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EVALUATION AND FIELD VERIFICATION OF STRENGTH AND STRUCTURAL IMPROVEMENT OF CHEMICALLY STABILIZED SUBGRADE SOIL

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16. Abstract

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The type and amount of chemical additive is dependent on the purpose or function of the treated material (i.e., improved physical properties or improved strength) and selection is based on accepted or standardized procedures. Questions then arise with regard to chemically treated subgrade soils about the rate of development and ultimate value of improvement. The purpose of this research is to develop relationships between rate of development and magnitude of strength (or physical property) improvement for chemically treated subgrade soils. The purpose of this interim report is to summarize the results of recent research into any existing relationships and current experience with chemically treated soils as used in the AASHTO-MEPDG.

No active or recently completed research involved with characterizing Level 2 and 3 design input parameters for AASHTO-MEPDG was found. The closest research topics involved the use of field and laboratory data to calculate structural layer coefficients. The research confirmed that when chemically treated subgrade soil layers were included in the structural design of the pavement, the layer coefficients were conservative and the resulting pavement layer thicknesses were reduced, as was the construction cost. Most of the recent research evaluation of performance of various chemical additives. Most of the evaluations were between different chemicals, i.e. lime vs. fly ash vs. CKD, etc. The research reaffirmed that chemicals such as lime, fly ash, CKD, and cement generally perform well in most fine-grained soils as long as the type and amount of the chemical are selected to meet the purpose of the treated layer and problem soils are identified and considered in the selection process.

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LIST OF SYMBOLS

E = Modulus of Elasticity, psi or ksi

M_r = Resilient Modulus, psi or ksi

q_u = Unconfined Compressive Strength, psi

MR = Modulus of Rupture

R = Resistance Value

CBR = California Bearing Ratio

P200 = Percent Passing U.S. No. 200 sieve

PI = Plasticity Index

DCP = Dynamic Cone Penetration

μ = Poisson's Ratio

a_2, a_3 = Structural Coefficients

SN = Structural Number

SUMMARY

Often subgrade soils exhibit properties, particularly strength and/or volume change properties that limit their performance as a support element for pavements. Typical problems include shrink-swell, settlement, collapse, erosion or simply insufficient strength. A common approach to subgrade soil support or stability problems involves chemical modification or stabilization with additives such as lime (hydrated or quick), fly ash (Class C from lignite coal), cement kiln dust (CKD) or Portland cement. Other additives are available, but this group constitutes the major products or by-products used on roadway construction in Oklahoma.

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Chapter 1

INTRODUCTION

Background

Often subgrade soils exhibit properties, particularly strength and/or volume change properties, that limit their performance as a support element for pavements. Typical problems include shrink-swell, settlement, collapse, erosion or simply insufficient strength. A common approach to subgrade soil support or stability problems involves chemical modification or stabilization (FHWA) with additives such as lime (hydrated or quick), fly ash (Class C from lignite coal), cement kiln dust or Portland cement. Other additives are available, but this group constitutes the major products or by-products used in roadway construction in Oklahoma.

The type and amount of chemical additive is typically selected using standardized procedures (ASTM, ODOT). In cases where the subgrade soil's strength is important in designing pavement thickness and predicting performance, ASTM D4609 test protocol is the best approach for selecting the type and defining the amount of soil additive.

Questions arise with regard to chemically treated subgrade soils about the rate of development and ultimate magnitude of improvement (strength increase or volume change stability) on construction projects. In other words, is the improvement response of field constructed soil layers the same as the laboratory mix design response? Potential differences between laboratory and field improvement responses may be the result of one or more of the following sources:

1. Normal variability of natural soils.
2. Variability (number and lateral extent) of soil types (i.e., assumption that one percentage of additive "fits" all the soils on the project).
3. Variability of field construction process (i.e., components, quality of workmanship).
4. Influence of climate

Typically, once the treated subgrade soil is compacted, the strength or volume change stability improvement is "assumed" to equal the laboratory mix design test results. The pavement is then designed using structural numbers based on historical, and sometimes, limited data reflecting the actual influence of the treated subgrade soil layer on the thickness and performance of the pavement. Often the strength improvement of the treated subgrade soil is simply ignored in the pavement design equation. Limited information is available on the rate of development and comparative magnitude of strength improvement of stabilized subgrade soils.

The "Guide for Mechanistic-Emperical Design of New and Rehabilitated Pavement Structures (MEPDG)" (AASHTO) uses an hierarchical (level) system for selecting or determining design inputs for pavement design. The system is based "on the philosophy that the level of engineering effort exerted in the pavement design process should be consistent with the relative importance, size, and cost of the design project". The three levels used in the MEPDG procedure are;

1. Level 1 is the most current implementable procedure available, normally involving comprehensive laboratory or field tests.
2. Level 2 requires that inputs are estimated through correlations with other material properties measured in the laboratory or field.
3. Level 3 requires an estimate of the most appropriate design input value of the material property based on experience with little or no testing.

