Research Report
KTC 93-4

CONSTRUCTION AND PERFORMANCE OF HIGHWAY SOIL SUBGRADES MODIFIED WITH ATMOSPHERIC FLUIDIZED BED COMBUSTION RESIDUE AND MULTICONE KILN DUST

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16. Abstract In an effort to increase the utilization of by-product materials in highway construction projects, the Kentucky Transportation Cabinet authorized the experimental use of residue from an atmospheric fluidized bed combustion (AFBC) process and multicone kiln dust (MKD), a by-product resulting from the production of lime, as subgrade soil modifiers. This report presents information relative to preconstruction and post-construction laboratory evaluations, construction procedures, construction monitoring activities, and performance evaluations of a highway subgrade soil modified using AFBC spent lime, MKD, Type 1P cement, and hydrated lime. An untreated section served as a control section for the project located on Kentucky Route 11 in Lee and Wolfe Counties.

The laboratory testing program consisted of determining select engineering properties of the soil in a natural state and in a state altered by the chemical admixtures. Index tests were performed, moisture density relationships were determined, and bearing ratio and swell tests were performed. Based on the laboratory unconfined compression tests and bearing capacity tests, the two waste by-products significantly improved the shear strength and bearing strength of the subgrade soil. Field monitoring activities were comprised of both construction monitoring and post-construction monitoring. Construction procedures were essentially the same for all admixture types and no significant problems were encountered. Satisfactory moisture and density were achieved. Construction activities were documented through moisture content and density compliance tests. In-place bearing capacity tests and Road Rater deflection tests were performed on the untreated subgrade and again after modification. The analyses indicated significant improvement in subgrade strength after admixture modification.

Post-construction monitoring included determining in-situ bearing capacities, assessing moisture conditions and determining soil classifications of the treated and untreated subgrade layers. Road Rater deflection tests were conducted to assess the structural condition of the pavement structure. Results of the field monitoring program confirmed that each chemically modified subgrade continued to exhibit greater strengths than the untreated subgrade section. However, because of non-uniform mixing, the soil-AFBC subgrade sections exhibited severe differential swelling shortly after construction. The bituminous pavement required milling to eliminate humps on the pavement surface. The pavement was overlaid and apparently the subgrade swelling has ceased. However, due to the expansive nature of the AFBC spent lime, future use as a soil modifier could not be recommended. Results of field monitoring activities indicated that MKD was a suitable soil modifier and future use was recommended.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS					APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol	Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH					**LENGTH**		
in.	inches	25.40000	millimetres	mm	mm	millimetres	0.03937	inches	in.
ft	feet	0.30480	metres	m	m	metres	3.28084	feet	ft
yd	yards	0.91440	metres	m	m	metres	1.09361	yards	yd
mi	miles	1.60934	kilometres	km	km	kilometres	0.62137	miles	mi
		AREA					**AREA**		
$in.^2$	square inches	645.16000	millimetres squared	mm^2	mm^2	millimetres squared	0.00155	square inches	$in.^2$
ft^2	square feet	0.09290	metres squared	m^2	m^2	metres squared	10.76392	square feet	ft^2
yd^2	square yards	0.83613	metres squared	m^2	m^2	metres squared	1.19599	square yards	yd^2
ac	acres	0.40469	hectares	ha	ha	hectares	2.47103	acres	ac
mi^2	square miles	2.58999	kilometres squared	km^2	km^2	kilometres squared	0.38610	square miles	mi^2
		VOLUME					**VOLUME**		
fl oz	fluid ounces	29.57353	millilitres	ml	ml	millilitres	0.03381	fluid ounces	fl oz
gal.	gallons	3.78541	litres	l	l	litres	0.26417	gallons	gal.
ft^3	cubic feet	0.02832	metres cubed	m^3	m^3	metres cubed	35.31448	cubic feet	ft^3
yd^3	cubic yards	0.76455	metres cubed	m^3	m^3	metres cubed	1.30795	cubic yards	yd^3
		MASS					**MASS**		
oz	ounces	28.34952	grams	g	g	grams	0.03527	ounces	oz
lb	pounds	0.45359	kilograms	kg	kg	kilograms	2.20462	pounds	lb
T	short tons (2000 lb)	0.90718	megagrams	Mg	Mg	megagrams	1.10231	short tons (2000 lb)	T
		FORCE AND PRESSURE					**FORCE**		
lbf	pound-force	4.44822	newtons	N	N	newtons	0.22481	pound-force	lbf
psi	pound-force per square inch	6.89476	kilopascal	kPa	kPa	kilopascal	0.14504	pound-force per square inch	psi
		ILLUMINATION					**ILLUMINATION**		
fc	foot-candles	10.76426	lux	lx	lx	lux	0.09290	foot-candles	fc
fl	foot-Lamberts	3.42583	candela/m^2	cd/m^2	cd/m^2	candela/m^2	0.29190	foot-Lamberts	fl
		TEMPERATURE (exact)					**TEMPERATURE (exact)**		
°F	Fahrenheit temperature	5(F-32)/9	Celsius temperature	°C	°C	Celsius temperature	1.8C + 32	Fahrenheit temperature	°F

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

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EXECUTIVE SUMMARY

Design and construction of highway pavements on fine-grained soils, such as clays and silty clays, or soils having poor or marginal engineering properties, are frequently encountered by pavement design engineers. The objective of this study was to evaluate the results of laboratory testing, construction of test sections, and the field performance of subgrade soils modified with four different chemical admixtures. This study was undertaken as part of a long-term effort to evaluate potential applications for by-product waste materials in highway construction. One potential application involves the use of by-product materials for highway subgrade soil modification. This report describes an extensive evaluation used to determine the suitability of using two waste by-products as soil modifiers: a residue resulting from the cracking of crude oil by an atmospheric fluidized bed combustion process (AFBC spent lime); and, a by-product, multicone kiln dust that results from the production of lime. The report also documents the laboratory testing, construction and field performance of subgrade soils modified with type 1P cement and hydrated lime.

The laboratory testing program consisted of determining select engineering properties of the soil in a natural state and in an altered state. Index tests were performed, moisture-density relationships were determined, and bearing ratio and swell tests were performed. A laboratory procedure was developed to determine the optimum percentage of admixture to add to the soil. When the optimum percentage of admixture is added to the soil, the maximum unconfined compressive strength is obtained. Increasing the percentage of admixture above the optimum amount did not significantly increase the unconfined compressive strength of the modified soil. Based on the laboratory unconfined compression tests and CBR tests, the two waste by-products significantly improved the shear strength and bearing strength of the subgrade soil.

Field monitoring included both construction monitoring and post-construction performance monitoring. Construction of the modified subgrade sections was documented, tests were conducted for moisture content and density compliance, in-place bearing ratio tests were performed, and Road Rater deflection data were obtained on the untreated subgrade and again, seven days after modification. The construction procedures that were utilized exemplified the inability to effectively assure that the proper amount of chemical admixture was being applied and mixed with the soil. However, relative densities were easily achieved through proper compaction and, generally the modified subgrades were compacted slightly dry of the optimum moisture content. Initial analyses of the deflection data indicated that the estimated average subgrade modulus of elasticity was about 24,000 psi for the untreated subgrade. Seven days after chemical admixture modification

the estimated subgrade moduli of the soil-AFBC spent lime sections averaged about 75,000 psi and about 93,000 psi for the soil-MKD section. Shortly after completion of the soil-AFBC spent lime subgrade sections, severe differential swell or heave of the pavement surface occurred.

Post-construction monitoring included re-examining the expansive characteristics of the soil-AFBC spent lime mixture, performing in-place bearing capacity tests, obtaining undisturbed samples to analyze in the laboratory relative to unconfined compressive strengths, moisture contents and soil classifications, monitoring pavement elevations, distresses and rutting characteristics. The preconstruction laboratory evaluation of soil swell attributes indicated a swell of about 3.1 percent when seven percent AFBC residue was combined with the natural soil. The natural soil exhibited 3.9 percent swell. Additional specimens of the soil-AFBC mixture were evaluated. A specimen remolded from a bag sample obtained from a trench that was opened to investigate the subgrade heave had less than one percent swell in the California Bearing Ratio (CBR) test. Specimens were remolded using soils obtained from stockpiles and extreme quantities of the AFBC residue. It was found that the volumetric swell of the remolded specimens containing 15 to 30 percent AFBC residue (by weight of the dry soil) ranged between 24 and 27 percent. Based on this investigation, it was concluded that the amount of AFBC residue mixed with the natural soil at locations where differential heave had occurred exceeded the specified seven percent necessary for soil modification. This excessive amount of AFBC spent lime residue proved to be detrimental to the pavement. The two sections wherein AFBC spent lime was used to modify the subgrade soils required some patching to keep the roadway passable. Ultimately, both sections were milled and overlaid.

Results of in-situ bearing capacity tests indicated elevated CBR's for all chemically modified sections. The latest data indicated average in-place CBR values on the order of 27 for the soil-AFBC spent lime modified subgrade and about 96 for the soil-MKD subgrade. The average in-place CBR of the untreated, or control section was about eight at this time. Undisturbed shelby tube specimens did not provide meaningful data since samples were difficult to obtain and extrude without creating shear planes. Values of the dry densities of the modified soils were largely obtainable only for the soil-AFBC spent lime modified subgrade section. The dry density of soil-AFBC spent lime specimens averaged 89.8 pcf and 94.9 pcf, respectively, during 1989 and 1991 investigations. These values are substantially less than values recorded during construction and are a direct result of the volumetric swell that the soil-AFBC mixture underwent after construction. The untreated soils underlying the AFBC modified soils had a dry density of about 119 pcf during both years. Results of index tests performed on extruded Shelby tube specimens indicated that the chemically modified soils were generally classified as SM in the Unified Classification System and soil from the untreated layer was classified as CL.

The pavement surface was monitored for changes in elevation. Significant elevation changes occurred only in the soil-AFBC sections. The experimental and control sections were visually surveyed periodically for observable pavement distress since the completion of construction. Factors such as rutting and cracking were of principal concern. Overall, the chemically modified subgrade sections are in good condition and exhibiting excellent performance. With the exception of one area within the soil-AFBC section near Station 563+00, no significant pavement distresses or heaving have been observed to date. Pavement rutting characteristics were monitored during the study. On average, the deepest rutting occurred in the control section. The absence of significant pavement rutting in the chemically modified subgrade sections is illustrative of the benefits of chemical admixture subgrade modification.

Analyses of subsequent deflection tests performed with the Road Rater to quantify the long-term benefits of the admixture modification indicated that each chemically modified subgrade section continues to exhibit higher strengths than the untreated control section. However, because a three-layer (bituminous pavement over dense graded aggregate over subgrade) solution was employed to analyze the Road Rater deflection data collected during the evaluation period, long-term elastic moduli values of the chemically modified subgrade layers were not specifically determined.

It was concluded that the AFBC spent lime admixture enhanced the overall bearing strength and shear strength characteristics of the natural soil. However, the construction procedures employed by the subcontractor could not prevent excessive amounts of the AFBC residue from being mixed with the natural soil. Future use of the AFBC spent lime for soil subgrade modification was not recommended because of the extremely expansive nature of this waste by-product. Further research is needed to identify and control the mechanism that causes the swelling of the soil-AFBC spent lime mixtures. It also was concluded that multicone kiln dust waste material used as a soil modifier provided increased the shear strength properties above those of the natural soil. The results of the in-situ field tests also indicated that the soil-MKD layer appeared to be gaining strength with time. Because of the available calcium oxide in the waste material (about 23 percent), the strength gain with time was expected. The soil-MKD modified subgrade section has performed excellently and further use of this waste by-product is warranted. Future use of multicone kiln dust as a subgrade soil modifier is encouraged based on the results of this successful field trial.

INTRODUCTION

Construction of highway pavements on fine-grained soils, such as clays and silty clays, or soils having poor or marginal engineering properties, are frequently encountered by geotechnical and pavement design engineers. The purpose of this study was to evaluate the results of laboratory testing, construction of test sections and the field performance of subgrade soils stabilized with AFBC spent lime, a by-product from an atmospheric fluidized combustion process, and multicone kiln dust, a by-product from the production of lime, and compare the performance of these experimental soil modifiers to the performance of pavements constructed on subgrade soils modified with more conventional chemical admixtures: portland cement and hydrated lime. More specifically, this report presents information concerning initial laboratory testing, construction of test sections utilizing various admixtures for soil modification, and the results of periodic performance monitoring activities.

BACKGROUND

The method normally used to modify fine-grained soil subgrades is mechanical compaction. Compaction specifications for soil subgrades usually require that placement dry density and moisture content conform to stated criteria. For example, many specifications require that placement dry density of the soil subgrade be 95 percent of the dry density obtained from the standard laboratory compaction procedure (AASHTO T-99 or ASTM D 698) and the placement moisture content not be two percent more or less than the optimum moisture content obtained from the standard laboratory compaction test. Many soils, when initially compacted to conform to such criteria, may have adequate strength to withstand, without failure, construction traffic loadings and traffic loadings shortly after the pavement is constructed.

However, the bearing strength of fine-grained soils is very sensitive to changes in moisture content. With regard to moisture content of soil subgrades, two problems may arise. First, if the moisture content of the compacted subgrade exceeds the optimum moisture content of the soil, that is, the placement water exceeds that necessary for optimum moisture content, then inadequate bearing strength may result. As the moisture content of the soil increases, there is a decrease in the undrained shear strength, or bearing strength. Compaction of soils having moisture contents exceeding the optimum moisture content is not uncommon. Secondly, when clay, or silty clay subgrades remain exposed, during construction, to rainfall and snowfall for a considerable time before the base stone and pavement are placed, they tend to absorb water, swell, and increase in volume. With an increase in moisture content and volume, the undrained shear strength, or bearing strength, decreases. Consequently, failures of the soil subgrade may occur under construction traffic loadings.

Atmospheric fluidized bed combustion (AFBC) is an advanced combustion process which provides a method of cracking crude oil more economically and in an environmentally acceptable manner. Sulfur dioxide, an undesirable by-product of cracking crude oil, is captured by calcium oxide formed from the limestone to produce calcium sulfate as a by-product of the AFBC process. Construction and operation of fluidized bed combustion units in Kentucky represent another high volume source of waste material that require disposal. The production of additional waste materials represents a large liability and operating expense. The spent lime by-product of the AFBC process may be disposed of by conventional methods at substantial costs; however, it may have a number of benefits. Mineral resources and construction materials could be conserved by replacement with AFBC spent lime which otherwise would have to be disposed of in a landfill. Road construction costs could be reduced by using less commercial lime and cement in soils. Utilization of coal fly ash could be expanded as a mixture with AFBC spent lime. Useful lives of landfills could be extended.

ADMIXTURE MATERIALS

The hydrated lime used in the subgrade section, Station 348+00 to 402+50, was produced and supplied by the Dravo Lime Company of Maysville, Kentucky. Chemical analysis of the hydrated lime is shown in Table 1. Total CaO is 72 percent and the amount available is 69 percent. The multicone kiln dust (MKD) used in the subgrade section, Station 402+50 to 429+50, was also supplied by Dravo Lime Company. Chemical analysis of the MKD is also shown in Table 1. Total CaO is 28 percent and the amount available is 23 percent. The type 1P cement used in the two soil-cement subgrade sections, Station 317+50 to 348+00 and Station 429+50 to 522+00, was supplied by the Kosmos Cement Company of Louisville, Kentucky. Type 1P cement contains about 20 percent fly ash. Chemical analysis of the type 1P cement is shown in Table 2. The AFBC spent lime waste by-product used in the two soil-AFBC subgrade sections, Station 260+00 to 317+50 and 532+00 to 576+50, was obtained from the Ashland Petroleum Company of Ashland Kentucky. Chemical analysis of this material is depicted in Figure 1. Results shown in Figure 1 represent x-ray diffraction tests on 21 test specimens. Amounts of compounds in the AFBC spent lime material vary widely. The amount of CaO ranged from about 62 to 80 percent and averaged 70 percent. Variability, when considered alone, would not limit the use of this material. Admixture designs could be based on the strength of the soil-AFBC spent lime mixtures using the lower percentages of the compounds which improve stability and strength.

TABLE 1. CHEMICAL AND PHYSICAL ANALYSES OF HYDRATED LIME AND MULTICONE KILN DUST*

Chemical Analysis		Physical Analysis	
Compound	Percent	Sieve Size	Percent Passing (%)
Hydrated Lime			
Total CaO	72.00	No. 20	100.0
Available CaO	69.00	No. 30	100.0
MgO	2.50	No. 50	100.0
SiO_2	1.60	No. 100	99.9
R O	0.75	No. 200	99.0
Fe_2O_3	0.15	No. 325	97.0
Al_2O_3	0.16		
Sulfur	0.045		
Multicone Kiln Dust			
$CaCO_3$	47.0	No. 50	90.1
CaO	28.0	No. 100	75.2
Available CaO	23.0	No. 200	63.0
MgO	4.6	No. 325	47.6
Sulfur	1.2		
SiO_2	8.8		
Fe_2O_3	0.7		
Al_2O_3	3.2		
CO	1.2		

* Courtesy of the Dravo Lime Company

LABORATORY TESTING PROGRAM

The laboratory testing program determined the suitability of using the by-product materials as soil modifiers. An extensive laboratory testing program conducted prior to construction began with obtaining samples of the natural soils. Disturbed samples of the natural soils were obtained from three stockpiles constructed by the contractor. The stockpiles were located at Stations 273+00, 334+00, and 574+00. Also, the Geotechnical Branch of the Division of Materials (Kentucky Transportation Cabinet) obtained samples of the soil subgrade every 500 feet along the entire length of the reconstructed roadway. Geology of the area consisted of interbedded layers of shales, sandstones, siltstones, and some coal. The soils at the construction site are residual and consist of derivatives of the shales, sandstones, siltstones, and coal.

