layers. Another tenet is that cracks that do occur in this material will cure due to a process called autogeneous healing, where in effect the cracks are cemented due to the continuing pozzolanic reactions of unreacted raw materials.

Reflective Cracking Due to Shrinkage Cracking in LFA Stabilized Soil Base Layer

One of the four older projects used in the current study is located on Hwy. 7 in Yalobusha County. This project was the focus of an earlier research effort conducted to evaluate the use of Class C fly ash in LFA stabilization (Ferguson, 1990). The LFA stabilized material was placed during the fall of 1989. The average time between placement of the LFA material and the placement of the first lift of HMA was approximately three weeks. A 400-ft. test section, from station 359+00 to station 363+00, was selected in which the cracks were mapped in the LFA stabilized material prior to placement of the HMA. The cracks that developed in this layer were attributed to shrinkage of this stabilized material. Twelve years later, in February of 2002, the cracks in the HMA were mapped throughout the length of this test section. A comparison of the two crack maps indicated that 82 percent of the cracks in the LFA layer reflected through the overlying HMA.

The reflective cracking observed in the Yalobusha project was probably exacerbated by the lack of a construction platform, since it was not included in the pavement design. Such a platform, as is currently required on all new pavement construction, would have reduced the load-induced pavement deflections and possibly allowed the stabilized material the required support to affect autogeneous healing of these shrinkage cracks. This might be considered analogous to placing a broken arm in a cast to prevent its movement while the break heals. Aside from being quite painful, if the arm were allowed to move freely about without a cast, the cracked bone would not heal.

Shrinkage cracking was also observed to occur in the LFA stabilized material during the construction of Hwy. 302 (George, 2001). Based on the shrinkage cracking observed in both the Hwy. 302 and Hwy. 7 projects, shrinkage cracking appears to be a problem in this type of stabilized material, and techniques should be implemented to mitigate its negative effects.

In the two referenced studies (George, 2002) and (Scullion, 2001), the cement stabilized soil was purposely precracked soon after construction of the stabilized layers. Subsequent testing indicated that these precracked layers regained strength and stiffness in two ways: similar to (1) either the gains recorded prior to the precracking operations, or (2) as compared to control sections. The project located on Hwy. 302 in Mississippi has a construction platform, and while the project in Texas does not, the latter pavement is located in a subdivision and thus not subjected to numerous relatively large load applications. Given the results of these two studies, it is postulated that even if an LFA stabilized soil base does crack due to the application of construction loads, this cracking will not significantly affect the long-term strength and stiffness gains of this stabilized material so long as it has adequate foundation support.

In summary, subsequent construction operations do not have to be detained until the stabilized material achieves some given level of strength and stiffness to avoid overloading and cracking this material. MDOT's current policy should be changed so that subsequent construction loading is applied soon after the placement of the LFA stabilized layer, even if the layer cracks due to this loading, provided a good construction platform is present.

Consideration for Heavy Truck Loading

The recommendation to load the LFA stabilized soil layer with construction equipment soon after its placement does not mean that the pavement should be immediately opened to heavy truck traffic. Such loading should be restricted such that the stress/strength ratio does not exceed 0.65 in the LFA stabilized base layer (American Coal Ash Association, 1991). The achievement of the requisite strength and stiffness to obtain this ratio varies based on many of the variables illustrated in Figure 34 of Chapter 9. The condition of field curing is a big factor. When using Class F fly ash in conjunction with late fall construction, the 0.65 requirement will preclude the opening of the pavement to heavy truck traffic until the following spring or summer.

In-Service Pavement Loading

Thus far the focus of this chapter as been directed towards the construction loading condition and several facets related to this condition. Now the focus will be directed towards the in-service loading condition with emphasis on the selection of an in-situ LFA Proctor UCS. The LFA stabilized soil base layer must be durable and meet the structural requirements for the pavement based on the anticipated loading. The selection of an in-situ LFA Proctor UCS is therefore based on the attainment of these two requirements. As with other topics addressed in the current study, information regarding soil cement and lime stabilization have been incorporated in the discussion for LFA stabilized soils due to the chemical similarities of these three stabilized materials. The benefit of increasing the LFA base layer design thickness from 6 inches to 8 inches is also addressed in the context of the current discussion.

Selection of an In-situ LFA Proctor UCS Based On Durability Requirements

Shrinkage cracking is a potential problem with both LFA and cement stabilized soils due to the reflection of these cracks through the overlying HMA layer. Interest has been expressed to reduce the required level of strength in soil cement stabilized layers to try to minimize this problem. The question as to how much strength is really needed to adequately carry the traffic loading has also generated interest in reducing the required level of strength (Crawley, 2002). While these are valid considerations, the quality of the material cannot be reduced to such an extent that it renders a nondurable material. A certain level of strength is necessary for the stabilized material to resist environmental affects, or weathering of this material. The importance of durability as it relates to determining an in-situ LFA Proctor UCS is highlighted with the following quote (Portland Cement Association, 1992):

> "The principal requirement of a hardened soil-cement mixture is that it withstand exposure to the elements. Strength might also be considered a principal requirement; however, since most soil-cement mixtures that possess adequate resistance to the elements also possess adequate strength, this requirement is secondary."

This reference considers two factors affecting the durability of the stabilized material. These include cycles of freezing and thawing, and cycles of wetting and drying. Freeze/thaw is not a significant problem in Mississippi; however, cycles of wetting and drying in an LFA base layer is a durability concern.

One weathering mechanism that may impact an LFA base layer is the leaching of lime from this layer. Lime may leach from this layer before the lime and fly ash have reacted to form cementitious compounds. Chapter 10 included discussion of the potential for leaching of the unreacted lime due to cycles of wetting and drying during the winter months following late season construction of the LFA base layer. While not verified in this study, it is anticipated that the rate of leaching is accelerated in base layers compacted to relatively low levels of density due to the higher levels of permeability that would correspond to these low density base layers.

The potential for leaching of lime from the pavement layer is not restricted to that of LFA and soil blends. Cyclic wetting and drying may cause leaching to occur in lime-treated soils and is considered a significant durability concern of these materials (Little, 1995). Research results cited by that author indicate that if a sufficient amount of lime is used to effect pozzolanic reactions with the corresponding formation of cementitious products, the effects of moisture are usually negligible; however, smaller amounts of lime may only effect flocculation and ion exchange with the clay minerals, and these may be reversed due to the leaching out of the calcium ions.

The reference by Little underscores the need that a sufficient amount of stabilizing agent(s) must be added to provide for the durability of the base layer. This suggests a minimum level of strength required to permanently alter the engineering properties of the layer in question.

The following quote suggests that chemically stabilized materials can also experience a reversal of the stabilization process due to leaching subsequent to significant gains in strength and stiffness (Scullion and Saarenketo, 1997):

> “The suction test has also been used successfully to evaluate the permeability/moisture flow characteristics of several heavily stabilized cement-treated bases, which contain between 5 and 6 percent cement by weight. Two of these bases had failed prematurely. In both cases the failed stabilized bases were shown to be permeable; the surface dielectric of 150-mm-high cores increased substantially after 2 days in the suction test. In both instances the cement-treated bases disintegrated under traffic, and field cores taken from the failed sections showed that the stabilization process was apparently being reversed via leaching. This was attributed to the fact that moisture was able to flow through the layer. In one case during the suction test, calcium carbonate crystals formed on the surface of the sample.”

Given the low levels of compacted density for LFA stabilized soil base layers, it is possible that these layers are sufficiently permeable to allow for the dissolution of chemical (cementitious) bonds in these layers subsequent to adequate curing. This provides an additional potential explanation for the premature failure of pavements that include this type of stabilized base layer within the pavement structure. Chapter 7 offered one recommendation to address this issue, which was to increase the compacted density of this layer. Increasing the density will reduce the permeability, thus slowing down the rate at which the lime could be removed from this layer.

Testing was not conducted in the present study to evaluate an in-situ LFA Proctor UCS based on durability considerations. It is recommended that such laboratory work be conducted on samples of LFA and soils blends that are compacted to a level of density commensurate with that achieved in the field. The test protocol should focus on the degradation of the chemically stabilized material due to the effects of moisture. Possible protocols include AASHTO T 135, Wetting-and-Drying Test of Compacted Soil-Cement Mixtures, which evaluates durability based on cycles of wetting and drying, or on the Tube Suction Test which evaluates moisture sensitivity of base, subbase or subgrade materials (Little and Yusuf, 2001, Scullion and Saarenketo, 1997).

In the interim, it is recommended that a minimum in-situ LFA Proctor UCS of 400 psi be used to account for the durability facet of required strength for the LFA stabilized soil base layer (NCHRP No. 37, 1976, American Coal Ash Association, 1991). These two references focused on the importance of sufficient strength in the chemically stabilized material to resist degradation due to cycles of freezing and thawing. Evaluations based on the aforementioned test protocols that focus on moisture considerations may require revision to the recommended strength for the conditions typically encountered in Mississippi.

Selection of In-situ LFA Proctor UCS Based On Structural Requirements

A major objective of pavement design and construction is to economically obtain a long-lasting adequately performing product with a minimum of maintenance. One potential way to achieve this objective is to construct a Perpetual Pavement. Since the majority of the new pavements constructed in Mississippi are flexible pavements, the following definition is used (Newcomb, 2002):

> "A Perpetual Pavement is defined as an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal in response to distresses confined to the top of the pavement."

The design of a Perpetual Pavement differs fundamentally from the flexible pavement design method currently utilized by MDOT in that the design life, by definition, is 50 years for the Perpetual Pavement; whereas the number of loads anticipated over typically a 10-year design life is entered into the MDOT design procedure. A complete discourse regarding the design and construction of a Perpetual Pavement is beyond the scope of this study; however, one aspect is pertinent to the current discussion. The base course of a Perpetual Pavement must be designed to carry a significantly larger number of loads than required by the current design method.

In this study the concept of a Perpetual Pavement is used in conjunction with the current MDOT design method to obtain an in-situ LFA Proctor UCS. The objective is to extend the design life of the base course to that corresponding to a Perpetual Pavement, with the constraint that the thickness of the overlying HMA is based on MDOT's current typical 10 year flexible pavement design life. To accomplish this objective, pavements were designed using the current MDOT LFA structural layer coefficient of 0.20. These pavements were then evaluated using a mechanistic approach to estimate the strength required for the given thickness of the base layers to carry a 50-year traffic loading.

The base course in a flexible pavement system serves to increase the stiffness of the pavement structure, extend the fatigue life of the overlying HMA layers, and reduce the vertical stresses on the underlying subgrade to a level that minimizes deep pavement foundation rutting. Subsequent to selection of the recommended in-situ LFA Proctor UCS the other layers in the pavement systems were checked to ensure satisfactory performance. Using this approach, subsequent full depth pavement reconstruction should be minimized and rehabilitation/reconstruction efforts predominantly confined to the HMA layers of the pavement structure.

The loading, level of subgrade support, and the thickness and stiffness of each of the layers in the pavement structure affect the magnitude of the calculated flexural stress at the bottom of the LFA base layer. In an effort to reasonably encompass as many of these variables as possible in determining an in-situ LFA Proctor UCS, more than one pavement design was considered in the evaluation. A revised pavement design for the test section of each of the five newer projects was utilized that incorporated a design CBR for the subgrade underlying the length of the given test section, instead of the design CBR used for the entire length of the corresponding project. Each of these five revised pavement designs included a 6-inch LFA stabilized soil base course with the thickness of the overlying HMA determined using the current MDOT flexible pavement design procedure. Table 29 includes the required 10-year traffic data, revised design CBR, applicable structural layer coefficients and resulting required thickness of HMA for each of the five newer projects included in this study. While not included in the pavement design as per MDOT design protocol, and not included in the five newer projects evaluated in the study, a 6-inch chemically stabilized subgrade layer was included in the mechanistic analyses of these pavements.

The WESLEA program was used for evaluating the revised pavement designs when the assumption of full bonding between the HMA and LFA layers was used in the analyses. The Bisar program was used when slip was assumed between these two layers. Two base course thicknesses and a range of base course stiffness were evaluated to determine an appropriate combination to achieve a “perpetual” base course to support a pavement designed under the current MDOT design procedure.

Table 30 includes a summary of the input data for the two programs. The lower limit of 106,300 psi for the LFA base layer backcalculated modulus is approximately the15th percentile of the collective values for the five newer projects. From Equation 3 this modulus value corresponds to a LFA Proctor UCS of 171.6 psi, which is noted as the lower limit in Table 30 under the heading “LFA Proctor UCS”. A LFA Proctor UCS of 700 psi corresponds to the upper limiting modulus value of 546,600 psi used in the calculations. It is not anticipated that a LFA Proctor UCS greater than 700 psi would ever be achieved with an acceptable level of reliability for typical MDOT pavement construction practices and the type of granular materials stabilized in Mississippi. Where the thickness of the base course is increased to 8 inches, the thickness of the overlying HMA is reduced by 1 inch to maintain an approximate “equivalent” pavement structure as indicated at the bottom of Table 30.

Figures 48 through 52 and Figures 54 through 58 provide the basis for selecting an in-situ LFA Proctor UCS based on structural considerations. Each of these figures corresponds to one of the five newer projects. The left end point of each curve in each of these figures corresponds to the LFA Proctor UCS of the 15th percentile LFA modulus as shown in Table 30. The right end point corresponds to an LFA Proctor UCS of 700 psi.

These ten figures include two horizontal lines labeled “10-Year Traffic” and “50-Year Traffic” respectively. For each of the five newer projects the traffic loading was estimated for both of these periods of time as number of 18-kip ESALs. These 18-kip values were then converted to equivalent numbers of 34-kip tandem-axle loads as shown in Table 31 using the indicated Equivalency Factors (Guide for Design of Pavement Structures, 1993). These equivalent 34-kip tandem-axle loads were then entered into Equation 8 to obtain an estimation of corresponding stress/strength ratios. The horizontal lines shown in the ten figures are plotted in accordance with these resulting stress/strength ratios.

The purpose of including the two horizontal lines in each of the ten figures is to enable a comparison of the stress/strength ratio of the LFA base layer, corresponding to various pavement layer thicknesses and modulus values, to the stress/strength ratios corresponding to the design traffic loadings. By varying the LFA modulus, and thus the corresponding LFA Proctor UCS via Equation 3, an estimation of the required in-situ LFA Proctor UCS value can be selected via mechanistic analyses that will provide sufficient strength to the LFA base layer to carry the anticipated traffic loading.

The curves in Figures 48 through 52 were developed assuming a 34-kip tandem-axle load and full bond between each of the pavement layers. The upper two curves in these figures illustrate the need for an underlying stabilized subgrade layer. Significantly higher levels of strength are required in LFA base layers which do not have the benefit of an underlying stabilized subgrade layer to carry the traffic loading for the design life of the pavement. For example, the 6-inch LFA base layer with no LTS requires a strength of 550 psi to carry the traffic for a 10-year design life in Clarke County, and 700 psi for Smith County. With the inclusion of a 6-inch LTS layer however, the figures indicate that much lower levels of strength would be required to carry the loads for this period of time and that the controlling factor for required strength would be based on the durability of the stabilized material instead of the structural requirement. This observation supports the quote by the Portland Cement Association that strength is a secondary issue relative to durability.

The 6-inch LFA base layer with no LTS requires a strength of in excess of 700 psi for all five projects to carry the traffic for a 50-year design life. With the inclusion of a 6-inch LTS layer the figures indicate that the required strengths would vary from 350 to 550 psi to carry the loads for this time period. This would necessitate the selection of a minimum 550 psi in-situ LFA Proctor UCS for routine LFA base layer construction QC/QA.

Discussion to Increase LFA Base Layer Design Thickness

Increasing the design thickness of the base course was suggested by the Blain Companies (Letter dated May 28,2002), and was also recommended in a technical memorandum by Little (Little, 2002). For the construction loading condition it was concluded that increasing the base layer thickness to 8 inches would not provide a sufficient reduction in the stress/strength ratio to warrant the added layer thickness. However, the in-service condition presents a completely different situation because the base layer is located deeper within the pavement structure, under the overlying HMA layer. This results in a reduction in the developed flexural stresses at the bottom of the base layer relative to when this layer is located at the surface. In addition, the LFA stabilized material has had time to cure and increase in stiffness, which also results in a reduction in the stress/strength ratio as evidenced by Figures 48 through 52.

The figures indicate that the use of an 8-inch base layer with no underlying LTS would require strengths from 550 to 650 psi to carry the loads for the 50-year traffic loading. With the inclusion of a 6-inch LTS layer the figures indicate that very low levels of strength would be required to carry the loads for this period of time and that the controlling factor for required strength would be based on the durability of the stabilized material instead of the structural requirement.

Consideration of Deficient Base Layer Thickness, Overload, and HMA – LFA Interface Bonding Condition

Based on the discussion of Figures 48 through 52 it appears that the use of 8 inches of base results in a overly conservative design for most cases encountered for the in-service loading condition; however, the stress/strength ratios in these figures have been calculated using the assumptions that (1) the design thickness of the layers are actually constructed in the field, (2) the pavement will not be subjected to overloading, and (3) there is 100 percent bonding between the pavement layers.

Tables 5 and 6 in Chapter 3 provide ample evidence that the in-situ layer thickness will vary from the design thickness, with some locations deficient by as much as 2 inches from the design thickness. Information from the MDOT Law Enforcement Division indicates that overloads of up to 8 kips have been recorded on tandem axles (Huff, 2003).

Figure 53 illustrates the increase in the stress/strength ratio due to the compounding affects of both deficient layer thickness and overload for the Bolivar County project. This figure illustrates these affects for up to a 2-inch deficient layer thickness and up to a 6-

kip overload. The curves in this figure were developed assuming a LFA modulus value of 275,700 psi, which corresponds to an in-situ LFA Proctor UCS of 400 psi, and full bonding between the HMA and LFA layers. The bottom curve illustrates the increase in the stress/strength ratio in the LFA base course as the deficiency in layer thickness is increased from 0 to 2 inches. Note that the ratio is increased by 23 percent given a 2-inch deficient thickness with no overload. The middle and upper curves represent the stress/strength ratios for varying deficient layer thickness and a 3- and 6-kip overload respectively. Note that given a 2-inch deficient base layer thickness, the stress/strength ratio for this pavement is increased by 44 percent when subjected to a 6-kip overload.

In terms of the effects on traffic loading, application of overloads result in a decrease in the number of design loads that can be applied to the pavement before effecting fatigue cracking in the LFA base layer. While not considered in this study, given sufficient data on the number and corresponding amount of overloads, the concept of fatigue consumption could be investigated for use in obtaining an estimate of this reduction in number of design loads.

Deficient LFA Base Course Layer Thickness

Comparing stress/strength ratios of a pavement with a deficiency in layer thickness to the stress/strength ratios corresponding to either the 10-year or 50-year design traffic loading can be more readily accomplished by assuming a uniform value for loading in the analyses. Figures 54 through 58 were developed assuming a 34-kip tandem-axle load and included a modeled 6-inch LTS layer in each of the pavement structures. These figures illustrate the increase in the stress/strength ratios in the LFA base course for each of the five newer projects given a modeled 1-inch deficient in-situ LFA base layer thickness relative to the 8-inch design thickness. The curves labeled “1” Def., Full Bond” represent the pavement with a 1-inch deficient base layer thickness, or an in-situ thickness of 7 inches instead of 8 inches, but with the same thickness of HMA as the corresponding curves labeled “8” Base, Full Bond.”

Assuming that all of the loads were at the legal limit and full bonding between the HMA and LFA layers, Figures 54 through 58 indicate that, even with a deficient 1-inch base layer thickness, relatively low strengths could be used in the base course to satisfy the structural requirements for the 10-year design period. As previously discussed, however, overloading would render a less conservative assessment for this design period. For the 50-year design period strengths from 350 to 500 psi would be required based on structural considerations.

Condition of Bonding Between HMA and LFA Base Layers

Figure 59 provides evidence of good bonding between the HMA and LFA base layers at this particular location since the two materials did not separate during either the coring or core extraction process. Good bonding between these layers is essential for the long term performance of a pavement structure.

Chapter 9 included a discussion regarding the potential formation of a dry crust, or layer, on top of a newly constructed LFA base layer due to the lack of proper curing. It was noted that such a dried layer prevents the formation of a good bond between the LFA base layer and the overlying HMA layer, which may cause shoving to occur within the pavement. The lack of a good bond also results in a dramatic increase in the level of developed flexural stresses at the bottom of both of these layers. Such an increase leads to reduced pavement life.

Figures 54 through 58 each include a curve entitled “8” Base, With Slip.” These curves represent the same pavement structure as the curves entitled “8” Base, Full Bond,” except that the pavement was modeled with slip between the HMA and LFA layers using a value of 1000 in the Bisar program. All five of these curves show an increase in the stress/strength ratio with increasing strength of the base course up to about 450 psi, and then a gradual decrease in the ratios with a continuing increase in strength. A cursory review of these figures would indicate that if relatively low strengths were maintained in the LFA base course, slippage between these layers would not present any problem; however, a more detailed examination leads to a contrary conclusion.

The pavement structure acts as a unit. Changes in the thickness and strength of any of the layers, or in the bonding condition between layers, effect changes in the developed stresses in all of the layers included in the pavement system. The occurrence of slip between the HMA and LFA layers causes a significant change in the flexural stresses developed at the bottom of the overlying HMA layer.

Figure 60 provides an illustration of this variation in stress for the Bolivar County project. The stresses were calculated for a point at the bottom of the HMA layer midway between two tires on a given tandem-axle assuming a 34-kip tandem-axle load. For both the 100 percent bonded and the slip conditions the flexural stresses in the HMA layer decrease with increasing base course strength. This indicates that as the LFA base course increases in stiffness, an increasing amount of the load on the pavement is being carried by the base layer and less by the HMA layer. While both curves parallel each other, the upper curve illustrates that much higher flexural stresses are developed in the HMA layer with slip, as opposed to no slip, between the two layers. Note that for the 100-percent bonded condition, the bottom of the HMA layer actually begins to act in compression instead of tension at a LFA base strength of about 275 psi.

Check of Allowable Number of Loads for each Layer in the Pavement Structure

Based on Figures 48 through 52 and 54 through 58, and assuming full bond between the HMA and LFA layers, it is tentatively concluded that an in-situ LFA Proctor UCS of 400 psi will provide adequate strength to the LFA base course for both the durability and structural requirements for the majority of the pavements constructed in Mississippi. The use of an 8-inch instead of a 6-inch LFA base thickness provides allowance for some deficiency in in-situ layer thickness and/or overload, but not for slippage between the HMA and LFA layers. These tentative conclusions, however, are based on the developed flexural stresses at the bottom of the LFA base layer. All of the layers within the pavement must be checked to ensure their performance over the design life of the pavement.

This check is accomplished by the use of transfer functions, which are used to estimate the number of 34-kip loads to effect flexural fatigue at the bottom of the HMA, LFA base and LTS layers using requisite mechanistic analyses output from each of the five revised pavement designs. These values are expressed as a percent of the 50-year design loading and constitute the basis of the comparison to this design period.

The transfer functions used in this study were not developed using materials derived from MDOT construction projects; therefore, they are limited to providing estimates for the allowable numbers of loads. MDOT is currently funding State Study No. 170 – Implement the 2002 Design Guide for MDOT (Phase II), which includes the calibration of performance equations using test data from materials typically used in MDOT construction projects. It is recommended that the five revised pavement designs be evaluated with the calibrated performance equations once they become available to ensure the basis of the recommendations included in the current study.

Equation 8 is used to check the LFA base layer since this transfer function was developed for LFA stabilized soil materials. Table 32 includes the comparison data for this layer for the various conditions shown in the table. In most cases an 8-inch LFA base design allows for some deficiency in in-situ base layer thickness assuming full bond between the layers and the pavement is not overloaded on a consistent basis. Note, however, that slippage between the HMA and LFA layers causes a significant reduction in the allowable 34-kip loads that can be applied to the pavement structure. The table indicates that an 8-inch LFA base layer in four out of the five pavements will not carry the traffic for even the 10-year design period. This underscores the need to properly cure the LFA layers in the field to minimize the occurrence of slip.

The WESLEA program includes a HMA fatigue equation that was utilized to check the loading capacity for the HMA layer. This particular fatigue equation is a modified version of an equation developed at the University of Illinois using Mn/Road fatigue crack data. This equation uses tensile strain instead of stress at the bottom of the HMA layer as shown below (Timm, Birgisson and NewComb, 1999):

$$N_f = 2.83*(10\text{^}-6)*((10\text{^}6/\varepsilon_t)\text{^}3.148) \quad \text{Equation 9}$$

Where: N_f = Number of repeated loads under current structural conditions before a fatigue crack will form

ε_t = Maximum horizontal tensile strain at bottom of first layer caused by one pass of current wheel configuration, expressed in microstrain

The following transfer function is used for the LTS layers (Little and Yusuf, 2001):

$$S = 0.923 - (0.058 * \log N) \quad \text{Equation 10}$$

Where: S = Stress/Strength Ratio

N = Number of load applications to failure

Table 33 provides a summary of the performance of the five revised pavement designs for both 6- and 8-inch LFA base layer thicknesses. The numbers are very high for both the HMA and LTS layers and should not be interpreted as meaning that these layers will actually provide for that many load repetitions. Flexural fatigue is the only failure mechanism checked for performance in both Tables 31 and 32, and the numbers in these tables do not account for environmental affects. The important point is that the transfer functions used for the five pavements indicate that the LFA base layer in each of them is the controlling layer regarding fatigue performance of these pavements.