This new or more organized approach to pavement design further highlights the need for a better understanding of the rate of development and comparative magnitude of strength improvement for stabilized subgrade soils, especially for Level 2 and 3 design inputs.

Objectives of Proposed Research

The purpose of the proposed research is to develop relationships between the rate of development and magnitude of strength improvement for chemically stabilized subgrade soils and pavement design input parameters. These relationships can be used to confirm and/or adjust pavement design input parameters currently recommended in the MEPDG to reflect Oklahoma soils, commonly used chemical additives, and pavement design experience.

The major objectives of the proposed research are:

1. Review existing correlations between chemically treated soils and AASHTO-MEPDG design input parameters.

2. Select roadway construction projects in grading and drainage stages of construction which represent different subgrade soil types and chemical additives used.
3. Collect representative soil samples from construction project locations for classification, quality control, and engineering property testing.
4. Collect representative chemically treated soil samples from construction project locations for engineering property testing.
5. Following compaction and acceptance of the chemically treated project locations conduct time sequenced field (tests) evaluation of strength and stiffness.
6. Using established time rate of development and maximum level of strength gain relationships, compare to previous/existing design input parameters correlations or experience-based lower limits and accept or adjust parameters accordingly.

The purpose of the interim report is to summarize the results of Objective 1.

Chapter 2

REVIEW OF RELEVANT EXPERIENCES

Background

Chemically treated subgrade soils provide support to pavements and enhance the performance of the pavement system. Chemically treated soils influence performance by one or both of the following methods:

1. Improved physical properties such as reduced plasticity, reduced moisture-holding capacity, reduced shrink-swell response, and improved stability. This occurs at "lower" percentages of the chemical additive and is generally referred to as chemical modification of the soil. Basic chemical reactions between the additive and soil, include cation (typically calcium or magnesium) exchange and agglomeration/flocculation.
2. Improved strength of the treated soil, which obviously increases the common strength characterization parameters, i.e., unconfined compressive strength, resilient modulus, and stiffness. This occurs at "higher" percentages of the chemical additive and is generally referred to as chemical stabilization of the soil. Basic chemical reactions between the additive and soil include the same cation exchange and agglomeration/flocculation that occurs in soil modification plus the development of pozzolanic reaction products that "stick" the soil particles together. The level of development of pozzolanic reaction products is dependent on the amount of chemical additive, time, pH, and temperature. The pozzolanic reaction products are strong, durable, and provide long-termed performance when properly selected and constructed.

Alternatively, chemical additives and the influence they have on subgrade soils may be characterized as non-cementitious or cementitious, which is similar in context to the modification versus stabilization categorization. Non-cementitious chemical additives provide a source of cations which interact with the soil minerals in the form of cation exchange and agglomeration/flocculation and any pozzolanic reaction products are limited because the necessary chemicals to form the reaction products must be "provided" by the soil mineral. Lime is considered a non-cementitious chemical. Cementitious chemical additives provide both a source of cations for modification reactions and a source of the "building blocks" for pozzalonic reaction products, specifically silica and alumina. In other words, at appropriate percentages of cementitious chemical additives, sufficient cations, silica, and alumina are available to modify the soils physico-chemical properties as well as stabilize the soils with pozzolanic reaction products. Flyash (class C), cement kiln dust (CKD), and Portland cement are cementitious chemicals.

Mechanical-Empirical Pavement Design Guide

In the Mechanical Empirical Pavement Design Guide (MEPDG), "the chemically stabilized materials group consists of lean concrete, cement stabilized, open grade cement stabilized, soil cement, lime-cement-flyash, and lime treated materials". Lean concrete, cement stabilized, open graded cement stabilized, and soil cement are high quality (i.e., high strength) materials consisting of mixtures of natural granular or graded coarse and/or fine aggregates and cement. Mix design procedures can confidently define the amount of strength improvement. Lime, cement (more often CKD) and flyash or combinations of additives are more commonly used in fine-grained soils, so whether strength improvement occurs or how much occurs is more difficult to determine. In the MEPDG, fine-grained soils treated with cementitious chemical additives would be considered in the "chemically stabilized materials group", while fine-grained soils treated with non-cementitious chemical additives would be considered in the "unbound granular and subgrade materials group".