The laboratory testing program consisted of determining select engineering properties of the soil in an untreated, or natural, state and in a state treated by a chemical admixture. The purposes of the laboratory study were to:

- classify the soils of Kentucky Route 11,
- develop the necessary data so that an appropriate chemical admixture could be selected,
- determine changes, if any, in the engineering properties of the soils after treatment with chemical admixtures, and
- determine the optimum percentage of a given chemical admixture to add to the soils.

The laboratory study consisted of performing the following tests:

- liquid and plastic limits
- specific gravity tests
- particle-size analyses
- soil classifications
- visual descriptions
- pH tests
- moisture-density relationships
- bearing ratio tests
- swell tests, and
- unconfined compression tests.

Index Tests

Liquid and plastic limit tests were performed according to procedures of ASTM D 4318. Particle-size analyses were performed according to ASTM D 854. The soil samples were classified using the Unified Soil Classification System, ASTM D 2487, and the AASHTO Classification System, M 145.

TABLE 2. CHEMICAL ANALYSIS OF THE TYPE 1P CEMENT*

Element	Percent
C	1.67
Na	0.60
Mb	1.63
Al	6.84
Si	25.94
P	659.30 ppm
S	2.44
K	0.77
Ca	53.87
Ti	0.37
Mn	352.02 ppm
Fe	5.67
Sr	917.48 ppm

* Courtesy of the Kosmos Cement Company

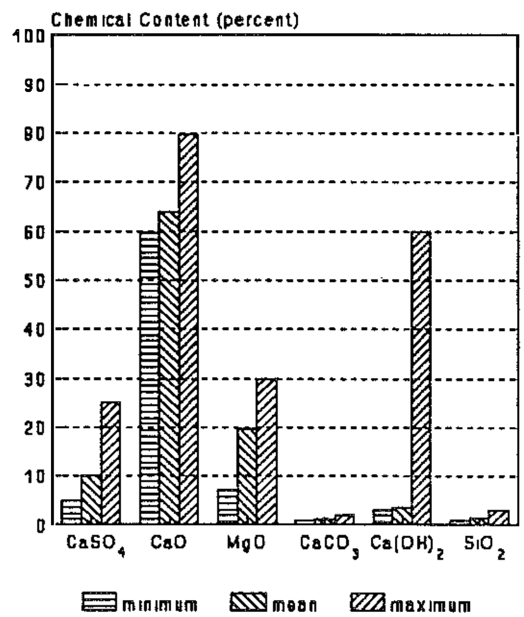

Figure 1. *Chemical analysis of the waste by-product obtained from the atmospheric fluidized bed combustion process.*

Moisture-Density Relationships

Moisture-density relationships of treated and untreated soils were determined according to ASTM D 698, Method A, or AASHTO T 99. The purpose of these tests was to determine the optimum water content and maximum dry density of the soils. Also, these tests were used to study the variation, if any, of optimum moisture content and maximum dry density of treated soils as the percentage of chemical admixture increased. The values obtained from these tests also were used to check field compaction of the chemically treated soil subgrade.

Optimum Percentage of Chemical Admixture

Various methods may be used to determine the optimum percentage of chemical admixture. These methods are as follows:

- unconfined compression tests,
- charts and tables by manufacturers of chemical admixtures, and
- pH tests.

Unconfined compression tests

One of the most widely used methods of determining the optimum percentage of chemical admixture to mix with a given type of soil is the unconfined compression test. In this approach, several soil specimens are remolded at different percentages of admixture and at optimum moisture content and maximum dry density (or at selected values of moisture content and dry density). Unconfined compression tests are performed on the specimens following procedures of ASTM D 2166 -- strain controlled technique. The unconfined compressive strengths are plotted as a function of the percentage of the chemical admixture. The optimum percentage of chemical admixture is a point where there is no significant increase in the unconfined compressive strength as the percent of chemical admixture is increased.

Charts and tables by product manufacturers

Several charts and tables have been devised by manufacturers for selecting the percent of chemical admixture. These have been described by Terrel. For example, the Portland Cement Association presents a table showing the cement requirements for various soil types based on the AASHTO and Unified Soil Classification Systems. Also, the National

Lime Association has published a graph for determining the percent of hydrated lime to use for a given type of soil. This graph makes use of the index properties of the soil. Results obtained from these tables and graphs were used to compare to results obtained from unconfined compression tests.

Use of pH tests

Tests to determine the pH of hydrated lime-soil mixtures were performed following a procedure proposed by Eades and Grimm. This is a quick method for determining lime requirements for lime stabilization.

Bearing Ratio Tests and Swell Tests

California Bearing Ratio (CBR) tests were performed. Soils from the KY 11 site were generally tested following procedures of Kentucky Method KM-64-501. In the Kentucky method, CBR specimens are molded using the values of optimum moisture content and maximum dry density, as determined from ASTM D 698.

During the course of performing CBR tests, swell measurements were made according to test procedures outlined by Kentucky Method KM-64-501. Additionally, a few selected swell tests were performed on the AFBC waste by-product.

LABORATORY TEST RESULTS

Laboratory Index Properties

Index test data and classifications of the untreated soils obtained from the soil subgrade at 500-foot intervals along KY 11 are summarized in Table 3. Liquid limits of the untreated soils ranged from 27 to 48 percent. Plasticity indices ranged from 7 to 21 percent and averaged 12 percent. The percent soil passing the No. 200 sieve ranged from about 43 percent to 87 and averaged about 69 percent. Based on a chart and guidelines by Epps and soil index data, it was determined that both hydrated lime and cement were suitable admixtures for the soils of KY 11. According to the AASHTO Classification System, the untreated soils classify as A-4, A-6, and A-7-6. Based on the Unified Classification System, the soils classify mainly as ML-CL.

TABLE 3. INDEX TEST DATA AND SOIL CLASSIFICATION OF UNTREATED SOILS

			Grain-Size Analysis Percent Finer Than:						Classification	
Station Number	Liquid Limit (%)	Plasticity Index (%)	No. 3/8-in. (%)	No.4 (%)	No.10 (%)	No.40 (%)	No.200 (%)	0.002mm (%)	AASHTO	Unified System
264	44	18	100.0	98.7	98.4	97.0	85.0	33.2	A-7-6(17)	ML-CL
269	32	9	100.0	98.0	97.7	96.4	82.7	17.6	A-4(7)	ML-CL
274	37	13	100.0	98.6	98.1	92.3	81.4	34.9	A-6(11)	ML-CL
279	42	17	100.0	98.0	97.2	91.7	80.6	31.5	*	*
284	37	14	100.0	82.4	82.5	82.5	74.9	22.2	A-6(10)	ML-CL
289	43	18	100.0	98.4	97.9	94.2	85.1	36.8	A-7-6(16)	ML-CL
294	47	20	100.0	98.7	98.0	91.2	83.0	37.4	A-7-6(18)	ML-CL
299	48	21	100.0	99.5	99.0	95.4	86.8	37.5	A-7-6(21)	ML-CL
304	33	16	100.0	99.2	98.8	94.8	81.6	27.9	A-6(12)	CL
309	31	9	100.0	98.7	98.2	95.2	83.5	22.2	A-4(7)	ML-CL
314				79.8						
319				89.5						
324				95.3						
329				95.7						
334				79.9						
344				68.8						
349				75.8						
354				80.9						
359				90.0						
364				89.5						
369	30	9	100.0	70.5	54.8	52.7	43.8	10.6	A-4(1)	GM-GC
374	30	10	100.0	77.3	63.8	60.8	51.2	13.7	A-4(2)	CL
379	28	9	100.0	94.3	91.0	86.7	67.1	17.6	A-4(4)	CL
384	31	11	100.0	90.7	83.7	78.6	64.0	16.6	A-4(5)	CL
389	30	10	100.0	92.9	83.1	79.7	67.2	17.8	A-4(5)	CL
394	33	9	100.0	89.1	81.2	75.2	69.6	14.9	A-4(5)	ML-CL
399	29	8	100.0	93.3	86.4	81.6	66.4	14.8	A-4(4)	ML-CL
404	34	10	100.0	92.1	85.0	78.6	71.5	14.6	A-4(6)	*
409	33	11	100.0	94.1	82.5	77.3	69.6	16.3	A-6(6)	ML-CL
459	36	8	100.0	95.4	95.4	84.4	75.8	15.0	A-4(6)	ML
464	43	15	100.0	94.3	80.2	76.4	72.6	24.4	A-7-6(11)	ML-CL
469	38	13	100.0	92.3	91.7	87.1	74.1	27.5	A-6(9)	ML-CL
474	40	16	100.0	92.6	89.3	80.6	69.9	25.4	A-6(10)	ML-CL
479	43	16	100.0	92.5	89.8	81.5	72.2	26.5	A-7-6(11)	ML-CL
484	30	8	100.0	92.9	91.3	84.1	60.6	15.3	A-4(3)	ML-CL

Index properties of the untreated and treated soils obtained from the three stockpiles (located at Stations 273+00, 334+00, and 574+00) are shown in Table 4. Liquid limits of the stockpiled soils ranged from 36 to 43 percent. Plasticity indices ranged from 12 to 15 percent. The percent passing the No. 200 sieve ranged from 70 to 74. Based on the

TABLE 4. INDEX PROPERTIES OF UNTREATED AND TREATED SOILS FROM STOCKPILES

Sample Type	Stockpile Location	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity	Grain-Size Analysis Percent Finer Than: No.4 (%)	No.10 (%)	No.40 (%)	No.200 (%)	Classification AASHTO	Unified System	Percent of Chemical Additive (%)
Untreated	273+00	39	14	2.69	100.0	91.2	82.4	74.0	A-6(10)	CL	0
Untreated	334+00	43	15	2.80	100.0	92.4	82.0	73.0	A-7-6(11)	CL	0
Untreated	574+00	36	12	2.72	100.0	89.7	81.0	70.0	A-6(8)	CL	0
Cement	273+00	NP	NP	2.65	100.0	91.2	60.0	39.4	A-4(0)	GM	10
Lime	273+00	45	10	2.80	100.0	91.2	65.7	54.0	A-5(4)	CL	6
AFBC	273+00	47	15	2.83	100.0	91.2	77.5	64.0	A-7-5(9)	CL	4
AFBC	273+00	51	13	2.80	100.0	91.2	75.6	64.4	A-7-5(9)	MH	7
AFBC	334+00	43	12	2.80	100.0	92.4	80.3	72.6	A-7-5(9)	CL	4
AFBC	334+00	49	14	2.80	100.0	92.4	63.6	48.5	A-7-5(5)	GM	7

AASHTO Classification System, the stockpiled soils at Stations 273+00, 334+00 and 574+00, classified as A-6(10), A-7-6(11), and A-6(8), respectively. The stockpiled soils classified as CL according to the Unified Soil Classification System.

Treatment of the clay soils with cement significantly affected the index properties. The clay soils became non-plastic and the percent passing a No. 200 sieve was reduced from 74 to 39. The AASHTO soils classification changed from A-6(10) to A-4(0). Classification of the soils by the Unified Classification system changed from CL to GM. Treatment with lime also produced some changes in the index properties. There was some reduction in the plasticity index. The percent passing the No. 200 sieve changed from 74 to 54. The AASHTO classification changed from A-6(10) to A-5(4) and the Unified classification remained the same, CL. Treatment with the waste by-product, AFBC produced mixed results. Plasticity indices showed little, or no change. The percent passing the No. 200 sieve changed from 74 to about 49. The Unified classification of the treated soils ranged from CL to GM. Treatment with AFBC did not appear to improve the classification. Originally, the use of multicone kiln dust was not included in the plans for this study but was proposed during construction of KY Route 11. Index tests on MKD-treated soil were not performed prior to the use of the MKD material as a soil modifier. Index tests were to be performed at a later date but unfortunately, were not.

Moisture-Density Relationships

The maximum dry density and optimum moisture content of the soils treated with hydrated lime and the AFBC spent lime changed significantly as the percent of either of these chemical admixtures increased. Maximum dry densities and optimum moisture

TABLE 5. MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENTS OF UNTREATED AND TREATED SOILS

			Untreated		Treated		
Sample Number	Stockpile Station Number	Type of Chemical Additive	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Chemical Additive (%)
11-A	273+00	Lime	106.1	18.0	103.9	18.7	6
11-B	273+00	Cement	106.1	18.0	107.7	16.8	10
11-C	273+00	AFBC	106.1	18.0	102.5	20.0	4
11-D	273+00	AFBC	106.1	18.0	101.7	20.5	6
11-E	273+00	AFBC	106.1	18.0	100.0	19.0	8
11-F	273+00	AFBC	106.1	18.0	99.5	19.0	12
11-G	334+00	AFBC	-	-	102.7	21.2	5
11-H	334+00	MKD	-	-	107.1	20.0	2
11-I	334+00	MKD	-	-	108.1	20.1	5
11-J	334+00	MKD	-	-	108.1	19.7	8
11-K	334+00	MKD	-	-	107.1	18.7	10
11-L	574+00	AFBC	112.8	16.3	105.7	16.2	5
11-M	574+00	AFBC	112.8	16.3	103.0	18.8	10

contents of untreated and treated soils from the three stockpiles are given in Table 5. There were no initial values of maximum dry density or optimum moisture content obtained for stockpile Station 334+00. The maximum dry density and optimum moisture content of the untreated soil from Station 273+00 were 106.1 pcf and 18.0 percent, respectively. Treatment with six percent hydrated lime reduced the maximum dry density by two percent and increased the optimum moisture content by about four percent. Treatment with ten percent cement increased the maximum dry density from 106.1 (untreated) to 107.7 pcf -- a 1.5 percent increase. Optimum moisture content decreased from 18.0 percent to 16.8 percent, or a change of 6.5 percent. A noticeable change occurred in the maximum dry density and optimum moisture content of soils mixed with the AFBC spent lime. Variations of the optimum moisture content and maximum dry density of specimens from the stockpile located at Station 273+00 and the percent AFBC are shown in Figure 2. Optimum moisture content and maximum dry density of specimens from the stockpile located at Station 574+00, as a function of the percent AFBC spent lime, are shown in Figure 3. As shown in Figures 2 and 3 (and Table 5), the maximum dry density decreased six to nine percent as the percent AFBC spent lime approached 10 to 12. The optimum moisture content increased six to 16 percent as the percent AFBC spent lime approached 10 to 12. The MKD waste material was combined with the soils from the stockpile at Station 334+00. Treatment with the MKD had little

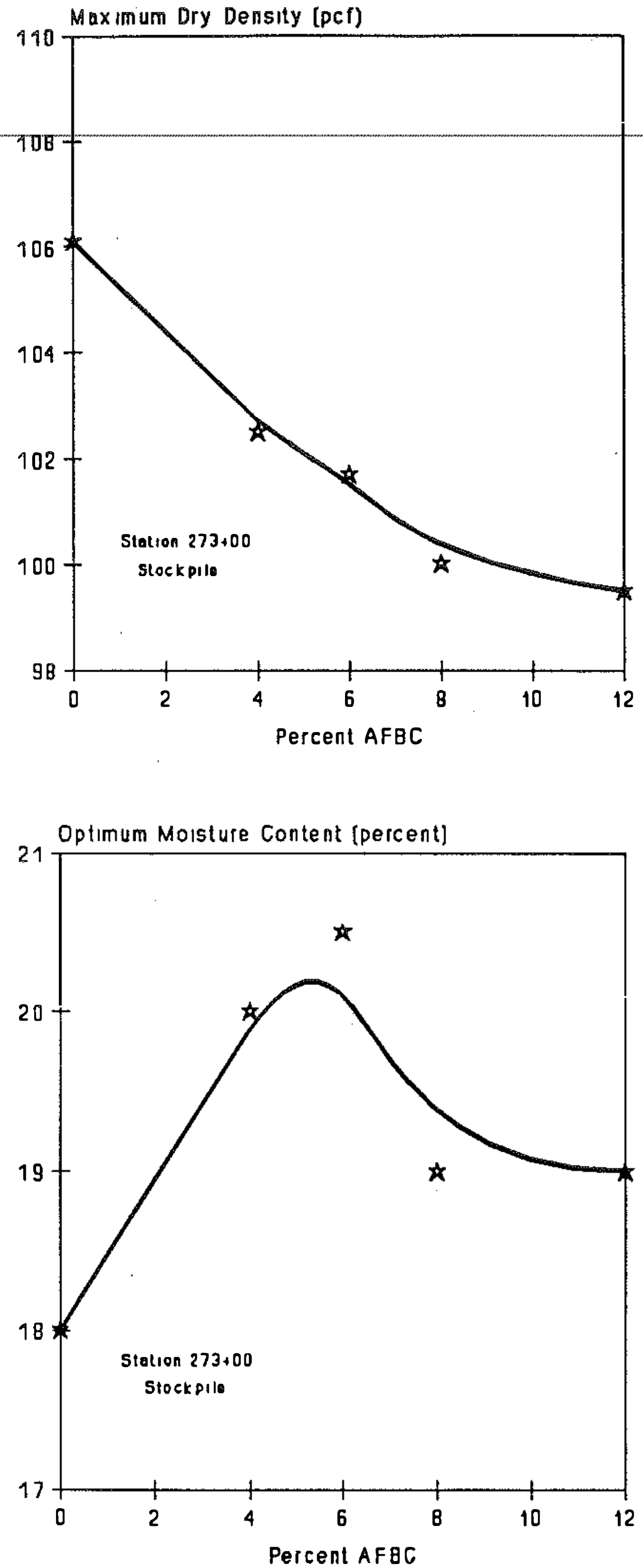

Figure 2. *Variation of the maximum dry densities and optimum moisture contents of soils from stockpile Station 273+00 with the percentage of AFBC spent lime.*