Conclusions for In-Service Loading Condition

It is concluded that an 8-inch LFA base layer on a 6-inch chemically stabilized subgrade layer provides for an adequate, but not overly conservative design. It is recommended that MDOT increase the required thickness of a LFA stabilized soil base course to 8 inches for routine pavement design. A value of 400 psi for the in-situ LFA Proctor UCS appears to be an acceptable requirement for the 8-inch base course based on both durability and structural considerations for most cases encountered in Mississippi.

Compatibility of Current MDOT LFA Design Structural Layer Coefficient and Required In-Situ Proctor UCS

The in-situ LFA structural layer coefficient and corresponding in-situ LFA Proctor UCS values were plotted from the available data of the five newer projects, but due to the wide scatter in the points, no meaningful relationship was obtained between these two parameters. Therefore, no correlation can be used from the data obtained in the current study to evaluate the compatibility of the current MDOT design LFA structural layer coefficient of 0.20 and the recommended in-situ LFA Proctor UCS of 400 psi. An idea of the reasonableness of the correspondence between these two values can be ascertained from Table 34. This table is a reproduction of the data included in Table 6-1 for soil cement stabilized soils (George, 2002). The information in Table 34 indicates that there is general agreement in the values proposed in this study with the values shown for several of the states included in that table.

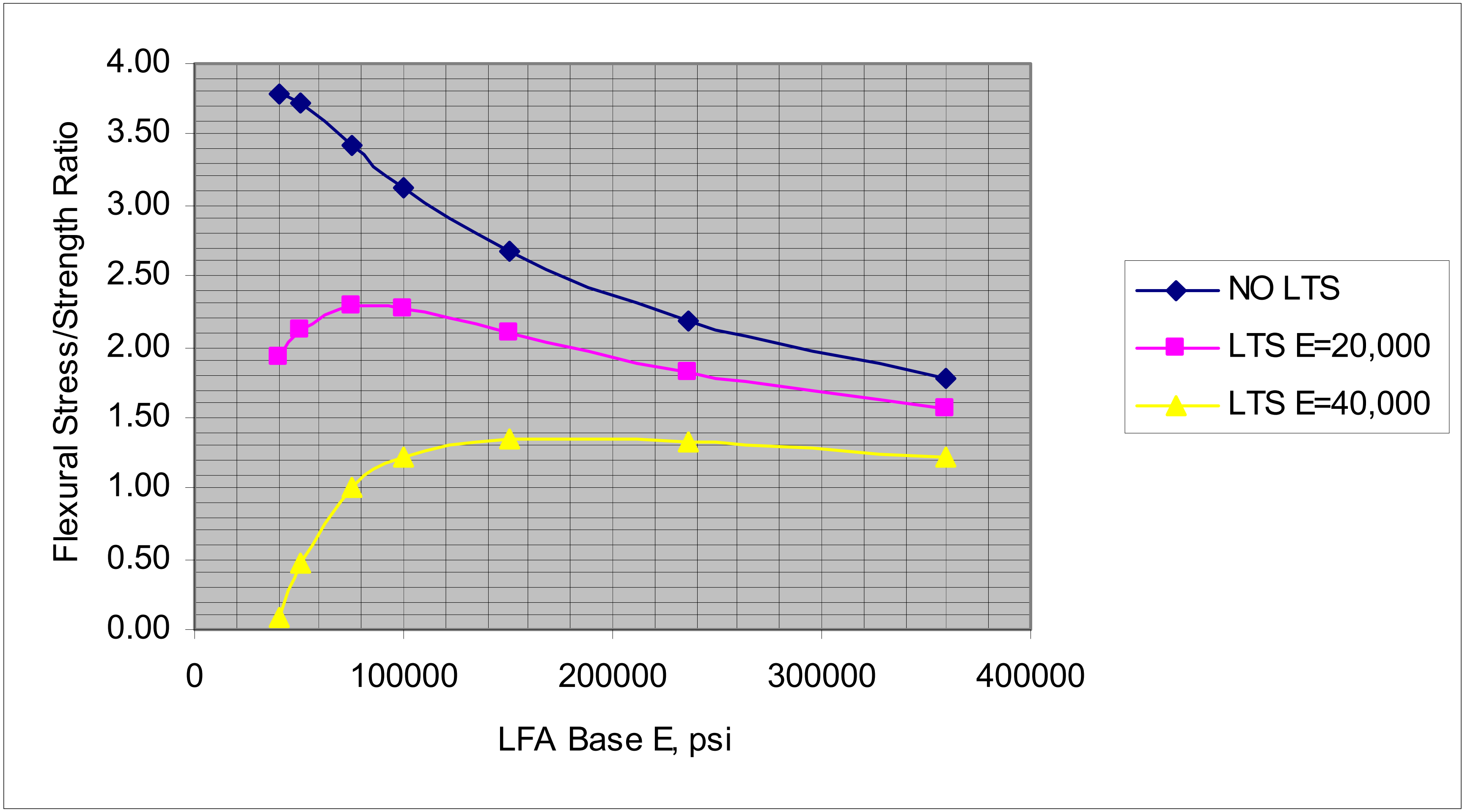


Figure 46. Flexural Stress/Strength Ratio at Bottom of LFA Base vs. LFA Base E During Construction Loading - 6" LFA Base, 6" LTS

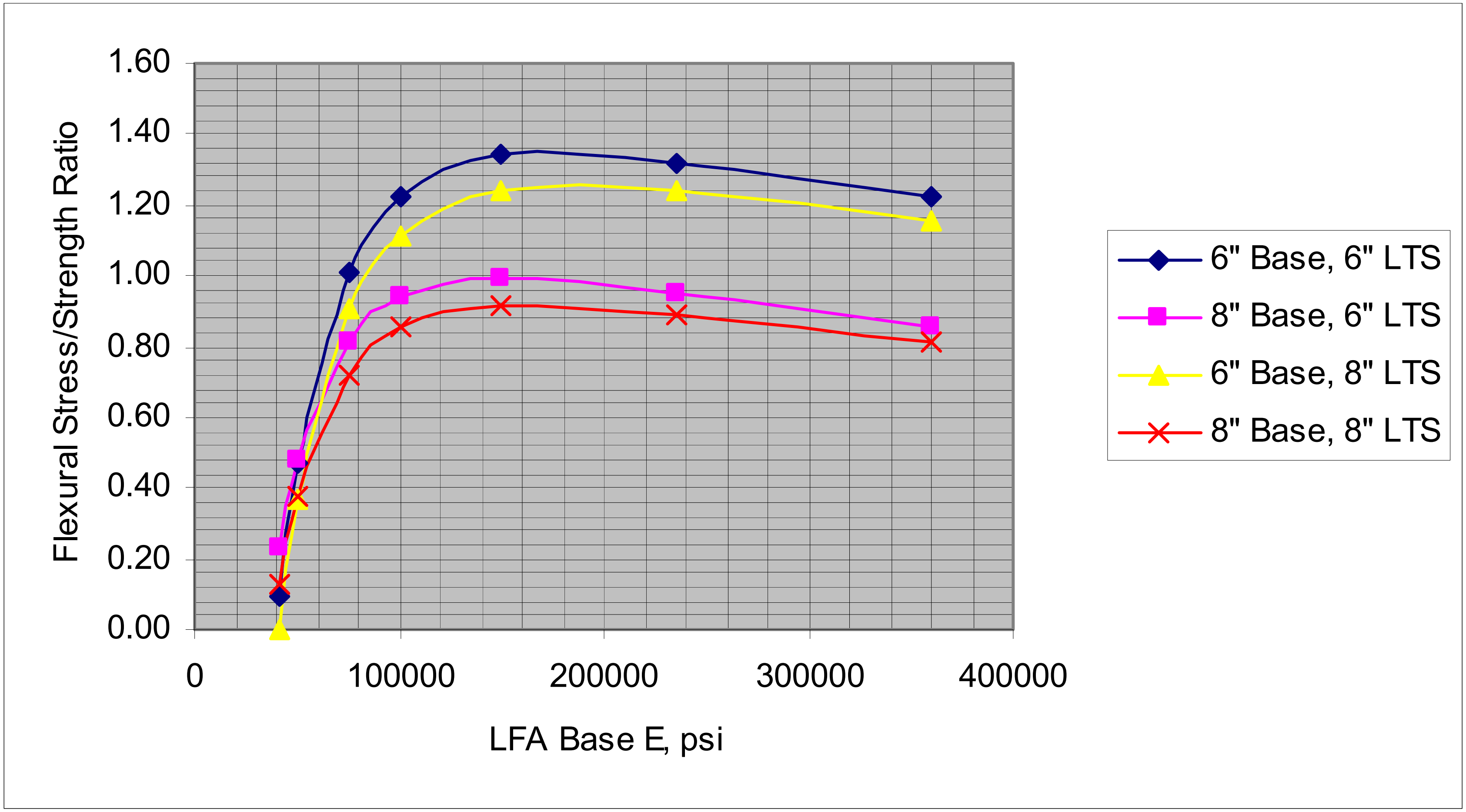


Figure 47. Flexural Stress/Strength Ratio at Bottom of LFA Base Vs. LFA Base E During Construction Loading - Variable Base and LTS Thickness, LTS E = 40,000 psi

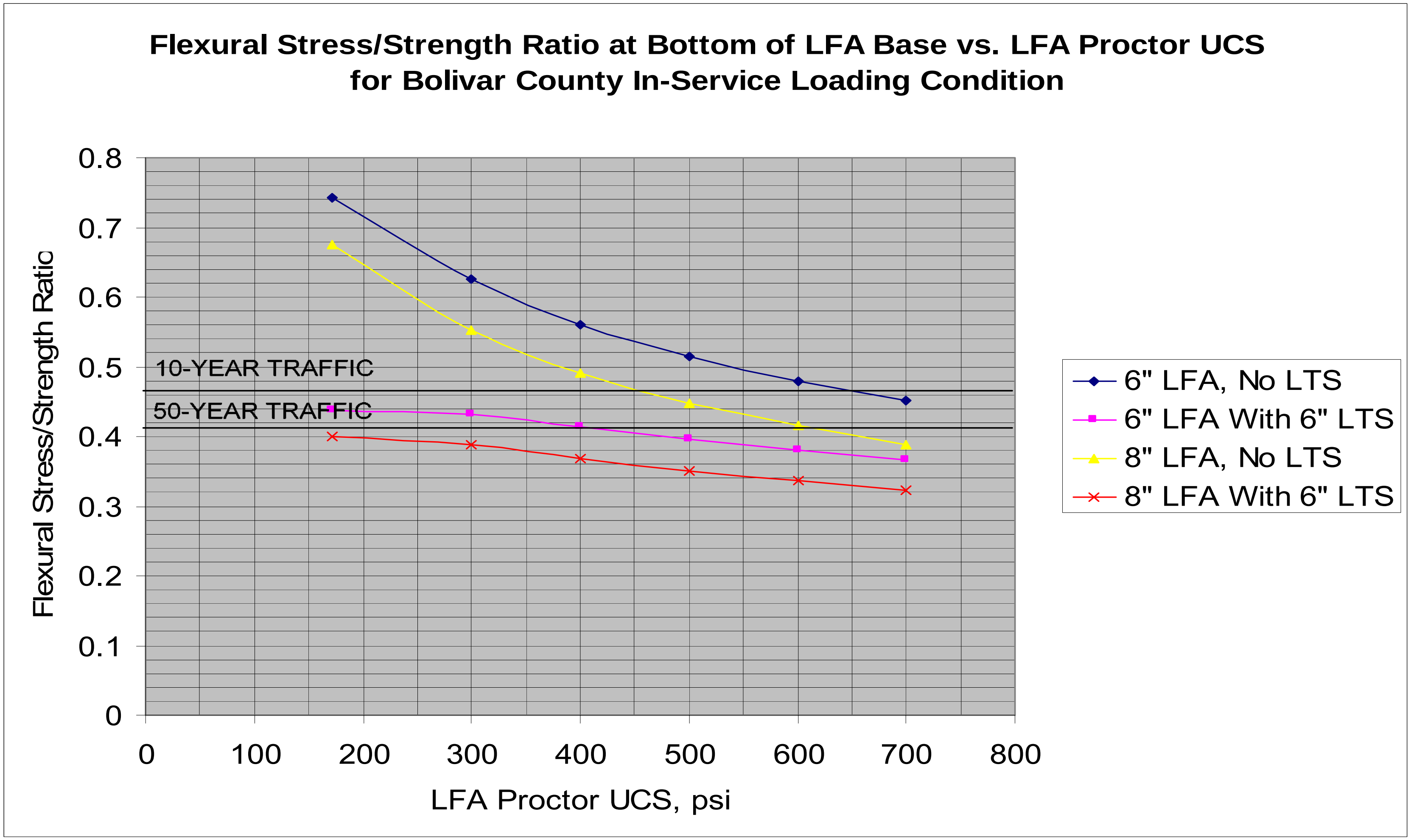


Figure 48. Flexural Stress/Strength Ratio at Bottom of LFA Base vs. LFA Proctor UCS for Bolivar County In-Service Loading Condition

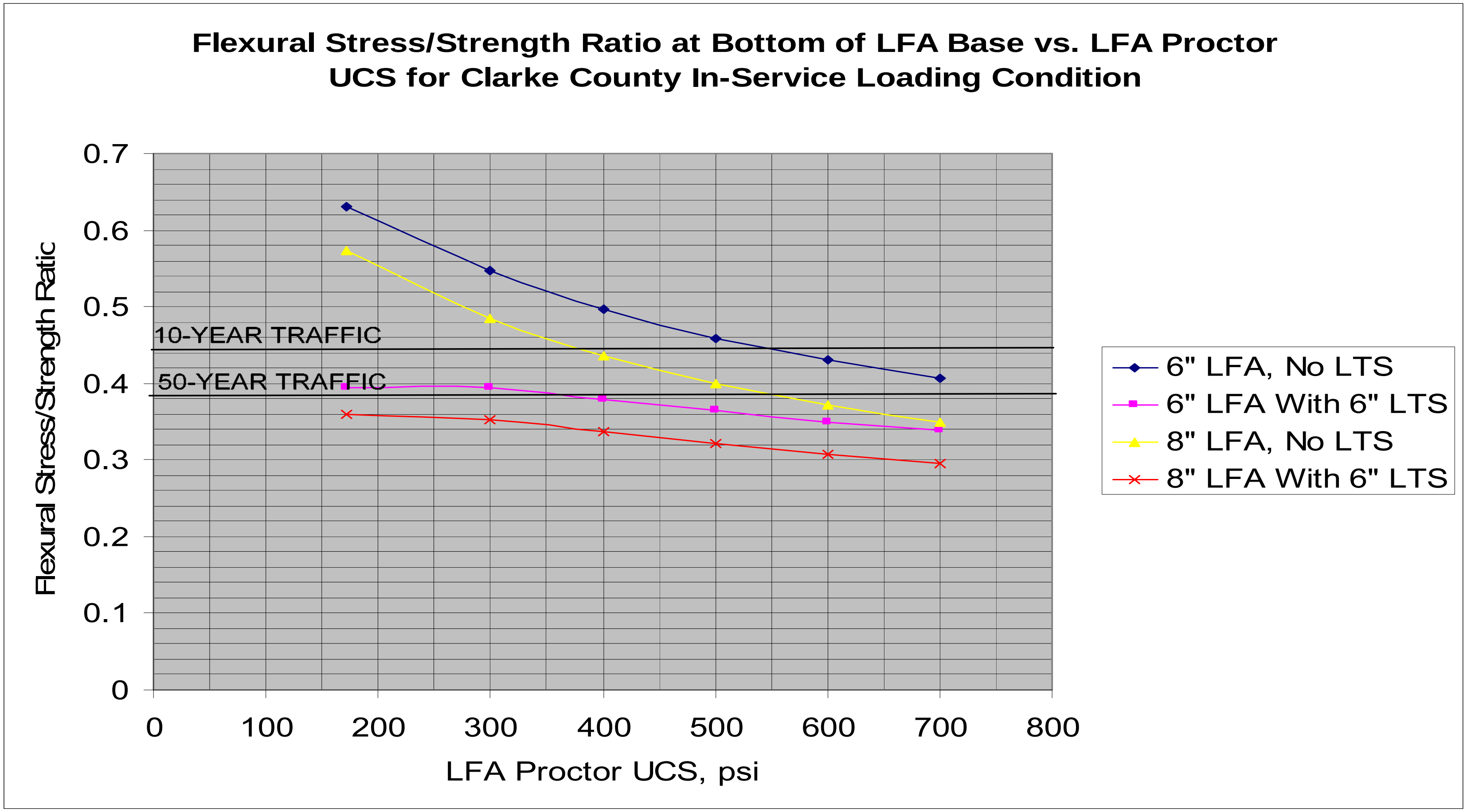


Figure 49. Flexural Stress/Strength Ratio at Bottom of LFA Base vs. LFA Proctor UCS for Clarke County In-Service Loading Condition

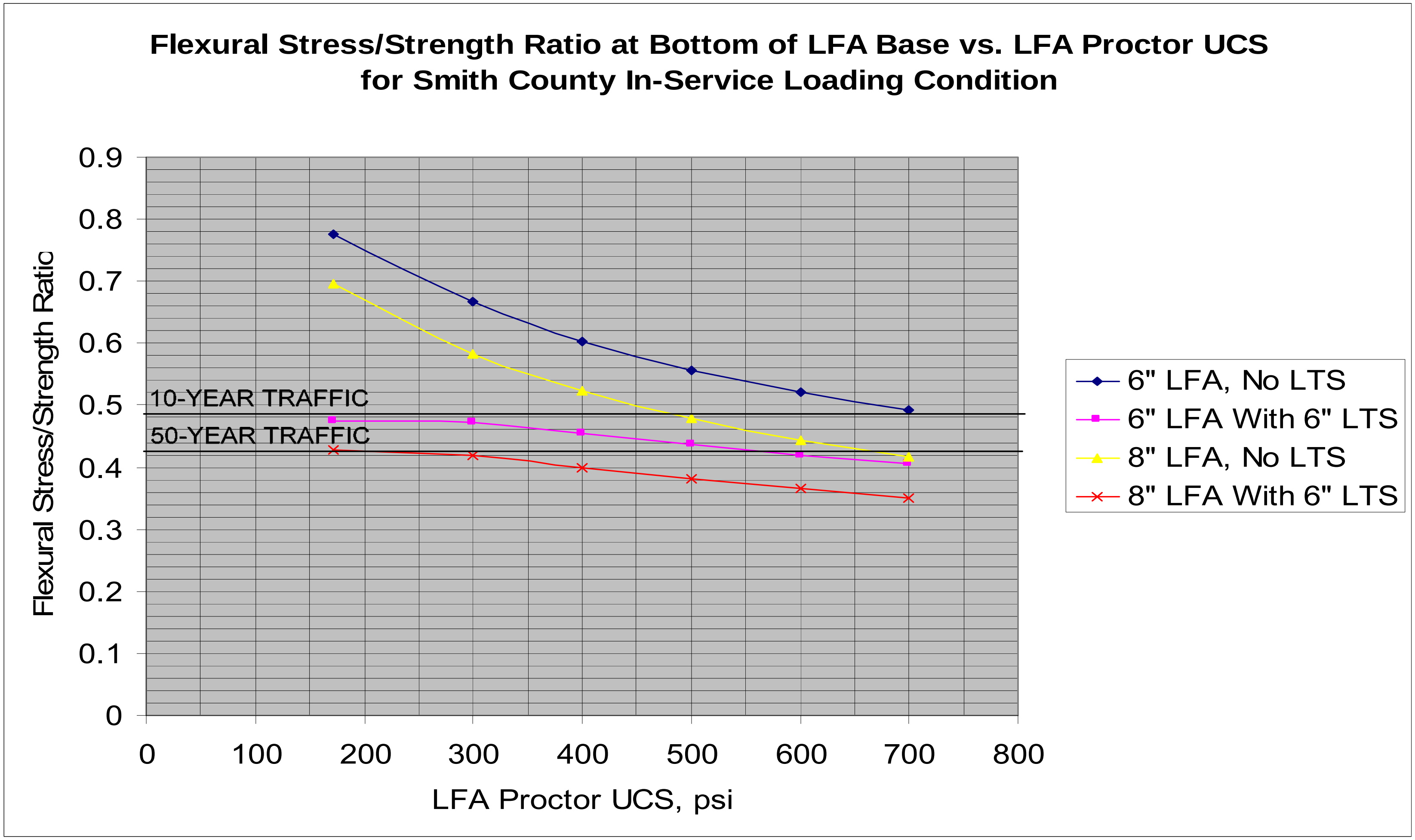


Figure 50. Flexural Stress/Strength Ratio at Bottom of LFA Base vs. LFA Proctor UCS for Smith County In-Service Loading Condition

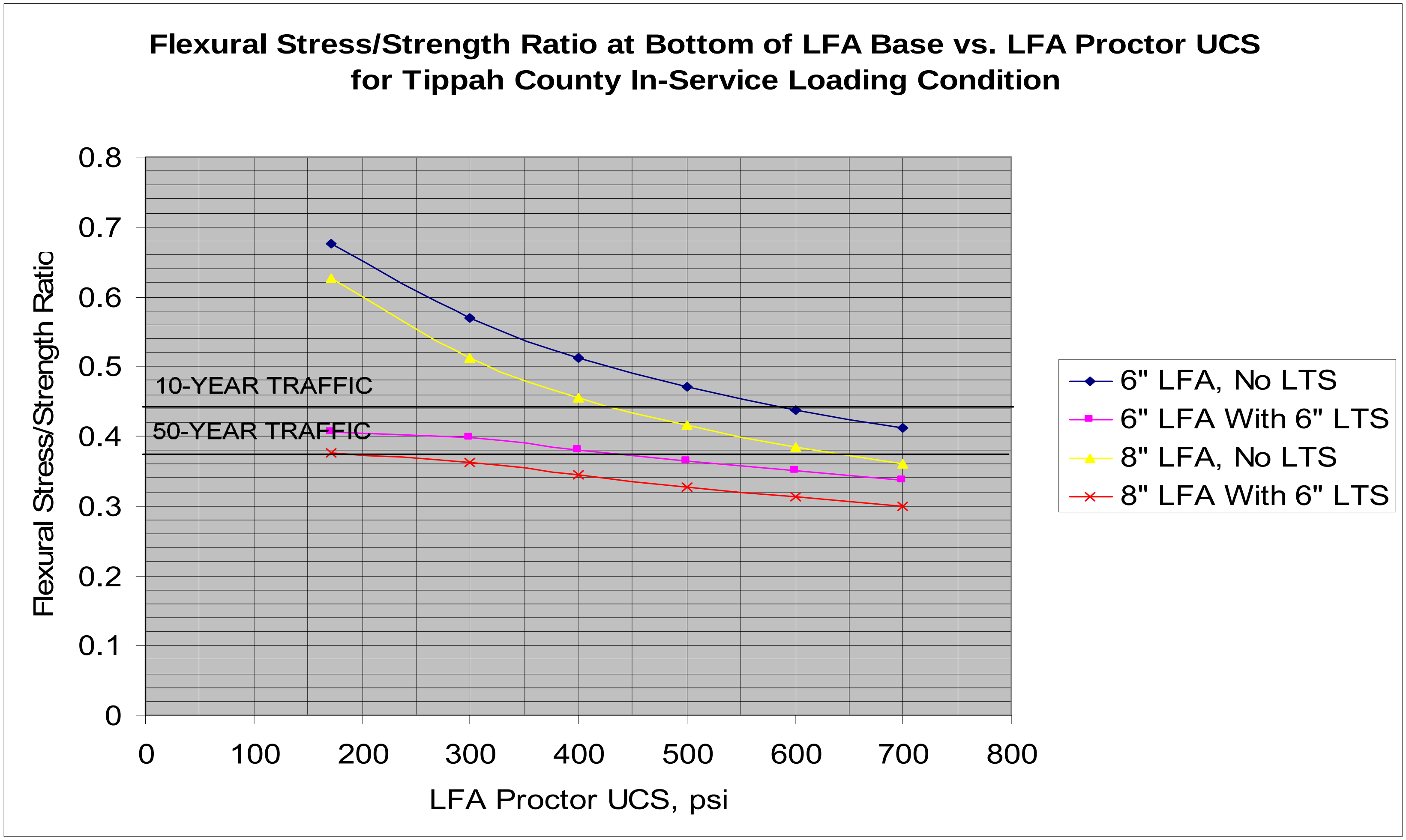


Figure 51. Flexural Stress/Strength Ratio at Bottom of LFA Base vs. LFA Proctor UCS for Tippah County In-Service Loading Condition