According to the MEPDG, input parameters for design for chemically stabilized materials group are:

- Elastic Modulus
- Resilient Modulus
- Modulus of Rupture
- Poisson's Ratio
- Thermal Conductivity
- Heat Capacity

For unbound granular and subgrade materials input parameters to design include:

- Resilient Modulus
- Poisson's Ratio
- Classification and other properties (for Climate Model)

The MEPDG uses more sophisticated models for analyzing the performance of chemically stabilized materials than for unbound granular and subgrade materials. A more detailed explanation of the different models appears in the MEPDG.

Since the purpose of the review of relevant experiences is to summarize laboratory and field research experiences of DOT's with regard to performance of chemical additives for modification/stabilization of subgrade soils, it's helpful to understand how the various measured laboratory and field properties relate to the required input parameters for the MEPDG. The input parameters discussed in subsequent paragraphs will emphasis Level 2 and 3 inputs since confirmation of these parameters for typical Oklahoma soils is a part of the purpose of this research project.

Level 2 correlations with other material properties or modulus values for the chemically stabilized material group are shown in Table 1 (MEPDG Table 2.2.42)

Table 1. Models/Relationships used for determining Level 2 E or M_r (from MEPDG)

Chemically Stabilized Material	Recommended Relationships*
Lean concrete[1] Cement treated aggregate[1]	$E=57000\sqrt{f_c^1}$ (*18*) where, E is the modulus of elasticity, psi; f^1_c=compressive strength, psi, tested in accordance with AASHTO T22
Open graded cement stabilized	No correlation are available
Soil cement[2]	$M_r=1200*q_u$(*18*) where, E is the modulus of elasticity, psi; q_u=unconfined compressive strength, psi, tested in accordance with ASTM D 1633, "Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders"
Lime-cement-flyash[2]	$E=500\div q_u$(*19*) where, E is the modulus of elasticity, psi; q_u=unconfined compressive strength, psi tested in accordance with ASTM C 593, "Standard Specifications for Fly Ash and Other Pozzolans for use with Lime"
Lime stabilized soils[2]	$M_r=0.124q_u+9.98$(*17*) where, M_r=resilient modulus, ksi q_u=unconfined compressive strength, psi, tested in accordance with ASTM D 5102, "Standard Test Method for Unconfirmed Compressive Strength of Compacted Soil-Lime Mixtures"

[1]Compressive strength f_c can be determined using AASHTO T22.
[2]Unconfined compressive strength q_u can be determined using the MDTP.

Level 3 typical modulus values for the chemically stabilized material group, are shown in Table 2, (MEPDG Table 2.2.43) and 3 (MEPDG Table 2.2.44)

Table 2. Summary of typical modulus values for chemically stabilized materials. (from MEPDG)

Chemically Stabilized Material	E or M_r Range, psi	E or M_r Typical, psi
Lean concrete	1,500,000 to 2,500,000	2,000,000
Cement stabilized aggregate	700000, to 1,500,000	1,000,000
Open graded cement stabilized aggregate	--	750,000
Soil cement	50,000 to 1,000,000	500,000
Lime-cement-flyash	500,000 to 2,000,000	1,500,000
Lime stabilized soils*	30,000 to 60,000	45,000

*For reactive soils with 25 percent passing No. 200 sieve and PI of at least 10.

Table 3. Summary of typical modulus values for deteriorated chemically stabilized materials. (from MEPDG)

Chemically Stabilized Material	Deteriorated M_r Typical, psi
Lean concrete	300,000
Cement stabilized aggregate	100,000
Open graded cement stabilized	50,000
Soil cement	25,000
Lime-cement-flyash	40,000
Lime stabilized soils	15,000

Level 2 correlations with other material properties for modulus of rupture (flexural strength) for the chemically stabilized material group are shown in Table 4 (MEPDG Table 2.2.46)

Table 4. Relationship between unconfined compressive strength and flexural strength for chemically stabilized materials. (from MEPDG)

Chemically Stabilized Material	Test Protocol	Typical MR, psi
Lean concrete	AASHTO T 22	MR can be conservatively estimated as being 20 percent of the q_u (*15*)
Cement treated aggregate		
Open graded cement stabilized aggregate	Not available	--
Soil cement	ASTM D 1633	MR can be conservatively estimated as being 20 percent of the q_u (*15*)
Lime-cement-flyash	ASTM C 593	
Lime stabilized soils	ASTM D 5102	

Level 3 typical modulus of rupture values for the chemically stabilized material group are shown in Table 5 (MEPDG Table 2.2.47)

Table 5. Typical flexural strength (MR) values for chemically stabilized materials. (from MEPDG)

Chemically Stabilized Material	Typical MR, psi
Lean concrete	450
Cement stabilized aggregate	200
Open graded cement stabilized	200
Soil cement	100
Lime-cement-flyash	150
Lime stabilized soils	25

Recommended ranges of Poisson's ratio for the chemically stabilized material group are shown in Table 6 (MEPDG Table 2.2.48)

Table 6. Recommended ranges of Poisson's ratio for chemically stabilized materials. (from MEPDG)

Material	Poisson's Ratio
Cement Stabilized Aggregate (including Lean Cement)	0.1 to 0.2
Soil cement	0.15 to 0.35
Lime-Fly Ash Materials	0.1 to 0.15
Lime stabilized soils	0.15 to 0.2

Thermal conductivity and heat capacity are inputs to the climate model used to estimate temperature and moisture profiles in the pavement structure and subgrade. More details on estimating thermal properties are available in the MEPDG.