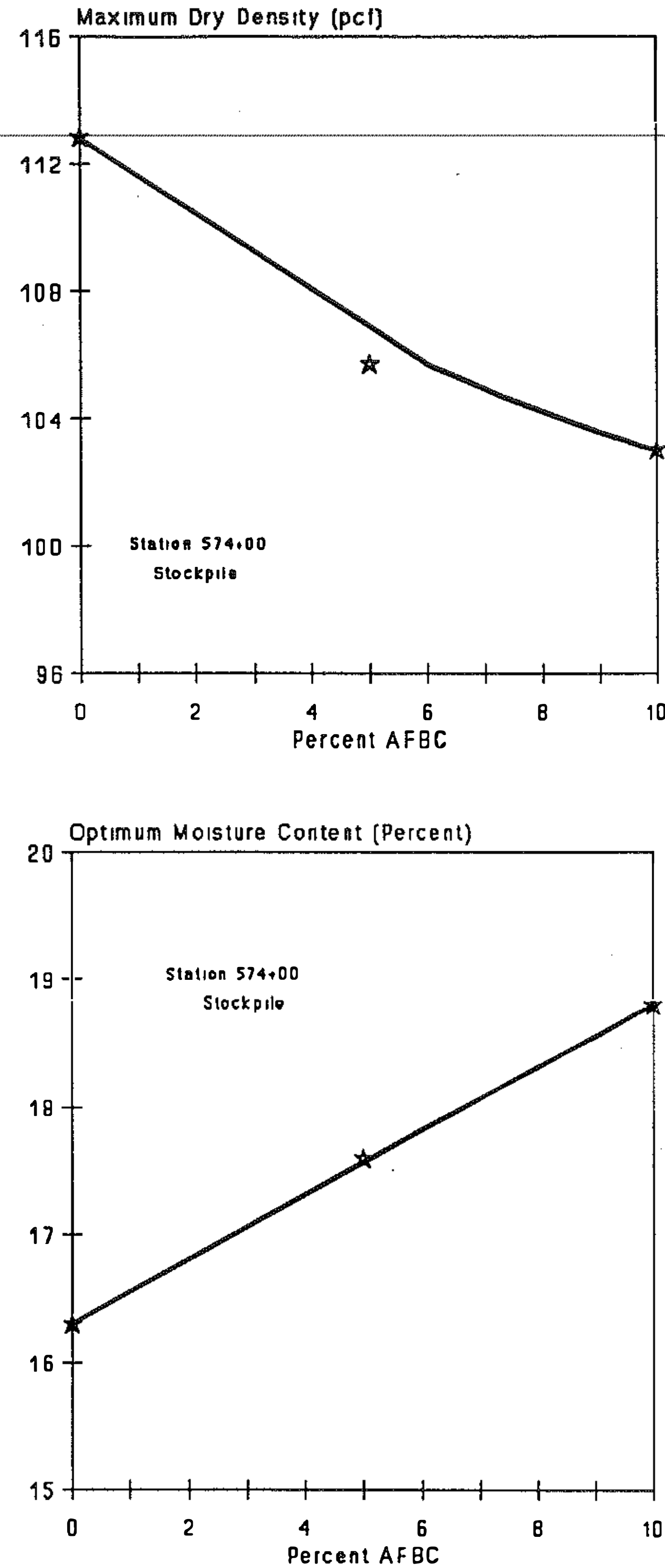

Figure 3. *Variation of the maximum dry densities and optimum moisture contents of soils from stockpile Station 574+00 with the percentage of AFBC spent lime.*

affect on the maximum dry density or optimum moisture content of soils. Variations of the optimum moisture content and maximum dry density of specimens from the stockpile located at Station 334+00 and the percent MKD are shown in Figure 4.

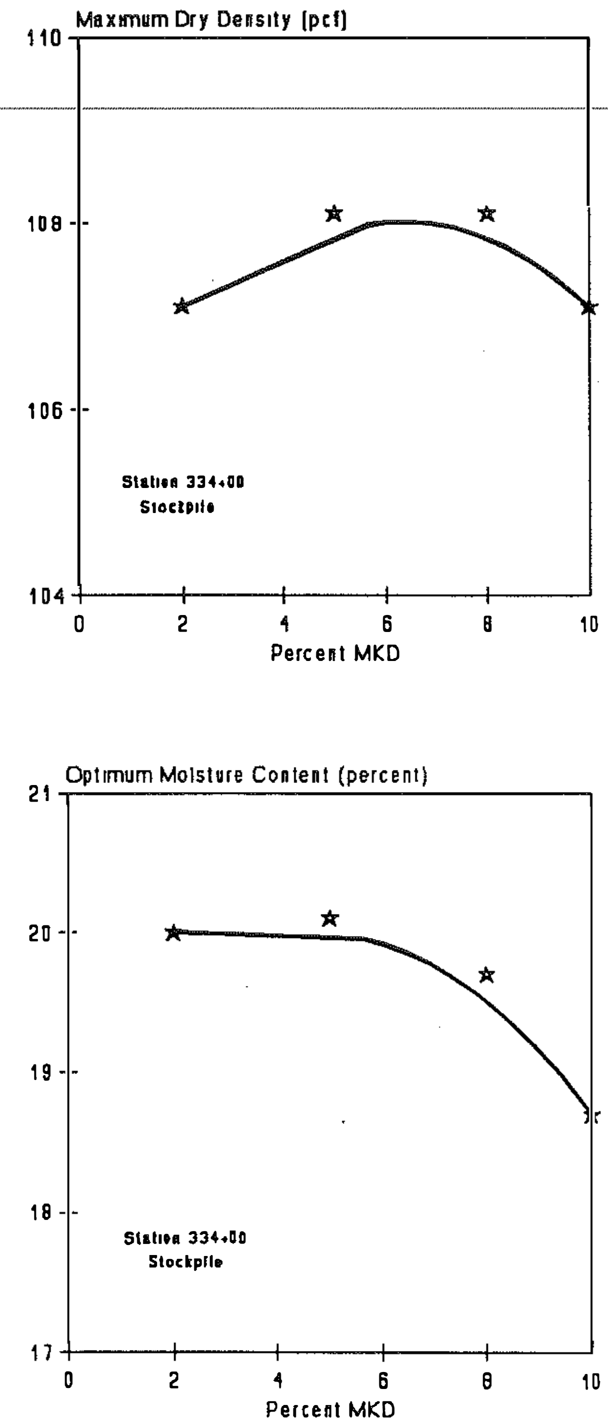

Figure 4. *Variation of the maximum dry densities and optimum moisture contents of soils from stockpile Station 334+00 with the percentage of multicone kiln dust.*

Optimum Percentage of Chemical Admixture

Results of the different methods used to determine the optimum percentage of chemical admixture are described below.

<u>Unconfined compression tests</u>

Unconfined compressive strengths of remolded specimens from the KY 11 site were used to determine the optimum percents of hydrated lime, cement, MKD, and AFBC spent lime. In these series of tests, all treated, remolded specimens were cured for seven days. Variation of the unconfined compressive strength as a function of percent hydrated lime is shown in Figure 5. Soils for the lime tests were obtained from the stockpile located at Station 273+00. Based on the curve in Figure 5, the optimum percent hydrated lime is about six or seven percent. The value selected and used in field stabilization was seven percent. As shown in Figure 5, the unconfined compressive strength of the untreated (unsoaked) specimen remolded to optimum moisture content and maximum dry density was about 40 psi. At an optimum value of seven percent hydrated lime, the unconfined strength of lime treated samples was some 150 percent greater than the strength of the untreated, remolded soil.

Two soil-cement series of unconfined compressive strength tests were performed on soils obtained from stockpiles located at Stations 273+00 and 574+00. Since the maximum dry density and the optimum moisture contents of untreated and treated specimens were similar, all unconfined compressive strength specimens were molded to maximum dry density and optimum moisture contents obtained from standard compaction tests on untreated specimens. For remolded specimens of soils from Station 273+00, the variation of unconfined compressive strength and the percent cement is shown in Figure 6. The optimum percent cement is about 10 to 12. The average, unconfined compressive strength of three, untreated (unsoaked) specimens of soils from the stockpile at Station 273+00 and remolded to optimum moisture content and maximum dry density was about 40 psi. At the optimum percent of cement, the unconfined compressive strength of the cement-treated soils was about 265 psi, or the strength of the cement-treated soils was about six to seven times greater than the strength of the untreated (unsoaked), remolded soil. The unconfined compressive strengths as a function of the percent cement of cement-treated soils from the stockpile located at Station 574+00 are shown in Figure 7. Based on this plot, the optimum percent cement is about 10 to 12. The unconfined compressive strength of a soil specimen remolded to optimum moisture content and maximum dry density and untreated

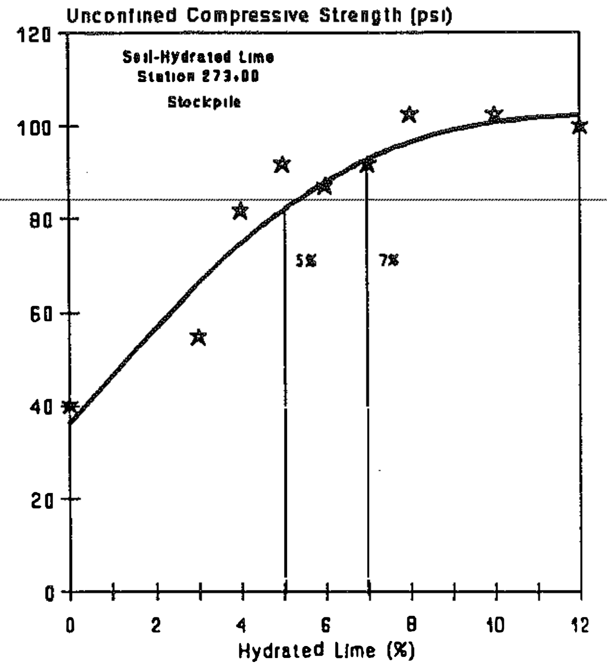

Figure 5. *Unconfined compressive strength of soil specimens from stockpile Station 273+00 as a function of the percentage of hydrated lime.*

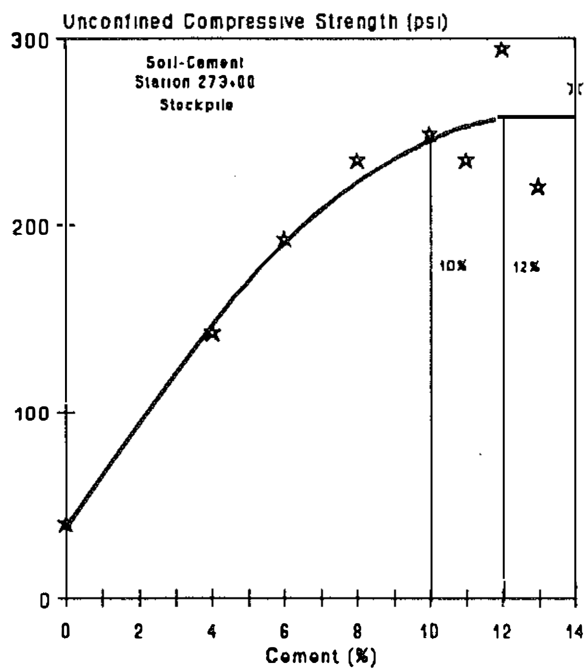

Figure 6. *Unconfined compressive strength of soil specimens from stockpile Station 273+00 as a function of the percentage of cement.*

(unsoaked) was 42.7 psi. At the optimum percent cement, the unconfined compressive strength of the soil-cement specimens was about 470 psi. This strength was about ten to 11 times the value of the untreated strength. Values selected for two field trials of cement stabilized soil subgrades were ten and seven percent.

Unconfined compressive strengths of remolded soils obtained from the stockpile located at Station 334+00 and treated with MKD are shown in Figure 8 as a function of the percent MKD. Based on these data, the optimum percent MKD was about eight to ten percent. A value of ten percent MKD per dry weight of the soil was used for the field trial.

Variations of unconfined compressive strength as a function of the percent AFBC spent lime are shown in Figures 9 and 10. The two series of unconfined compression tests were performed on remolded soils from stockpiles located at Stations 273+00 and 574+00. As shown in Figure 9, the optimum percent AFBC spent lime for soils from Station 273+00 was about five or six. For soils from Station 574+00, the optimum percentage of AFBC was about six percent. For the two trial sections constructed in the field, a value of seven percent was used. Unconfined compressive strengths at the optimum percentage of AFBC spent lime were about four times greater than the strengths of untreated (unsoaked) remolded specimens.

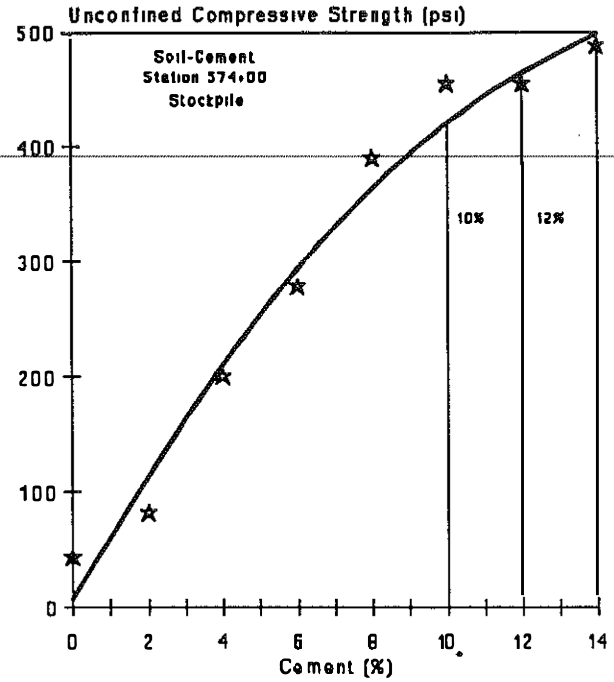

Figure 7. *Unconfined compressive strength of soil specimens from stockpile Station 574+00 as a function of the percentage of cement.*

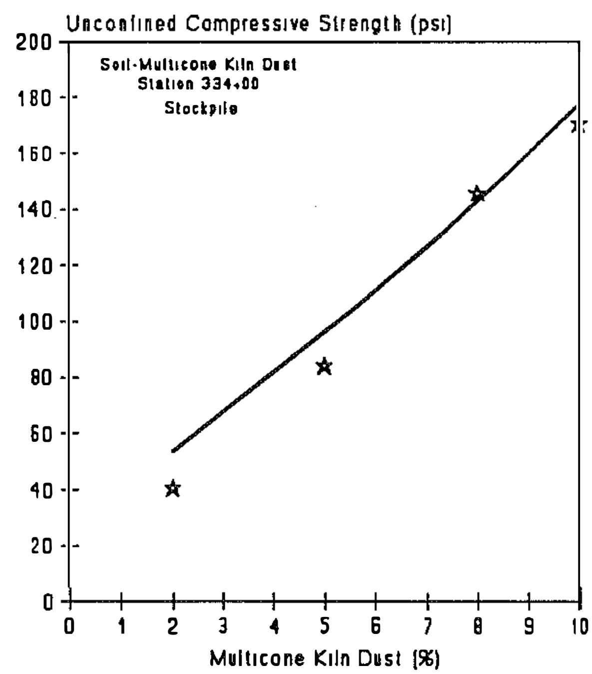

Figure 8. *Unconfined compressive strength of soil specimens from stockpile Station 334+00 as a function of the percentage of multicone kiln dust.*

Determinations based on pH tests

Results of pH tests performed on soil-hydrated lime mixtures are shown in Figure 11. These results indicate that the optimum percent of hydrated lime is about five percent. This value is somewhat lower than the six to seven percent optimum value obtained from the unconfined compressive strength tests. Although the procedure by Eades and Grimes was devised specifically for determining the optimum percent hydrated lime, the method was used with the AFBC spent lime material to determine if it was applicable to this material. Results of the pH tests on soil-AFBC spent lime mixtures are shown in Figure 12. Values of pH as a function of the percent of AFBC spent lime indicate that five percent of the AFBC spent lime is an optimum value. This compares reasonably well with the optimum values obtained from the unconfined compression tests shown in Figures 9 and 10.