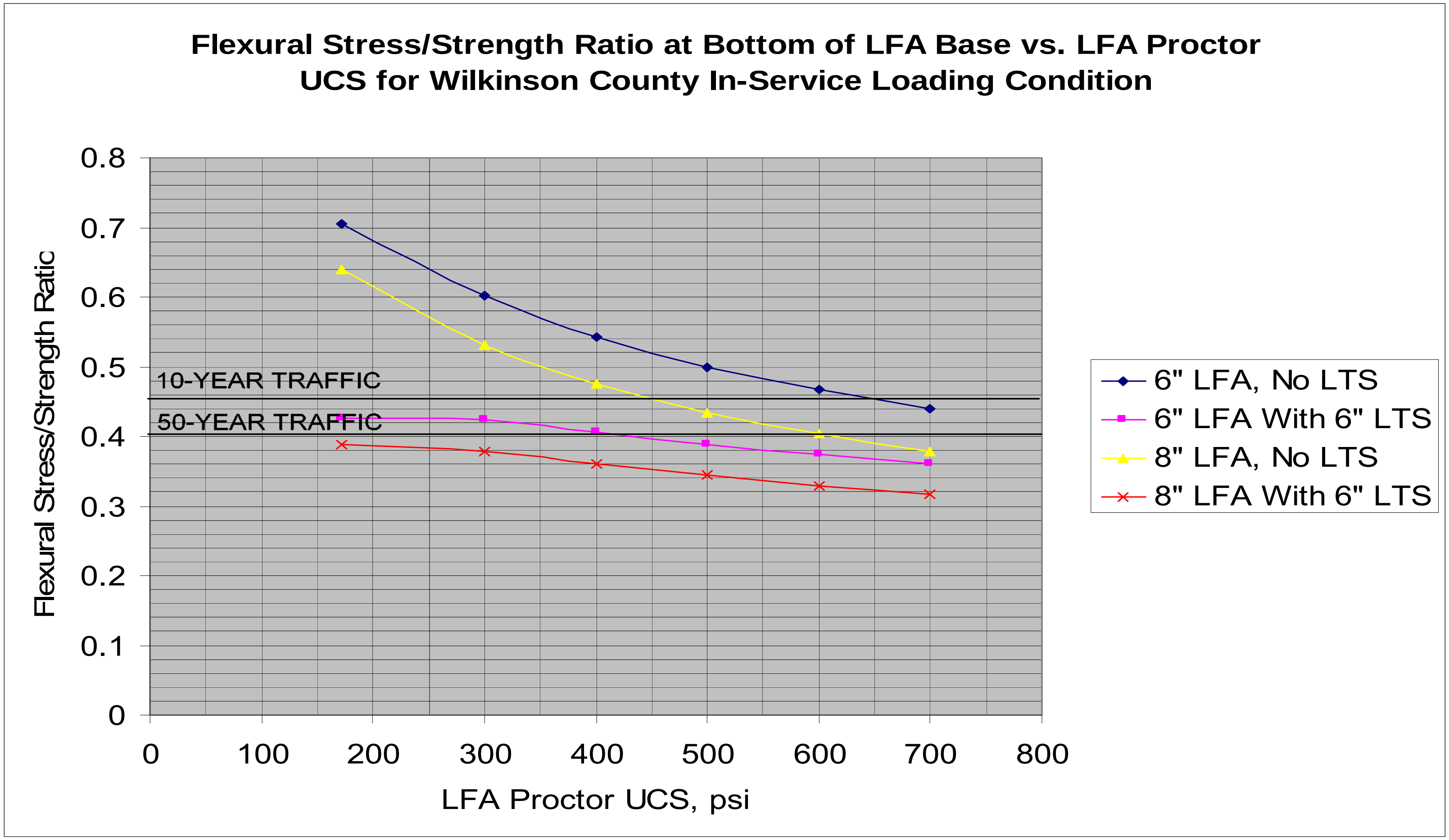


Figure 52. Flexural Stress/Strength Ratio at Bottom of LFA Base vs.LFA Proctor UCS for Wilkinson County In-Service Loading Condition

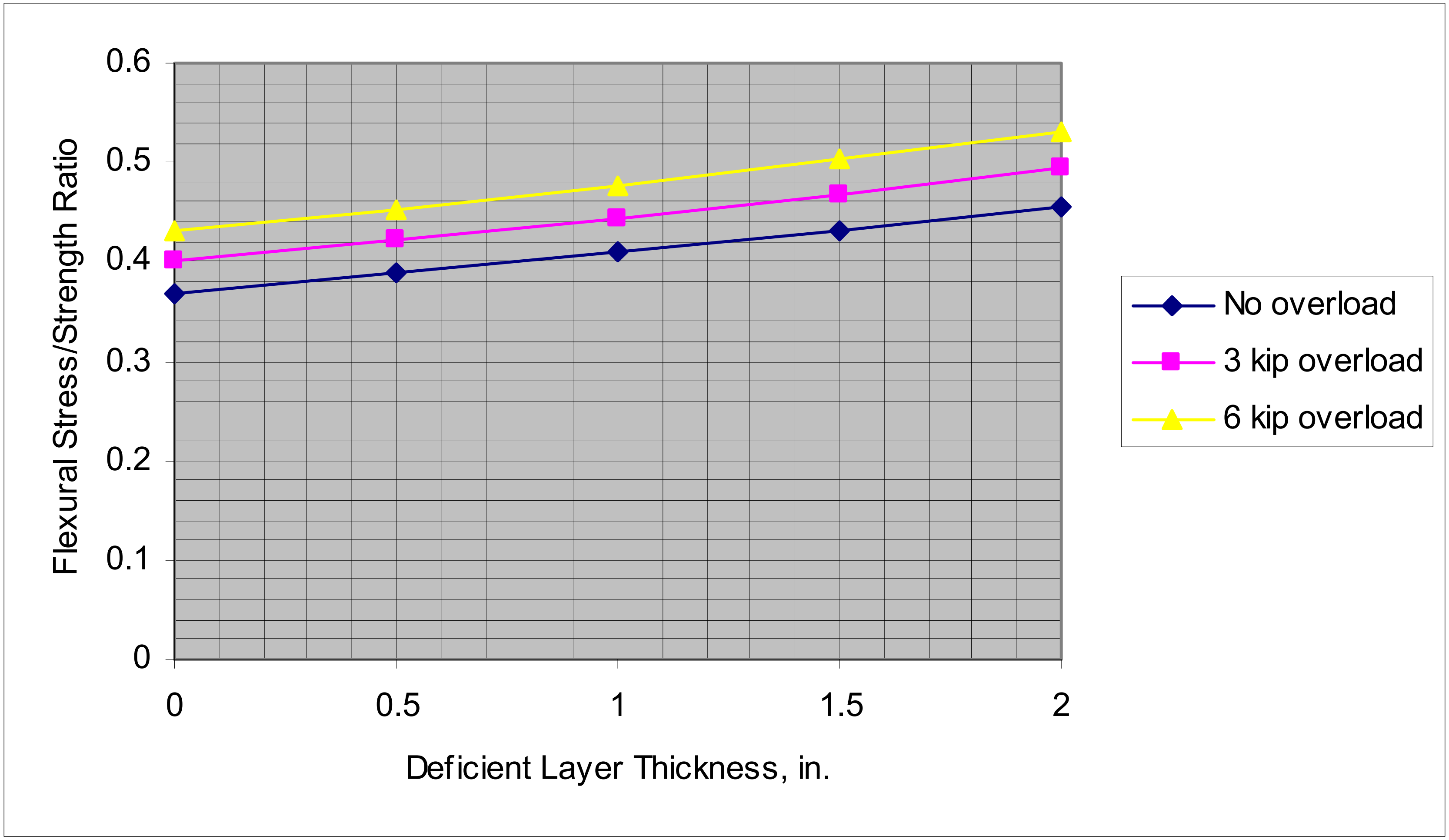


Figure 53. Flexural Stress/Strength Ratio vs. Deficient Layer Thickness for Varying Levels of Overload in Bolivar County Project In-Service Loading Condition

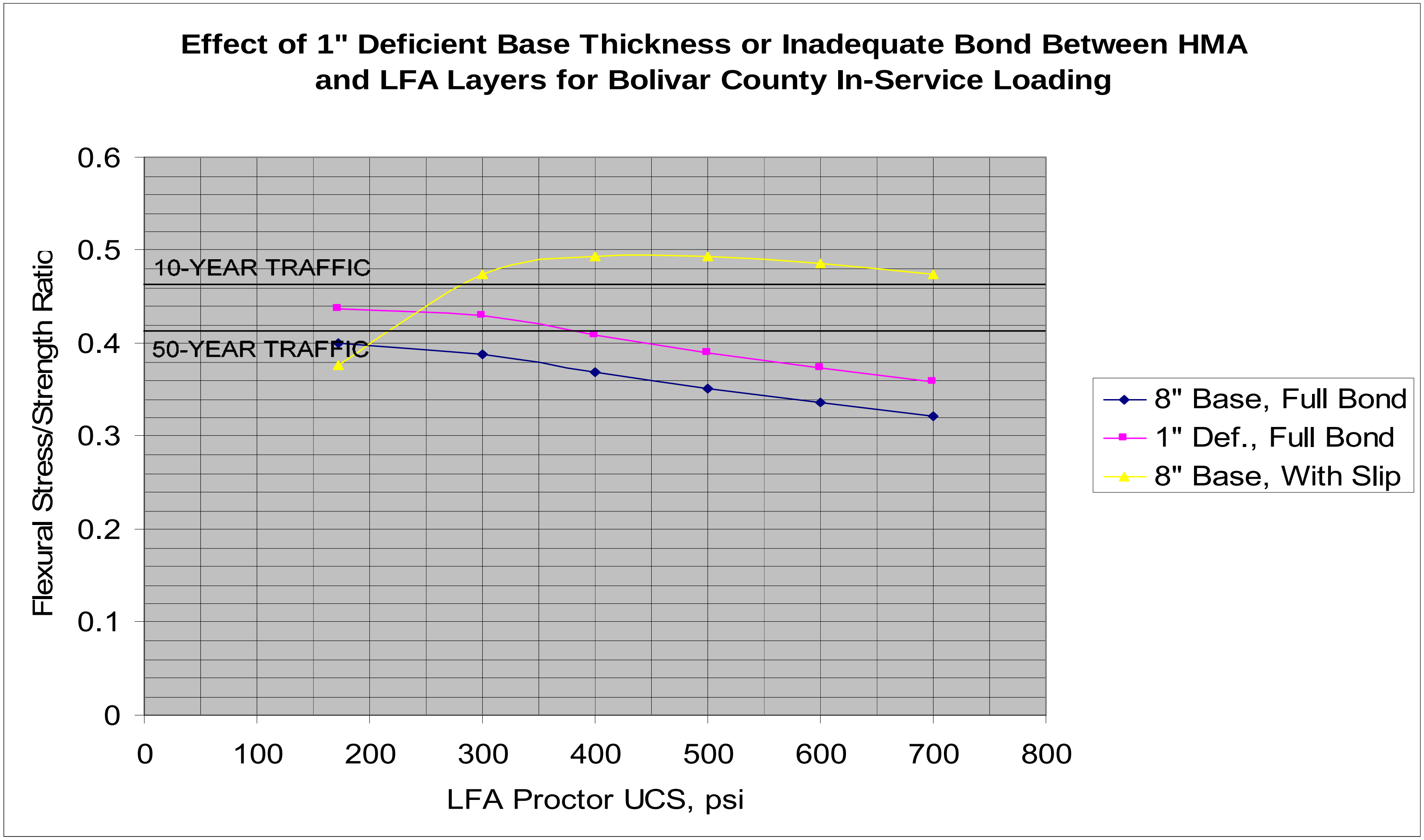


Figure 54. Effect of 1" Deficient Base Thickness or Inadequate Bond Between HMA and LFA Layers for Bolivar County In-Service Loading

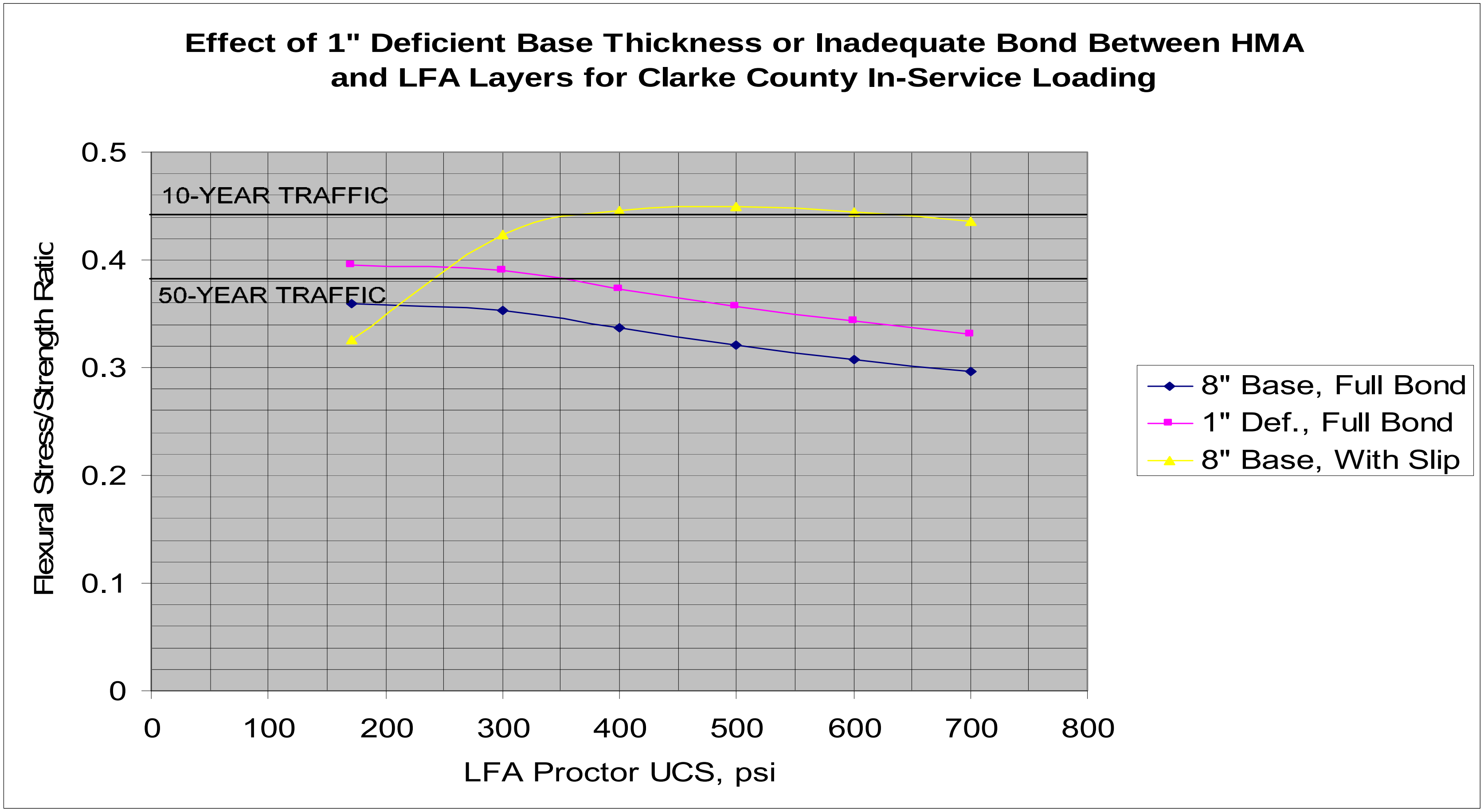


Figure 55. Effect of 1" Deficient Base Thickness or Inadequate Bond Between HMA and LFA Layers for Clarke County In-Service Loading

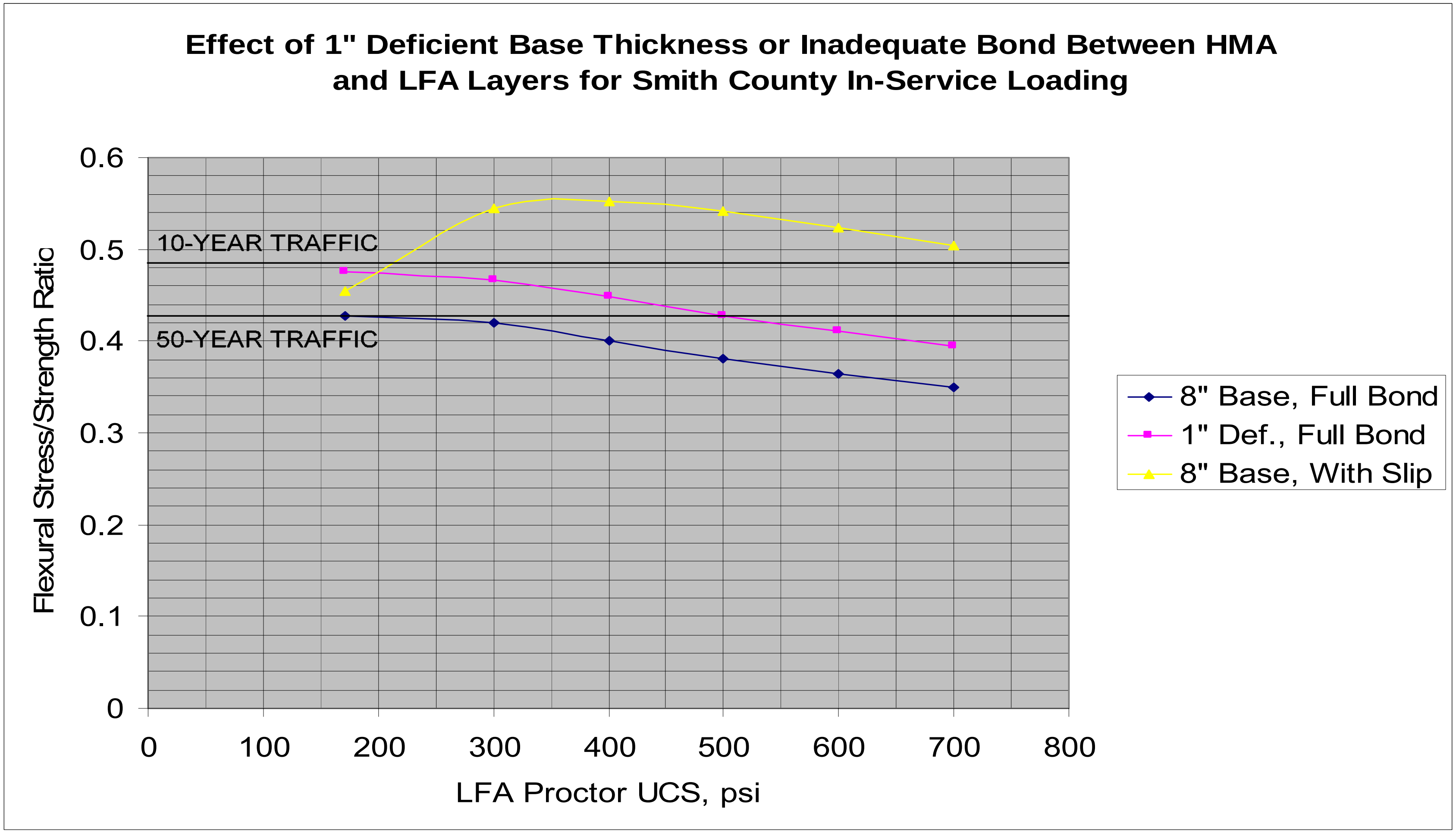


Figure 56. Effect of 1" Deficient Base Thickness or Inadequate Bond Between HMA and LFA Layers for Smith County In-Service Loading

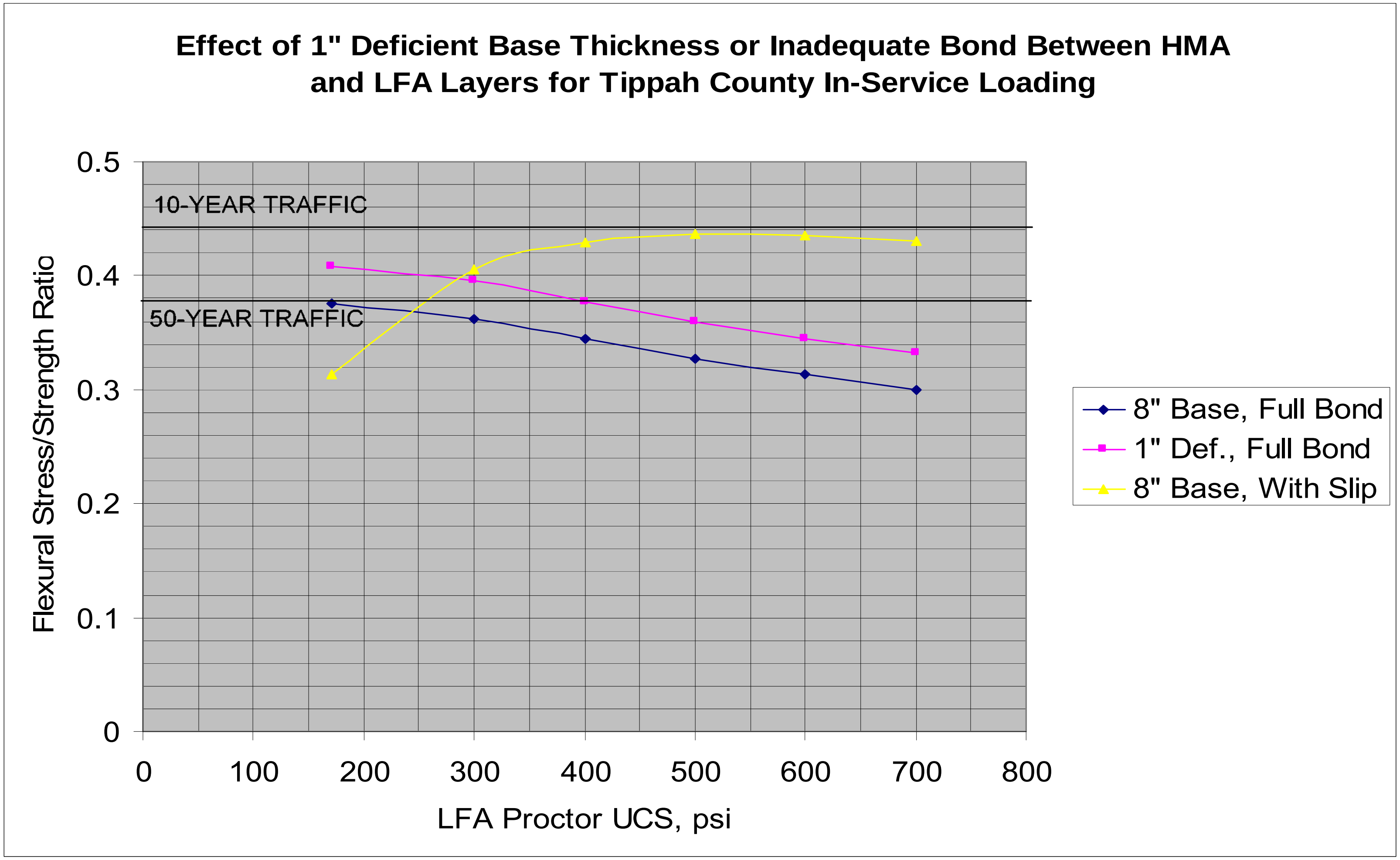


Figure 57. Effect of 1" Deficient Base Thickness or Inadequate Bond Between HMA and LFA Layers for Tippah County In-Service Loading

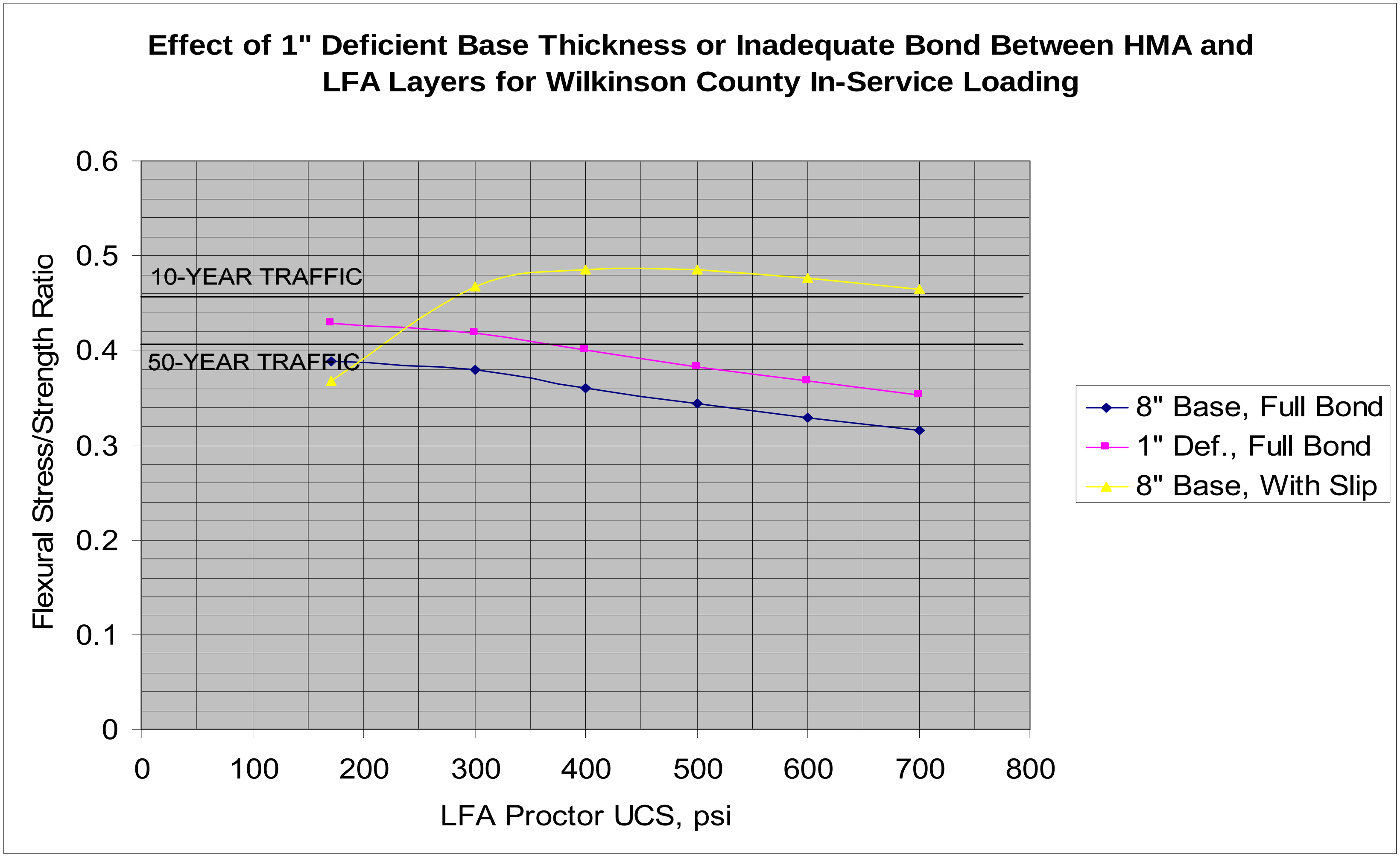


Figure 58. Effect of 1" Deficient Base Thickness or Inadequate Bond Between HMA and LFA Layers for Wilkinson County In-Service Loading