Level 2 correlations with other material properties for resilient modulus for the unbound granular and subgrade materials group are shown in Table 7 (MEPDG Table 2.2.50)

Table 7. Models relating material index and strength properties to M_r. (from MEPDG)

Strength/Index Property	Model	Comments	Test Standard
CBR	$M_r=2555(CBR)^{0.64}$(TRL) M_r, psi	CBR –California Bearing Ratio, percent	AASHTO T 193, "The California Bearing Ratio"
R-value	$M_r=1155+555R$(*20*) Mr, psi	R=R-value	AASHTO T 190, "Resistance R-Value and Expansion Pressure of Compacted Soils"
AASHTO layer coefficient	$M_r=30000\left(\frac{a_i}{0.14}\right)$(*20*) M_r, psi	a_i=AASHTO layer coefficient	AASHTO Guide for the Design of Pavement Structures
PI and gradation*	$CBR=\frac{75}{1+0.728(wPI)}$ (see Appendix CC)	wPI=P200*PI P200-percent passing No. 200 sieve size PI=plasticity index, percent	AASHTO T 27. "Sieve Analysis of Coarse and Fine Aggregates" AASHTO T 90. "Determining the Plastic Limit and Plasticity Index of Soils"
DCP*	$CBR=\frac{292}{DCP^{1.12}}$	CBR=California Bearing Ratio, percent DCP=DCP index, mm/blow	ASTM D 6951, "Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Application"

*Estimates of CBR are used to estimate M_r

Level 3 typical resilient modulus values for the unbound granular and subgrade material group are shown in Table 8 (MEPDG Table 2.2.51)

Table 8. Typical resilient modulus values for unbound granular and subgrade materials (modulus at optimum moisture content) (Appendix CC). (from MEPDG)

Material Classification	M_r Range	Typical M_r
A-1-a	38,500-42,000	40,000
A-1-b	35,500-40,000	38,000
A-2-4	28,000-37,500	32,000
A-2-5	24,000-33,000	28,000
A-2-6	21,500-31,000	26,000
A-2-7	21,500-28,000	24,000
A-3	24,500-35,500	29,000
A-4	21,500-29,000	24,000
A-5	17,000-25,500	20,000
A-6	13,500-24,000	17,000
A-7-5	8,000-17,500	12,000
A-7-6	5,000-13,500	8,000
CH	5,000-13,500	8,000
MH	8,000-17,500	11,500
CL	13,500-24,000	17,000
ML	17,000-25,500	20,000
SW	28,000-37,500	32,000
SP	24,000-33,000	28,000
SW-SC	21,500-31,000	25,500
SW-SM	24,000-33,000	28,000
SP-SC	21,500-31,000	25,500
SP-SM	24,000-33,000	28,000
SC	21,500-28,000	24,000
SM	28,000-37,500	32,000
GW	39,500-42,000	41,000
GP	28,000-40,000	34,500
GW-GC	28,000-40,000	34,500
GW-GM	35,500-40,500	38,500
GP-GC	28,000-39,000	34,000
GP-GM	31,000-40,000	36,000
GC	24,000-37,500	31,000
GM	33,000-42,000	38,500

Significant caution is advised when selecting resilient modulus values from Table 8 because the values are "very approximate". Levels 1 or 2 are strongly preferred.

The MEPDG does not provide correlations with other material properties for Poisson's ratio (i.e. Level 2) for unbound granular and subgrade material group. It recommends using "local knowledge and experience." Level 3 typical Poisson's ratio values for unbound granular and subgrade materials group are shown in Table 9 (MEPDG Table 2.2.52).

Table 9. Typical Poisson's ratio values for unbound granular and subgrade materials. (From MEPDG)

Materials Description	**μ Range**	**μ Typical**
Clay (saturated)	0.4—0.5	0.45
Clay (unsaturated)	0.1—0.3	0.2
Sandy clay	0.2—0.3	0.25
Silt	0.3—0.35	0.325
Dense sand	0.2—0.4	0.3
Coarse-grained sand	0.15	0.15
Fine-grained sand	0.25	0.25
Bedrock	0.1—0.4	0.25

Classification and other properties are used in the climate model to estimate temperature and moisture profiles in the pavement structure and subgrade. More details on measuring or estimating the required input parameters are available in the MEPDG.