Bearing Ratio and Swell Tests

Results obtained from bearing ratio tests performed on remolded specimens treated with hydrated lime, cement, and the AFBC spent lime material are shown in Table 6. The CBR value for the untreated soil was 3.7 and 4.1 at penetrations of 0.1 inch and 0.5 inch, respectively. Total swell strain of the untreated sample was 3.9 percent (see Figure 13). The KYCBR value of the soil-cement (10%) sample was 300 at 0.1-inch penetration and 111 at 0.5-inch penetration. Swell strain of this specimen was essentially

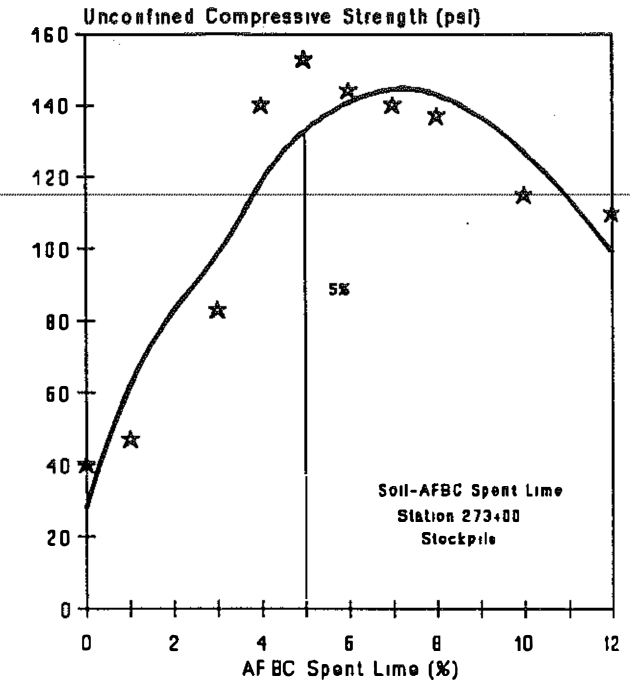

Figure 9. *Unconfined compressive strength of soil specimens from stockpile Station 273+00 as a function of the percentage of AFBC spent lime.*

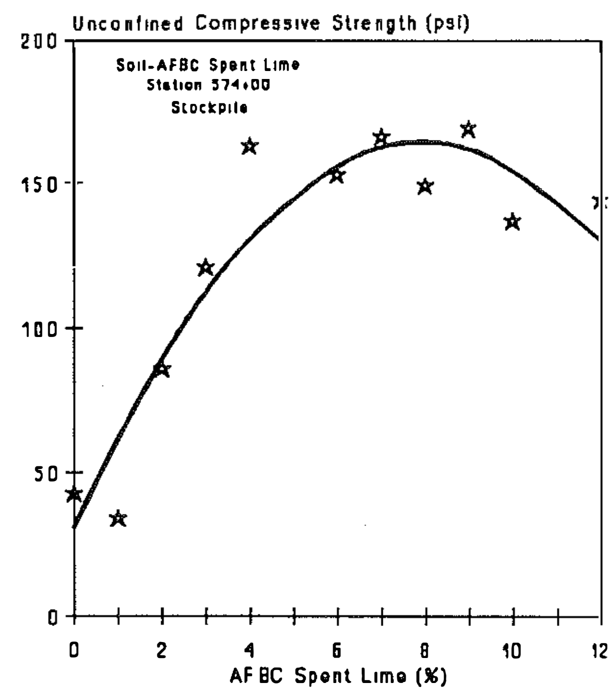

Figure 10. *Unconfined compressive strength of soil specimens from stockpile Station 574+00 as a function of the percentage of AFBC spent lime.*

zero (0.02 percent). The soil-hydrated lime (6%) sample had KYCBR values of 67 and 45, respectively, at penetration values of 0.1 inch and 0.5 inch. The total swell strain of this specimen was 0.2 percent. Consequently, when the soils were treated with cement or hydrated lime, there was a considerable reduction in swell strains (see Figure 14) when compared to the swell strain of the untreated soil.

The values of KYCBR of an AFBC-soil sample remolded at seven percent of AFBC spent lime were 48 and 33 at 0.1-inch and 0.5-inch penetrations, respectively. These values are some nine to 13 times larger than the KYCBR values of the untreated specimen. Total swell (strain) of the soil-AFBC mixture (7% admixture) was 3.1 percent. The value of swell was slightly lower than the swell strain (3.9 percent) observed for the untreated specimen (see Figure 13).

CONSTRUCTION PROCEDURES

The experimental project is located on Kentucky Route 11 in Lee and Wolfe Counties, approximately seven miles north of Beattyville (see Figure 15). The route extends from Station 260+00 to 576+50 and is about six miles in length. The sections of stabilized soil subgrades on KY 11, types of admixture stabilizers, beginning and ending station numbers, and lengths of the sections are shown in Table 7. Included in the subgrade modification were two sections of

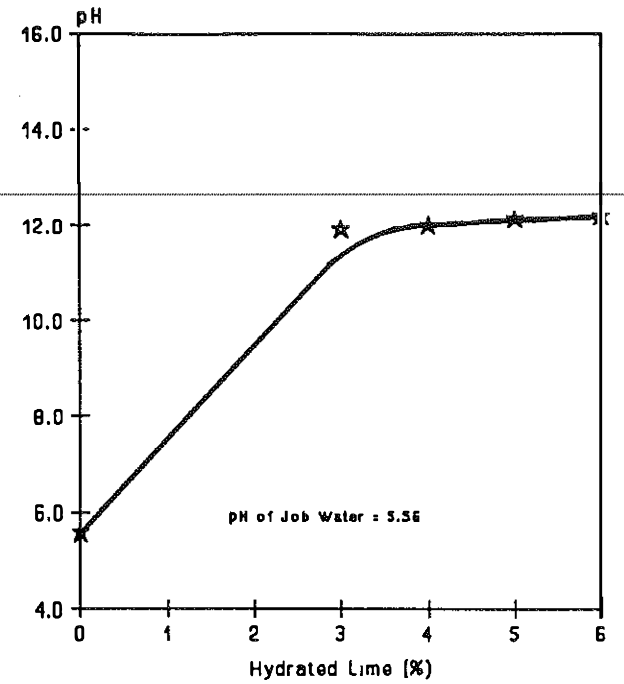

Figure 11. *Variation of the pH value of soil-hydrated lime mixtures with the percentage of hydrated lime.*

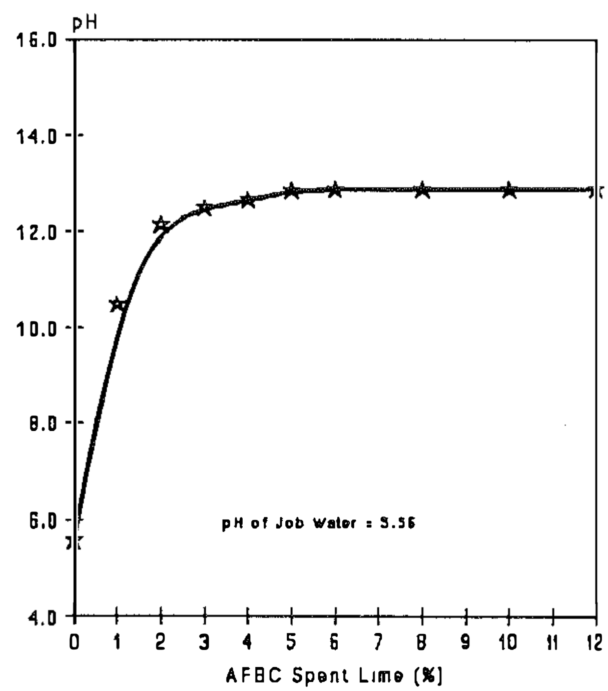

Figure 12. *Variation of the pH value of soil-AFBC spent lime mixtures with the percentage of AFBC spent lime.*

TABLE 6. SOAKED, KYCBR-VALUES OF UNTREATED AND TREATED SOILS

Sample Number	Curing Period (days)	Type and Percentage of Chemical Admixture	Soaked Penetration Value 0.1-inch	Soaked Penetration Value 0.5-inch	Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Moisture Content (%)	Total Swell (%)
U-1	-	None	3.7	4.1	115.6	16.9	110.1	19.9	3.9
C-10	4	Cement-10%	300	111.1	123.6	15.9	116.6	13.0	0.02
HL-1	-	Hydrated Lime-6%	67.3	44.7	112.6	18.7	112.8	18.6	0.02
AFBC-7	4	AFBC-7%	47.7	32.7	114.0	17.0	107.4	20.2	3.1

soil-AFBC subgrade, two sections of soil-cement subgrade, one section of soil-lime subgrade, and one section of soil-multicone kiln dust. A short section extending from Station 522+00 to 532+00 was not modified in any manner so that it could serve as the control section for comparison purposes.

The pavement sections of KY 11 were initially proposed for construction as 8.5-inches asphaltic concrete and 17.0 inches of dense graded aggregate (DGA) base. The decision to employ soil modification procedures was made after the initial design process. It was proposed to use the various chemical admixtures documented herein. Past experience of design personnel had indicated that the thickness of DGA could be reduced as the thickness of the modified soil layer was increased. The thickness design was initially modified to include: 8.5-inches asphaltic concrete, 5.0-inches DGA and 12.0 inches of modified soil subgrade. Preliminary analyses of the modified soil mixtures indicated the soil-cement appeared to be stronger than the other modified soil mixtures. It was decided, during construction, to further modify thicknesses of the upperlying materials so as to have more equivalent thickness designs from a structural perspective. The thicknesses of the DGA and asphaltic concrete within each section are detailed in Table 7.

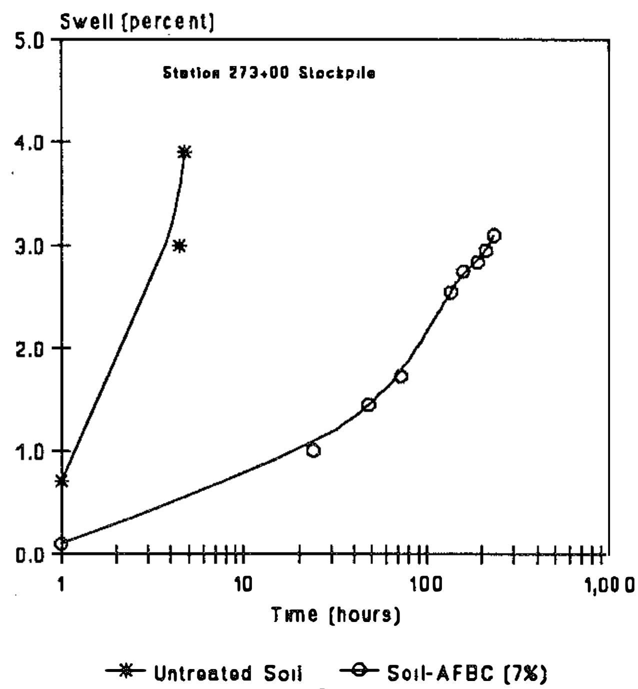

Figure 13. *Swell of untreated soils and AFBC spent lime (7%)-treated soils as a function of the logarithm of time.*

The contract for the combined grade, drain, and surfacing project was awarded to Elmo Greer and Sons, Incorporated, of London, Kentucky. Soil modification procedures for the project were subcontracted to Mount Carmel Sand and Gravel Company, of Mount Carmel, Illinois. Preparation of the untreated subgrade soil was completed in May 1987. This involved compacting and shaping the sudgrade to grade elevation or somewhat below grade to accommodate anticipated volume increases due to the incorporation of the chemical admixtures. The soil modification procedures varied somewhat for each chemical admixture type primarily because of the experimental nature of the project. Equipment used included Ray-Go soil

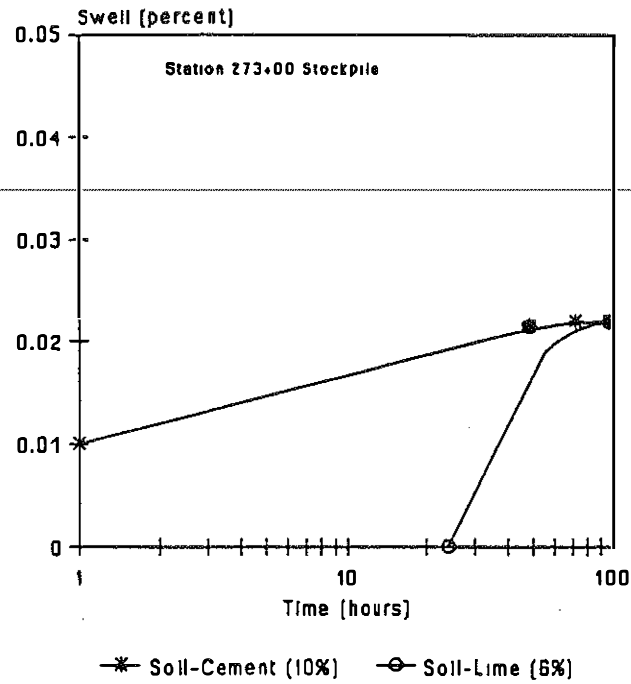

Figure 14. *Swell of hydrated lime-treated and cement-treated soils as a function of the logarithm of time.*

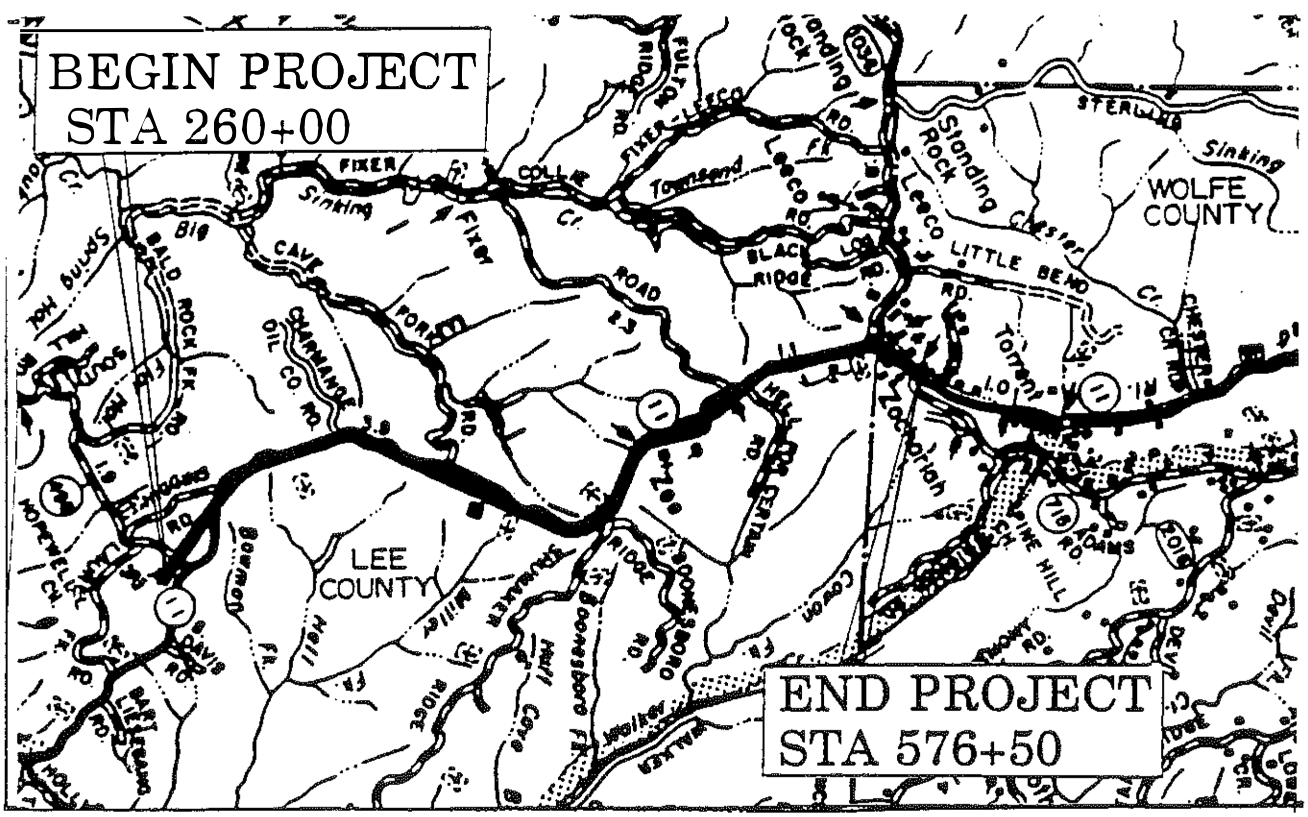

Figure 15. *Location of project.*

pulverizers, sheepsfoot and smooth-wheel vibratory compactors, spreader trucks for distributing the chemical admixtures over the subgrade, water trucks, small bulldozers, and motor graders. All soil modification sections were treated to a depth of 12 inches. Construction procedures are described below.

TABLE 7. MODIFIED SOIL SUBGRADE SECTIONS AND PAVEMENT LAYER THICKNESSES

			Thicknesses		
Beginning Station	Ending Station	Length (miles)	Crushed Stone (in.)	Asphaltic Concrete (in.)	Stabilizing Admixture
260+00	317+50	1.080	5.0	8.5	AFBC Spent Lime (7%)
317+50	348+00	0.587	5.0	6.0	Portland Cement (10%)
348+00	402+50	1.032	5.0	8.5	Hydrated Lime (7%)
402+50	429+50	0.511	5.0	8.5	Multicone Kiln Dust (10%)
429+50	522+00	1.752	5.0	6.0	Portland Cement (7%)
522+00	532+00	0.189	5.0	11.0	Untreated
532+00	576+50	0.843	5.0	8.5	AFBC Spent Lime (7%)

Soil-AFBC Subgrade Section -- STA 260+00 to STA 317+50

Construction requirements for AFBC residue roadbed modification are summarized in Kentucky Department of Highways' Special Note for AFBC Residue Roadbed Stabilization (Experimental). The Special Note, reproduced in Appendix A of this report, was developed by engineers exclusively for this project. The Special Note requires primary and final mixing. Specifically, the Special Note requires that two thirds of the AFBC spent lime be placed initially. The moisture content of the modified soil should be no less than optimum, and no more than five percent greater than the optimum moisture content. After primary mixing, the modified soil layer is shaped to the approximate cross section, and lightly compacted to minimize evaporation loss. Following primary mixing, the modified layer is to be cured for at least 48 hours. This permits the spent lime and water to break down the clay clods. During the preliminary curing phase, the surface of the subgrade is to be kept moist to prevent drying and cracking. Immediately after the preliminary curing phase, the remaining one third of the AFBC spent lime should be spread and the stabilized layer completely mixed and pulverized again. Final mixing continues until all clods are broken down so that 100 percent, exclusive of rock particles, passes a one-inch sieve and at least 60 percent passes a No. 4 sieve. However, it is stated that if the pulverization requirement can be met during the primary mixing phase, the preliminary curing and final mixing steps can be eliminated, which was the situation on this project. After the subgrade is brought to grade elevation, a bituminous curing seal is required to prevent excessive loss of moisture. No vehicular traffic is allowed to traverse the subgrade after placement of the curing seal until after a period of seven days.