Figure 59. Example of Good Bonding Between HMA and LFA Layers

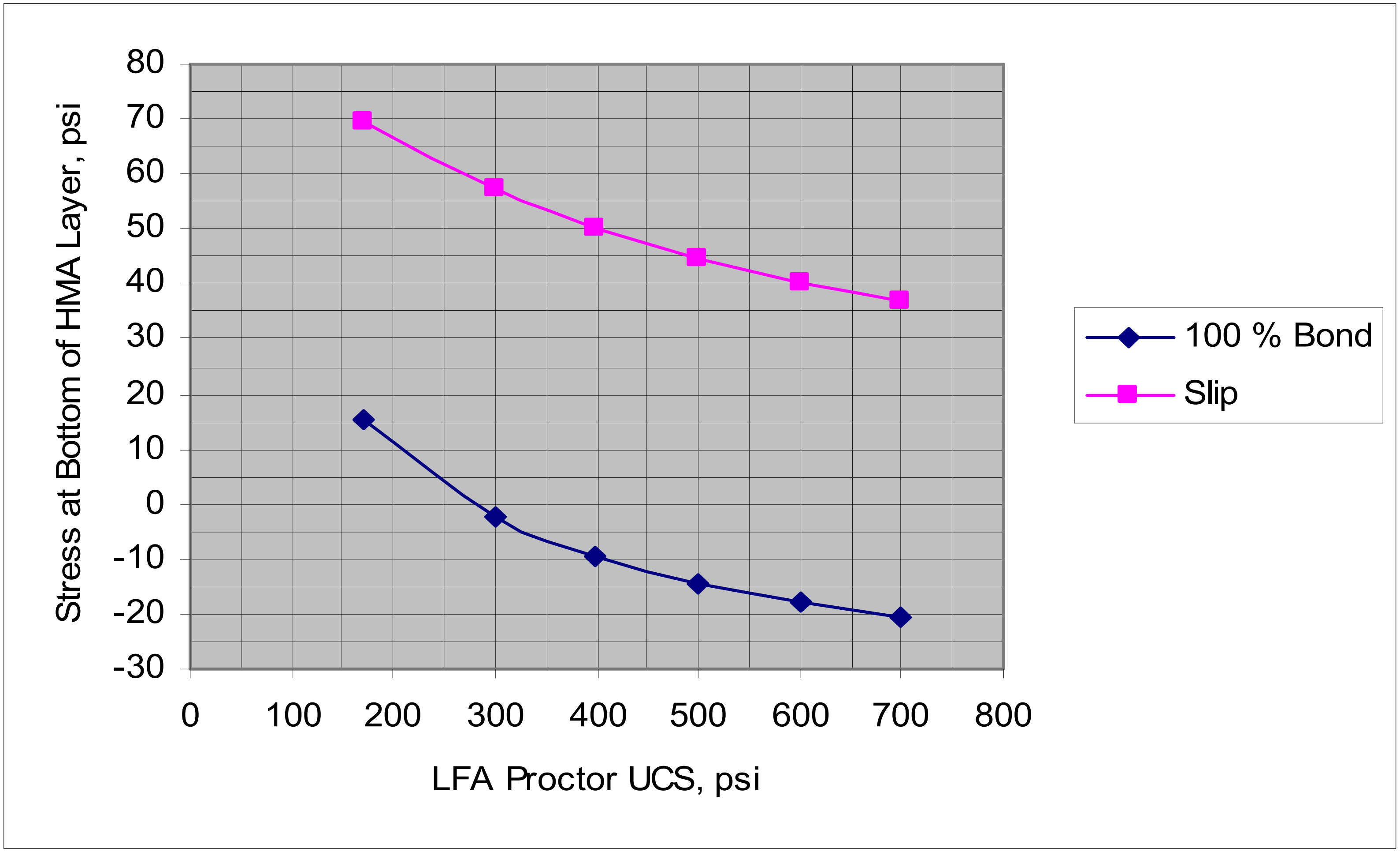


Figure 60. Variation in Stress at Bottom of HMA Layer Between Tires on Given Axle vs. LFA Proctor UCS - Comparing Full Bond and Slip Between HMA and LFA Layers for Bolivar County Project In-Service Condition

Table 28. Summary of Modulus, Poisson's Ratio, LFA Proctor UCS, and Layer Thickness Used to Compute Stress/Strength Ratios for the Construction Loading Condition

Material	Modulus (psi)	Poisson's Ratio	LFA Proctor UCS (psi)
LFA Base Course	41,200 - 359,900	0.3	70 - 350
Lime Treated Subgrade	No LTS to 40,000	0.3	
Subgrade	12,000	0.4	

Layer Thicknesses, in.

County	LFA	LTS
Bolivar	6.0(1) - 8.0	6.0 - 8.0
Clarke	6.0(1) - 8.0	6.0 - 8.0
Smith	6.0(1) - 8.0	6.0 - 8.0
Tippah	6.0 - 8.0(1)	6.0 - 8.0
Wilkinson	6.0(1) - 8.0	6.0 - 8.0

(1) Original Design LFA Layer Thickness

Table 29. Revised Pavement Design for Each of the Five Newer Projects

County	ADT (Year)	10th Year ADT (Year)	DHV	D (%)	T (%)	Revised Design CBR	Flex Rate	Computed Design ESALS	Required SN	Design HMA a_1	Design Thickness HMA (in.)	Design LFA a_2	Design Thickness LFA (in.)
Bolivar	(1998) = 4000	(2008) = 5119	670	55	14	4	1260	1,614,552	4.48	0.44	7.50	0.2	6
Clarke	(2001) = 5500	(2011) = 7700	1200	50	19	6	1260	2,883,573	4.54	0.44	7.75	0.2	6
Smith	(1998) = 2100	(2008) = 2825	410	55	15	8	1260	934,295	3.60	0.44	5.50	0.2	6
Tippah	(2000) = 6300	(2010) = 9050	1300	55	15	3	1260	2,912,003	5.14	0.44	8.25	0.2	6
Wilkinson	(1997) = 4180	(2007) = 5618	940	55	15	5	1260	1,858,717	4.39	0.44	7.25	0.2	6

ADT = Average Daily Traffic
DHV = Design Hourly Volume
D = Directional Factor
T = Truck Traffic
Flex Rate = Average 18 KIP Axle Loads Per 1000 Vehicles

Table 30. Summary of Modulus, Poisson's Ratio, LFA Proctor UCS, and Layer Thickness Used to Compute Stress/Strength Ratios for the In-Service Loading Condition

Material	Modulus (psi)	Poisson's Ratio	LFA Proctor UCS (psi)
HMA	235,300	0.35	
LFA Base Course	106,300 - 546,600	0.15	171.6 - 700
Lime Treated Subgrade	40,000	0.15	
Subgrade:			
Bolivar	11,930	0.4	
Clarke	15,990	0.4	
Smith	18,640	0.4	
Tippah	8,820	0.4	
Wilkinson	14,400	0.4	

	Layer Thicknesses, in.		
	HMA	LFA	LTS
Bolivar	6.5 - 7.5(1)	6.0(2) - 8.0	6
Clarke	6.75 - 7.75(1)	6.0(2) - 8.0	6
Smith	4.5 - 5.5(1)	6.0(2) - 8.0	6
Tippah	8.25(1) - 9.25	6.0 - 8.0(2)	6
Wilkinson	6.25 - 7.25(1)	6.0(2) - 8.0	6

(1) Corresponds to Design HMA Layer Thickness Shown in Table 28, Not Original Design Thickness Shown in Table 2 from Chapter 2

(2) Original Design LFA Layer Thickness

Table 31. Conversion of Design Traffic Loading in ESALs to Equivalent 34-kip Tandem-Axle Loads

County	10-Year Design SN	Equivalency Factor	10-Year Design ESALs	Equivalent 10-Year 34 kip Loads	50-Year Design ESALs	Equivalent 50-Year 34 kip Loads
Bolivar	4.48	1.100	1614552	1467775	6276840	5706218
Clarke	4.54	1.099	2883573	2623815	15326767	13946103
Smith	3.6	1.110	934295	841707	4306294	3879544
Tippah	5.14	1.089	2912003	2674016	17010921	15620680
Wilkinson	4.39	1.102	1858717	1686676	8539735	7749306

Table 32. Comparison of Number of 34-kip Tandem-Axle Loads to Effect Fatigue Cracking at the Bottom of the LFA Base Layer for Various Combinations of Base Layer Thickness and HMA - LFA Interface Bonding Conditions, Expressed as Percent of 50 - Year Design Traffic Loading

	50 - Year Design Traffic	10 - Year Design Traffic	6" Design Base Full Bond	8" Design Base Full Bond	8" Design Base 1" Deficient Base Thickness Full Bond	8" Design Base With Slip	8" Design Base 1" Deficient Base Thickness With Slip
Bolivar	100	26	101	358	118	11	6
Clarke	100	19	111	359	129	17	10
Smith	100	22	48	221	59	3	1
Tippah	100	17	93	263	103	25	20
Wilkinson	100	22	93	331	109	10	5

Table 33. Check of 6" and 8" LFA Base Designs - Number of Loads to Effect Fatigue Failure in HMA, LFA, and LTS Layers - Expressed as Percent of 50-Year Design Loading

Check 6" LFA Base Design

	50 - Year Design Loading (%)	HMA (%)	LFA (%)	LTS (%)
Bolivar	100	9473	101	9.28E+09
Clarke	100	4076	111	6.88E+09
Smith	100	9035	48	2.18E+10
Tippah	100	4789	93	2.76E+09
Wilkinson	100	6548	93	9.44E+09

Check 8" LFA Base Design

	50 - Year Design Loading (%)	HMA (%)	LFA (%)	LTS (%)
Bolivar	100	11478	358	1.16E+10
Clarke	100	4601	359	8.14E+09
Smith	100	10506	221	2.84E+10
Tippah	100	6982	263	3.52E+09
Wilkinson	100	7449	331	1.14E+10

Table 34. Soil Cement Structural Layer Coefficients and Corresponding UCS Values Used by Various State DOTs

State	Layer Coefficient	Compressive Strength Requirements, psi
Alabama	0.23	650
	0.20	400 - 650
	0.15	Less than 400
Arizona	0.28	For cement-treated base with minimum 800 psi (plant mixed)
	0.23	For cement-treated subgrade with minimum 800 psi (mixed-in-place)
Delaware	0.20	
Florida	0.15	300 (mixed-in-place)
	0.20	500 (plant mixed)
Georgia	0.20	350
Louisiana	0.15	200 min
	0.18	400 min
	0.23	Shell and sand with minimum 650 psi
Montana	0.20	400
New Mexico	0.23	650 min
	0.17	400 - 650
	0.12	Less than 400
Pennsylvania	0.20	650 min (mixed-in-place)
	0.30	650 min (plant mixed)
Wisconsin	0.23	650 min
	0.20	400 - 650
	0.15	Less than 400

Chapter 12 Summary of Conclusions and Recommendations

Conclusions

The soils typically stabilized in Mississippi with LFA are granular with the plasticity index (PI) of these soils limited to 10 or less. A review of 182 LFA mix designs was conducted which included designs for both base course construction and subgrade stabilization. This review indicated that five percent of the soils were classified as A-1-a, three percent as A-1-b, 82 percent as A-2-4, and six percent as the fine grain soil type, A-4.

The review of the 182 mix designs indicated that 76 percent included a 1:3 lime/fly ash ratio and 20 percent included a 1:4 ratio. A review of the mix designs accepted for construction from between April 15, 1999, to November 10, 2000, indicated that 77 percent included Class F fly ash and that 23 percent included Class C fly ash.

Visual Examination of LFA Cores

Significant variation exists in the quality and properties of a given LFA base course. Evidence of this variation has been documented both visually and numerically. A LFA core rating scheme was developed and used to visually classify the relative quality and suitability for UCS testing of extracted cores of the LFA material on a scale from one to six. The LFA core ratings recorded for each of the nine projects provide visual evidence of this variation.

Table 7 in Chapter 3 illustrates that all five of the newer projects include LFA stabilized soil that ranged from well cemented material providing excellent testable cores to relatively poorly cemented material from which no core could be obtained for testing. Based on this visual classification scheme, the LFA material in 62 percent of the tested locations within the newer pavements was in excellent condition with an assigned value of one. Two out of the 63 newer pavement test locations, both of these in the same project, had very poor LFA material present in the pavement and were assigned a value of six.

The LFA material in 62 percent of the tested locations within the four older pavements was also assigned a value of one. No LFA material in any of these pavements was classified as a six.

LFA Core UCS Test Results

The UCS test device, illustrated in Appendix B, had an upper loading limit of about 10,000 pounds, which corresponds to 795 psi for the four-inch diameter cores. Quite unexpectedly, the strength of many of the cores exceeded the loading capacity of this testing device. The use of this UCS test device did not allow for calculation of either the average or coefficient of variation in in-situ LFA strength; however, the upper loading limit of this device did allow the applied stress to exceed the 500 psi LFA base course design value. The in-situ strength of 41 percent of the LFA stabilized material in the newer pavements and 56 percent in the older pavements exceeded the design value. The in-situ strength of 21 percent of the LFA stabilized material in the newer pavements and 31 percent in the older pavements exceeded 795 psi. The greater percentages associated with the older pavements are attributed to the continuing strength gain of LFA stabilized material with time.

LFA Backcalculated Modulus

LFA backcalculated modulus and in-situ structural layer coefficient values provide numerical evidence of LFA material variability. For the five newer projects the average backcalculated modulus was 423.6 ksi with a coefficient of variation of 72.3 percent. For the four older projects the average backcalculated modulus was 169.5 ksi with coefficient of variation of 67.7 percent. The decrease in the average modulus value between the newer and older pavements is not surprising considering that the FWD test procedure actually measures an effective modulus of a given pavement layer. With time the pavement layer in question cracks due to traffic and environmental effects which reduce the stiffness, or modulus, of the layer. The coefficients of variation for both the newer and older pavements are similar in magnitude and indicate significant variation in this material property.

LFA In-Situ Structural Layer Coefficients – Newer Pavements

MDOT uses the AASHTO Interim Guide for the Design of Rigid and Flexible Pavements – 1972 for its flexible pavement design methodology. The structural layer coefficient is the primary input parameter reflecting the quality of the pavement materials in this design procedure. In this study the basis of evaluation for the LFA material is the development of in-situ LFA structural layer coefficients. The average normalized LFA structural layer coefficient for the five newer pavements is 0.232 with 67 percent of the tested locations exceeding the design value of 0.20. The average exceeds the design value, and taken on this merit alone, indicates excellent early performance of the LFA stabilized soil base courses. However, the coefficient of variation for these pavements is 32 percent, indicating a significant variation in the in-situ properties of this stabilized material. The large variation in the quality of the in-situ material suggests a significantly lower level of performance than the average values indicate when the concept of reliability is introduced into the evaluation scheme. Given an average of 0.232 at the current level of variability, the corresponding design value should be 0.14 to provide a 90 percent level of confidence. This value is 30 percent less than the design value currently used by MDOT.

LFA In-Situ Structural Layer Coefficients – Older Pavements

It is difficult to assign a structural layer coefficient to materials that have experienced degradation due to the effects of traffic loading and the environment. The data from the five newer projects was used to develop a relationship between in-situ LFA structural layer coefficient and backcalculated LFA modulus. This relationship was then used to determine the in-situ LFA layer coefficient values for the four older projects. The average for all four older pavements was 0.165 with a coefficient of variation of 23.3 percent. This average is less than the design of 0.2 and is expected due to traffic loading and environmental effects on these older pavements. The variability calculated for the older pavements is less than the variability calculated for the newer pavements. This reduction in variability can probably be attributed to the use of the relationship to calculate the LFA layer coefficients rather than an actual reduction in variability, since both the older and newer pavements were constructed using similar field-mixed-in-place methods.

Based upon visual observation, and backcalculated moduli and in-situ structural layer coefficient values, it is concluded that MDOT LFA stabilized soil base courses possess highly variable material properties. It can be numerically demonstrated that variations in the LFA material modulus from one location to another within a given pavement result in differential performance throughout the length of that pavement. An example of a documented project that experienced premature pavement failure due to highly variable LFA and HMA material properties is the phase two project constructed in 1985-1986 on US 84/98 in Adams County. Details of this project are included in Chapter 1.

Variation in In-Situ LFA Layer Thickness

There is significant variation in the in-situ LFA stabilized soil base layer thickness within the majority of the pavements cored for this study (Table 21). Two of the newer pavements have a difference between minimum and maximum layer thickness of at least 4 inches, which is 67 percent of the design layer thickness. As with variations in material properties, it can be numerically demonstrated that variations in the LFA stabilized soil base layer thickness from one location to another within a given pavement results in differential performance throughout the length of that pavement.

Difference in Curing Temperatures Used for LFA Mix Design Cylinders and Field-Mixed Material Cylinders

The UCS values of the field-mixed material cylinders are always lower than the laboratory-mixed material cylinders associated with the corresponding LFA mix design(s). As a result, the strengths of the field-mixed cylinders are recorded, but not utilized for quality control, unless the strengths are abnormally low when compared to the strengths normally obtained for field-mixed material. A review of the UCS test results from 139 field-mixed cylinders, representing six projects under construction during the 2001 construction season, indicated an average UCS of 70.4 psi with a coefficient of variation of 58 percent. This average value is 86 percent below the design value of 500 psi. This variation in UCS test values is in general agreement with the variability noted in Chapter 4 for the backcalculated modulus values of both the older and newer pavements.

Several factors contribute to the total discrepancy noted between field-mixed and laboratory-mixed cylinder UCS test results. A significant factor, as discussed in Chapter 6, is the difference in curing temperature. Laboratory-mixed cylinders used for LFA mix designs are cured for 28 days at 100 ^{0}F prior to UCS testing; however, field-mixed cylinders are cured for the same time period, but at 73 ^{0}F. Field-mixed material cylinders cured at the lower temperature developed approximately 20 percent of the UCS developed in field-mixed material cylinders cured at the higher temperature for the same duration of curing.

Another significant factor is the difference in the degree of proportioning and blending between the laboratory and in the field. Laboratory proportioning of LFA and soil blends result in much more uniform blends than that possible in the field using the current field-mixed-in-place method of blending the materials.

Laboratory Investigation of Effect of Increased Compaction on Developed LFA Stabilized Soil Strength

A 50-percent increase in UCS was observed by increasing the compaction level from 94 to 100 percent standard density for the particular LFA and soil blend included in this laboratory investigation.

The relationship between material design and construction requirements need to be clearly understood. For an LFA mix design cylinders are compacted in the laboratory to 100 percent standard density. Acceptance of the mix is based upon the achievement of a 500 psi Proctor UCS. Construction specifications at the initiation of this study required that the LFA base course would be compacted to a minimum of 94 percent standard density when using the red sand topping material considered in the laboratory investigation. These specifications were in effect requiring a field Proctor UCS of 392 psi, or about 22 percent less than the laboratory mix design required strength.

LFA and Soil Blends Placed at the End of a Given Construction Season – Benefit of Increased Compaction

As discussed in Chapter 6, LFA stabilized material requires time and temperatures exceeding 40 ^{0}F for effective strength gain to occur, especially when a Class F fly ash is used in the blend. This is an important consideration for late season LFA construction given the relatively cool temperatures of late fall and winter. The saturation of compacted LFA and soil mixtures, before the occurrence of significant strength gain, was identified as one of the reasons for several premature pavement failures in Mississippi.

Increased levels of compaction reduce the potential amount of absorbed water. For a given LFA and soil blend, increasing the density from 94 to 100 percent standard density resulted in an 18 percent reduction in the amount of absorbed water. Compaction above 100 percent standard density would result in an even greater reduction in the amount of absorbed water.

LFA and Soil Blends Placed at the End of the Construction Season – Initial Performance as an Unbound Material

The red sand topping used in the laboratory investigations had 23 percent non-plastic fines. When the 3 percent lime and 12 percent Class F fly ash were added for stabilization, an additional 15 percent “fines” were mixed into this soil. Initially, before any pozzolanic reactions occur, the strength and behavior of this material in a pavement layer corresponds to essentially that of a silty sand soil, or an unbound granular material. Given sufficient time and curing temperatures the blend experiences pozzolanic reactions and becomes more like a cement-bound material. If this type of blend is placed in late fall and little strength gain occurs during the following winter months, the response of this material to increases in moisture content will be more like that of an unbound granular material, not a cemented material. For cement bound materials, the presence or absence of moisture has no effect on the direct response of this material during deflection testing. However, for unbound materials, at a given density and stress level, moisture content is probably the most significant factor affecting the modulus of this material. The modulus of an unbound material can decrease by several factors with increasing moisture content.

Based on the foregoing discussion, increasing the level of required density of the LFA and soil blend will result in an increase in the unbound strength of the base layer at the time of placement. This will aid in reducing the incidence of premature pavement failures due to saturation of this layer prior to significant pozzolanic-induced strength gain.

Strength Gain of LFA and Soil Blends Following Late Season Placement

Twelve cylinders were fabricated in the laboratory using the same blend of soil and LFA. These cylinders were divided into two sets of six cylinders, and then each set was subjected to a different curing regime. Six cylinders were cured for 90 days at 50 ^{0}F to try to simulate the effect of the cool winter temperatures that typically occur during the months of December, January, and February. The average UCS for these cylinders was 66 psi, indicating that little strength development can be expected during the cool winter months, which is an important consideration for late season LFA stabilized soil base course construction utilizing Class F fly ash.

The second set of six cylinders was cured for 90 days at 50 ^{0}F followed by 28 days of curing at 100 ^{0}F. The objective was to see if the LFA stabilized soil, placed at the end of one construction season, and experiencing little increase in strength over the subsequent winter months while subjected to saturating moisture conditions, would gain strength with increase in temperature during the following construction season. The average UCS of these cylinders was 441 psi, 75 percent of the strength obtained with a curing regime corresponding to that for an LFA mix design, and 88 percent of the design strength of 500 psi. This is a significant improvement over the 66 psi recorded for the 90-day curing at 50 ^{0}F, and illustrates that pozzolanic reactions do activate subsequent to an extended dormant time period, given a sufficient increase in temperature.

A significant difference between the LFA stabilized material cured in the laboratory, and that cured over the winter and following spring and summer months in the field, is that the laboratory cured material is not subjected to any loading. The material in the field will likely be subjected to at least minimal construction loading during this time period.

In-Situ Relative Quality of LFA and Soil Blends Placed During Late Season Construction Versus Placement During the Following Construction Season

Three of the five newer pavements included LFA stabilized soil base material that was placed both in the fall and following spring/summer time periods which allow a limited evaluation for potential difference due to late fall construction. The Proctor strengths could not be utilized in the comparison analyses due to the upper limit imposed by the UCS testing machine; however, the backcalculated LFA moduli and normalized LFA layer coefficient values are available for use in these analyses. The statistical F and T tests were selected to compare each set of data from each of the three projects using a level of significance, or alpha, of 0.10. A summary comparison is included at the end of Table 26. For both the Bolivar and Clarke County projects there is no statistical difference between the properties of the LFA stabilized soil material placed in the fall of 1998 relative to the material placed the following construction season. For the Smith County project there is a statistical difference in LFA material properties between these two time periods; however, the data indicates that the material placed during the fall of 1998 is a better quality material than that placed the following summer. For these three projects it can be concluded that there are no detrimental effects to the LFA stabilized soil due to late season construction.

Use of Degree Days to Establish Construction Cut-Off Date for LFA Stabilized Soil Base Construction

Given the results of a previous field investigation conducted in the MDOT 2nd District and the fact that the occurrence of freeze/thaw events are generally not a major issue in

Mississippi, no degree day requirements are recommended for LFA stabilized soil base construction in Mississippi.