The correlations with other material properties and typical values presented in the MEPDG are referred to a number of sources and appear to be compilations and summaries of relationships and limiting values presented in or interpreted from the various sources. In other words, the information in the tables does not appear, in the form presented, in any of the reference sources. As far as research into calibrating or characterizing input parameter for stabilized materials or unbound subgrade soils is concerned, members of the MEPDG Implementation Group (contacted and replied by email) were not aware of any research directed at the input parameters. Several research projects involving laboratory and field performance of chemical additives along with some comparative evaluations of mechanistic empirical pavement designs including (or omitting) MEPDG design input were obtained using various information search websites as well as federal and state DOT websites. The remainder of this chapter summarizes several of those documents.

Contribution of Treated Soil Layers in Pavement

Qubain, Seksisnky, and Li evaluated the influence a lime-stabilized subgrade soil layer had on the resultant pavement thickness for a project involving widening and reconstruction of a section of the Pennsylvania Turnpike. The subgrade soils were fairly uniform medium to stiff clays. The effects of lime stabilized layers were incorporated in the pavement design (AASHTOWare DARWin 3.01 computer program) using three options:

1. Using an appropriate resilient modulus for the lime stabilized layer
2. Using a CBR of 15 for the treated layer
3. Considered lime treated layer as subbase and assigned it a structural number.

Laboratory test result on the subgrade soil are presented in Table 10 (Qubain, et al)

Table 10. Laboratory Testing Results (from Qubain, et al).

Sample	USCS	Water Content (%)	Liquid Limit (%)	Plasticity Index (%)	Dry Density (g/cm^3)	Optimum Water (%)	Swell (%)	CBR (%)	M_R (kPa)
Natural	CL	16.9	37	13	1.73	17.7	1.4	8	64,000
Lime-treated	--	16.4	34	10	1.73	15.1	1	37	250,000

Using consistent pavement design input parameters for each of the three cases, the resulting pavement thicknesses are summarized in Table 11 (Qubain, et al).

Table 11. Pavement Comparisons for Different Design Approaches

Pavement Layers	Layer Thickness (mm) Lime-Stabilized Subgrade			Layer Thickness (mm) Non-Stabilized Subgrade		
	M_R= 165000kPa	CBR = 15	Structural Coefficient = 0.11	Preliminary CBR = 5	M_R = 60000 kPa	CBR = 8
AC	130	130	130	130	130	130
BCBC	130	130	170	330	300	250
ATPBC	100	100	100	100	100	100
PennDOT 2A	130	150	150	200	200	200
Total Thickness	490	510	550	760	730	680

The lime stabilized layer CBR of 15 should be compared to the untreated subgrade CBR of 8 (Column 2 vs. Column 6) for a reduction in thickness of 170 mm. A more dramatic difference occurs between resilient modulus inputs (Column 1 vs. Column 5) or 220 mm. Over the length of the project the authors estimated a 20% saving in cost or about $4.5

million by using lime and incorporating the change in the subgrade properties in the design of the pavement.

Shafee Yusut, Little, Sarkor completed a research project for the Mississippi DOT to assess material properties and performance of lime-treated subgrades. Soil samples were collected from four project locations and field tests were conducted on the pavements, each of which had a lime-treated subgrade layer. Laboratory tests (Unconfined Compression and Resilient Modulus Tests) were conducted on stabilized and unstabilized soil specimens. Field tests (Falling Weight Deflectometer and Dynamic Cone Penetrometer) were conducted on the lime-treated layers and the untreated subgrade soil beneath the lime-treated layer. Comparisons between stabilized and unstabilized soils were made for both laboratory and field tests. Table 12 summarizes the laboratory test results and Table 13 summarizes the field test results.

Table 12. Unconfined Compressive Strength and Resilient Moduli of Mississippi Soils (from Yusut, et al)

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

Note: NT = Not Tested

Table 13. Results for LTS and Unstabilized Subgrade for Mississippi Pavements. (From Yusut, et al)

Pavements	GPT Results		FWD Results			DCP Results			
	Layer Thickness (mm)	Dielectric Constant	Subgrade Moduli (MPa)	LTS Moduli (MPa)	Ratio (LTS/ Subgrade)	Layer Thickness (mm)	Subgrade CBR	LTS CBR	Ratio (LTS/ Subgrade)
US 61 N Washington County	HMA: 250 LTS: 150	9 – 13	79	425	4.38	LTS: 125	15	500	33.33
US 82 E Washington County	HMA: 325 LTS: 150	6 – 8	119	2466	20.72	LTS: 125	12	150	12.50
US 82 W Lowndes County	HMA: 363 LTS: 150	7 – 10	123	1350	10.98	LTS: 150	4	47	11.75
US 45 N Kemper County	HMA: 250 LTS: 250	No Data	125	1482	11.86	LTS: 275	10	133	13.30

The ratios of stabilized to unstabilized properties, with one exception for laboratory testing, all show moderate to significant improvements in measured properties when lime was used.