The AFBC spent lime was trucked to the job site from the Ashland Oil Company refinery in Catlettsburg in covered tractor-trailer trucks. Initially, the AFBC spent lime materials were dumped into a storage pit and covered prior to using to modify the soil subgrade. A front-end loader was used to load the AFBC spent lime material into modified spreader trucks. The tops of the spreader trucks had been removed to facilitate the loading operation. Treatment of the subgrade began at Station 260+00 and proceeded in a northerly direction. The AFBC spent lime was spread over the smooth subgrade in approximately 200-foot lengths (see Figure 16). The amount of AFBC spent lime used in this section was approximately seven percent (by dry weight of the soil). The prepared subgrade was virtually dry and had a very hard crust. Much of the water applied for mixing ran off the grade and into ditches because of the hard crust on the smooth subgrade. Figure 17 shows the Ray-Go soil pulverizer in action. A water truck is positioned behind the soil pulverizer to add additional water to the soil-AFBC spent lime mixture. The soil pulverizers ground up the soil to a depth of 12 inches. The pulverization requirement was met during the first pass of the soil pulverizing equipment. A sheepsfoot vibratory roller was used for initial compaction immediately behind the soil pulverizers. Figure 18 is typical of the soil pulverization and compaction sequence. A smooth-wheel vibratory roller was used for final compaction. Personnel of the Kentucky Department of Highways conducted moisture/density tests using a Troxler nuclear density gage to ensure proper moisture content and compaction of the modified soil subgrade. After determining that moisture and density requirements had been achieved, a road grader was used to cut the subgrade to grade elevation (see Figure 19). The required bituminous curing seal was placed after achieving proper grade. Figure 20 shows an overview of the completed subgrade. In the background is a recently placed bituminous seal. The AFBC stockpile is to the right, just prior to the cut in the hillside.

This construction sequence continued throughout the first 2,500 to 3,000 feet of the approximate 5,750 soil-AFBC spent lime section. Toward the end of the first section, the materials hauler began to dump the AFBC spent lime material directly onto the subgrade in lieu of placing it in the stockpile. The subgrade subcontractor spread the AFBC spent lime directly on the subgrade with the front-end loader. This appeared to work just as well as distributing the materials with the spreader trucks. However, there was virtually no control relative to how much AFBC spent lime material was being incorporated.

Difficulties encountered on the first AFBC spent lime modification section included flow of the material, having to cut the modified subgrade to final grade elevation, and obtaining correct moisture/density measurements. Because of the fine-grained nature of the AFBC spent lime, the material flowed much like a liquid. Because of this problem,

Figure 16. *Spreader truck distributing AFBC spent lime materials over the surface of the prepared subgrade.*

Figure 17. *A soil pulverizer mixes the AFBC spent lime with the natural soil. A water truck adds water necessary to achieve proper moisture content.*

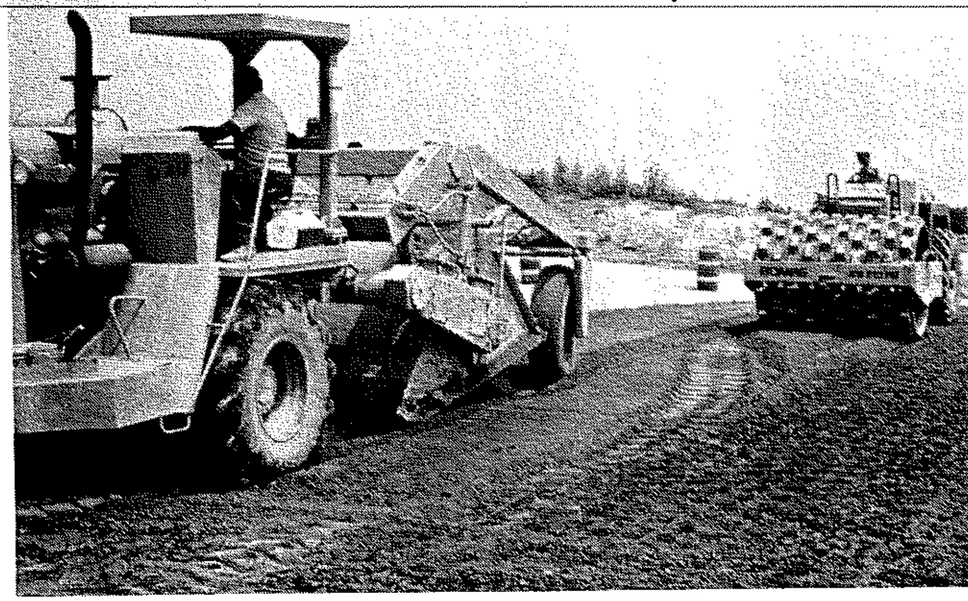

Figure 18. *Illustrative of a typical soil pulverization and compaction sequence.*

Figure 19. *A road grader was used to trim the modified subgrade to elevation grade.*

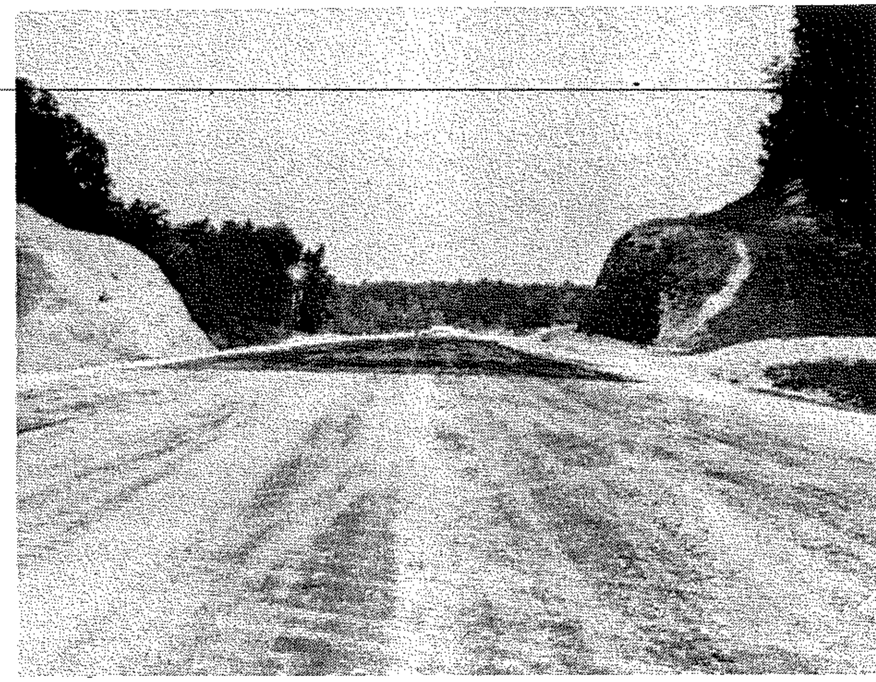

Figure 20. *An overview of the completed AFBC spent lime modified subgrade near Station 275+00.*

it was necessary for the subcontractor to construct windrows along the subgrade shoulders to contain the AFBC spent lime material and water on the subgrade. When the water was placed on the AFBC spent lime, a significant amount of steam was produced, reducing visibility to near zero. Another problem was the absence of cut-off valves inside the cabs of the water trucks. On several occasions, a water truck would become stuck in the mud and discharge excessive water onto the subgrade. In the cab cut-off valves were installed after the first day. The subcontractor spent considerable time cutting the modified soil to grade elevation. Because the incorporation of the AFBC spent lime increased the soil volume, nearly four inches of the modified soil had to be trimmed to obtain proper grade elevation. However, the modified soil subgrade was easily trimmed even 24 to 30 hours after final compaction.

Department of Highways' inspectors experienced some difficulties in obtaining correct moisture readings from the nuclear density gage. The difficulties were attributed to the high amount of hydrocarbons contained in the modified soil. After the problem was identified, the inspectors determined the actual moisture content by applying a moisture-content correction factor. The correction factor was determined by field drying a soil sample to determine the correct moisture content and entering the correction factor into the nuclear density machine.

Soil-Cement Subgrade Section -- STA 317+50 to STA 348+00

Construction requirements for portland cement roadbed modification are summarized in Section 304 of the Kentucky Department of Highways' Standard Specification for Road and Bridge Construction. The specification requires scarification and pulverization of the soil subgrade prior to the application of the cement and recommends that pulverization continue during the mixing process until 100 percent of the soil passes a one-inch sieve and at least 80 percent of the soil passes a No. 4 sieve. The specification states that the moisture not be below the specified optimum moisture nor more than one fifth above the specified optimum moisture content. Compaction of the soil-cement mixture subgrade is at least 95 percent of the standard Proctor.

The spreader trucks had their tops replaced and were loaded directly from pneumatic tanker trucks from the Kosmos Cement Company of Louisville. The application rate for the type 1P Portland cement was ten percent (by dry weight of the soil) for this section. The construction procedures used within this section did not appear to vary significantly from those employed in the soil-AFBC spent lime modified section. Contrary to the specified construction procedures, the cement and water were spread over the prepared subgrade and mixed. The pulverization requirement was easily achieved. Required density and moisture of the compacted subgrade were within specified tolerances. The subcontractor had no problems with the Portland cement modification other than the fact that the soil-cement set rather quickly. Because of the quick setting time, the subgrade had to be cut to final grade elevation within five hours after the mixing operation began. A bituminous curing seal was placed to prevent excessive evaporation after the subgrade was cut to grade.

Soil-Hydrated Lime Subgrade Section -- STA 348+00 to STA 402+50

The soil-hydrated lime section was also deemed an experimental section within this project. The initial experimental soil-hydrated lime roadbed had been constructed during 1986 on the AA highway in northeastern Kentucky. Construction requirements for the hydrated lime subgrade modification are outlined in Kentucky Department of Highways' Special Note for Lime Roadbed Stabilization (Experimental). The Special Note is contained in Appendix B of this report. The Special Note for lime roadbed stabilization is similar in detail to that for AFBC residue roadbed stabilization. The Special Note requires two-thirds of the specified quantity of lime to be spread and thoroughly mixed into the soil. Moisture is added to the soil so that the moisture content is no less than optimum, nor more than five percent above optimum. After this primary mixing phase, the Special Note states that the modified layer be shaped to the approximate section and lightly compacted and allowed to mellow for a period of 48 hours. Following the mellowing period, the remaining one-third of the lime is to be spread and mixed until 100 percent of the modified soil passes a one-inch sieve and at least 60 percent passes a No. 4 sieve. If necessary, additional water is added to raise the moisture content before final compaction.

The prepared subgrade within this section was dry and very hard. The spreader trucks were loaded directly via pneumatic tanker trucks which transported the material from Dravo Lime Company's Maysville plant (see Figure 21). The application rate for the hydrated lime was seven percent by dry weight of the soil. The hydrated lime was spread full width by the spreader trucks over the prepared subgrade. Water was added and the mixture was pulverized in sections not exceeding 200 feet. It was not determined whether the subgrade subcontractor utilized the primary mixing and preliminary curing (mellowing) period as detailed in the Special Note.

Because of the fineness of the hydrated lime, windrows were constructed along the shoulders to keep the hydrated lime and water from escaping over the side slope. The subcontractor indicated that the hydrated lime was harder to work with than the AFBC spent lime, primarily due to the fineness of the hydrated lime. The setting time of the soil-hydrated lime mixture was somewhat longer than the soil-AFBC mixture. The subcontractor had very little trouble cutting the treated subgrade to grade elevation even 48 hours after incorporating the hydrated lime into the subgrade.

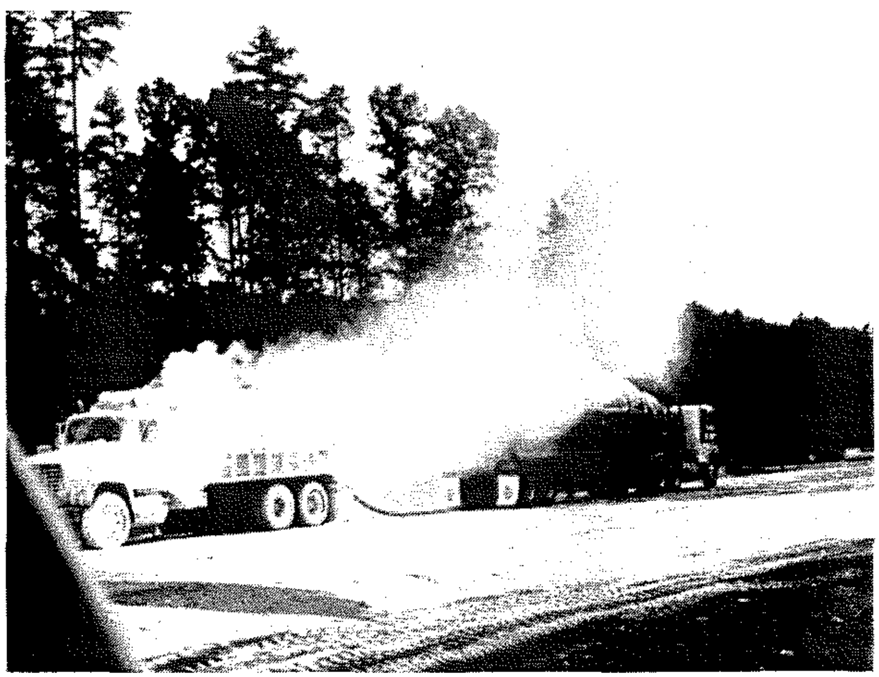

Figure 21. *The spreader trucks were filled with hydrated lime directly from pneumatic tanker trucks.*

Soil-Multicone Kiln Dust Subgrade Section -- STA 402+50 to STA 429+50

The soil-multicone kiln dust subgrade modification section was experimental and this was the first use of the waste material to modify a soil subgrade. The construction requirements for the multicone kiln dust (MKD) subgrade modification are outlined in Kentucky Department of Highways' Special Note for MKD Roadbed Stabilization (Experimental). The Special Note for MKD soil subgrade modification is contained in Appendix C of this report. The Special Note was developed by engineers exclusively for this project. The Special Note requires primary and final mixing. Two thirds of the MKD was specified to be placed initially during the primary mixing phase. The moisture content of the modified soil during the primary mixing phase should be no less than optimum, and no more than five percent greater than the optimum moisture content. After primary mixing, the modified soil layer is shaped to the approximate cross section and lightly compacted to minimize evaporation loss. Following primary mixing, preliminary curing (mellowing) is required for at least 48 hours. During the preliminary curing phase, the surface of the subgrade is to be kept moist to prevent drying and cracking. Immediately after the preliminary curing phase, the remaining one third of the MKD is spread and the stabilized layer completely mixed and pulverized again. Final mixing continues until all clods are broken down so that 100 percent, exclusive of rock particles, passes a one-inch sieve and at least 60 percent passes a No. 4 sieve. However, it is stated that if the pulverization requirement can be met during the primary mixing phase, the preliminary curing and final mixing steps can be eliminated. After compaction and shaping to grade elevation, a bituminous curing seal is required to prevent excessive loss of moisture. No vehicular traffic is allowed to traverse the subgrade after placement of the curing seal until after a period of seven days.

The MKD waste materials were supplied by the Dravo Lime Company's Maysville plant. The spreader trucks were filled directly using pneumatic tanker trucks. Unlike previous sections, the prepared subgrade was ripped with a road grader having a ripper attachment prior to placing the MKD. Figure 22 shows a spreader truck distributing MKD on the ripped subgrade. Ripping the soil subgrade prior to placing the MKD and water permitted easier incorporation of the chemical admixture and moisture in the soil. The spreader trucks distributed the MKD material on the ripped subgrade. Water was then added and the soil subgrade was pulverized. The sheepsfoot roller provided initial compaction. Inspectors checked for moisture content after initial compaction. When there was not sufficient moisture in the subgrade, then additional water was added, the soil was pulverized and compacted again. The working area for the MKD section was shoulder to shoulder, and approximately 200 feet in length.

Figure 22. *The prepared subgrade was ripped prior to distributing the multicone kiln dust with the spreader truck.*

Ripping the subgrade prior to placing the MKD material precluded the need to construct windrows to contain the stabilizing agent and water on the subgrade. The pulverization requirement was met with one pass, thereby eliminating the need for a mellowing period. Required density and moisture of the compacted subgrade were easily achieved. The subcontractor indicated that the MKD was very easy to work with. The setting time was very similar to that of the soil-hydrated lime mixture and somewhat longer than the soil-AFBC mixture. The subcontractor had no trouble cutting the treated subgrade to grade elevation 48 hours after incorporating the MKD into the subgrade.

Soil-Cement Subgrade Section -- STA 429+50 to STA 522+00

The construction procedures were similar to those used beforehand on the previously constructed section except that the subgrade was scarified prior to applying the cement. Seven percent type 1P Portland cement was incorporated into the soil. There were no construction difficulties observed within this section.

Untreated Soil Subgrade Section -- STA 522+00 to STA 532+00

A 1,000-foot section of the subgrade was not stabilized and served as a control section for the project. Conventional compaction methods were employed to construct the subgrade within this section.