Construction Loading

For LFA stabilized material cured for one month under spring or fall curing conditions, and assuming a subgrade CBR of 5 and good construction platform, no tensile stresses are developed at the bottom of the LFA stabilized soil layer; hence, the stress/strength ratio is 0. The LFA material has not cured enough to carry a significant amount of the load and is in effect being “cradled” by the underlying construction platform and subgrade. The first lift of HMA can be placed without overstressing the base material.

Summer curing temperatures are significantly higher than spring or fall curing temperatures thus effecting a greater degree of developed strength and stiffness in the base layer. This increased stiffness of the LFA material allows it to carry some of the load with resulting tensile stresses developing at the bottom of the LFA layer. Placement of the first lift of HMA would overstress the base material and cause some cracking within this pavement layer.

Based on stress/strength calculations, given LFA base modulus values less than about 75,000 psi and a minimum LTS modulus of 40,000 psi, Figure 46 from Chapter 11 indicates that construction loading will not crack the bottom of the LFA base layer for the current typical MDOT pavement design/construction practice of using 6 inches of LFA stabilized base overlying 6 inches of chemically stabilized design soil. For lower quality construction platforms; i.e., LTS modulus values less than 40,000 psi, and LFA modulus values exceeding 75,000 psi, Figure 46 illustrates that the LFA stabilized base layer will crack due to construction loading, even at the American Coal Ash Association recommended equivalent LFA Proctor UCS value of 500 psi.

Given the results of two studies it is postulated that even if a LFA stabilized soil base does crack due to the application of construction loads, this cracking will not significantly affect the long-term strength and stiffness gains of this stabilized material so long as it has adequate foundation support to facilitate autogeneous healing of this stabilized material.

Reflective Cracking Due to Shrinkage Cracking in LFA Stabilized Soil Base Layer

In the Yalobusha County project 82 percent of the shrinkage cracks in the LFA stabilized soil base layer reflected through the overlying HMA. The reflective cracking observed in this project was probably exacerbated by the lack of a construction platform, since it was not included in the pavement design. Shrinkage cracking was also observed to occur in the LFA stabilized material during the construction of Hwy. 302 (George, 2001). Based on these observations shrinkage cracking appears to be a problem in this type of stabilized material.

Benefit of Chemically Stabilized Subgrade Layer

Figures 48 through 52 in Chapter 11 illustrate the benefit of the use of a chemically stabilized subgrade by reducing the stress/strength ratios for the overlying LFA base layer and extending the service life of this layer. These figures support the Department's decision to chemically stabilize the subgrade for all new pavement construction.

Increase LFA Base Layer Thickness for Routine Pavement Design/Construction

A major objective of pavement design and construction is to economically obtain a long-lasting product with a minimum of maintenance. One potential way to achieve this objective is to construct a Perpetual Pavement. The design of a Perpetual Pavement differs fundamentally from the flexible pavement design method currently utilized by MDOT in that the design life, by definition, is 50 years for the Perpetual Pavement, whereas the number of loads anticipated over typically a 10-year design life is entered into the current MDOT design procedure. Therefore, the base course of a Perpetual Pavement must be designed to carry a significantly larger number of loadings.

In this study the concept of a Perpetual Pavement is used in conjunction with the current MDOT design methodology. The objective is to extend the design life of the base course to that corresponding to a Perpetual Pavement, with the constraint that the thickness of the overlying HMA is based on MDOT's current typical 10-year flexible pavement design life. Using this approach subsequent full depth pavement reconstruction should be minimized and rehabilitation/reconstruction efforts predominantly confined to the HMA layers of the pavement structure.

Figures 48 through 52 in Chapter 11 illustrate a significant extension in the performance life of the LFA base layer by increasing the design thickness of this layer from 6 inches to 8 inches. These figures indicate that the additional 2 inches is too conservative; however, Figures 55 through 59 illustrate the need for this increased design thickness to allow for deficiencies in in-situ LFA layer thickness. The added thickness also helps to offset the effects of overloading on the pavement.

Field LFA Proctor UCS

The LFA stabilized soil base layer must both be durable and meet the structural requirements based on the anticipated loading. The referenced literature recommended a minimum of 400 psi to ensure the durability of this stabilized material.

Figures 55 through 59 in Chapter 11 suggest that an in-situ LFA Proctor UCS of 400 psi will provide adequate strength for a Perpetual Pavement base layer provided: a 6-inch chemically stabilized subgrade layer is included under the pavement structure, the LFA base layer thickness is increased from 6 inches to 8 inches, and full bonding is ensured between the pavement layers.

LFA Design Structural Layer Coefficient

The revised design for each of the five newer projects utilized the current MDOT LFA structural layer coefficient of 0.20. This value can be maintained assuming the in-situ LFA Proctor UCS value of 400 psi, in conjunction with the three aforementioned provisions, are achieved in the LFA base layer and pavement structure. This combination of design structural layer coefficient and strength reasonably agree with the requirements for soil cement in the States of Alabama, Georgia, Louisiana, Montana, and Wisconsin.

Recommendations

Fly Ash QC/QA

Both the physical and the chemical properties of the fly ash used in the LFA mix design for a given project should be maintained in all of the shipments of ash to that project during field construction. This will aid in producing a consistent product along the length of that project with a quality corresponding to its design. The fly ash specifications and associated quality conformance testing necessitates the development of an effective Quality Control/Quality Assurance (QC/QA) program to control the quality of fly ash shipped to MDOT projects. The development of a pozzolanic reaction test has been suggested to fulfill the requirements of a QC/QA fly ash test. For this test a blend of lime and fly ash, with the same proportions as that required in the corresponding LFA mix design, is made into cubes, subjected to an accelerated rate of curing for two days, and then tested for UCS. The MDOT LFA mix design process requires up to 28 days before a proposed mix design is found acceptable for use. A pozzolanic reaction test could be used in screening potential combinations of lime and fly ash that do not sufficiently react before their use in the more time consuming LFA mix design process. During the course of field construction samples of the lime and fly ash being delivered to the project site could be obtained and tested using this procedure to ensure the same reactivity as that observed during the design process. It is recommended that a research study be initiated to develop a pozzolanic reaction test to establish acceptance/rejection criteria of a given LFA blend.

Loss On Ignition

Loss on ignition (LOI) is another chemical parameter associated with fly ash. LOI is a measure of the unburned carbon or coal remaining in the ash. The Mississippi Standard Specifications for Road and Bridge Construction allow a maximum of 10 percent LOI for soil stabilization. Class F fly ash with an LOI of 16 percent was successfully utilized in a stabilized base course of a ramp in Delaware, and a 12 percent LOI fly ash was successfully used in Michigan for a base course. This limited data supports the current MDOT requirement for LOI when the fly ash is used for soil stabilization; however, it is recommended that research be conducted to quantitatively evaluate the impact of LOI on the reactivity of the fly ash.

Correlate Laboratory LFA Mix Design Compaction Effort With Specified Field Compaction Level

The laboratory-mixed material cylinders should be compacted with a modified Proctor compactive effort in accordance with AASHTO T-180, with the exception that the blows per layer will be adjusted so that the compacted density is approximately 96 percent modified density.

Note: The blows per layer will be a fixed value for every LFA mix design performed at the MDOT Central Laboratory. This number will be determined for the most prevalent type of soil stabilized with LFA in Mississippi; i.e., an A-2-4 soil type. This number of blows per layer will also be applied to the field mixed soil cylinders fabricated in conjunction with the QC/QA program.

LFA Mix Design With Class C Fly Ash

Chapter 6 included a discussion on the fundamental difference between the strength gain characteristics of a LFA stabilized soil when using a Class C ash as opposed to using a Class F ash. In addition to the pozzolanic reactions that both classes of ash experience, Class C ash has a hydration component that can potentially increase the early strength gain of the LFA stabilized soil. The initial gain in strength associated with the hydration of this ash occurs at a greater rate than that of Portland cement. Delayed compaction can cause a reduction in the strength achieved in the field relative to the design strength since LFA mix design cylinders are fabricated immediately after mixing of the materials in the laboratory. It is recommended to maintain the same delay in compaction during the laboratory design phase as the delay in compaction during construction. This requirement may lead to the incorporation of a greater percentage of a given Class C ash, or possibly the exclusion of the particular ash; however, the laboratory derived strength will more closely model that being obtained in the field.

Increase Compaction of LFA Stabilized Soil Base Course to Increase Average In-Situ Layer Coefficient

Three approaches were considered to achieve the current MDOT design value of 0.20. The third approach is a combination of both the first and second approaches, which include increasing the average value of the in-situ LFA structural layer coefficient and reducing the variability in this design parameter. Increasing the required level of field compaction is one way to increase this value. Based on a review of compaction data obtained from the construction of the Nissan Plant near Canton Mississippi, suggested levels of required compaction found in the literature for both lime and LFA stabilized soils, and a review of the current research by Dr. Dallas Little, it is recommended to increase the required level of compaction to 96 percent modified Proctor density for LFA stabilized soil base courses. The adoption of this recommendation is contingent upon MDOT requiring a bottom-to-top improvement in compacted densities of the layers comprising the pavement foundation.

Compaction of Basement and Design Soils

At the onset of this study the required density for basement and design soils was 94 and 96 percent standard density respectively. In response to the bottom to top approach for pavement foundation improvement, it is recommended to increase the basement and design soil requirements to 96 and 98 percent standard density respectively.

Special consideration should be made for high volume change soils when they are encountered in the design soil prism. When high volume change soils are compacted to relatively high levels of density these soils are subject to changes in volume with changes in moisture content. In these cases the 98 percent standard density requirement may be too high, and consideration should be given to possibly lowering this recommended density requirement. This evaluation should be performed on a case by case basis rather than automatically reducing the required level of compaction for every situation encountered in the field.

Compaction of Lime Stabilized Subgrade

At the onset of this study the required level of compaction for a lime stabilized fine-grained soil was 95 percent standard density. Given the prevalence of weak subgrade soils throughout Mississippi, especially in the northern and central regions of the State, and the low levels of required compaction in the basement and design soils, 95 percent was a reasonable value. However, increasing the level of compaction in the basement and design soils improves the strength of these pavement foundation soils, thereby allowing an increase in the level of required compaction for the overlying chemically stabilized subgrade layer. It is recommended to increase the required compacted density of the lime-treated subgrade layer to 100 percent standard density for all new pavement construction that includes a design soil CBR equal to or in excess of five.

In cases where the lack of locally available better quality material has required the use of on-site materials with a design CBR of less than 5, or the use of high volume change soils requiring a reduction in recommended density, in the design soil prism a sufficiently stiff soil foundation may not be available to support the recommended increase in level of compaction for the overlying lime stabilized subgrade layer. In these cases it may be necessary to maintain the current 95 percent standard density requirement for this stabilized layer. However, an evaluation should be performed on a case by case basis rather than automatically reducing the required level of compaction for every weak foundation condition encountered in the field.

In those cases where the lime stabilized subgrade layer cannot be compacted to 100 percent standard density, the resulting pavement foundation may not be stiff enough to support the recommended 96 percent modified density in the overlying LFA stabilized soil base course. A corresponding reduction in the recommended base course density may be required; however, as with situations involving the lime stabilized subgrade layer, this should also be decided on a case-by-case basis. Reducing the required base course density will reduce the quality of the base course material, which should be reflected in the pavement design process by the use of a lower structural layer coefficient for this material.

Compaction of LFA or Cement Stabilized Subgrade

At the onset of this study the required levels of compaction for a LFA or cement stabilized soil was based on the pavement layer under consideration and the type of soil included in the stabilized blend. Subgrade soils to be stabilized with either cement or LFA typically possess greater inherent strength than fine-grained soils requiring stabilization with lime; therefore, these foundation soils will typically support greater levels of compaction in overlying layers. It is recommended to compact cement or LFA stabilized subgrade layers to 100 percent standard density.

Reduce Variability

There are a multitude of potential sources affecting variation in the properties of a LFA stabilized soil base course (Figure 34). The primary focus in this study to address this issue is on field construction procedures. Two potential methods to reduce variability are improving the current method of field-mixed-in-place, and plant mix with placement of the blended material via a paver. The former method constitutes the predominant discussion included in this study because the in-state soil stabilization contractors have made substantial investments in pulvamixers for field-mixed-in-place construction.

Improvements to the Current Field-Mixed-In-Place Method of Construction

A primary source of variability in the in-situ LFA stabilized soil properties is the current method of field-mixed-in-place construction. The following recommendations are included to improve this method:

1. Evaluate the use of the Vane Feeder Spreader to more evenly distribute the fly ash across the surface of the roadbed. The Vane Feeder Spreader was developed by Cutrell Trucking of Amarillo, Texas and offers a solution for reducing both the variability in the spread and the magnitude of the dusting problem associated with spreading fly ash.

2. Increase the target fly ash content applied in the field by 2 percent over that required in a given LFA design.

3. Evaluate the use of the Vane Feeder Spreader to more evenly distribute the lime across the surface of the roadbed and reduce the dusting problem.

4. Increase the target lime content applied in the field by 0.5 percent over that required in a given LFA design.

5. Evaluate the construction practice of adjusting the moisture content of the raw soil for the base course to within 100 to 120 percent of the optimum moisture content for the LFA and soil blend immediately prior to spreading the fly ash and lime.

6. Evaluate the method of nursing to add these controlled amounts of water to the raw soil; i.e., add the water through the pulvamixer. This method will aid in obtaining a more uniform distribution of moisture without creating a hazard to construction personnel as discussed in Chapter 9.

7. Change MDOT's current field pulverization requirement for LFA stabilized material to 100 percent of the blended material, excluding gravel or stone, passing the 1-inch sieve.

8. The use of an autograde trimmer, operated off a string-line, is recommended to control the extent of surface undulations in the finished base course.

9. It is recommended that the QC/QA program include measuring the in-situ LFA base layer thickness the same day that the base course is constructed to ensure that the design layer thickness is achieved in the field. The dynamic cone penetrometer (DCP) could be employed for this purpose. Remedial action for inadequate base layer thickness depends upon the type of fly ash used in the blend. When Class F ash is used in the blend, the base layer can be remixed with the pulvamixer to a depth sufficient to obtain the desired layer thickness. When Class C fly ash is used in the blend, remixing is not advised, and the inadequate base layer thickness should be compensated with an additional thickness of overlying pavement layer material.

Note: Excessive variability in LFA stabilized soil pavement layer properties is not a problem unique to LFA base course construction. Currently there is significant interest within MDOT to construct chemically stabilized soil base courses using Portland cement as the stabilizing agent due to problems associated with the use of LFA as the stabilizing agents. Portland cement is an excellent alternate for LFA, but its use does not automatically eliminate problems with variability because both materials are spread and incorporated into the soil using the field-mixed-in-place method. Similar problems are encountered with the use of soil cement. It is recommended that MDOT adopt similar procedures for improving the current field-mixed-in-place method for soil cement construction as well as LFA construction.

Reduction in Variability and Impact on LFA Structural Layer Coefficient

It is estimated that adopting the recommendations included in this study for field-mixed-in-place construction would reduce the variability to 75 percent of the current level. Adopting the recommended increase in density to 96 percent modified density and adopting the recommendations for improving the current field-mixed-in-place construction method could allow the potential increase in LFA design layer coefficient to 0.22, or a 10 percent increase in this design parameter over the current design value.

Using Plant-Mixed Approach to Reduce Variability

A central mixing plant was used for blending the lime, fly ash, soil and water for the US 84/98 project in Adams County as discussed in Chapter 1. A judgment regarding the veracity of using a plant mix approach for blending these materials should not be made based on this project since an old plant was used that experienced problems with proportioning. Modern mixing plants used in HMA and Portland cement concrete production are fully automated and produce tons of high quality mix for road construction. Use of a mixing plant is the recommended method of blending the lime, fly ash, soil and water because it allows greater control in the proportioning of these materials and yields a more uniform product.

It is estimated that using plant mixed LFA and soil blends would result in a 50 percent reduction from the current levels of variability. Adopting the recommended increase in density to 96 percent modified density and adopting the recommendation for using the plant mixed approach could allow the potential increase in LFA design layer coefficient to 0.24, or a 20 percent increase in this design parameter over the current design value.

Placement of Plant-Mixed Material With A Paver

Table 22 includes the average in-situ HMA layer thickness for each of the nine projects. HMA is placed with a paver. By comparing Tables 21 and 22 a reduction in the value of the coefficient of variation for the HMA layer thickness relative to the LFA layer thickness is observed for the majority of the projects. The greatest difference observed between the maximum and minimum LFA layer thickness among the nine projects is 4.7 inches, whereas the greatest difference for the HMA layer thickness is 2.25 inches. These observations indicate that the placement of LFA and soil blends with a paver instead of the current field-mixed-in-place construction method may reduce the variability in LFA layer thickness.

Increase LFA Base Layer Thickness for Routine Pavement Design/Construction

It is recommended that MDOT increase the thickness of a LFA stabilized soil base course from 6 inches to 8 inches for routine pavement design.

Requirement for In-Situ LFA Proctor UCS

It is recommended that MDOT require a minimum in-situ LFA Proctor UCS of 400 psi for all LFA stabilized soil base course construction, subject to the following verification:

Testing was not conducted in the present study to evaluate an in-situ LFA Proctor UCS based on durability considerations. It is recommended that such laboratory work be conducted on samples of LFA and soils blends that are compacted to a level of density commensurate with that achieved in the field. The test protocol should focus on the degradation of the chemically stabilized material due to the effects of moisture. Possible protocols include AASHTO T 135, Wetting-and-Drying Test of Compacted Soil-Cement Mixtures, which evaluates durability based on cycles of wetting and drying, or on the Tube Suction Test, which evaluates moisture sensitivity of base, subbase or subgrade materials.

Curing of LFA Stabilized Soil Base Course

1. The segment of LFA base course placed on a given day will be covered the same day of placement with a bituminous seal coat. In the interim time period between placement of the base course and the bituminous seal, the surface will be maintained in a continuously moist condition.

2. If a dry crust forms over the LFA stabilized soil base course, it will be removed with either a motor grader or auto trimmer just prior to placement of the bituminous seal.

Testing of Field-Mixed Material

1. The original LFA design Proctor should not be used for daily quality control. Either a full Proctor test or the one-point Proctor test should be performed each day of LFA base course construction and the results of that test used for that day's field compaction control.

2. Chapter 6 included a discussion on the fundamental difference between the strength gain characteristics of a LFA stabilized soil when using a Class C ash as opposed to using a Class F ash. When using Class C fly ash for a LFA stabilization project, it is recommended to maintain the same delay in compaction when developing the daily Proctor curve for controlling field densities as the delay in compaction during construction. This recommendation is particularly important when applied to materials being compacted to 96 percent modified density.

3. Field-mixed material cylinders should be fabricated in 4-inch diameter split-mold Proctor molds.

4. The field-mixed material cylinders should be compacted with a modified Proctor compactive effort in accordance with AASHTO T-180, with the exception that the blows per layer be adjusted so that the compacted density is approximately 96 percent modified density.

5. Fabricate two UCS cylinders for every 8000 square yards of LFA stabilized material placed, with a minimum of two per day's production when that day's production does not exceed 8000 square yards.

6. Field-mixed cylinders should be transported in the split molds to the laboratory where they will be cured. The mold containing the LFA cylinder should be enclosed in a plastic bag, or some other method adopted, to minimize moisture loss from the sample.

7. The field-mixed cylinders should be cured for 28 days at 100 ^{0}F instead of 73^{0}F.

8. The field-mixed cylinder UCS test results should be used for QC/QA of LFA base course construction.

9. After a five-hour soaking period, the required UCS of the field-mixed cylinders should be 400 psi for base course construction.

10. The UCS test result reported and compared to the 400 psi requirement should be the average of the two cylinders.

11. MDOT should develop a specification for QC/QA of LFA stabilized soil base courses that includes corrective work to be performed when the 400 psi strength requirement is not achieved in the stabilized field material.

Late Season Construction

1. Use of LFA stabilized soil as a base course material is acceptable for late fall construction provided that the material is compacted to a minimum 100 percent standard density to minimize amount of absorbed water and

2. Construction loading is kept to a minimum and the pavement will not be open to traffic during the winter months immediately following construction.

3. The exclusive use of Class C ash in the LFA stabilized soil blend may provide sufficient strength for traffic loading during the winter months immediately following construction if (1) the Class C fly ash has a self-cementing component of strength gain to provide acceptable performance of the base course under traffic loading until the pozzolanic strength gain reactions are initiated during the following spring and summer months and (2) field compaction can be performed in an expedient manner to take advantage of this potential self-cementing component of strength gain.

4. If no Class C fly ash is available with sufficient self – cementing characteristics and the pavement must be opened to traffic, a different chemical stabilizing agent, such as cement, should be used for stabilizing the base course.

5. The stabilized base course should not be exposed at the surface throughout the winter months immediately following construction. This pavement layer should be covered with, as a minimum, the next course within the given pavement structure to minimize the potential for degradation due to F/T events.

Early Construction Loading

MDOT's current policy of waiting seven days prior to commencement with HMA placement should be changed so that subsequent construction loading is applied as soon as possible after the placement of the LFA stabilized layer, before significant curing has occurred, to minimize potential overloading and cracking of this stabilized material.

Heavy Truck Loading Subsequent to LFA Stabilized Soil Base Layer Construction

The recommendation to load the LFA stabilized soil layer with construction equipment soon after its placement does not mean that the pavement should be immediately opened to heavy truck traffic. Such loading should be restricted such that the stress/strength ratio does not exceed 0.65 in the LFA stabilized base layer. When using Class F fly ash in conjunction with late fall construction, the 0.65 requirement will preclude the opening of the pavement to heavy truck traffic until the following spring or summer.

Construction of Pavements to Validate the Recommendations Included in the Current Study

It is recommended that several projects be constructed using the recommendations included in this study for modifying the current field-mixed-in-place method, and several additional projects constructed with plant mixed material placed with a paver. Evaluation of these projects would enable a determination of the actual reduction in in-situ LFA material property variability relative to the current method of field construction. This evaluation should also include the determination of the in-situ LFA structural layer coefficients of these projects to substantiate any revisions for the current LFA design structural layer coefficient. These projects should incorporate an 8-inch LFA base layer with a minimum in-situ LFA Proctor UCS of 400 psi.

Additional Research Effort Required to Better Define the Relationship Between LFA Backcalculated Modulus and LFA Proctor UCS

Due to the low R^2 value associated with the data to derive Equation 3, additional research needs to be performed to better define this relationship.

References

1. American Association of State Highway and Transportation Officials. (1993). *AASHTO guide for design of pavement structures*. Washington, D.C.
2. American Coal Ash Association. (1991). *Flexible Pavement Manual*. Washington D.C.: Author.
3. American Coal Ash Association. (1995). *Fly Ash Facts for Highway Engineers* (Technical Report, FHWA-SA-94-081). Washington D.C.
4. Barstis, W. F., & Crawley, A. B. (2000). *FHWA Demonstration Project 59 The Use of Fly Ash in Highway Construction U.S. 84/98 Adams County* (Final Report 84-DP59-MS-05). Jackson, MS: Mississippi Department of Transportation.
5. Bates, R. L., & Jackson, J. A. (Eds.). (1984). *Dictionary of Geological Terms* (3rd ed.). New York: DoubleDay.
6. Chemical Lime Group, (1992). *Lime Uses in Transportation Construction*. Fort Worth, TX: Author.
7. Chen, D. Bilyeu, J. Lin, H. & Murphy, M. (2000). Temperature Correction on Falling Weight Deflectometer Measurements. *Transportation Research Record No 1716*, 30-39.
8. Coleman, H. W., & Steele, W. G. (1989). *Experimentation and Uncertainty Analysis for Engineers* (2nd ed.). New York: John Wiley & Sons.
9. Crawley, A. B. (1984). *The Use of Fly Ash in Highway Construction* (Interim Report FHWA-DP-59) Jackson, MS: Mississippi Department of Transportation.
10. Crawley, A. B. (1990). Lime-Fly Ash-Aggregate Base and Subbase Courses. In B Suprenant (Ed.), *Proceedings of the First Materials Engineering Congress: Vol 2. Serviceability and Durability of Construction Materials* 1276-.

11. Crawley, A. B. (1998). *Impact of Water Migration Through Pavement on Pavement Performance – Recent Episodes of the Bathtub Design*. Presentation made at the 1998 Mississippi Transportation Conference, Biloxi, Mississippi, October 1998.

12. Crawley, A. B. (2002). *Soil-Cement Structural Behavior Design and Durability Requirements*. Presentation made at the Mississippi Department of Transportation Soil-Cement Seminar, Raymond, Mississippi, June 27, 2002.

13. De Shong, M. D. (2002). [Promotional video of Cutrell Trucking Company, Inc. received from Murphy De Shong of same] Contact: Cutrell Trucking Company, Inc. P.O. Box 50656 Amarillo, TX 79159 (1-800-625-7184).

14. Ferguson, G. & Levorson, S. M. (1999). *Soil and Pavement Base Stabilization with Self-Cementing Coal Fly Ash*. Alexandria, VA: American Coal Ash Association.

15. Ferguson, P. J. (1990). *The Use of Class C Fly Ash in Highway Construction* (State Study 97, Design and Construction Report MSHD-RD-90-97). Jackson, MS: Mississippi State Highway Department.

16. Ferguson, P. J., & Avent, J. (1993). *Lime-Fly Ash-Aggregate Base Course with Class C Fly Ash* (State Study 97, Final Report MDOT-RD-93-97). Jackson, MS: Mississippi Department of Transportation.