A subsequent paper by Mallela, Quontas, and Smith accumulates experience on the use of lime to treat highway subgrades and correlates the information with the MEPDG. This is a good reference which expands some of the topics included in the MEPDG relating to treatment of subgrade soils with lime.

Performance of Chemically Treated Subgrade Soils

Over the past seven years, a number of laboratory and field research studies have evaluated short- and long-term performance of chemical additives for modifying and stabilizing subgrade soils. The following paragraphs describe the reported results of these studies.

Kentucky Experiences

The Kentucky Transportation Cabinet (KYTC) and the University of Kentucky Transportation Center (UKTC) (Hopkins, et al) undertook a research study of 20 selected roadway sections on 14 roadway projects that had been stabilized with lime or cement to improve the subgrade soils' properties and pavement performance. The research involved a forensic evaluation of the stabilized subgrade soils in which borings through the pavement and subgrade were conducted to obtain soil samples and run in situ CBR tests. Falling Weight Deflector (FWD) tests were run on the pavements prior to the borings. Some of the "significant findings and recommendations" are summarized as follows:

1. Measured in situ CBR values at the 85th percentile value for various soil additives were:

Lime Kiln Dust (LKD)	24
Hydrated Lime	27
Hydrated Lime/Portland Cement	32
Portland Cement	59
Atmospheric Fluidized Bed Composition (AFBC) Ash	9
Untreated	2

   Age of pavement test sections varied between 8 and 15 years. The study concluded "that chemically treated subgrades are very durable and long lasting".

2. Structural credit of chemically stabilized soil subgrades in the design of pavements was established based on a relationship published by AASHTO relating CBR and structural layer coefficient, a_3. Using the 85th percentile CBR values from above, the structural coefficient, a_3, of subgrades mixed with LKD, hydrated lime, hydrated lime/Portland cement, Portland cement, and AFBC ash were 0.1, 0.106, 0.11, 0.13 and 0.08, respectively. For pavement test sections , "backcalculated" or "in service" structural coefficients were:

Soil – hydrated lime subgrades (4)	0.05, 0.09, 0.10 and 0.19
Soil – Portland cement subgrades (3)	0.10, 0.16 and 0.18
Soil – LKD subgrades (1)	0.10
Soil – AFBC subgrades (2)	0.09 and 0.15

   Age of pavement test sections varied between 12 to 15 years. Positive structural layer coefficients indicate that thinner pavement sections could be (and were) used for the pavement design. The remaining pavement test sections in the study had back calculated structural coefficients, a_3, of 0 to 0.03. No structural credit would have been given for these stabilized subgrades in the pavement design.

3. "Chemical stabilization substantially increased the elastic modulus of untreated soils at all sites. Back-calculated values of modulus obtained from FWD tests of subgrades mixed with chemical admixture are about two times greater than back-calculated values of modulus of untreated soils."
4. Chemical stabilization represents a very economical means of improving the poor engineering strengths of Kentucky soils. Based on structural number, SN, required by the 1981 Kentucky flexible pavement design curves, the cost of pavement sections constructed on stabilized soil subgrades are less than equivalent pavement sections constructed on non-stabilized soil subgrades. Moreover, the thickness of a pavement resting on a treated subgrade can be thinner than the thickness if a pavement resting on an untreated subgrade. For a flexible pavement measuring 36 feet in width, the average cost savings for soil-hydrated lime and soil-cement subgrade stabilization was 19,100 dollars per mile.

OU/ODOT Research

The OU School of Civil Engineering conducted a study (Miller, et al) to evaluate cement kiln dust (CKD) as a soil stabilizer, which compared the laboratory and field behavior of CKD from three sources in Oklahoma and calcium oxide (quick lime). The field study involved construction of four test sections along a rural highway in Oklahoma. Soil samples were collected before, during field mixing, and following compaction for laboratory testing. Field testing included Dynamic Cone Penetration (DCP) of the treated subgrade and FWD testing after the pavement construction. In addition, an in-depth laboratory study was conducted on a clay and sand soil taken adjacent to the field test section. The laboratory study included plasticity, unconfined compressure strength, durability (freeze-thaw, wet-dry), swell and CBR. Results of the field study showed that the performance of CKD varied with the source (i.e., characteristics) of the CKD. The laboratory study showed that overall, CKD was at least as effective if not more effective than quick lime for the clay soil. For sand, CKD (a cementitious material) was clearly a more effective stabilizer than quick lime. CKD performed similarly to quick lime for reducing plasticity. CKD treated soils were more durable than quick lime treated soils. The study recommended that because of the variability of CKD, its use as a chemical additive should be evaluated on a case-by-case basis.