Soil-AFBC Subgrade Section -- STA 532+00 to STA 576+50

This section was conceived after construction difficulties were encountered on the first AFBC spent lime modified subgrade section. The construction procedures were altered from those used on the initial section to include ripping the prepared subgrade prior to spreading the AFBC spent lime material. After the AFBC spent lime materials were spread over the ripped subgrade, water was added and the soil was pulverized. The application rate of the AFBC spent lime was seven percent by dry weight of the soil for this section. The subgrade was checked for the proper moisture content and dry density after initial compaction with a vibratory sheepsfoot roller. When the moisture content was within a range of +/- two percent and the dry density was equal to or greater than 95 percent of maximum dry density, final compaction was completed using the smooth-wheel vibratory roller. After completing compaction requirements, the modified subgrade was cut to elevation grade and a curing seal of bituminous emulsion was sprayed. Generally, vehicular traffic was prevented from traversing the subgrade for a period of seven days.

CONSTRUCTION EVALUATIONS

Investigations relative to the engineering properties of the modified soil subgrades continued during construction of the modified subgrade. Field testing consisted of moisture content / dry density tests for construction compliance, and in-place bearing capacity tests, moisture content tests and Road Rater deflection tests performed on the subgrade both before and after modification with chemical admixtures. Test procedures and results are described herein.

Field Density and Moisture Content Compliance Tests

Field density tests of the compacted soil subgrade were performed by Kentucky Department of Highways' personnel using a Troxler moisture-density nuclear gage. The manufacturer's recommended procedures were followed while conducting the field density tests.

Results of the field moisture-density tests obtained from the nuclear density gage are summarized in Tables 8 through 10. Maximum dry density and optimum moisture contents of subgrade samples collected at various station numbers are shown in the left portion of those tables. Maximum dry densities obtained from standard compaction tests (ASTM D 698-78) may be adjusted on the basis of the percent material retained on the No. 4 sieve and according to a nomograph in the Kentucky Department of Highways' (KDOH) manual of Kentucky Test Methods (1983). In some instances, KDOH personnel adjusted the laboratory maximum dry density for oversized material. However, adjustments in laboratory optimum moisture contents were not always made because the percent oversized material retained on the No. 4 screen was very small in many cases. The adjusted, or reference, values of maximum dry densities were used to compare to values of dry density obtained from the nuclear density meter. Relative compaction is the ratio of adjusted, maximum dry density to field dry density, and is shown in Tables 8 through 10. Based on 84 nuclear density tests taken on all sections of KY 11, the relative compaction averaged 98.2 percent with a standard deviation of +/- 2.6 percent. Since specifications required that all field dry densities be 95 percent of maximum dry density, all subgrade sections were compacted according to the dry density specification.

With regard to the moisture content, compaction specifications generally required that the field moisture content be no less than the optimum moisture content nor more than five percent above optimum moisture. Differences between field and optimum moisture contents are shown in the right-most column of Tables 8 through 10. A negative sign in front of the difference indicates that the field moisture content was less than the optimum moisture content and a positive value indicates that the field moisture content was greater than optimum moisture content. An average value of the differences between field and optimum moisture contents was -0.2 percent. This means that generally the subgrades were compacted at moisture contents only slightly dry of optimum. In 39 cases, field values of moisture content met moisture content specifications. In 45 of 84 tests conducted along the subgrades of KY 11, the moisture content specification was not met. In 44 of the 45 failure cases, the field moisture content was dry of the optimum moisture content. In one of the 45 failure cases, the field moisture content was more than optimum moisture plus five percent.

TABLE 8. DRY DENSITY AND MOISTURE CONTENT COMPLIANCE DATA FOR MODIFIED SOIL SUBGRADES

	AASHTO T-99 Laboratory Data		Adjusted Laboratory Data			Field Moisture-Dry Density Tests from Nuclear Gage			
Station Number	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	+ No. 4 Material (%)	Reference Maximum Dry Density (pcf)	Location of Field Test (STA. No.)	Dry Density (pcf)	Moisture Content (%)	Relative Compaction (%)	Field Water Content Minus Optimum Water Content (%)
AFBC Spent Lime Modified Subgrade Soil, Station 260+00 to Station 317+50									
264+00	105.0	16.0	23.2	113.0	262+00	109.2	16.4	96.6	0.4
			23.2	113.0	264+10		17.1	96.6	1.1
			17.8	111.0	268+00		18.5	98.4	2.5
			17.8	111.0	273+00		16.3	98.4	0.3
269+00	112.0	15.0							
274+00	109.0	15.0	22.1	116.2	276+00	110.7	17.1	95.3	2.1
			22.1	116.2	277+00	110.5	17.2	95.1	2.2
279+00	108.0	15.0	15.5	113.0	280+00	108.0	16.9	95.6	1.9
284+00	113.0	15.0	6.9	115.0	285+00	109.4	16.1	95.1	1.1
289+00	106.0	16.0							
294+00	108.0	18.0	12.5	112.0	296+00	107.8	17.9	96.3	-0.1
299+00	105.0	16.0	14.9	110.0	312+00	109.4	16.1	99.5	0.1
				110.0	315+00	104.5	20.3	95.0	4.3
304+00	113.0	14.0							
309+00	116.0	14.0							
Cement Modified Subgrade Soil (10%), Station 317+50 to Station 348+00									
314+00	116.0	9.0	21.2						
				116.0	318+50	113.1	9.8	97.5	0.8
319+00	116.0	12.0	11.5						
324+00	113.0	11.0	4.7	116.0	320+00	116.0	14.2	100.0	3.2
329+00	113.0	11.0	4.3						
334+00	121.0	11.0	20.1						
344+00	126.0	10.0	31.2						
Hydrated Lime Modified Subgrade Soil (7%), Station 348+00 to Station 402+50									
349+00	122.0	9.0	24.2						
354+00	123.0	9.0	19.1						
359+00	118.0	13.0	10.0						
364+00	121.0	12.0	11.5						
Multicone Kiln Dust Modified Subgrade Soil (7%), Station 402+50 to Station 429+50									
404+00	110.0	18.0	7.9						
409+00	112.0	14.0	5.9						
414+00	112.0	19.0	5.4						
419+00	115.0	15.0	8.4	111.0	422+50	110.0	15.8	99.1	0.8
424+00	115.0	11.0	6.5	115.0	426+00	110.0	14.9	95.6	3.9
429+00	117.0	10.0	15.4						

* indicates moisture content not within specification limits.

TABLE 9. DRY DENSITY AND MOISTURE CONTENT COMPLIANCE DATA FOR SOIL-CEMENT (7%) SECTION -- STA. 429+50 to STA. 522+00

	AASHTO T-99 Laboratory Data		Adjusted Laboratory Data			Field Moisture-Dry Density Tests from Nuclear Gage			
Station Number	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	+ No. 4 Material (%)	Reference Maximum Dry Density (pcf)	Location of Field Test (STA. No.)	Dry Density (pcf)	Moisture Content (%)	Relative Compaction (%)	Field Water Content Minus Optimum Water Content (%)
434+00	111.0	16.0	9.2						
	112.0	16.0	7.1						
439+00	111.0	15.0	9.2						
444+00	111.0	16.0	5.2	111.0	445+00	108.1	18.0	97.4	2.0
				111.0	443+00	108.9	15.2	98.1	-0.8*
449+00	116.0	14.0	8.2	116.0	434+00	114.9	12.5	99.1	-1.5*
				116.0	436+50	115.2	13.4	99.3	-0.6*
				116.0	438+50	114.3	12.9	98.5	-1.1*
				116.0	441+00	110.4	13.7	95.2	-0.3
				116.0	448+25	110.6	10.8	95.3	-3.2*
				116.0	442+50	110.0	16.5	94.8	2.5
				116.0	445+50	111.4	12.2	96.0	-1.8*
				116.0	447+50	112.5	15.5	97.0	1.5
454+00	113.0	15.0	4.8	113.0	453+00	111.7	14.2	98.8	-0.8*
459+00	117.0	14.0	5.1	117.0	459+00	110.2	15.6	94.2	1.6
				117.0	456+00	110.6	14.9	94.5	0.9
464+00	110.0	16.0	1.5	110.0	466+00	112.8	14.1	102.5	-1.9*
				110.0	462+50	106.4	13.9	96.7	-2.1*
469+00	111.0	17.0	7.6	111.0	465+00	109.3	16.2	98.5	-0.8*
				111.0	469+00	110.7	14.7	99.7	-2.3*
				111.0	469+20	119.1	13.6	107.3	-3.4*
474+00	112.0	15.0	1.8	112.0	473+00	107.4	18.0	95.9	3.0
				112.0	470+00	106.1	14.1	94.7	-0.9*
479+00	112.0	18.0	6.9	112.0	473+50	106.9	18.5	95.4	0.5
484+00	116.0	14.0	8.2						
449+00	116.0	14.0	8.2	116.0	464+00	113.6	14.1	97.9	0.1
					461+00	115.8	14.0	99.8	0.0
					456+00	111.3	14.3	95.9	0.3
					453+50	115.4	14.9	99.5	0.9
					448+50	116.7	13.1	100.6	-0.9*
484+00	116.0	14.0	8.2	116.0	490+00	110.5	14.3	95.3	0.3
				112.0	500+00	114.5	14.2	102.2	0.2
				112.0	499+50	112.7	13.3	100.6	-0.7*
				112.0	494+00	112.6	12.9	100.5	-1.1*
				112.0	496+50	112.5	11.3	100.4	-2.7*
509+00	112.0	15.0	18.5	117.0	512+00	111.5	19.6	95.3	4.6
				112.0	505+00	114.9	12.2	102.6	-2.8*
				112.0	506+25	112.2	13.2	100.2	-1.8*
				112.0	510+75	117.1	13.0	104.6	-2.0*
519+00	116.0	11.0	5.9	116.0	518+50	111.4	14.5	96.0	3.5
				116.0	519+00	112.1	14.3	96.6	3.3
				116.0	521+50	113.7	14.5	98.0	3.5
				116.0	514+00	110.6	8.7	95.3	-2.3*
				116.0	516+00	111.3	14.0	95.9	3.0

* indicates moisture content not within specification limits.

TABLE 10. DRY DENSITY AND MOISTURE CONTENT COMPLIANCE DATA FOR UNTREATED SOIL SECTION AND SOIL-AFBC SECTION -- STA. 532+00 to STA. 576+50

	AASHTO T-99 Laboratory Data		Adjusted Laboratory Data			Field Moisture-Dry Density Tests from Nuclear Gage			
Station Number	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	+ No. 4 Material (%)	Reference Maximum Dry Density (pcf)	Location of Field Test (STA. No.)	Dry Density (pcf)	Moisture Content (%)	Relative Compaction (%)	Field Water Content Minus Optimum Water Content (%)
Untreated Subgrade Soil, Station 522+00 to Station 532+00									
524+00	113.0	16.0	6.8	113.0	526+00	108.1	21.1	95.7	5.1*
				113.0		115.1	14.9	101.9	-1.1*
529+00	111.0	17.0	7.0	115.5	529+15	117.6	13.5	101.8	-3.5*
				115.5	524+85	116.0	15.1	100.4	-1.9*
				117.2	531+75	111.3	12.7	95.0	-4.3*
AFBC Spent Lime Modified Subgrade Soil, Station 532+00 to Station 576+00									
534+00	116.0	14.0	9.9	116.0	547+25	114.6	13.9	98.8	-0.1
					545+50	111.9	15.7	96.5	1.7
					542+00	117.7	11.8	101.5	-2.2*
539+00	110.0	13.0	5.7						
544+00	115.0	14.0	8.4						
545+00	113.0	14.0	7.9	113.0	540+00	112.1	13.5	99.2	-0.5*
				113.0	537+00	116.5	17.0	103.1	3.0
				113.0	532+75	113.6	13.8	100.5	-0.2
				113.0	537+00	112.9	15.2	99.9	1.2
				113.0	534+50	114.3	14.0	101.2	0.0
554+00	117.0	13.0	10.7	117.0	549+00	113.0	13.9	96.6	0.9
				117.0	554+25	113.0	12.8	96.6	-0.2
				117.0	551+25	119.2	8.8	101.9	-4.2*
559+00	110.0	17.0	3.8	114.0	561+50	108.4	18.6	95.1	1.6
				114.0	557+10	111.8	15.6	98.1	-1.4*
				114.0	561+50	108.2	12.9	94.9	-4.1*
				114.0	565+50	109.9	16.5	96.4	-0.5
				114.0	574+00	109.7	14.6	96.2	-3.4*
569+00	114.0	15.0	6.8	114.0	572+50	113.7	13.8	99.7	-1.2*
				114.0	569+50	114.0	13.5	100.0	-1.5*
				114.0	576+00	116.7	13.0	102.4	-2.0*
				114.0	573+00	113.9	11.8	99.9	-3.2*
				114.0	570+50	113.0	12.5	99.1	-2.5*
				114.0	569+00	111.3	16.0	97.6	1.0
				114.0	565+00	111.4	10.4	97.7	-4.6*
				114.0	565+25	111.4	10.4	97.7	-4.6*

* indicates moisture content not within specification limits.

In-Place CBR Tests

In-place CBR tests were generally performed before and shortly after admixture modification of the subgrade soil. Unfortunately, this was not the case for all sections. In-place CBR tests after the seven-day curing period were performed only on the two soil-

AFBC modified subgrade sections. Measurement of the subgrade bearing capacity, by KTC personnel, was in general accordance with ASTM D 1883-87, except that the tests were performed on the soil in its actual in-situ condition. Moisture content of the soil was determined in accordance with ASTM D 2216-80. Values of the in-situ CBR and corresponding subgrade moisture content of the natural soil are contained in Table 11. The in-situ bearing strength and moisture content of the untreated subgrade materials were generally high and low, respectively. The moisture content of the untreated clayey subgrade soil was quite low. The low moisture content effected an elevated subgrade bearing capacity of the natural soil prior to admixture modification.

TABLE 11. COMPARISON OF FIELD CBR VALUES FOR SOIL-AFBC SECTION, STA. 260+00 TO STA. 317+50

Station Number	Age at Test (days)	In-Situ CBR	In-Situ Moisture Content (%)
267+00 lt	Before treatment	37	7.6
267+00 lt	After seven days	*	6.1
268+00 rt	Before treatment	26	6.1
268+00 rt	After seven days	39	6.9
270+50 rt	Before treatment	*	7.4
270+50 rt	After seven days	47	5.3
273+00 lt	Before treatment	*	13.5
273+00 lt	After seven days	36	13.4
278+00 rt	Before treatment	42	18.1
278+00 rt	After seven days	34	7.8
280+50 lt	Before treatment	20	11.8
280+50 lt	After seven days	*	11.1
293+00 rt	Before treatment	22	12.5
293+00 rt	After seven days	36	11.4
293+50 lt	Before treatment	30	16.1
293+50 lt	After seven days	53	19.0

* indicates insufficient data for CBR computation.

The untreated subgrade of the soil-AFBC section, from Station 260+00 to Station 317+50, had an average in-situ CBR of 30 and ranged from about 20 to 42 for the eight tests performed. The moisture content of the untreated subgrade averaged 11.6 percent and ranged from about six percent to 18 percent. Seven days after chemical admixture modification, the tests were repeated at the same locations. In-place CBR's ranged from 34 to 53 and the in-situ moisture content ranged from five to 19 percent. Overall, the average in-place CBR increased to about 41 while the in-situ moisture content decreased to 10.1 percent. The decrease in the moisture content of the soil can only be attributable to the incorrect moisture content readings obtained from the use of the nuclear density device. The optimum moisture content for this section was about 16 percent. The moisture content of the soil-AFBC spent lime mixture averaged 17.3 percent as measured with the nuclear density gage during construction. Some moisture readings recorded prior to applying a moisture-content correction factor. Five of the eight locations tested after seven days exhibited an increase in bearing strength after soil modification. There was not a significant change in the moisture content of the treated versus untreated soils.

The in-situ CBR of the untreated subgrade within the soil-lime section could not be ascertained due to insufficient data obtained for CBR computation. Field personnel conducted six in-situ CBR tests within the section. It was found that the untreated subgrade soil was extremely hard and exhibited high bearing capacity. The moisture content of the natural soil ranged from 5.8 percent to 8.6 percent and averaged 6.8 percent. Seven in-situ CBR tests were performed within the soil-multicone kiln dust section prior to admixture modification. The untreated subgrade had an average in-situ CBR of 30 and ranged from about 10 to 40. The moisture content of the untreated subgrade averaged 8.8 percent and ranged from 6.8 percent to 12.0 percent. Only two in-situ CBR tests were performed in the soil-cement section, from Station 429+50 to Station 522+00. The untreated subgrade within this section had an in-situ CBR range from about 11 to 13. The moisture content of the untreated subgrade ranged from 14.2 percent to 15.5 percent. Two in-situ CBR tests also were performed in the untreated subgrade section. The subgrade within this section had an in-situ CBR range of 15 to 44. The moisture content of the untreated subgrade ranged from 16.5 percent to 24.9 percent.

Two in-situ CBR tests were performed in the second soil-AFBC section (Station 532+00 to Station 576+50) prior to soil modification procedures. Eight CBR tests were performed seven days after modification procedures, although KTC personnel could not perform the tests in the locations tested previously. The two tests performed prior to modification indicated an in-situ CBR range of 7 to 48. The moisture content of the untreated subgrade ranged from 10.7 percent to 15.5 percent. The in-situ CBR value of the eight tests performed after the seven-day curing period averaged 56 and ranged from 37 to 73. The moisture content of the modified soil averaged 18.9 percent and ranged from 13.3 percent to 22.1 percent. The number of data points preclude any comparison of untreated in-situ CBR and moisture content values with values seven days after modification.