17. George, K.P. (2001). *Soil Stabilization Field Trial* (State Study 133, Interim Report). University, MS: The University of Mississippi.

18. George, K. P., Personal communication, The University of Mississippi, University, MS, January 10, 2001.

19. George, K. P. (February, 2002). *Soil Stabilization Field Trial* (State Study 133, Interim Report II). University, MS: The University of Mississippi.

20. George, K. P. (2002). *Minimizing Cracking in Cement-Treated Materials for Improved Performance* (Research and Development Bulletin, RD123). Skokie, Il: Portland Cement Association.

21. George, K.P. and Uddin, W. (2000). *Subgrade Characterization for Highway Pavement Design* (State Study 131, Final Report, FHWA/MS-DOT-RD-00-131). University, MS: University of Mississippi.

22. Golden, D., Personal Communication, Combustion By-Products Use, Electric Power Research Institute, April 29, 2002.

23. Gomez, M. & Thompson, M. R. (1983). *Structural Coefficients and Thickness Equivalency Ratios* (FHWA/IL/UI-202). Urbana, IL: University of Illinois at Urbana-Champaign.

24. Graham, S. N., Letter from the Blain Companies (MT. Olive, MS) to Mr. Richard Owens and Mr. David Trevathon of the Mississippi Road Builders Association regarding the subject “Proposed Revisions to Lime and Lime-Fly Ash Specifications”, dated May 31, 2002.

25. Hadley, W. O. (1991). Material Characterization and Inherent Variation Analysis of Soil-Cement Field Cores. *Transportation Research Record No 1295,* 23-36.

26. Halsted, G. E., Personal Communication, Portland Cement Association, White, GA. 2002.

27. Huff, W., Personal Communication, Mississippi Department of Transportation, Jackson, MS, December 12, 2003.

28. Humphrey, H., Personal Communication, American Coal Ash Association, January 9, 2002.

29. Johnson, S. (2000). *FWD Data Processors Users Guide*. Jackson, MS Mississippi Department of Transportation; Unpublished manuscript.

30. Jordan, W. S., Mississippi State Highway Department Inter-Departmental Memorandum - Project No/Subject: 88-0103-00-804-10 Lime-Ash Stabilization with Escatawpa Conditioned Ash (laboratory test report with recommendation), January 5, 1989.

31. Little, D. N. (1995). *Stabilization of Pavement Subgrades & Base Courses with Lime*. Dubuque, IA: Kendall/Hunt.

32. Little, D. N., *Review of Recommendations on Lime-Fly Ash (LFA) Mixtures* (Draft of Technical Memorandum dated August 24, 2002). College Station, TX, Texas Transportation Institute.

33. Little, D. N., & Yusuf, F. S. (2001). *An Example Problem Illustrating the Application of the National Lime Association Mixture Design and Testing Protocol to ascertain Engineering Properties of Lime-Treated Subgrades for Mechanistic Pavement Design/Analysis* (Final Report FHWA/MS-DOT-RD-01-129). College Station, TX: Texas Transportation Institute.

34. Mateos, M. & Davidson, D. T. (1963). Compaction Characteristics of Soil-Lime-Fly Ash Mixtures. *Highway Research Record No 29,* 27-41.

35. Mitchell, J. K., & Dermatas, D. (1992). Clay Soil Heave Caused by Lime-Sulfate Reactions. *Innovations and Uses for Lime, ASTM STP 1135*, D.D. Walker, Jr., T.B. Hardy, D.C. Hoffman, and D.D. Stanley, Eds., American Society for Testing and Materials, Philadelphia, 41-64.

36. Mohseni, A. (1998). *LTPP Seasonal Asphalt Concrete (AC) Pavement Temperature Models* (Final Report FHWA-RD-97-103). Rockville, MD: Pavement Systems (PavSys).

37. National Cooperative Highway Research Program. (1976). *NCHRP Synthesis of Highway Practice 37 Lime-Fly Ash-Stabilized Bases and Subbases* (Project 20-5 FY'74 Topic 6-06). Washington D.C.: Author.

38. National Highway Institute. (1994). *Pavement Deflection Analysis - Participant Workbook* (Publication No FHWA-HI-94-021, NHI Course No 13127). Washington D.C., Federal Highway Administration.

39. National Lime Association. (1991). *Lime Stabilization Construction Manual* (10th ed.) Bulletin 326. Arlington, VA: Author.

40. Newcomb, D. (2002). *Perpetual Pavements - A Synthesis* (APA 101). Landum, MD: Asphalt Pavement Alliance.

41. Petry, T. M., & Little, D. N. (1992). Update on Sulfate-Induced Heave in Lime and Portland Cement Treated Clays: Determination of Potentially Problematic Sulfate Levels. *Transportation Research Record No 1362*, 51-55.

42. Portland Cement Association. (1984). *Soil-Cement Inspector's Manual* (2nd ed.). Skokie, IL: Author.

43. Portland Cement Association. (1992). *Soil-Cement Laboratory Handbook*. Skokie, IL: Author.

44. Rollings, M. P., & Rollings, R. S.Jr. (1996). *Geotechnical Materials in Construction*. New York: McGraw-Hill.

45. Scullion, T. (2001). *Field Investigation: Pre-Cracking of Soil-Cement Bases to Reduce Reflection Cracking*. A Paper Submitted for Presentation and Publication in the 2002 Annual Meeting of the Transportation Research Board, Washington D.C., August 2001.

46. Scullion, T. & Saarenketo, T. (1997). Using Suction and Dielectric Measurements as Performance Indicators for Aggregate Base Materials. *Transportation Research Record No 1577,* 37-44.

47. The Blain Companies, Letter from the Blain Companies (MT. Olive, MS) to Mr. Richard Owens and Mr. David Trevathon of the Mississippi Road Builders Association regarding the subject “Proposed Revisions to Lime-Fly Ash, Granular Base Density Specification”, dated May 28, 2002.

48. Timm, D., Birgisson, B. & Newcomb, D. (1999). WESLEA for Windows (Version 3.0) [Computer software]. Auburn University, AL: Authors.

49. Turner, M. (2003). [Hwy. 61 Penetrometer Results on Lime-Fly Ash]. Unpublished raw data.

50. Viskochil, R. K., Hardy, R. L., & Davis, D. T. (1958). Effect of Density on Strength of Lime-Fly Ash Stabilized Soil. *Highway Research Board Bulletin 183,* 5-15.

51. Von Quintus, H. & Killingsworth, B. (1998). *Analyses Relating to Pavement Material Characterizations and Their Effects on Pavement Performance* (Final Report FHWA-RD-97-085). Austin, TX: Brent Rauhut Engineering Inc.

52. Yoder, E. J., & Witczak, M. W. (1975). *Principles of Pavement Design* (2nd ed.). New York: John Wiley & Sons.

Appendix A

Codes for Visual Examination of Hot Mix Asphalt Cores

Code	Description
01	Intact core; excellent condition; suitable for testing.
02	Hairline cracks on the surface of the core; suitable for testing.
03	Cracks and/or voids visible along the side of the core; core is suitable for testing.
04	Badly cracked or damaged core; unsuitable for testing except for maximum specific gravity or asphalt content.
07	Core extremely damaged from sampling, shipping, or laboratory handling; unsuitable for testing.
09	Core consisted of two or more AC layers. Appropriate layer numbers to be assigned to each layer.
10	One or more asphaltic concrete layers have become separated due to sampling, shipping or laboratory handling; appropriate layer numbers to be assigned to each layer.
11	Segregation of coarse and fine aggregate is observed over 25% or more of the surface area of the core.
12	Voids in the matrix of the AC mixture are observed along the sides of the core.
13	Voids due to loss of coarse and fine aggregate are observed along the sides of the core.
14	Core is missing significant portions and cannot be considered a coherent cylindrical core; unsuitable for testing.
15	Coarse aggregate along the face of the core contains 50% or more of crushed materials with fractured faces.
16	Coarse aggregate along the face of the core is a mixture of uncrushed gravel and crushed gravel or stone.
17	More than 10% of the surface area of the core contains soft and deleterious aggregate particles or clay balls. Soft is defined as those aggregates that can be easily scratched with a knife.

18 Slight stripping. Stripping is defined as the displacement of asphalt cement film from the surface of the aggregate. Slight stripping is identified when the asphalt cement film has been displaced from and/or discoloration is observed on less than 25% of the surface area of the aggregate(s) showing signs of stripping.

19 Severe stripping. A loss of coarse or fine aggregate has been noted over 25% or more of the core face and the asphalt film has been displaced from 25% or more of the surface area of the aggregate(s).

20 Slight bleeding. 5% or less of the asphalt matrix portion of the core is in a non-hardened condition and exhibits shiny and sticky surface.

21 Severe bleeding. More than 5% of the asphalt matrix portion of the core is in a non-hardened condition and exhibits shiny and sticky surface.

99 Other comment (describe in a brief note).

Note: References to Codes 19 through 21 should also identify the specific layer(s) in which the problem is occurring.

Appendix B

Unconfined Compression Testing of LFA Cores

This appendix describes the procedure used to prepare and test LFA cores to determine core unconfined compressive strength (UCS).

The cored test specimens had an approximate four-inch diameter and variable height. The ends of each core were squared and trimmed to provide a smooth surface for uniform bearing against loading platens. This was accomplished by first precutting the ends with a saw (Figure B1) to create surfaces perpendicular to the axis of the core and then trimming with the aid of a trimming ring (Figure B2). The trimming ring could be adjusted along the length of the core to control the depth of final trimming with a beveled strike-off bar. These rings, 2 inches wide and fitted with an adjustable clamping device, were fabricated at a local metal shop. Intact cores having a minimum height of 4 inches following squaring and trimming were selected for UCS testing. This squaring and trimming method worked well for LFA stabilized sandy topping materials typically encountered in MDOT soil stabilization work and many of the finished cores could be tested without capping.

In some cases the soil used for LFA stabilization contained coarse sand and larger-sized particles, which required capping of the core ends to provide a uniform bearing surface for testing. In these cases the trimming ring was used as a casting ring. Since this ring can be adjusted along the length of a core, extremely thin plaster of Paris caps can be fabricated (Figure B3). For this study, the measured length of a core and core density calculations include the capping material. Given the minimal amount of material used to cap the core ends, it was assumed that minimal error would be introduced into the calculated core density values.

The prepared cores were soaked in water for 48 hours prior to UCS testing. Following soaking, the surface water was removed from the core with a towel and the core then mounted in the UCS testing device (Figure B4). Note that the core is enclosed in a plastic bag that is open at the top to allow the upper platen direct contact with the core. The bag was used to capture the entire core for oven drying following UCS testing. It was assumed that the thickness of the bottom of the bag would have negligible affect on the UCS measured for the core.

The loading device was equipped with both load and deformation measuring capability that allowed these measurements to be obtained every 30 seconds throughout the duration of testing. This enabled a plot of loading stress vs. strain to be developed for each core, and where a reasonable curve was obtained, an estimate of the LFA Young's Modulus obtained from the slope of that curve.

Following UCS testing all of the material from a given core was placed in a pan and oven dried. The computations associated with this testing included a determination of core density, an estimate of percent Proctor density where the proctor data was available, moisture content of core following the 48-hour soak period, core UCS as tested, and an equivalent Proctor UCS. Figure B5 is a sample of the data sheet used for recording data and the core stress vs. strain curve.

Figure B1. Saw Used to Trim Ends of LFA Cores

Figure B2. Trimming Ring Used with a Strike-Off Bar to Complete Trimming of LFA Cores

Figure B3. Example of Very Thin Plaster of Paris Caps

Figure B4. UCS Testing Device

Figure B5. Sample of UCS Data Sheet

LABORATORY MEASUREMENTS

County: Clarke

Station Number: 42+50

Laboratory ID Number: 19

Load/Strain data file name: CL4250

Date UCCS test performed: 15-Feb-01

Asphalt Core Rating: 3,10

LFA Core Rating: 1

Note: Measured length includes the thickness of the caps

L1 (in) =	5.305	D1 (in) =	4.004
L2 (in) =	5.298	D2 (in) =	4.001
L3 (in) =	5.291	D3 (in) =	4.001
Avg. L (in)	5.298	Avg. D (in)	4.002

Cross sectional area = 12.57894151

Failure Load, lbs. = 9801

Failure Stress, psi = 779.1593585

In order to get the "Proctor Equivalent" Strength of samples whose L/D ratio is different from 1.15, the strength determined should be divided by a correction factor.

L/D ratio = 1.3238381

Correction Factor = 0.952563718

"Proctor Equivalent" Strength, psi = 817.9604

Weight of pan, g = 15

Weight of core plus pan after performing UCCS test, g = 2261.3

Weight of core plus pan after oven-drying, g = 1984.9

Dry unit weight, lbs./cu.ft. = 112.6053

Standard Proctor Density, lbs./cu.ft. = 114.5

Figure B5 cont'd. Sample of UCS Data Sheet

Core density expressed as percent standard proctor density, % = 98.34524

Water Content after 48 hour soak, % = 14.03117

Load, lbs.		Stress, psi
0	0	0
19	0.0014	1.510460954
1204	0.0151	95.71552572
2882	0.0199	229.1130774
4673	0.024	371.4938968
6417	0.0285	510.1383128
8142	0.0345	647.2722678

	Stress	Strain
Upper pt.	650	0.0325
Lower pt.	30	0.0135

E = 32631.579 psi

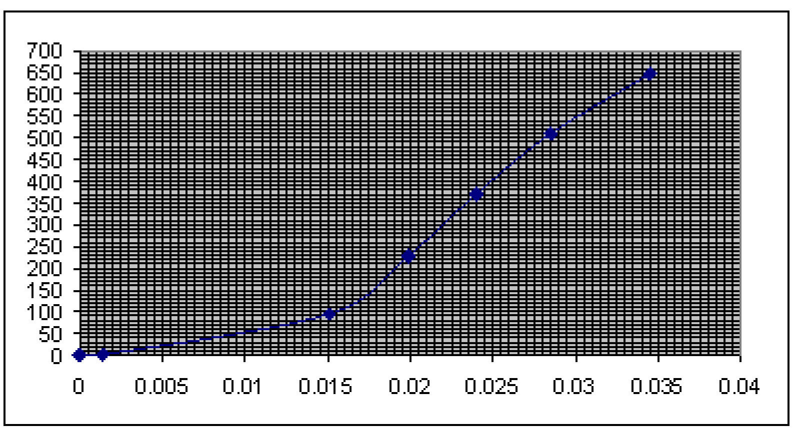
Channel:	CH17			
Description	HM2000, 10,000# PR			
Span	1055			
Zero	0			
F.S.D.	10000			
Offset	0			
Units:	lbs.			
Linear	Interval:	0:00:30		
G.Than	Reference:	CH18	Divisions:	5

Nu.	Date	Time	Seconds	Base	Divs	Value
1	FEB17/01	11:12:23	0	0	0	0
2	FEB17/01	11:12:29	6	6	2	19
3	FEB17/01	11:12:59	36	36	127	1204
4	FEB17/01	11:13:29	66	66	304	2882
5	FEB17/01	11:13:59	96	96	493	4673
6	FEB17/01	11:14:29	126	126	677	6417
7	FEB17/01	11:14:59	156	156	859	8142
8	FEB17/01	11:15:29	186	186	1016	9630
9	FEB17/01	11:15:41	198	198	16	152

Figure B5 cont'd. Sample of UCS Data Sheet

Channel: CH18
Description HM2000, Deflection
Span 2500
Zero 0
F.S.D. 0.25
Offset 0
Units: inches
Linear Interval: 0:00:30
G.Than Reference: CH18 Divisions: 5

Nu.	Date	Time	Seconds	Base	Divs	Value
1	FEB17/01	11:12:24	0	0	0	0
2	FEB17/01	11:12:29	5	5	14	0.0014
3	FEB17/01	11:12:59	35	35	151	0.0151
4	FEB17/01	11:13:29	65	65	199	0.0199
5	FEB17/01	11:13:59	95	95	240	0.024
6	FEB17/01	11:14:29	125	125	285	0.0285
7	FEB17/01	11:14:59	155	155	345	0.0345
8	FEB17/01	11:15:29	185	185	470	0.047
9	FEB17/01	11:15:41	197	197	1432	0.1432



Appendix C

Deflection Data for Newer Projects

County	Date Tested	FWD Test Location	HMA Mid Depth Temp (^{0}F)	Load (lbs.)	R1	R2	R3	R4 (mils)	R5	R6	R7
Bolivar	16-Jan-01	290+00	47.1	9737	6.91	5.75	4.43	3.36	2.5	1.93	1.54
		295+00	47.1	9713	7.24	5.96	4.56	3.41	2.51	1.94	1.54
		300+00	47.1	9657	7.71	6.44	4.84	3.59	2.63	2.01	1.61
		305+00	47.1	9836	7.36	6.2	4.75	3.54	2.58	1.97	1.53
		335+00	47.1	10028	5.57	4.76	3.93	3.21	2.55	2.05	1.63
		340+00	47.1	10152	7.41	6.36	4.97	3.75	2.74	2.05	1.53
		345+00	47.1	9972	4.66	3.93	3.2	2.58	2.04	1.65	1.35
		350+00	47.1	10036	5.02	4.1	3.26	2.54	1.94	1.52	1.22
		713+00	48.1	9505	6.52	5.6	4.54	3.57	2.74	2.14	1.69
		718+00	48.1	9516	6.13	5.17	4.09	3.16	2.4	1.9	1.51
		723+00	48.1	9452	7.59	6.64	5.47	4.37	3.39	2.69	2.14
		728+00	48.1	9532	6.41	5.59	4.64	3.77	3	2.43	1.96
		733+00	48.1	9545	6.63	5.78	4.81	3.9	3.07	2.49	2.01
		738+00	48.1	9692	6.27	5.51	4.62	3.81	3.06	2.49	2.04
		743+00	48.1	9705	7.05	6.13	4.97	3.94	3.05	2.42	1.96
		748+00	48.1	9688	6.83	5.97	4.94	3.98	3.13	2.49	1.99

Appendix C (cont'd)

County	Date Tested	FWD Test Location	HMA Mid Depth Temp (^{0}F)	Load (lbs.)	R1	R2	R3	R4 (mils)	R5	R6	R7
Clarke	10-Jan-01	39+50	48.1	9953	7.50	5.72	4.17	2.93	1.86	1.35	1.02
		40+00	48.1	9988	6.61	5.40	4.03	3.01	2.24	1.70	1.30
		40+50	48.1	9844	6.07	5.19	4.15	2.97	2.19	1.62	1.22
		41+00	48.1	9892	6.87	5.77	4.06	2.87	2.00	1.43	1.06
		41+50	48.1	9884	5.50	4.53	3.55	2.67	1.90	1.40	1.10
		42+00	48.1	9740	5.91	4.99	3.92	2.94	2.08	1.51	1.18
		42+50	48.1	9804	4.96	4.11	3.22	2.40	1.73	1.24	0.94
		43+00	48.1	9705	6.13	4.81	3.49	2.37	1.64	1.22	0.91
		752+00	48.1	10819	7.74	6.09	4.07	2.65	1.75	1.26	0.95
		755+00	48.1	10696	7.14	5.83	4.30	3.06	2.22	1.72	1.37
		758+00	48.1	10624	9.10	7.06	4.97	3.51	2.49	1.89	1.53
		761+00	48.1	10632	6.10	4.94	3.71	2.81	2.15	1.68	1.37
		764+00	48.1	10360	6.40	5.47	4.05	2.87	2.13	1.60	1.21
		767+00	48.1	10228	6.87	5.65	4.39	3.24	2.26	1.68	1.29
		770+00	48.1	9936	5.50	4.67	3.87	3.07	2.37	1.84	1.41
		773+00	48.1	10048	6.05	4.94	3.87	3.02	2.32	1.80	1.41
Smith	8-Jan-01	493+00	48.4	9956	5.73	4.35	3.31	2.50	1.85	1.40	1.12
		498+00	48.4	9820	5.27	4.33	3.35	2.57	1.92	1.49	1.16
		503+00	48.4	9904	4.92	3.91	2.90	2.15	1.59	1.20	0.98
		508+00	48.4	9908	5.85	4.83	3.65	2.70	1.96	1.49	1.13
		518+00	48.4	9604	3.51	2.60	1.88	1.36	0.96	0.69	0.50
		522+00	48.4	9556	6.02	4.86	3.72	2.80	2.01	1.54	1.20
		528+00	48.4	9449	4.56	3.67	2.73	2.02	1.50	1.19	0.97
		610+00	48.4	9852	5.10	3.97	2.72	1.82	1.24	0.97	0.81
		613+00	48.4	10068	4.52	3.52	2.43	1.66	1.13	0.85	0.67
		616+00	48.4	9932	5.07	3.84	2.63	1.74	1.15	0.84	0.66
		619+00	48.4	9913	5.93	4.79	3.58	2.60	1.89	1.48	1.19
		622+00	48.4	9817	4.95	3.94	2.73	1.80	1.20	0.86	0.67
		625+00	48.4	9713	6.78	5.20	3.39	2.02	1.23	0.87	0.70
		628+00	48.4	9740	6.76	5.28	3.69	2.50	1.68	1.23	0.98
		631+00	48.4	9649	9.21	6.63	4.14	2.55	1.63	1.16	0.93

Appendix C (cont'd)

County	Date Tested	FWD Test Location	HMA Mid Depth Temp (⁰F)	Load (lbs.)	R1	R2	R3	R4 (mils)	R5	R6	R7
Tippah	30-Jan-01	163+00	46.2	9844	4.55	3.48	2.41	1.62	1.14	0.85	0.70
		167+00	46.2	9860	5.72	4.45	3.16	2.24	1.63	1.24	1.00
		171+00	46.2	9964	6.35	5.35	4.20	3.20	2.39	1.82	1.39
		175+00	46.2	9964	5.87	5.01	4.13	3.32	2.55	1.99	1.53
		179+00	46.2	9992	6.80	5.80	4.62	3.61	2.74	2.12	1.68
		183+00	46.2	10056	5.55	4.31	3.02	2.12	1.54	1.20	0.99
		187+00	46.2	9873	11.99	10.46	8.46	6.54	4.84	3.56	2.63
		191+00	46.2	9808	3.80	2.97	2.23	1.66	1.26	0.98	0.81
Wilkinson	25-Jan-01	164+00	47.3	9449	6.76	5.61	4.28	3.25	2.43	1.88	1.52
		169+00	47.3	9676	8.15	6.30	4.34	2.96	2.11	1.63	1.33
		174+00	47.3	9281	6.62	5.45	4.11	3.08	2.28	1.70	1.31
		179+00	47.3	9249	4.87	3.91	3.14	2.47	1.91	1.48	1.15
		184+00	47.3	9172	9.45	7.28	4.76	3.03	2.00	1.45	1.19
		189+00	47.3	9284	5.15	4.26	3.09	2.17	1.49	1.11	0.87
		195+00	47.3	9777	4.97	4.22	3.52	2.72	2.03	1.57	1.25
		200+00	47.3	9777	9.11	7.33	5.20	3.57	2.41	1.69	1.30

Appendix D

Deflection Data for Older Projects

County	Date Tested	FWD Test Location	HMA Mid Depth Temp (^{0}F)	Load (lbs.)	R1	R2	R3	R4 (mils)	R5	R6	R7
Forrest/Perry	28-Feb-01	288+00	65.6	9377	8.46	6.25	4.51	3.24	2.34	1.73	1.31
		293+00	65.6	9380	7.09	5.30	3.88	2.92	2.15	1.59	1.22
		298+00	65.6	9356	7.94	6.69	4.33	2.91	2.07	1.59	1.31
		303+00	65.6	9292	7.99	6.34	4.55	3.23	2.27	1.72	1.31
		330+00	65.6	9084	14.29	9.49	5.56	3.31	2.00	1.30	0.99
		335+00	65.6	9068	13.15	8.79	4.51	2.61	1.68	1.15	0.88
		340+00	65.6	9236	7.52	5.76	3.81	2.48	1.60	1.14	0.89
		345+00	65.6	9212	8.35	6.73	4.46	2.80	1.81	1.30	0.99
		516+00	65.6	9249	10.80	7.28	4.54	2.95	1.99	1.38	1.07
		519+00	65.6	9444	5.41	4.11	2.97	2.10	1.51	1.12	0.89
		524+00	65.6	9372	9.57	6.69	4.14	2.69	1.89	1.45	1.15
		528+00	65.6	9308	9.07	6.13	3.70	2.30	1.52	1.15	0.89
		530+00	65.6	9388	8.05	6.09	3.99	2.68	1.87	1.40	1.11
		535+00	65.6	9356	8.80	7.08	5.00	3.43	2.37	1.68	1.27
		540+00	65.6	9212	10.83	8.08	5.26	3.31	2.19	1.53	1.19
		545+00	65.6	9241	8.52	6.19	3.95	2.63	1.93	1.55	1.30

Appendix D (cont'd)