Kansas Experience

In a laboratory study conducted at Kansas University, (Milburn and Parsons) evaluated the performance of eight soils with lime (hydrated), cement, fly ash (Class C) and a proprietary liquid chemical (PermaZyme 11-X). The study evaluated the influence of the chemicals on the plasticity, strength, durability (freeze-thaw, wet-dry), leaching, and stiffness properties if the soils tested. Some of the general conclusions reached during the study are summarized in the following:

1. Lime, fly ash, and cement were effective in improving the plasticity characteristics of the soils tested, with lime showing the most influence on PI.
2. Lime, fly ash, and cement dramatically lowered the swell of CL soils. Most of the CH soils, with sulfates, swelled the same or more than the untreated soils.
3. Lime, fly ash, and cement treated soils exhibited significant strength improvements while the enzyme-treated soils showed modest strength gains. Most strength gains were retained after durability and leaching testing. Lime and cement treated soils performed best after durability testing.
4. Lime and cement treated soils exhibited higher stiffness than fly ash treated soils.
5. The liquid enzyme stabilizer did not substantially improve soil performance.

The study recommended that the function of the chemical additive (reduced plasticity, reduced swell, increased strength) should be considered in selecting the type of additive. Selection of the amount of chemical additive should be based on various guidelines for each of the various chemicals (i.e. ASTMD6276 for lime). Caution should be taken for treatment of sulfate rich soils with calcium based chemicals.

A research study at Kansas State University (Romanoschi, et al) conducted at the Civil Infrastructure Systems Laboratory (CISL) using accelerated pavement testing (APT) methods evaluated the performance of Portland cement, fly ash, lime and a commercial product (EMC^2) as a soil additive to one soil in an accelerated traffic loading environment. Four flexible pavement test sections were constructed and tested under full scale traffic loading conditions. A companion laboratory study was conducted to characterize the influence of each of the chemical additives on the test soil. Some of the major findings of the research are summarized in the following:

1. Under traffic loading conditions lime was the most effective stabilizer for the soil used in the research. Cement was next most effective followed by fly ash and the commercial product. The evaluation was based on "vertical compressive stresses at the top of the unbound clayey subgrade" which were measured during testing.
2. From the laboratory testing programs lime, Portland cement, and fly ash reduced swell and increased the unconfined compressive strength of the soil. The highest unconfined compressive strength was exhibited by cement, followed by lime and fly ash.

The study recommended the use of lime as the chemical stabilizer for clayey non-sulfate soil, with properties similar to that used in the traffic test. "Stabilization with lime leads to better pavement performance than stabilization with cement, even though soil-cement has higher compressive strength than that of lime stabilized soil."

Mississippi Experience

The Mississippi Department of Transportation (MDOT) conducted a study (Bartes and Bartes and Metcalf) of the long term performance of lime-fly ash (LFA) stabilized soil as a base course material. The field portion of the study included performing FWD tests on newer and older pavements and coring the pavements at each FWD location to observe layer conditions, pavement thicknesses and obtain cores for laboratory unconfined compressive strength (UCS) testing. The measured parameters were used to calculate layer (LFA layer) structural layer coefficients. The field and laboratory testing programs were conducted at nine sites using similar soils for the LFA treated base courses (mostly A-2-4). Table 14 from Bartes and Metcalf shows the calculated structural layer coefficient for the LFA stabilized layers.

Table 14. Summary of Structural Layer Coefficients (from Bartes and Metcalf)

Route	Number of Tested Locations	Normalized LFA a_2 Average	Normalized LFA a_2 Coef. of Var.	HMA a_1 Average
Newer Projects				
Bolivar	16	0.216	25.4	0.465
US-61				
US-45	16	0.273	18.9	0.462
Hwy. 35	15	0.214	22.7	0.423
US-72	8	0.177	30.7	0.44
Wilkinson US-61	8	0.259	50.3	0.448
Summary	63	0.232	32	0.451
Route	Number of Tested Locations	LFA a_2 Based on Equation 3 Average	LFA a_2 Based on Equation 3 Coef. of Var.	Revised HMA a_1 Calculated from Revised SN_{eff}
Older Projects				
Forrest US-98	16	0.18	22.3	0.38
George US-98	8	0.186	25.1	0.312
US-84	16	0.155	13.7	0.434
Hwy. 7	16	0.152	36.6	0.434
Summary	56	0.165	23.3	0.401

Typically MDOT design uses a structural layer coefficient of 0.2 for a LFA mix design based on a UCS of 500 psi. All but one of the newer projects met or exceeded the typical value while all of the older projects were less than the typical value. This leads to the following conclusions drawn by the authors from the results of the study.