Road Rater Deflection Tests and Analyses

The Kentucky Transportation Center Model 400B Road Rater was used for structural evaluation of each admixture section and the control (untreated) section. The Model 400B Road Rater is a dynamic pavement testing device which applies a steady state vibratory load to the pavement. The magnitude of the steady state vibratory load is a function of the frequency and amplitude of the vibrating mass. The mass for the Model 400B Road Rater is constant at 300 pounds. Frequency may be varied from 0.0 to 0.1 inch. The steady state vibratory load applied by the Road Rater impulses the pavement. The forced motion of the pavement is measured by velocity sensors. The vibrating mass of the Road Rater is suspended by a system of rubber bellows which distribute the load concentrically about the lifting cylinder. A second set of bellows is used to provide for equal distribution of the loading to two feet.

The Road Rater is a hydraulically actuated system with both the raising and lowering of the system to the roadway surface and the vibratory motion of the mass controlled and actuated by an electro-hydraulic system. Data acquisition is computer controlled using a Hewlett-Packard 85B microcomputer with data storage on magnetic tape and also a paper tape. Override mechanisms are available which permit manual operation and manual data collection. A range of dynamic loadings is possible depending upon the selection of frequencies and amplitudes of vibration. Practically, the loading limits of the Road Rater 400B system are 0 to 2,400 pound-force. A frequency of 25 Hz was selected for all testing activities. Past experience has indicated that this frequency generally results in consistent response characteristics for all velocity transducers. Amplitudes were varied to produce dynamic forces of 600 and 1,200 pound-force for testing. Loadings are transmitted to the roadway surface via two load feet. Road Rater deflection tests were performed on the prepared natural subgrade prior to treatment with an admixture. Deflection tests were later conducted after placement and mixing of each subgrade section with a chemical admixture. Subsequent deflections were obtained after placement of the crushed stone layer and after placement of each course of the asphaltic concrete pavement. The deflection data were used to estimate the elastic moduli of the subgrade layer.

The Road Rater vibratory loading is approximately sinusoidal. The dynamic loading (sine wave) of the Road Rater has been approximated as a square wave. Superposition principles may be used to compute the surface deflection at each velocity transducer location. Deflections are computed for the loadings associated with one of the load feet. By symmetry and superpositioning, the deflections for one load foot may be doubled to represent the deflections associated with the two load feet.

The Road Rater applies a dynamic loading to the pavement. In theory, dynamic and/or wave propagation analyses techniques should be used for analysis of deflections. However, for the sake of simplification, the measured deflection basins have been interpreted in terms of static analyses and layer elastic theory. More specifically, measured Road Rater deflection basins have been assumed to have resulted from a static load with a peak to peak vibratory load superimposed on the static load. In this situation, the static load used in analysis and interpretation of the dynamic deflections is the peak-to-peak magnitude of the square wave.

Elastic layer principals may be used to compute theoretical deflections for the applied loadings and the specific locations of each Road Rater velocity transducer. There are a number of multi-layer elastic computer programs which may be used to compute deflections, stresses and strains in pavements. The Chevron N-Layer computer program, used in Kentucky to model pavement behavior, was used for initial analysis of the deflection data. The following input parameters are required as input into the Chevron N-Layer computer program:

- thickness of each layer (inches),
- Young's modulus of elasticity for each layer (psi),
- Poisson's ratio for each layer,
- the coordinate of each required answer point (corresponding to the location of each velocity transducer),
- loading applied to the road surface,
- contact pressure (applied load / contact area) for one loading foot of the Road Rater.

An array of layer moduli was used in combination with the constructed layer thicknesses and assumed values of Poisson's ratio. These parameters were entered into the Chevron N-Layer computer program to generate a matrix of simulated surface deflection basins corresponding to a number of combinations of layer moduli.

Simulations were initially determined for two different conditions:

- simulation No. 1: deflections on untreated subgrade (12 inches) over a semi-infinite untreated layer; and
- simulation No. 2: deflections on treated subgrade (12 inches) over a semi-infinite layer of untreated subgrade.

Each simulation utilized a multi-layered elastic approach to compute theoretically expected deflections. Simulation No. 1 was used in combination with measured deflections on untreated subgrades to determine the elastic stiffness of the untreated subgrade prior to admixture modification. Equations were determined for each Road Rater sensor location wherein deflection was related to elastic stiffness or modulus of elasticity. Measured deflections corresponding to each sensor location were used as input into appropriate equations to determine associated elastic moduli (stiffnesses) corresponding to each sensor. Using simulation No. 1, the average elastic stiffness or modulus of the untreated soil subgrade (Station 262+00 to 562+00), was 24 ksi.

Simulation No. 2 was used in combination with measured deflections on treated subgrade materials to determine the elastic stiffness of the treated subgrade layer. Equations relating elastic stiffness and deflections were determined for each Road Rater sensor location. Measured deflections were used to determine associated elastic moduli (stiffnesses). Resulting mean moduli of the chemically modified layers are summarized in Table 12 and are compared with the estimated layer stiffnesses resulting from in-situ CBR tests. The results of these analyses were checked by comparing the deflection basins for the mean of the measured deflections (for each section) versus the modelled deflection basins from the elastic layer simulations. Results of subsequent Road Rater deflection tests performed on the various layers of crushed stone and asphaltic concrete materials are presented in the discussion of post-construction evaluations.

TABLE 12. ESTIMATED LAYER MODULI FROM ROAD RATER DEFLECTIONS

Section	Stations Tested		Mean Moduli Estimated from Road Rater Tests					Estimated Stiffness of In-situ CBR Tests (ksi)
	Beginning Station	Ending Station	Treated (ksi)	Untreated (ksi)	Modular Ratio Treated/Untreated	CBR Laboratory	CBR In situ	
Before Treatment								
Subgrade	262+00	562+00		24		4	32	48
Seven Davs After Treatment								
AFBC	263+00	292+00	73	24	3.1	48	56	84
Cement	326+00	338+50	137	24	5.7	300		
Hydrated Lime	376+00	401+00	46	24	1.9	67		
Multicone Kiln Dust	422+50	429+50	93	24	3.9			
AFBC	532+00	540+00	77	24	3.2	48	46	69

POST-CONSTRUCTION EVALUATIONS

After construction of the highway sections, researchers continued monitoring of the experimental and control sections. The initial post-construction analysis involved re-examining the expansive characteristics of the soil-AFBC spent lime mixture. Performance monitoring of the chemically modified subgrade soils included performing in-place bearing capacity tests on the subgrades, obtaining undisturbed samples to analyze in the laboratory relative to unconfined compressive strength, moisture content, and soil classification, monitoring pavement elevations for swell attributes, and performing Road Rater deflection measurements on the completed pavement structure.

Expansive Characteristics of the Soil-AFBC Spent Lime Mixture

Approximately two months after construction of the soil-AFBC spent lime subgrades, severe differential swell or heave occurred as shown in Figures 23 and 24. The swell or humps occurred almost immediately after rainy periods. A close-up view of a swell area and pavement crack is shown in Figure 25. The subgrade swelling was unexpected since prior laboratory tests of the soil-AFBC spent lime mixture indicated total swell of only 3.1 percent compared to the swell of the natural soil of 3.9 percent. Additionally, during construction, a specimen molded from a bag sample obtained during field mixing of the AFBC spent lime and soil (near Station 262+25) was determined to have a total swell of slightly less than three percent.

To investigate the pavement heave, a trench was excavated at Station 279+80. The soil-AFBC spent lime subgrade had heaved or swelled considerably. Both undisturbed and disturbed soil samples were obtained from the trench. Also, field moisture-density measurements were performed on the swollen AFBC-soil subgrade. To determine the swelling potential of compacted soil-AFBC spent lime mixtures, additional CBR-swell tests were performed. Using a bag sample of the soil-AFBC spent lime mixture obtained from the trench at Station 279+80, a sample, identified as 7-1FT in Table 13, was remolded in a CBR mold to the average values of field moisture content and dry density measured on the soil-AFBC spent lime subgrade. These values were 26.4 percent and

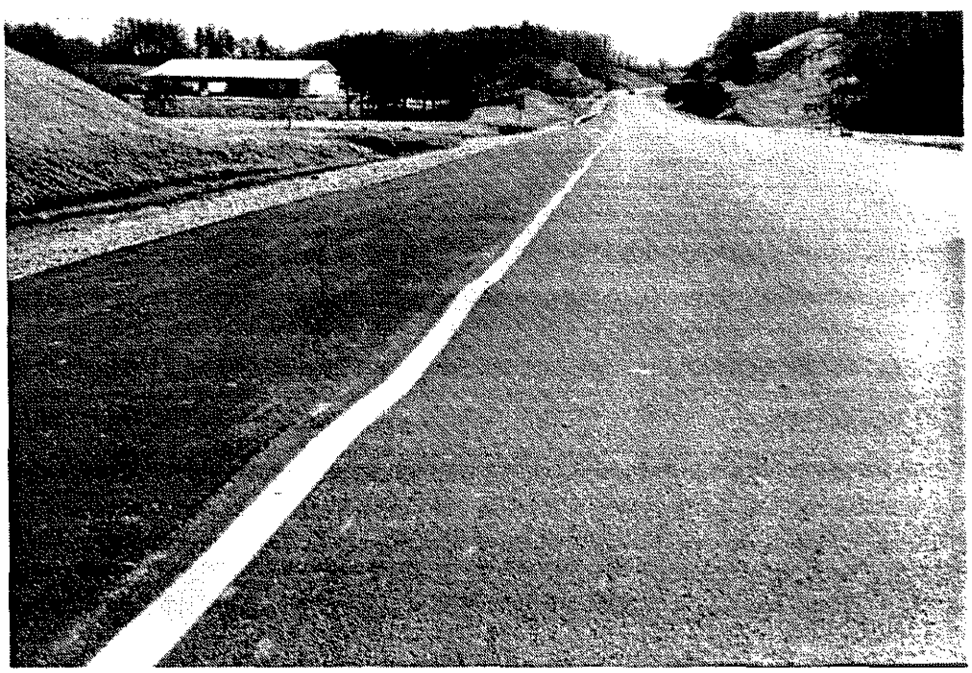

Figure 23. *Differential heave of the pavement surface was observed in the northbound lane near station 282+00.*

Figure 24. *View of humps on pavement surface near Station 560+00.*

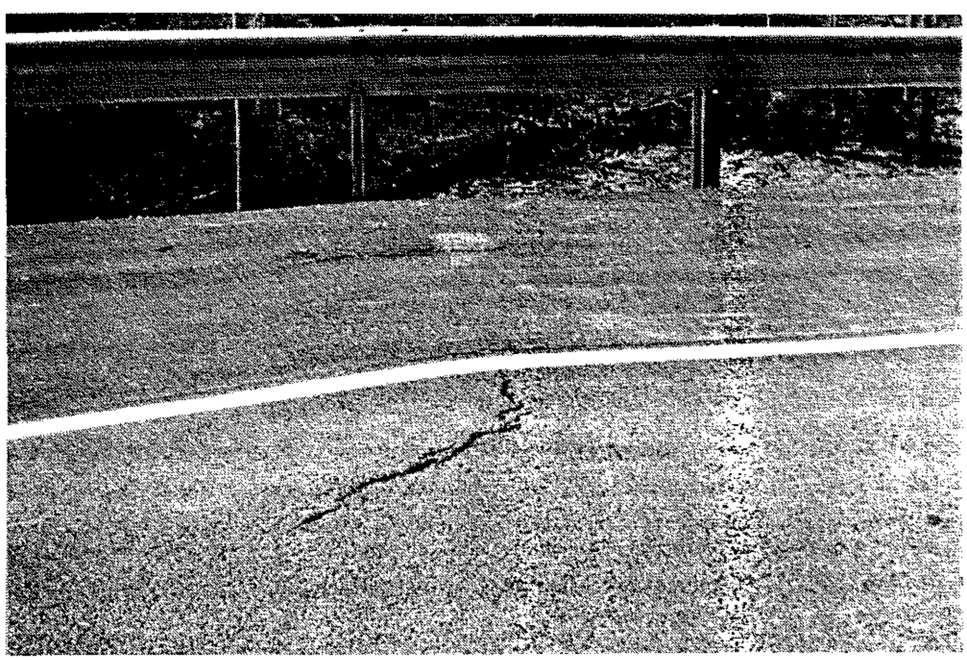

Figure 25. *Close-up view of cracked pavement and hump in southbound lane near Station 292+00.*

TABLE 13. KYCBR AND EXPANSION VALUES OF REMOLDED SOIL-AFBC SPECIMENS

		Soaked Penetration CBR Value		At Compaction		After Test		
Sample Number	Type and Percentage of Chemical Admixture	0.1-inch	0.5-inch	Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Moisture Content (%)	Total Swell (%)
7-1FB	AFBC (7%) Bag Sample	11.3	8.3	99.6	23.5		28.8	2.4
7-1FT	AFBC (7%) Trench Sample	57.7	39.5	93.6	25.9		31.2	0.8
15-LAB1	AFBC (15%)			104.0	12.6			25.7
15-LAB3	AFBC (15%)			97.9	13.9			26.3
30-LAB1	AFBC (30%)			94.4	15.1			26.5
30-LAB3	AFBC (30%)	1.7		91.0	11.5	79.8	41.9	24.3

98.1 pcf, respectively. The ASTM bearing ratio values at 0.1-inch and 0.5-inch penetrations were 58 and 39, respectively. Swell (strain) of this sample as a function of the logarithm of time is shown in Figure 26. Total swell of this sample in a period of about 48 days was only 0.8 percent. In the ASTM bearing ratio method, Designation D 1883 (78), specimens are soaked for 96 hours and then the bearing ratio test is performed. However, sample 7-1FT was allowed to swell until the difference between consecutive readings was less than 0.003 inch. Based on the curve in Figure 26, the primary portion of the swell appeared to have ceased and secondary swell strain measurements were not obtained.

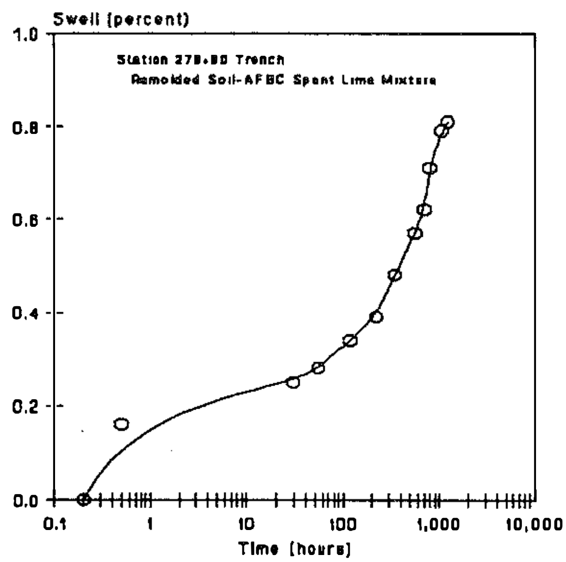

Figure 26. *Swell of a remolded specimen from the trench at Station 279+80 as a function of the logarithm of time.*

To examine the swelling nature of soil-AFBC spent lime mixtures, four additional CBR swell tests were performed on remolded specimens (stockpile Stations 273+00 and 574+00), using 15 percent and 30 percent of the AFBC material. These tests are identified in Table 13 as 15-LAB1, 15-LAB3, 30-LAB1, and 30-LAB3. Specimen designations ending in "1" were remolded using material from the stockpile at Station

273+00. Specimen designations ending in "3" were remolded using material from the stockpile at Station 574+00. The percent AFBC material used in these tests was higher than the seven percent used in the field. Hence, swell strains measured in these tests may be higher than strains observed for a lesser percent of the AFBC material. Swell strains as a function of the logarithm of time (in hours) are shown in Figures 27 through 30 for specimens identified as 15-LAB1, 15-LAB3, 30-LAB1, and 30-LAB3, respectively. In each of these tests, large swell strains occurred. Volumetric swell of sample 15-LAB1 was near 26 percent and primary swell continued for several months after the sample was immersed in water. Results of a second swell test performed on sample 15-LAB3 are shown in Figure 28. Total swell of this sample in a period of four months was 26.3 percent. This test had to be discontinued shortly after four months when it was observed that the swelling pressure had sheared one of the mold clamps (these clamps fasten the bottom of the mold to the mold base). Once the mold clamp had sheared, the vertical swell moved the bottom of the mold upward and invalid swell strains were obtained. However, as shown in Figure 28, the swell measurements indicated that primary swell was completed at a time of about 1,487 hours (about 62 days) for sample 15-LAB3 and secondary swell started before the clamp was sheared. A sufficient number of measurements of swell was obtained after completion of primary swell to establish the trend of secondary swell. As shown in Figure 28, the relationship of secondary swell and the logarithmic of time is linear.