County	Date Tested	FWD Test Location	HMA Mid Depth Temp (^{0}F)	Load (lbs.)	R1	R2	R3	R4 (mils)	R5	R6	R7
George	23-Jan-01	84+96	48.1	9468	12.35	8.85	5.13	2.93	1.83	1.38	1.10
		124+04	48.1	9420	7.94	6.07	4.25	2.76	1.65	1.04	0.79
		242+45	48.1	9473	27.67	12.52	4.78	2.72	1.94	1.55	1.23
		272+44	48.1	9428	6.98	5.25	3.43	2.25	1.51	1.20	0.91
		315+00	48.1	9108	12.56	9.50	6.10	3.92	2.56	1.73	1.25
		351+36	48.1	9105	15.50	9.59	5.54	3.62	2.57	1.96	1.53
		390+21	48.1	9124	21.88	11.82	5.06	3.17	2.34	1.81	1.45
		424+57	48.1	9308	9.13	5.96	3.05	1.63	0.94	0.65	0.51
Jones/Wayne	11-Jan-01	102+00	48.2	9260	6.89	6.02	4.72	3.59	2.64	1.94	1.44
		107+00	48.2	9377	6.61	4.73	3.25	2.26	1.54	1.07	0.81
		112+00	48.2	9225	9.77	7.32	4.91	3.22	2.07	1.41	1.04
		169+00	48.2	9316	11.28	8.95	6.31	4.37	2.94	2.07	1.47
		170+00	48.2	9329	11.02	8.42	5.66	3.68	2.33	1.57	1.19
		171+00	48.2	9257	9.91	7.82	5.00	3.36	2.30	1.63	1.24
		172+00	48.2	9452	12.59	9.52	5.70	3.52	2.35	1.57	1.20
		376+00	48.2	9452	6.46	5.17	3.54	2.37	1.57	1.13	0.91
		381+00	48.2	9681	7.71	5.90	3.83	2.37	1.46	1.05	0.83
		386+00	48.2	9609	7.55	6.13	4.28	2.85	1.85	1.25	0.96
		411+00	48.2	9580	8.02	6.52	4.45	2.95	1.99	1.39	1.07
		416+00	48.2	9513	10.35	7.40	4.50	2.59	1.48	0.95	0.74
		421+00	48.2	9548	9.70	7.80	5.50	3.76	2.42	1.62	1.09
		456+00	48.2	9556	6.71	5.44	3.87	2.74	1.93	1.45	1.17
		461+00	48.2	9716	10.61	8.20	5.31	3.39	2.19	1.56	1.15
		466+00	48.2	9668	7.18	5.76	3.92	2.54	1.68	1.23	0.97

Appendix D (cont'd)

County	Date Tested	FWD Test Location	HMA Mid Depth Temp (^{0}F)	Load (lbs.)	R1	R2	R3	R4 (mils)	R5	R6	R7
Yalobusha	31-Jan-01	340+04	46.6	9300	12.04	8.39	5.43	3.60	2.44	1.71	1.42
		341+96	46.6	9289	15.87	12.12	6.35	3.70	2.23	1.49	1.13
		360+04	46.6	9340	13.76	11.50	8.54	4.75	3.38	2.57	2.06
		361+96	46.6	9241	10.20	7.33	4.31	2.71	1.90	1.45	1.17
		402+04	46.6	9804	11.87	9.35	6.66	4.54	3.11	2.22	1.67
		403+96	46.6	9396	15.18	11.94	8.11	5.04	3.40	2.37	1.74
		420+04	46.6	9465	12.20	9.72	6.63	4.38	2.86	2.02	1.53
		421+96	46.6	9985	10.20	8.06	5.63	3.78	2.59	1.86	1.38
		455+04	46.6	9425	14.21	11.77	9.19	6.87	4.86	3.41	2.56
		456+96	46.6	9672	12.76	10.21	7.61	5.41	3.87	2.91	2.21
		488+04	46.6	9329	12.65	9.70	6.62	4.48	3.10	2.20	1.73
		488+96	46.6	9716	12.61	9.88	5.50	3.57	2.31	1.56	1.14
		492+04	46.6	9497	6.73	5.51	4.35	3.34	2.46	1.83	1.30
		493+96	46.6	9644	12.67	8.07	5.29	3.44	2.17	1.66	1.30
		507+04	46.6	9508	11.03	8.46	5.76	3.67	2.49	1.82	1.40
		508+96	46.6	9796	8.14	6.93	5.12	3.63	2.63	1.97	1.53

Appendix E

Supporting Data/Results of Computations for Determining LFA Structural Layer Coefficients for 5 Newer Projects

County	FWD Test Location	SN_{eff}	HMA E_{back} at Field Test Temp. (ksi)	HMA Lab. Equiv. at Field Test Temp. (ksi)	HMA Lab. Equivalent at 68^0F (ksi)	HMA a_1	In-Situ HMA Thickness (in.)	Granular Subbase a_3	In-Situ Granular Subbase Thickness (in.)	In-Situ LFA Thickness (in.)	In-Situ LFA a_2	Normalized LFA a_2
Bolivar	290+00	5.0874	1626	1183.8	507.5	0.458	7.75	0.09	0	9.50	0.162	0.257
	295+00	4.3070	1222	889.7	381.4	0.409	7.00	0.09	0	6.00	0.240	0.240
	300+00	4.1543	1517	1104.4	473.4	0.446	7.00	0.09	0	5.75	0.180	0.172
	305+00	4.3519	1667	1213.7	520.3	0.462	7.50	0.09	0	5.75	0.155	0.148
	340+00	4.6708	1922	1399.3	599.8	0.486	8.00	0.09	0	6.50	0.121	0.131
	345+00	5.7706	1631	1187.4	509.0	0.458	9.00	0.09	0	6.50	0.254	0.275
	350+00	5.4691	1486	1081.9	463.8	0.442	8.00	0.09	0	7.50	0.257	0.322
	713+00	4.9894	1803	1364.4	555.6	0.473	8.00	0.09	0	7.00	0.173	0.201
	718+00	4.8082	1442	1091.2	444.3	0.435	7.63	0.09	0	6.25	0.238	0.248
	723+00	4.9070	2010	1521.0	619.3	0.491	8.00	0.09	0	7.50	0.131	0.163
	728+00	5.2228	1880	1422.6	579.3	0.480	7.75	0.09	0	7.50	0.201	0.251
	733+00	5.0403	1767	1337.1	544.5	0.469	8.50	0.09	0	6.00	0.175	0.175
	738+00	5.4101	2298	1738.9	708.1	0.500	7.50	0.09	0	8.00	0.208	0.277
	743+00	4.6815	1773	1341.6	546.3	0.470	7.50	0.09	0	5.50	0.210	0.193
	748+00	5.1507	2117	1602.0	652.3	0.500	8.00	0.09	0	7.50	0.154	0.192

Appendix E (cont'd)

County	FWD Test Location	SN_{eff}	HMA E_{back} at Field Test Temp. (ksi)	HMA Lab. Equiv. at Field Test Temp. (ksi)	HMA Lab. Equivalent at 68^0F (ksi)	HMA a_1	In-Situ HMA Thickness (in.)	Granular Subbase a_3	In-Situ Granular Subbase Thickness (in.)	In-Situ LFA Thickness (in.)	In-Situ LFA a_2	Normalized LFA a_2
Clarke	40+00	4.0563	1328	966.8	414.5	0.423	5.13	0.09	0	6.63	0.284	0.314
	41+00	3.8616	2390	1740.0	745.9	0.500	5.25	0.09	0	6.00	0.206	0.206
	41+50	4.3414	1847	1344.7	576.4	0.479	5.25	0.09	0	6.00	0.305	0.305
	42+00	4.2133	2498	1818.7	779.6	0.500	5.25	0.09	0	5.50	0.289	0.265
	42+50	4.5179	2042	1486.7	637.3	0.496	5.75	0.09	0	5.75	0.290	0.278
	43+00	4.0111	1477	1075.3	461.0	0.441	6.00	0.09	0	6.00	0.227	0.227
	752+00	3.5547	2032	1479.4	634.2	0.495	5.00	0.09	0	5.75	0.188	0.180
	755+00	3.7527	1929	1404.4	602.0	0.486	4.50	0.09	0	5.00	0.313	0.261
	758+00	3.3872	1168	850.4	364.5	0.402	4.50	0.09	0	6.00	0.263	0.263
	761+00	4.3777	1130	822.7	352.7	0.396	6.25	0.09	0	6.50	0.292	0.317
	764+00	4.0869	2412	1756.1	752.8	0.500	5.25	0.09	0	5.50	0.266	0.244
	767+00	4.0570	1863	1356.4	581.4	0.480	5.25	0.09	0	6.00	0.256	0.256
	770+00	5.1943	1868	1360.0	583.0	0.481	6.75	0.09	0	6.50	0.300	0.325
	773+00	4.8655	1067	776.8	333.0	0.387	6.75	0.09	0	7.00	0.322	0.376
Smith	508+00	4.5341	1651	1188.5	517.3	0.461	7.00	0.09	0	6.75	0.194	0.218
	522+00	4.3795	1283	923.6	402.0	0.418	7.00	0.09	0	6.50	0.223	0.242
	528+00	4.7151	1430	1029.4	448.0	0.437	7.50	0.09	0	6.00	0.240	0.240
	610+00	4.5971	1216	875.4	381.0	0.409	7.38	0.09	0	7.13	0.222	0.263
	613+00	4.7266	1427	1027.3	447.1	0.436	7.75	0.09	0	6.25	0.215	0.224
	616+00	4.7566	1140	820.7	357.2	0.389	8.25	0.09	0	7.25	0.213	0.257
	619+00	4.8538	1521	1094.9	476.6	0.447	7.50	0.09	0	7.75	0.194	0.250
	622+00	4.4107	1724	1241.1	540.2	0.468	7.50	0.09	0	5.50	0.164	0.150
	625+00	3.6527	1134	816.3	355.3	0.398	7.25	0.09	0	4.75	0.162	0.128
	628+00	4.2200	968	696.8	303.3	0.371	8.75	0.09	0	6.00	0.162	0.162

Appendix E (cont'd)

County	FWD Test Location	SN_{eff}	HMA E_{back} at Field Test Temp. (ksi)	HMA Lab. Equiv. at Field Test Temp. (ksi)	HMA Lab. Equivalent at 68^0F (ksi)	HMA a_1	In-Situ HMA Thickness (in.)	Granular Subbase a_3	In-Situ Granular Subbase Thickness (in.)	In-Situ LFA Thickness (in.)	In-Situ LFA a_2	Normalized LFA a_2
Tippah	163+00	4.6321	1288	1010.6	392.5	0.414	7.50	0.09	0	6.50	0.235	0.191
	167+00	4.3300	1074	842.7	327.3	0.384	6.75	0.09	0	7.00	0.249	0.218
	171+00	5.1018	1896	1487.6	577.8	0.479	7.50	0.09	0	8.50	0.177	0.188
	175+00	5.2822	1962	1539.4	597.9	0.485	7.50	0.09	0	7.25	0.227	0.206
	179+00	4.9865	1673	1312.6	509.9	0.458	8.50	0.09	0	7.00	0.156	0.136
	183+00	4.6529	1205	945.4	367.2	0.403	7.00	0.09	0	8.25	0.222	0.229
	187+00	4.1109	1636	1283.6	498.6	0.455	7.75	0.09	0	7.25	0.081	0.074
Wilkinson	164+00	3.9085	1385	1039.8	427.8	0.429	5.50	0.09	0	5.50	0.282	0.258
	169+00	3.5248	1259	945.2	388.9	0.413	4.80	0.09	0	7.00	0.221	0.257
	174+00	3.8007	1435	1077.4	443.3	0.435	5.00	0.09	0	5.30	0.307	0.271
	179+00	5.2638	1156	867.9	357.1	0.398	5.50	0.09	0	9.00	0.341	0.512
	184+00	2.9106	1865	1400.2	576.1	0.479	5.00	0.09	0	4.30	0.120	0.086
	189+00	4.0365	2229	1673.5	688.5	0.500	6.00	0.09	0	4.30	0.241	0.173
	195+00	5.0696	2044	1534.6	631.4	0.494	6.00	0.09	0	5.50	0.383	0.351
	200+00	3.5874	1419	1065.4	438.3	0.433	6.00	0.09	0	6.50	0.152	0.165

Appendix F

Supporting Data/Results of Computations for Determining LFA Structural Layer Coefficients for 4 Older Projects

County	FWD Test Location	SN_{eff}	In-Situ HMA Thickness (in.)	LFA E_{back} (ksi)	LFA a_2 Based on Equation 6	In-Situ LFA Thickness (in.)	Granular Subbase a_3	In-Situ Granular Subbase Thickness (in.)	HMA a_1 Calculated from SN_{eff}	Revised SN_{eff}	Revised HMA a_1 Calculated from Revised SN_{eff}
Forrest/Perry	303+00	4.6029	7.50	216.4	0.188	5	0.09	4.00	0.440	3.8823	0.392
	330+00	3.0921	6.00	88.5	0.140	6	0.09	3.00	0.330	2.6673	0.304
	335+00	2.9964	5.50	52.8	0.118	6	0.09	3.00	0.367	2.5507	0.335
	340+00	4.2142	6.25	311.8	0.213	5	0.09	4.00	0.447	3.5831	0.403
	345+00	4.1377	6.25	49	0.115	6	0.09	3.00	0.508	3.6783	0.478
	516+00	3.4907	6.00	213.5	0.188	6.25	0.09	2.75	0.345	3.0689	0.316
	524+00	3.5996	5.50	196.9	0.183	6.25	0.09	2.75	0.402	3.1715	0.369
	528+00	3.5915	5.50	197.7	0.183	6.5	0.09	2.50	0.396	3.1856	0.363
	530+00	3.9471	5.25	312	0.213	6	0.09	3.00	0.457	3.4888	0.422
	535+00	4.1787	6.00	301.3	0.210	5	0.09	4.00	0.461	3.5757	0.421
	540+00	3.5653	5.50	246.8	0.197	5	0.09	4.00	0.404	3.0202	0.370
	545+00	3.8071	5.50	314.9	0.213	5	0.09	4.00	0.433	3.203	0.389
George	84+96	4.3418	4.50	138.1	0.162	5.5	0.09	12.50	0.516	2.5295	0.364
	242+45	2.9959	4.00	34.6	0.103	5	0.09	13.00	0.328	1.4125	0.225
	272+44	5.6508	5.25	519.9	0.252	5.5	0.09	12.50	0.598	3.4002	0.384
	315+00	4.5300	5.00	357.9	0.222	4.25	0.09	13.75	0.469	2.5916	0.329
	351+36	3.8929	4.00	236.2	0.194	4	0.09	14.00	0.464	1.9514	0.294
	390+21	3.2277	3.50	46.9	0.114	3.75	0.09	14.25	0.434	1.4324	0.288
	424+57	4.6458	4.00	527.8	0.253	4.25	0.09	13.75	0.583	2.2919	0.302

Appendix F (cont'd)

County	FWD Test Location	SN_{eff}	In-Situ HMA Thickness (in.)	LFA E_{back} (ksi)	LFA a_2 Based on Equation 6	In-Situ LFA Thickness (in.)	Granular Subbase a_3	In-Situ Granular Subbase Thickness (in.)	HMA a_1 Calculated from SN_{eff}	Revised SN_{eff}	Revised HMA a_1 Calculated from Revised SN_{eff}
Jones/Wayne	112+00	4.4650	5.50	105.4	0.149	8	0.09	6.50	0.489	3.4244	0.407
	169+00	4.6378	5.25	168.4	0.173	6.25	0.09	9.75	0.510	3.1561	0.395
	170+00	4.5297	5.25	143.8	0.165	6	0.09	10.00	0.503	3.0315	0.389
	171+00	4.5781	4.50	179.6	0.177	6.5	0.09	9.50	0.572	3.0875	0.430
	172+00	4.0203	4.00	32.3	0.100	6.5	0.09	9.50	0.628	2.6813	0.507
	376+00	5.5157	6.75	163.6	0.172	6	0.09	8.50	0.551	3.9464	0.432
	381+00	4.9258	6.00	161.8	0.171	6	0.09	8.50	0.522	3.5046	0.413
	386+00	5.1710	6.00	130.1	0.159	6	0.09	8.50	0.575	3.7236	0.461
	411+00	4.9914	5.75	159.2	0.170	6	0.09	8.50	0.557	3.5812	0.445
	416+00	4.1119	5.00	84	0.138	6.75	0.09	7.75	0.497	2.9746	0.409
	421+00	4.6994	5.50	96.2	0.144	7	0.09	7.50	0.548	3.4929	0.452
	456+00	5.5785	6.63	181.4	0.178	7	0.09	7.50	0.552	4.1352	0.436
	461+00	4.3904	5.50	82.8	0.137	7.5	0.09	7.00	0.497	3.3118	0.415
	466+00	5.2774	6.25	86.7	0.139	7.25	0.09	7.25	0.578	4.0012	0.479
Yalobusha	340+04	2.7112	4.75	295.7	0.209	6	0.09	0.00	0.307		
	341+96	2.2147	4.00	13.9	0.076	5.5	0.09	0.00	0.449		
	361+96	2.8949	4.00	53.1	0.118	7.5	0.09	0.00	0.502		
	402+04	3.2197	4.50	140.7	0.163	7.5	0.09	0.00	0.443		
	403+96	2.6228	3.50	124.7	0.157	6	0.09	0.00	0.480		
	420+04	2.9143	4.25	163.5	0.172	6	0.09	0.00	0.443		
	421+96	3.3480	5.00	171	0.174	6.8	0.09	0.00	0.433		
	455+04	3.2937	5.00	74.5	0.132	8	0.09	0.00	0.447		
	456+96	3.3987	4.50	117.4	0.154	9	0.09	0.00	0.447		
	488+04	2.9920	4.00	54.7	0.120	7.8	0.09	0.00	0.515		
	488+96	2.7410	4.25	124.7	0.157	7	0.09	0.00	0.386		
	493+96	2.6523	4.00	216.6	0.188	7	0.09	0.00	0.333		
	507+04	3.2060	4.50	109.3	0.150	7.8	0.09	0.00	0.452		

Appendix G

Material Property and Loading Inputs for the Layered Elastic Computer Programs WESLEA and Bisar

This appendix provides details regarding the selection of material property and loading inputs into the layered elastic computer programs WESLEA and Bisar for the calculation of flexural stresses in the HMA, LFA base, and LTS layers.

Poisson's Ratio Values

Poisson's Ratio values are stress dependent (Little, 1995, NCHRP No. 37, 1976). The construction loading condition imposes greater levels of stress on the LFA base and LTS layers than the in-service loading condition because in the latter case these layers are located deeper within the pavement structure under the HMA layers. The values included in Tables 28 and 30 take into account this stress dependency with the LTS and LFA base layers both assigned the value of 0.30 for the construction loading condition and 0.15 for the in-service loading condition.

Backcalculated Versus Laboratory Derived Modulus Values

The backcalculated modulus values obtained from the evaluation of FWD deflection data via the Modulus 5.1 computer program do not correspond to the modulus values obtained from laboratory testing of similar materials due to many factors. The question arises as to which modulus values should be entered into the programs, backcalculated or laboratory derived values? This question is asked because The AASHTO 2002 Pavement Design Guide requires the use of laboratory derived, or laboratory based modulus values as input into the design procedure, and in a couple of instances in the current study backcalculated values were converted to equivalent laboratory values. For example, in Chapter 2, discussion addressed the conversion of backcalculated subgrade modulus values to laboratory values to facilitate the estimation of a subgrade CBR value for each of the five newer projects. A second example requiring such a conversion is provided in Chapter 5. The HMA backcalculated modulus values were converted to equivalent laboratory values to facilitate the estimation of in-situ HMA structural layer coefficients from Figure 21. In both of these examples, a laboratory modulus value was correlated to a given parameter of interest; e.g., CBR or HMA structural layer coefficient; therefore, the backcalculated values were equated to laboratory values. The use of such a conversion introduces more variability in a given modulus input value, which results in less reliability of the output from the given program or correlation. However, the use of such a conversion is often necessary to proceed with a study in a somewhat timely and cost-effective manner.

For this study it is assumed that the resulting output of flexural stresses from the programs are more representative of the actual flexural stresses occurring in the pavement structure by the use of backcalculated values that have not been corrected to equivalent laboratory values. The backcalculated values better model the response of the in-situ pavement materials under traffic loading than laboratory values for two reasons. First, the backcalculated values are obtained from deflection data that was acquired using the FWD. The FWD utilizes a load/loading rate which approximates an 18-kip single axle load moving between 40 and 50 mph (George and Uddin, 2000) and provides a somewhat reasonable loading model of what is actually being placed on a given pavement. Second, the effects of cracks within each of the pavement layers, as well as the variability in the materials comprising these layers, are incorporated in the backcalculated values, thus providing more realistic, or effective, input values for modulus relative to laboratory derived values.

Subgrade Modulus Values

The uncorrected backcalculated subgrade modulus values obtained from the test locations within each of the five newer projects were used to evaluate a unique 10th percentile subgrade modulus value for each of these projects. For the short-term construction loading condition, an uncorrected backcalculated subgrade modulus value of 12,000 psi was selected based on the average of the three lowest 10th percentile values corresponding to the Bolivar, Tippah, and Wilkinson County projects. While not substantiated based on soil Atterberg limit and gradation tests, it is assumed that these three projects would have had a LTS had such a stabilized layer been required at the time of construction. The Clarke and Smith County projects were not included in this average because it was assumed, based on backcalculation results, that these two projects were constructed on a more granular subgrade than the other three projects and would require LFA as the subgrade stabilizing agent.

For the in-service loading condition each of the five newer projects was considered separately with the uncorrected unique 10th percentile subgrade soil modulus value, corresponding to the given project being utilized in the programs.

LTS Modulus Values

The stiffness of the chemically stabilized subgrade layer has a significant impact on the level of flexural stresses developed at the bottom of the overlying base layer. No lime or LFA stabilized subgrade layer was included in any of the projects evaluated in the current study; therefore, no backcalculated modulus values for chemically stabilized subgrade material are available from these projects for input into the programs. Estimated values for this material were obtained from backcalculated data that was available from previously conducted studies.

Limited data was available for the construction loading condition. In a previous study one Mississippi pavement, Hwy. 302 in Marshall County, provided backcalculated LTS modulus data for test sections and a control section after about four months of field curing (George, 2001). This period of curing was during the particularly hot summer of 2000. The high curing temperatures are reflected in the average backcalculated modulus of these sections of 73,350 psi. The coefficient of variation was 81 percent. The 10^{th} percentile value for this data is a negative value, which has no physical significance other than it indicates that a conservative estimate for the LTS backcalculated modulus of these sections would be equivalent to that of the untreated subgrade material.

High levels of field curing with the concurrent development of the modulus values observed in the referenced project do not constitute the typical case for spring and fall construction. This observation, in conjunction with the observed variability in the referenced project, indicates that a much lower value for consideration as an upper limit of LTS modulus should be used for the construction loading condition. The use of

40,000 psi for the lower curve in Figure 46 is an estimated value. Consistently achieving this high of a value is probably not a realistic expectation, but its use does serve to illustrate that this degree of stiffness is required in the chemically stabilized subgrade layer to reduce to an acceptable level the flexural stresses in the overlying base course.

For the in-service loading condition backcalculated LTS modulus data was obtained from a previous study that involved the evaluation of three Mississippi pavements. These included US Hwy. 45 N in Kemper County, US Hwy. 82 W in Lowndes County, US Hwy. 61 N in Washington County, and US HWY. 82 E in Washington County. The focus of that study was on the characterization of the LTS layer that was included in each of those pavement structures (Little and Yusuf, 2001). These pavements had been in service for between 15 and 20 years at the time of evaluation. The average LTS backcalculated modulus of each of these pavements varied from 61,600 psi to 357,650 psi. This large range in average values is indicative of the high variability in the in-situ engineering properties of this chemically stabilized material, making the selection of a unique representative input value difficult for use in the programs.

Little (1995) indicates that for pavement design the in-situ LTS modulus can be expected to vary between 20,000 and 70,000 psi for fine-grained soils. Based on this reference and the data from the four Mississippi pavements, an assumed LTS value of 40,000 psi was selected for the in-service loading condition.

LFA Modulus Values

Uncorrected backcalculated modulus values for the LFA stabilized soil from the five newer projects were either used directly, or were derived via Equation 3, to characterize this material in the programs for both loading conditions. Details of limits used for this material in the various graphs illustrating the two loading conditions are included in Chapter 11.

HMA Modulus Values

HMA is a viscoelastic material. FWD testing was conducted predominantly during the winter months for this study; therefore, a correction to the backcalculated HMA modulus values was necessary to reflect the lower modulus values typical of this material during the hot summer months. This correction was limited to a consideration for temperature and does not include the additional consideration of rate of loading as was the case in Chapter 5 for obtaining HMA in-situ structural layer coefficients from backcalculated modulus values. The consideration for rate of loading was required because the backcalculated modulus values had to be converted to equivalent laboratory modulus values to use the relationship. For the current discussion, no conversion is required because, as previously discussed, the backcalculated modulus more closely models the modulus that the pavement would experience under traffic loading. Tables G1 through G7 are used to facilitate the discussion on the methodology employed to estimate a unique HMA modulus value for input into the programs from the backcalculated HMA modulus values of all five of the newer projects.

Tables G1 through G5 include air temperature data for each of the five newer projects and the results of calculations estimating in-situ HMA temperatures with depth from these atmospheric temperatures. The daily average maximum and minimum air temperatures for each month of the year were obtained from a weather station located in close proximity to a given project. For example a weather station at Cleveland, Mississippi, was the closest station to the Bolivar County project, and air temperature data was obtained from that weather station for the values shown in Table G1. These monthly average maximum/minimum values were then averaged together to obtain an average monthly air temperature as shown in this table.