1. LFA base courses, used on over 600 projects, are a "variable product in terms of structural value and thickness". Changes in construction practice were recommended to ensure more uniform placement.
2. Placement conditions were increased to 100% Standard Proctor density and in situ UCS of 400 psi.
3. The typical LFA stabilized base course layer was increased from 6 to 8 inches and a 6-inch chemically stabilized subgrade layer was required for additional support. The LFA layer structural coefficient was maintained at 0.2.

Nebraska Experience

The Nebraska Department of Roads (NDOR) in conjunction with the University of Nebraska is currently investigating the performance of lime, CKD, and fly ash for use as stabilizing agents with a variety of Nebraska soils. The research study has as its objective to develop guidance for use and a draft set of specifications for incorporating these chemicals into local soils to improve stability, increase soil strength, and reduce swell characteristics of subgrade.

Chapter 3
SUMMARY

The objective of Task 1 of the research project was to review existing correlation between chemically treated soils and the AASHTO-MEPDG design input parameters. Interestingly, no active or recently completed research involved with characterizing Level 2 and 3 design input parameters was found. The closest research to the topic was evaluation using field and laboratory data to calculate structural layer coefficients (a_3 or in some cases a_2) used in the structural number (SN) design equation. For the most part, this research confirmed that when chemically treated subgrade soil layers are included in the structural design of the pavement, the layer coefficients are conservative. Also, when the chemically treated soil layer is included in the pavement design, the resulting thickness of the surface and base layers are reduced and the cost of construction is lower. Yet most state DOTs do not include the treated layer in pavement design. A large portion of recent research involved evaluation of performance, particularly long-term performance, of various chemical additives in different soil types. Most of the comparisons were between the different chemicals, i.e. lime vs. CKD vs. cement, with some proprietary chemicals included in a couple of the studies. This research addresses issues such as permanence, durability, and general improvement of various engineering properties, but does not answer concerns about the correlations with material properties or typical values for design input parameters.

However, research into the performance of chemical additives as soil treatments does reaffirm (or confirm) some of the accepted knowledge on selection of type and amount of additive, proprietary additives, and problem soils. The following paragraphs summarize observations relative to these accepted trends.

Selection of Chemical Additive

Most involved with treatment of soils with chemical additives agree that the chemical selected should address the function of the improved soil. As discussed in Chapter 2, if the desired function is improvement of physical properties, workability, or stability (volume change), then modification with a non- cementitious chemical such as lime would be appropriate. If the desired function is strength improvement (UCS, M_r, stiffness) then stabilization with cementitious chemical such as cement, fly ash, or CKD would be appropriate. This is essentially the approach that TxDOT has taken in their guidelines for modification and stabilization of soils and base for use in pavement structures (Texas DOT). It serves as a guideline for selection rather than a formal selection process. Opportunity to use judgment based on local materials and preferences for chemical additives is available. It's possible to use fly ash or CKD to modify soils (i.e. using lower percentages of each) but often these chemicals are not as efficient at achieving the function.

Selection of Amount of Chemical Additive

Selection of the appropriate amount of chemical additive obviously varies with the intended function of the chemical, but the testing protocol outlined in ASTM D 4609 (ASTM) provides a logical testing sequence for selecting an appropriate percentage of chemical additive for either modification or stabilization of subgrade soils. The lime-soil pH test (ASTM D 6276) can be effectively extended to other calcium-ion based products such as fly ash, CKD, or cement. The test defines the calcium saturation point for a given soil based on pH, so the form that the calcium comes is irrelevant. The results of the "chemical"-soil pH test provide a very good estimate of the amount of chemical additives required to modify the soil. The extent to which the chemical additive achieves that function is typically confirmed by Atterberg limits and swell testing at selected chemical additives percentages. The "chemical"-soil pH test results also provide an estimate of the amount of chemical additive required for stabilization (Rule-of-Thumb: Required stabilization percentage is about twice the modification percentage.).

For stabilization levels, the ASTM D 4609 protocol establishes mix design procedures including, specimen preparation, curing test and final selection process. This would be analogous to the MEPDG Level 1 input. Where full mix designs are not justified experience based criteria such as ODOT OHD L-50 and L-51 are a suitable alternative to selecting an appropriate percentage of chemical additive.

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