An LTPP High Pavement Temperature Model was used to perform the conversion of summer air temperatures to summer HMA pavement temperatures (Mohseni, 1998). The intent of this model is to evaluate the higher HMA temperatures expected at a given project locale to facilitate the selection of a binder for that project. However, an average maximum/minimum monthly air temperature was used for input into this model instead of the average maximum monthly air temperature. The reason for this is that the objective for this study is to obtain the most representative, or average, HMA temperature that the pavement will experience over the entire month, as opposed to just the higher end of the temperature range. Rutting and the selection of an appropriate grade of binder to resist this rutting, are not the issues here; the issue is the selection of an average structural response of the pavement to loading over a period of time which is modeled in the programs as an average HMA modulus for that period of time.

The use of this model is restricted to air temperatures in excess of 20 ^{0}C. The average of the maximum/minimum air temperatures for all five of the newer project locations exceeds this value for the months of May through September; therefore, these are the months included for the evaluation. The remaining seven months of the year are associated with relatively cooler air temperatures and the corresponding HMA modulus values are greater than those of the hot summer months. Greater HMA modulus values result in less tensile stress development at the bottom of the LFA layer, so the inclusion of these seven months would result in a less conservative estimate for the HMA modulus value.

As observed in Table G1, the temperature of the HMA is calculated for each 25-mm increment of depth, and then averaged to obtain a monthly average HMA temperature for the given project. The LTPP High Pavement Temperature Model used for these computations is shown in the reference as:

T_{pav} = 54.32 + (0.78 * T_{air}) – (0.0025 Lat^2) – (15.14 * $\log_{10}$ * (H + 25)) Equation G1

Where: T_{pav} = High AC pavement temperature below the surface, ^{0}C

Note: For the current study the average monthly daily temperature was obtained instead of the high temperature

T_{air} = High air temperature, ^{0}C

Note: For the current study the average monthly daily temperature was entered instead of the average high temperature

Lat = Latitude of the section, degrees

H = Depth to surface, mm

An average monthly HMA temperature of 37.8 ^{0}C is calculated for the Bolivar County project for the month of May, with this average increasing to a high of 42.5 ^{0}C for the month of July. The same process was repeated for the remaining four newer project locations as shown in Tables G2 through G5.

The average monthly HMA temperatures are then used as shown in Table G6 to convert the HMA backcalculated modulus values, shown adjacent to the column entitled

“Stations” in this table, to monthly values for each test location in each of the five newer project test sections for the five hottest months of the year. This conversion is accomplished by use of Equation 5 for each test location within each project for each of the five months. Note that the average monthly HMA modulus values for the Bolivar County project vary within a relatively narrow range from a high of 282,361 psi for the month of May to a low of 231,506 psi for the month of July. These values are significantly lower than those observed during the cool winter months. The same process was repeated for the remaining four newer project locations, but the results of these computations have not been included in this appendix.

An average monthly composite HMA modulus value was obtained by using the calculations from all five of the newer projects for each of the five months considered in this evaluation. These values are shown in Table G7. Since the project locations were fairly well dispersed throughout the state, a representative HMA modulus value for the entire state can be selected by obtaining the average of these five composite monthly averages. A value of 235,300 psi was selected as representative of the HMA modulus in Mississippi for the five hottest months of the year.

Input for Loading

The loading was modeled in the WESLEA program as a tandem axle with dual wheels for both the construction and in-service considerations. The assumptions include a 34,000-pound load evenly distributed among the eight tires with a tire pressure of 110 psi. Unless otherwise noted, full bonding is assumed between the pavement layers. The spacing between the axles is 54 inches and 13.5 inches between the dual wheels.

The flexural stresses for each combination of material property and pavement geometry considered were checked at the following locations:

- At the top of the LFA layer midway between two of the transversely oriented wheels
- At the bottom of the LFA layer midway between two of the transversely oriented wheels
- At the top of the LFA layer directly beneath one of the wheels
- At the bottom of the LFA layer directly beneath one of the wheels
- At the top of the LFA layer at the center of the area bounded by the four tires
- At the bottom of the LFA layer at the center of the area bounded by the four tires
- At the top of the LFA layer midway between two of the longitudinally oriented tires
- At the bottom of the LFA layer midway between two of the longitudinally oriented tires

Table G1. Variations in HMA temperature with depth versus months of the year for Bolivar County

Month	Daily Avg Air Temp. ^{0}F Maximum	Daily Avg Air Temp. ^{0}F Minimum	Daily Avg Air Temp. ^{0}F	Daily Avg Air Temp. ^{0}C	Latitude	Depth, mm	HMA Temp. at Given Depth, ^{0}C
May	82.3	62.1	72.2	22.3	33.7	25	43.2
	82.3	62.1	72.2	22.3	33.7	50	40.5
	82.3	62.1	72.2	22.3	33.7	75	38.6
	82.3	62.1	72.2	22.3	33.7	100	37.2
	82.3	62.1	72.2	22.3	33.7	125	36.0
	82.3	62.1	72.2	22.3	33.7	150	34.9
	82.3	62.1	72.2	22.3	33.7	175	34.1
						Average	37.8
June	89.2	70.1	79.7	26.5	33.7	25	46.4
	89.2	70.1	79.7	26.5	33.7	50	43.7
	89.2	70.1	79.7	26.5	33.7	75	41.8
	89.2	70.1	79.7	26.5	33.7	100	40.4
	89.2	70.1	79.7	26.5	33.7	125	39.2
	89.2	70.1	79.7	26.5	33.7	150	38.2
	89.2	70.1	79.7	26.5	33.7	175	37.3
						Average	41.0
July	92.7	73.5	83.1	28.4	33.7	25	47.9
	92.7	73.5	83.1	28.4	33.7	50	45.2
	92.7	73.5	83.1	28.4	33.7	75	43.3
	92.7	73.5	83.1	28.4	33.7	100	41.9
	92.7	73.5	83.1	28.4	33.7	125	40.7
	92.7	73.5	83.1	28.4	33.7	150	39.7
	92.7	73.5	83.1	28.4	33.7	175	38.8
						Average	42.5
August	91.8	71.5	81.7	27.6	33.7	25	47.3
	91.8	71.5	81.7	27.6	33.7	50	44.6
	91.8	71.5	81.7	27.6	33.7	75	42.7
	91.8	71.5	81.7	27.6	33.7	100	41.2
	91.8	71.5	81.7	27.6	33.7	125	40.0
	91.8	71.5	81.7	27.6	33.7	150	39.0
	91.8	71.5	81.7	27.6	33.7	175	38.2
						Average	41.9

Daily max./min. air temperature values obtained from the Cleveland, Mississippi, reporting station.

Table G1 Continued. Variations in HMA temperature with depth versus months of the year for Bolivar County

Month	Daily Avg Air Temp. ^0F Maximum	Daily Avg Air Temp. ^0F Minimum	Daily Avg Air Temp. ^0F	Daily Avg Air Temp. ^0C	Latitude	Depth, mm	HMA Temp. at Given Depth, ^0C
September	86.1	65.1	75.6	24.2	33.7	25	44.7
	86.1	65.1	75.6	24.2	33.7	50	42.0
	86.1	65.1	75.6	24.2	33.7	75	40.1
	86.1	65.1	75.6	24.2	33.7	100	38.6
	86.1	65.1	75.6	24.2	33.7	125	37.4
	86.1	65.1	75.6	24.2	33.7	150	36.4
	86.1	65.1	75.6	24.2	33.7	175	35.5
						Average	39.2

Daily max./min. air temperature values obtained from the Cleveland, Mississippi, reporting station.

Table G2. Variations in HMA temperature with depth versus months of the year for Clarke County

Month	Daily Avg Air Temp. ^0F Maximum	Daily Avg Air Temp. ^0F Minimum	Daily Avg Air Temp. ^0F	Daily Avg Air Temp. ^0C	Latitude	Depth, mm	Temperature at Given Depth, ^0C
May	82.4	58.6	70.5	21.4	32	25	42.7
	82.4	58.6	70.5	21.4	32	50	40.1
	82.4	58.6	70.5	21.4	32	75	38.2
	82.4	58.6	70.5	21.4	32	100	36.7
	82.4	58.6	70.5	21.4	32	125	35.5
						Average	38.6
June	88.2	65.7	77.0	25.0	32	25	45.5
	88.2	65.7	77.0	25.0	32	50	42.8
	88.2	65.7	77.0	25.0	32	75	41.0
	88.2	65.7	77.0	25.0	32	100	39.5
	88.2	65.7	77.0	25.0	32	125	38.3
						Average	41.4
July	90.1	69.0	79.6	26.4	32	25	46.6
	90.1	69.0	79.6	26.4	32	50	44.0
	90.1	69.0	79.6	26.4	32	75	42.1
	90.1	69.0	79.6	26.4	32	100	40.6
	90.1	69.0	79.6	26.4	32	125	39.4
						Average	42.5
August	89.8	67.9	78.9	26.0	32	25	46.3
	89.8	67.9	78.9	26.0	32	50	43.7
	89.8	67.9	78.9	26.0	32	75	41.8
	89.8	67.9	78.9	26.0	32	100	40.3
	89.8	67.9	78.9	26.0	32	125	39.1
						Average	42.2
September	85.2	62.5	73.9	23.3	32	25	44.2
	85.2	62.5	73.9	23.3	32	50	41.5
	85.2	62.5	73.9	23.3	32	75	39.6
	85.2	62.5	73.9	23.3	32	100	38.1
	85.2	62.5	73.9	23.3	32	125	36.9
						Average	40.1

Daily max./min. air temperature values obtained from the Quitman, Mississippi, reporting station.

Table G3. Variations in HMA temperature with depth versus months of the year for Smith County

Month	Daily Avg Air Temp. ^0F Maximum	Daily Avg Air Temp. ^0F Minimum	Daily Avg Air Temp. ^0F	Daily Avg Air Temp. ^0C	Latitude	Depth, mm	Temperature at Given Depth, ^0C
May	83.5	58.7	71.1	21.7	32	25	43.0
	83.5	58.7	71.1	21.7	32	50	40.3
	83.5	58.7	71.1	21.7	32	75	38.4
	83.5	58.7	71.1	21.7	32	100	37.0
	83.5	58.7	71.1	21.7	32	125	35.8
	83.5	58.7	71.1	21.7	32	150	34.7
	83.5	58.7	71.1	21.7	32	175	33.9
						Average	37.6
June	89.7	66.6	78.2	25.6	32	25	46.0
	89.7	66.6	78.2	25.6	32	50	43.4
	89.7	66.6	78.2	25.6	32	75	41.5
	89.7	66.6	78.2	25.6	32	100	40.0
	89.7	66.6	78.2	25.6	32	125	38.8
	89.7	66.6	78.2	25.6	32	150	37.8
	89.7	66.6	78.2	25.6	32	175	36.9
						Average	40.6
July	92.1	69.8	81.0	27.2	32	25	47.2
	92.1	69.8	81.0	27.2	32	50	44.6
	92.1	69.8	81.0	27.2	32	75	42.7
	92.1	69.8	81.0	27.2	32	100	41.2
	92.1	69.8	81.0	27.2	32	125	40.0
	92.1	69.8	81.0	27.2	32	150	39.0
	92.1	69.8	81.0	27.2	32	175	38.1
						Average	41.8
August	91.5	69.2	80.4	26.9	32	25	47.0
	91.5	69.2	80.4	26.9	32	50	44.3
	91.5	69.2	80.4	26.9	32	75	42.4
	91.5	69.2	80.4	26.9	32	100	41.0
	91.5	69.2	80.4	26.9	32	125	39.8
	91.5	69.2	80.4	26.9	32	150	38.8
	91.5	69.2	80.4	26.9	32	175	37.9
						Average	41.6

Daily max./min. air temperature values are from the Bay Springs, Mississippi, reporting station.

Table G3 Continued. Variations in HMA temperature with depth versus months of the year for Smith County

Month	Daily Avg Air Temp. ^{0}F Maximum	Daily Avg Air Temp. ^{0}F Minimum	Daily Avg Air Temp. ^{0}F	Daily Avg Air Temp. ^{0}C	Latitude	Depth, mm	Temperature at Given Depth, ^{0}C
September	87.2	64.2	75.7	24.3	32	25	45.0
	87.2	64.2	75.7	24.3	32	50	42.3
	87.2	64.2	75.7	24.3	32	75	40.4
	87.2	64.2	75.7	24.3	32	100	38.9
	87.2	64.2	75.7	24.3	32	125	37.8
	87.2	64.2	75.7	24.3	32	150	36.7
	87.2	64.2	75.7	24.3	32	175	35.9
						Average	39.6

Daily max./min. air temperature values are from the Bay Springs, Mississippi, reporting station.

Table G4. Variations in HMA temperature with depth versus months of the year for Tippah County

Month	Daily Avg Air Temp. ^{0}F Maximum	Daily Avg Air Temp. ^{0}F Minimum	Daily Avg Air Temp. ^{0}F	Daily Avg Air Temp. ^{0}C	Latitude	Depth, mm	Temperature at Given Depth, ^{0}C
May	81.0	61.2	71.1	21.7	34.9	25	42.5
	81.0	61.2	71.1	21.7	34.9	50	39.8
	81.0	61.2	71.1	21.7	34.9	75	37.9
	81.0	61.2	71.1	21.7	34.9	100	36.5
	81.0	61.2	71.1	21.7	34.9	125	35.3
	81.0	61.2	71.1	21.7	34.9	150	34.3
	81.0	61.2	71.1	21.7	34.9	175	33.4
						Average	37.1
June	89.3	68.9	79.1	26.2	34.9	25	46.0
	89.3	68.9	79.1	26.2	34.9	50	43.3
	89.3	68.9	79.1	26.2	34.9	75	41.4
	89.3	68.9	79.1	26.2	34.9	100	39.9
	89.3	68.9	79.1	26.2	34.9	125	38.7
	89.3	68.9	79.1	26.2	34.9	150	37.7
	89.3	68.9	79.1	26.2	34.9	175	36.8
						Average	40.6
July	92.3	72.9	82.6	28.1	34.9	25	47.5
	92.3	72.9	82.6	28.1	34.9	50	44.8
	92.3	72.9	82.6	28.1	34.9	75	42.9
	92.3	72.9	82.6	28.1	34.9	100	41.5
	92.3	72.9	82.6	28.1	34.9	125	40.3
	92.3	72.9	82.6	28.1	34.9	150	39.2
	92.3	72.9	82.6	28.1	34.9	175	38.4
						Average	42.1
August	90.8	71.1	81.0	27.2	34.9	25	46.8
	90.8	71.1	81.0	27.2	34.9	50	44.1
	90.8	71.1	81.0	27.2	34.9	75	42.2
	90.8	71.1	81.0	27.2	34.9	100	40.7
	90.8	71.1	81.0	27.2	34.9	125	39.5
	90.8	71.1	81.0	27.2	34.9	150	38.5
	90.8	71.1	81.0	27.2	34.9	175	37.6
						Average	41.4

Daily max./min. air temperature values are from the Memphis, Tennessee, reporting station.

Table G4 Continued. Variations in HMA temperature with depth versus months of the year for Tippah County

Month	Daily Avg Air Temp. ^{0}F Maximum	Daily Avg Air Temp. ^{0}F Minimum	Daily Avg Air Temp. ^{0}F	Daily Avg Air Temp. ^{0}C	Latitude	Depth, mm	Temperature at Given Depth, ^{0}C
September	83.9	64.5	74.2	23.4	34.9	25	43.8
	83.9	64.5	74.2	23.4	34.9	50	41.2
	83.9	64.5	74.2	23.4	34.9	75	39.3
	83.9	64.5	74.2	23.4	34.9	100	37.8
	83.9	64.5	74.2	23.4	34.9	125	36.6
	83.9	64.5	74.2	23.4	34.9	150	35.6
	83.9	64.5	74.2	23.4	34.9	175	34.7
						Average	38.4

Table G5. Variations in HMA temperature with depth versus months of the year for Wilkinson County

Month	Daily Avg Air Temp. ^{0}F Maximum	Daily Avg Air Temp. ^{0}F Minimum	Daily Avg Air Temp. ^{0}F	Daily Avg Air Temp. ^{0}C	Latitude	Depth, mm	Temperature at Given Depth, ^{0}C
May	83.1	62.7	72.9	22.7	31.2	25	43.9
	83.1	62.7	72.9	22.7	31.2	50	41.2
	83.1	62.7	72.9	22.7	31.2	75	39.3
	83.1	62.7	72.9	22.7	31.2	100	37.9
	83.1	62.7	72.9	22.7	31.2	125	36.7
						Average	39.8
June	88.6	69.1	78.9	26.0	31.2	25	46.5
	88.6	69.1	78.9	26.0	31.2	50	43.8
	88.6	69.1	78.9	26.0	31.2	75	41.9
	88.6	69.1	78.9	26.0	31.2	100	40.4
	88.6	69.1	78.9	26.0	31.2	125	39.2
						Average	42.4
July	91.0	72.1	81.6	27.5	31.2	25	47.6
	91.0	72.1	81.6	27.5	31.2	50	45.0
	91.0	72.1	81.6	27.5	31.2	75	43.1
	91.0	72.1	81.6	27.5	31.2	100	41.6
	91.0	72.1	81.6	27.5	31.2	125	40.4
						Average	43.5
August	90.8	71.3	81.1	27.3	31.2	25	47.4
	90.8	71.3	81.1	27.3	31.2	50	44.8
	90.8	71.3	81.1	27.3	31.2	75	42.9
	90.8	71.3	81.1	27.3	31.2	100	41.4
	90.8	71.3	81.1	27.3	31.2	125	40.2
						Average	43.3
September	86.7	66.1	76.4	24.7	31.2	25	45.4
	86.7	66.1	76.4	24.7	31.2	50	42.7
	86.7	66.1	76.4	24.7	31.2	75	40.8
	86.7	66.1	76.4	24.7	31.2	100	39.4
	86.7	66.1	76.4	24.7	31.2	125	38.2
						Average	41.3

Daily max./min. air temperature values are from the Natchez, Mississippi, reporting station.

Table G6. Average Monthly HMA Modulus Values for Bolivar County

Station	HMA $E_{(back)}$ Modulus (ksi)	HMA $E_{(back)}$ Modulus (MPa)	HMA Mid-depth Temp (°F) at time of FWD test	HMA Mid-depth Temp (°C) at time of FWD test	May 37.8	June 41	July 42.5	August 41.9	September 39.2
					Average Monthly HMA Temperature, °C				
					HMA Modulus Per Month, psi				
290+00	1626	11211.3	48.1	8.9	271079	236387	222256	227762	255074
295+00	1222	8425.7	48.1	8.9	203726	177654	167033	171171	191698
300+00	1517	10459.7	48.1	8.9	252907	220541	207357	212493	237975
305+00	1667	11494.0	48.1	8.9	277914	242348	227860	233505	261506
340+00	1922	13252.2	48.1	8.9	320427	279420	262715	269224	301508
345+00	1631	11245.7	48.1	8.9	271913	237114	222939	228462	255859
350+00	1486	10246.0	48.1	8.9	247739	216034	203119	208151	233112
713+00	1803	12431.7	47.1	8.4	285530	248989	234104	239903	268672
718+00	1442	9942.6	47.1	8.4	228361	199136	187231	191869	214878
723+00	2010	13859.0	47.1	8.4	318311	277575	260981	267446	299518
728+00	1880	12962.6	47.1	8.4	297724	259622	244101	250149	280146
733+00	1767	12183.5	47.1	8.4	279829	244017	229429	235113	263307
738+00	2298	15844.7	47.1	8.4	363920	317347	298375	305767	342434
743+00	1773	12224.8	47.1	8.4	280779	244846	230208	235911	264201
748+00	2117	14596.7	47.1	8.4	335256	292351	274874	281683	315462
				Average	282361	246225	231506	237241	265690

Table G7. Determination of HMA Modulus for Input Into WESLEA and Bisar

County	Station	HMA $E_{(back)}$ Modulus (ksi)	HMA $E_{(back)}$ Modulus (MPa)	HMA Mid-depth Temp (°F) at time of FWD test	HMA Mid-depth Temp (°C) at time of FWD test	May	June	July	August	September
Bolivar	290+00	1626	11211.3	48.1	8.9	271079	236387	222256	227762	255074
	295+00	1222	8425.7	48.1	8.9	203726	177654	167033	171171	191698
	300+00	1517	10459.7	48.1	8.9	252907	220541	207357	212493	237975
	305+00	1667	11494.0	48.1	8.9	277914	242348	227860	233505	261506
	340+00	1922	13252.2	48.1	8.9	320427	279420	262715	269224	301508
	345+00	1631	11245.7	48.1	8.9	271913	237114	222939	228462	255859
	350+00	1486	10246.0	48.1	8.9	247739	216034	203119	208151	233112
	713+00	1803	12431.7	47.1	8.4	285530	248989	234104	239903	268672
	718+00	1442	9942.6	47.1	8.4	228361	199136	187231	191869	214878
	723+00	2010	13859.0	47.1	8.4	318311	277575	260981	267446	299518
	728+00	1880	12962.6	47.1	8.4	297724	259622	244101	250149	280146
	733+00	1767	12183.5	47.1	8.4	279829	244017	229429	235113	263307
	738+00	2298	15844.7	47.1	8.4	363920	317347	298375	305767	342434
	743+00	1773	12224.8	47.1	8.4	280779	244846	230208	235911	264201
	748+00	2117	14596.7	47.1	8.4	335256	292351	274874	281683	315462
Clarke	40+00	1328	9156.6	48.1	8.9	213792	189887	181522	183751	200491
	41+00	2390	16479.1	48.1	8.9	384760	341740	326686	330697	360823
	41+50	1847	12735.1	48.1	8.9	297344	264098	252464	255564	278845
	42+00	2498	17223.7	48.1	8.9	402147	357183	341448	345641	377128
	42+50	2042	14079.6	48.1	8.9	328737	291981	279118	282545	308285
	43+00	1477	10183.9	48.1	8.9	237779	211193	201889	204368	222986
	752+00	2032	14010.6	48.1	8.9	327127	290551	277751	281162	306775
	755+00	1929	13300.5	48.1	8.9	310545	275823	263672	266910	291225
	758+00	1168	8053.4	48.1	8.9	188034	167009	159652	161613	176335
	761+00	1130	7791.4	48.1	8.9	181916	161576	154458	156355	170598
	764+00	2412	16630.7	48.1	8.9	388302	344886	329693	333741	364144
	767+00	1863	12845.4	48.1	8.9	299920	266386	254651	257778	281261
	770+00	1868	12879.9	48.1	8.9	300725	267101	255334	258470	282016
	773+00	1067	7357.0	48.1	8.9	171774	152568	145847	147638	161087
Smith	508+00	1651	11383.6	48.4	9.1	281941	247805	235773	237720	258503
	522+00	1283	8846.3	48.4	9.1	219098	192571	183220	184734	200884
	528+00	1430	9859.9	48.4	9.1	244201	214634	204213	205900	223901
	610+00	1216	8384.3	48.4	9.1	207656	182514	173652	175087	190394
	613+00	1427	9839.2	48.4	9.1	243688	214184	203784	205468	223431
	616+00	1140	7860.3	48.4	9.1	194677	171107	162799	164144	178494
	619+00	1521	10487.3	48.4	9.1	259741	228293	217208	219002	238149
	622+00	1724	11887.0	48.4	9.1	294407	258762	246198	248231	269933
	625+00	1134	7818.9	48.4	9.1	193653	170207	161942	163280	177555
	628+00	968	6674.4	48.4	9.1	165305	145291	138236	139378	151563

Table G7 Continued. Determination of HMA Modulus for Input Into WESLEA and Bisar

County	Station	HMA $E_{(back)}$ Modulus (ksi)	HMA $E_{(back)}$ Modulus (MPa)	HMA Mid-depth Temp (°F) at time of FWD test	HMA Mid-depth Temp (°C) at time of FWD test	May	June	July	August	September
Tippah	163+00	1288	8880.8	46.2	7.9	200697	172527	162146	166878	189526
	167+00	1074	7405.2	46.2	7.9	167351	143862	135205	139151	158036
	171+00	1896	13072.9	46.2	7.9	295436	253969	238686	245652	278991
	175+00	1962	13528.0	46.2	7.9	305720	262809	246995	254203	288703
	179+00	1673	11535.3	46.2	7.9	260688	224098	210613	216759	246177
	183+00	1205	8308.5	46.2	7.9	187764	161409	151697	156124	177312
	187+00	1636	11280.2	46.2	7.9	254922	219142	205955	211966	240733
Wilkinson	164+00	1385	9549.6	47.3	8.5	203259	182443	174536	175937	190865
	169+00	1259	8680.8	47.3	8.5	184767	165846	158657	159931	173501
	174+00	1435	9894.3	47.3	8.5	210596	189030	180836	182288	197756
	179+00	1156	7970.6	47.3	8.5	169651	152278	145677	146847	159307
	184+00	1865	12859.2	47.3	8.5	273702	245673	235024	236911	257013
	189+00	2229	15369.0	47.3	8.5	327122	293622	280895	283150	307176
	195+00	2044	14093.4	47.3	8.5	299972	269252	257582	259650	281681
	200+00	1419	9784.0	47.3	8.5	208248	186922	178820	180256	195551
			Monthly average for all 5 Counties			261529	230067	218280	221916	244861

May	261529	
June	230067	
July	218280	
August	221917	
September	244861	
Overall Average	235331	psi