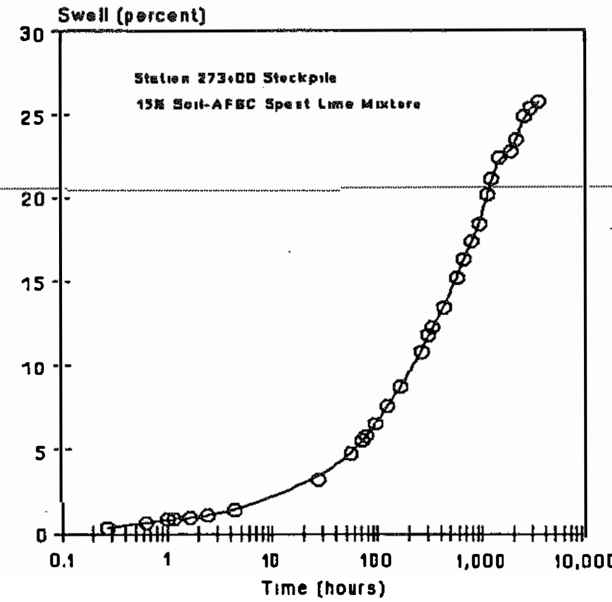

Figure 27. *Swell-logarithm of time curve of a soil specimen from stockpile Station 273+00 treated with 15 percent AFBC spent lime.*

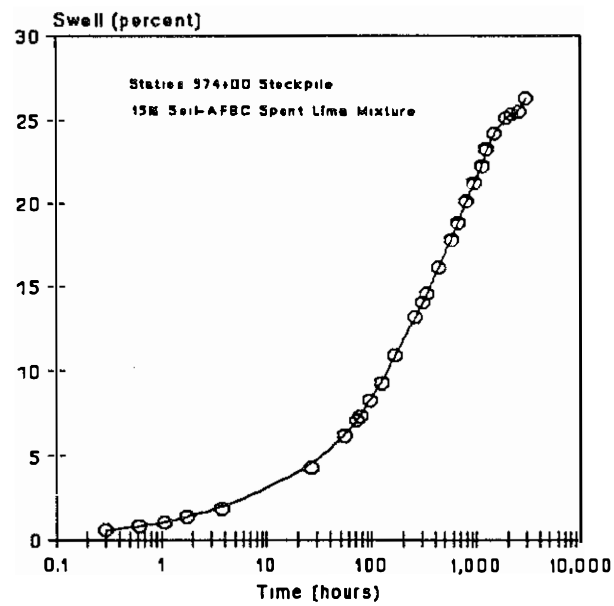

Figure 28. *Swell-logarithm of time curve of a soil specimen from stockpile Station 574+00 treated with 15 percent AFBC spent lime.*

Swell measurements of specimens 30-LAB1 and 30LAB3, which were remolded and mixed with 30 percent AFBC spent lime, are shown in Figures 29 and 30, respectively. In both cases, the total swell was 24 to 27 percent. The total swell in both cases was probably greater than the measured values because in each case the swell pressure of each sample was sufficient to shear one of the mold clamps of each mold. Once this occurred, the measurements of swell were invalid since the bottom of the mold and the top of the mold moved upward as the sample swelled vertically. The CBR value of sample 30-LAB3 was about two. The initial moisture content and dry density at compaction was 11.5 percent and 91.0 pcf, respectively. Upon testing, the moisture content in the top inch of the sample was determined to be 73 percent. The water content increased dramatically from the initial state to the final state of compaction. Also, the dry density decreased from 91.0 pcf to 79.3 pcf. As the moisture content increased, there was a decrease in dry density. As the dry density decreased (the volume increased), there was a large decrease in the shear strength and bearing ratio.

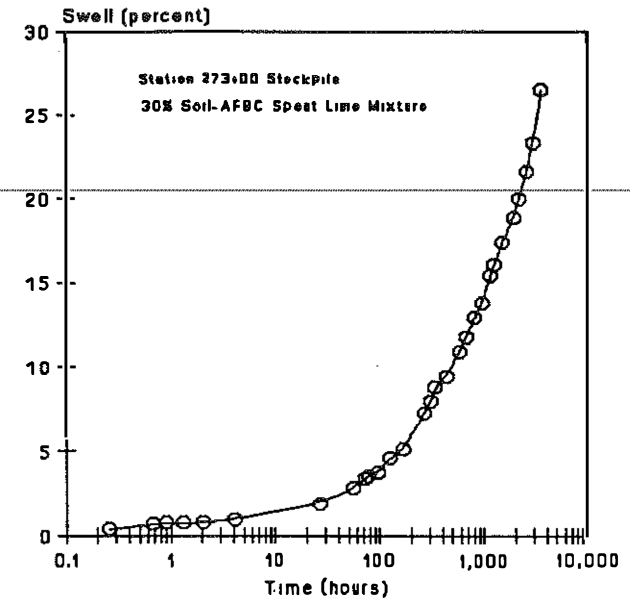

Figure 29. *Swell-logarithm of time curve of a soil specimen from stockpile Station 273+00 treated with 30 percent AFBC spent lime.*

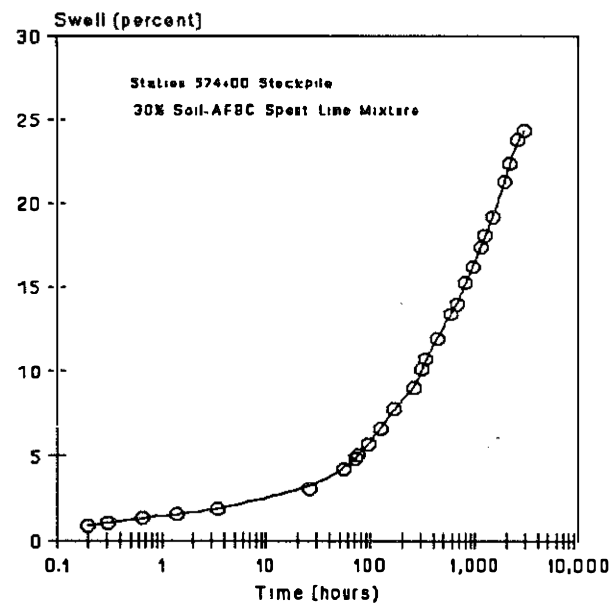

Figure 30. *Swell-logarithm of time curve of a soil specimen from stockpile Station 574+00 treated with 30 percent AFBC spent lime.*

As a means of estimating the time required for completion of primary swell of the soil-AFBC spent lime mixture in the field, the swell versus logarithm of time curves of specimens 7-1FT and 15-LAB3 were analyzed to determine a coefficient of swell (opposite of a coefficient of consolidation). Based on the curve presented in Figure 13 and the equation:

$$c_{ps} = \frac{TH^2}{t_{100}} \tag{1}$$

where C_{ps} = coefficient of primary swell (square inch per hour),
T = dimensionless time parameter,
H = thickness of laboratory specimen (inches), and
t_{100} = time to primary swell (hours).

The coefficient of primary swell for the soil-AFBC spent lime mixture is:

$$c_{ps} = \frac{(0.9)(4.504)^2\ inch^2}{1050\ hrs}$$

$$c_{ps} = 0.018\ \frac{inch^2}{hr}$$

Using the curve in Figure 28, the coefficient of primary swell of the soil-AFBC mixture is:

$$c_{ps} = \frac{(0.9)(4.0)^2\ inch^2}{1050\ hrs}$$

$$c_{ps} = 0.0097\ \frac{inch^2}{hr}$$

Hence, the coefficient of primary swell of the soil-AFBC spent lime mixture is approximately 0.018 to 0.0097 square inch per hour. Using the value of the later coefficient of swell, the time for completion of primary swell in the field may be approximated as follows (rearranging equation 1):

$$t_{100} = \frac{TH^2}{C_{ps}} \tag{2}$$

and

$$t_{100} = \frac{(0.9)(12)^2\ inch^2}{0.0097\ inch^2/hrs}$$

$$t_{100} = 13{,}361\ hrs = 556.7\ days = 1.53\ years$$

where H = thickness of the soil-AFBC spent lime layer in the field = 12 inches.

The first section of the soil-AFBC spent lime subgrade, Station 260+00 to 317+50 was constructed on August 8, 1987. From the above calculation, it was estimated that the completion of primary swell would occur around February 15, 1989, or 557 days after construction. This estimate is based on laboratory analyses and should be looked upon with a certain degree of caution and skepticism because field behavior of the material may be entirely different. For example, laboratory specimens were subjected to a continuous source of water while the source of water in the field varies or fluctuates due to wet and dry periods and some time is required during the early life of the soil-AFBC spent lime subgrade to reach a steady-state moisture environment. The completion of primary swell may occur over a longer time period than indicated by these theoretical calculations.

Another problem associated with the soil-AFBC spent lime mixture is illustrated in Figure 28. Swell of the soil-AFBC spent lime mixture exhibits secondary swell which occurs after completion of the primary swell. As shown in this plot, the relationship between the secondary swell and the logarithm of time (hours) is linear. Based on this curve, a coefficient of secondary swell, $\mathbf{c_{ss}}$, may be approximated as:

$\mathbf{c_{ss}}$ = slope of the swell-logarithm of time relationship.

Using this coefficient, the magnitude of swell that would occur between the time of completion of primary swell and some selected time after completion of primary swell may be approximated from the relationship:

$$H_{ss} = c_{ss}\, H \log \left(\frac{t_p}{t_{100}}\right) \tag{3}$$

where H_{ss} = secondary swell over a given time period (inches)
H = thickness of soil-AFBC spent lime layer (inches)
t_p = selected time after completion of primary swell (days), and
t_{100} = time of completion of primary swell (days).

Letting t_p equal five years and t_{100} equal 557 days (the estimated time to complete primary swell), then:

$$H_{ss} = (0.062)(12 \text{ in.})(\log(1{,}825 / 557)) = 0.4 \text{ inch.}$$

From five years after completion of primary swell to 27.4 years after construction, the secondary swell is equal to:

$$H_{ss} = (0.062)(12 \text{ in.})(\log(9{,}444 / 1{,}832)) = 0.9 \text{ inch.}$$

Therefore, from the time of completion of primary swell (557 days) to a time of about 25 years after construction, the total predicted secondary swell would amount to about 0.9 inch. These calculations indicate that secondary swell of the soil-AFBC spent lime mixture will be a problem in the future, but this problem may be controllable. However, estimates of secondary swell should be viewed cautiously since field and laboratory behavior of the mixture may be completely different. Surface elevations of the pavement were monitored to validate the model of predicted secondary swell of the soil-AFBC spent lime subgrade.

In-Place CBR Tests and Laboratory Analyses of Field Specimens

After the pavement surface of the AFBC sections began showing signs of non-uniform swelling, the asphaltic concrete pavement was cored to perform in-place CBR's on the treated subgrade layer and obtain moisture content samples. Two areas were targeted within the soil-AFBC sections for testing and were identified as "humped area" and "non-humped" area. The humped area (STA 279+79) had an in-place CBR of 38 and a corresponding moisture content of 23.9 percent. The non-humped area had an in-place CBR of 9 and an in-situ moisture content of 24.0 percent. As a follow-up, a trench was cut nearly one week later in an area where the pavement had formed a hump (near STA 279+82). Two in-place CBR's were performed in the trench. One test area was in the right wheel path of the southbound lane and the other test area was located near the shoulder's edge of the southbound lane. The in-place CBR for the right wheel path was determined to be 30 and the in-situ moisture content at this location was 26 percent. The in-place CBR near the edge of the lane was 40 and the in-situ moisture content equalled 24 percent. The moisture contents of the modified soil were fairly consistent and were determined to be 50 percent higher than the optimum moisture content of about 16 percent. The increase in volume and moisture content did not appear to reduce the bearing capacity of the AFBC spent lime modified soil.

Two in-place CBR's were performed during April, 1988 within the first AFBC section. The pavement had been milled within both AFBC spent lime modified sections due to the expansion of the modified soil. The milling activity facilitated definitive location of areas where there were humps in the subgrade. In-place CBR and moisture content of the treated soils were obtained. Measurements also were made of the bituminous core to determine the remaining thickness of the milled bituminous base material. The humped area, located near Station 305+55, had an in-place CBR of 13 and an in-situ moisture content of 36.1 percent. The non-humped area had an in-place CBR of 37 and an in-situ moisture content of 27.0 percent. These results were the reverse of those determined previously. However, the bearing capacity of the weaker area was still three times greater than that of the untreated soil, even at 36 percent moisture.

Additional CBR and moisture content data were obtained during the fall of 1988. During this investigation, tests were performed in each of the experimental sections with the exception of the untreated section and the AFBC spent lime section extending from Station 532+00 to Station 576+50. The moisture contents of the AFBC spent lime modified subgrade remained quite high at 34.5 and 29.8 percent. The corresponding in-place CBR's were 32 and 19, respectively. Bearing capacities for all sections were elevated. Soil moisture contents varied considerably throughout each section monitored. A summary of the results of these testing activities is presented in Table 14. The soil-MKD section had bearing capacities ranging from a CBR of 37 to a CBR of 97. The corresponding moisture contents were found to be 10.7 percent and 5.7 percent, respectively.

TABLE 14. RESULTS OF IN-SITU CBR TESTING; SEPTEMBER 1988

Station Number	In-Situ CBR	In-situ Moisture Content (%)	Type and Percent of Admixture
312+50 rt	32	34.5	AFBC (7%)
316+50 rt	19	29.8	AFBC (7%)
321+50 rt	47	16.6	Cement (10%)
325+50 rt	75	28.5	Cement (10%)
371+50 rt	30	19.1	Lime (7%)
381+75 rt	41	20.7	Lime (7%)
425+00 rt	97	10.7	MKD (10%)
428+00 rt	37	5.7	MKD (10%)
433+75 rt	39	5.1	Cement (7%)

Kentucky Transportation Center personnel continued to monitor bearing capacity and in-situ moisture characteristics of the experimental and control sections. Additional CBR and moisture content data were obtained during the spring of 1989 and during the spring of 1991. Additionally, Shelby tube samples were obtained for laboratory evaluations. Field tests were performed in each of the experimental sections and the control section during the investigations. The testing was performed during March of each year because the subgrade soil normally exhibits weakest conditions during the spring season due to moisture accumulated during the winter months.

Laboratory evaluations of the Shelby tube samples included performing unconfined compressive strength tests, determining moisture content, dry density and determining soil classifications of the extruded specimens. The Shelby tubes were difficult to push through the treated soil layer. Specimens that were obtained were difficult to extrude in the laboratory and many were disturbed, and even destroyed, during the extrusion process. Values obtained for the unconfined compressive strength during the laboratory evaluations were not considered to be representative of the true character of either the treated or untreated soils. It was concluded that the unconfined compressive strength specimens had suffered damage during the extrusion process. Results of the unconfined compressive strength tests that were performed on the extruded soil specimens also were inconclusive because of the limited number of test specimens. Therefore, results of the unconfined compressive strength tests will not be discussed herein but are included in the tables. Generally the treated soil layers were classified as SM and the untreated layers as CL by the Unified Classification System. Results of field testing activities and associated laboratory tests are contained in Tables 15 and 16 for the investigations conducted during 1989 and in Tables 17 and 18 for the investigations conducted during 1991.

It was determined during the March 1989 surveys that the in-situ moisture contents of ~~the soil-AFBC subgrade sections remained quite high. The in-situ moisture content of the~~ soil-AFBC sections ranged from 14.6 percent to 31.2 percent. The in-situ moisture content of the AFBC spent lime modified subgrade soil averaged 23.5 percent, nearly 50 percent higher than the designed optimum moisture content of about 16 percent. The in-place CBR values obtained in 1989 appeared to decrease from previous determinations, ranging from two to 27. The average value of 13 in-place CBR tests of the soil-AFBC subgrade was only 13. Unexpectedly, the lowest CBR value in the soil-AFBC sections also corresponded to the lowest moisture content. The average CBR for the AFBC spent lime modified subgrade increased to 27 during 1991. The CBR values ranged from 14 to 43. The in-situ soil moisture content also increased during 1991 to an average of 28.2 percent. The in-situ moisture content of the AFBC spent lime modified soil ranged from 23.2 to 31.4 percent.

Moisture contents of extruded tube specimens of the AFBC spent lime treated soil ranged from 25.6 to 37.8 percent and averaged 31.2 percent in 1989. During 1991, moisture contents of the treated soil specimens extruded from Shelby tubes ranged from 25.5 to 33.2 percent and averaged 28.9 percent. The moisture contents of extruded Shelby tube samples of the untreated soil subgrade layer ranged from 10.6 to 21.0 percent and averaged 16.4 percent in 1989. Moisture contents of the untreated soil extruded from Shelby tubes ranged from 14.5 to 19.1 percent and averaged 17.6 percent in 1991. Dry densities of AFBC spent lime treated soil specimens ranged from 82.4 pcf to 99.6 pcf and averaged 89.9 pcf in 1989. During 1991, dry densities of the treated soils ranged from 90.2 pcf to 97.4 pcf and averaged 94.9 pcf. The dry density of the untreated soil ranged from 109.3 pcf to 125.8 pcf and averaged 119.1 pcf in 1989. During 1991, the dry density of the untreated soil ranged from 116.6 pcf to 124.6 pcf and averaged 119.7 pcf.

In addition to investigating the bearing capacity and in-situ moisture content of the chemically modified subgrade layer during the 1991 site investigation, KTC personnel also performed in-place bearing capacity tests on the surface of the untreated soil subgrade and obtained moisture content samples to facilitate comparisons of the bearing capacity of treated and untreated subgrade layers. Bearing capacity tests conducted on the natural soil beneath the AFBC spent lime modified layer indicated an average CBR of only four and an average in-situ soil moisture content of 22.5 percent. Consequently, the AFBC spent lime treated layer exhibited a bearing capacity of 27 and the untreated soil subgrade layer below the treated layer had an in-place CBR of four. The moisture content of the treated layer averaged nearly six percent higher than the untreated layer.

TABLE 15. RESULTS OF IN-SITU CBR TESTING; MARCH 1989

			Shelby Tube Samples			
Station Number	In-Situ CBR	In-situ Moisture Content (%)	Unconfined Compressive Strength (psi)	Moisture Content (%)	Dry Density (pcf)	Type and Percent of Admixture
AFBC Spent Lime Modified Sections						
SITE 5	27	18.8				AFBC (7%)
301+77 rt	2	14.6				AFBC (7%)
301+77 rt			17.9	10.6	125.8	Untreated Layer
301+77 rt				17.9	117.5	Untreated Layer
305+53 rt	15	23.4	38.9	25.6	99.6	AFBC (7%)
305+53 rt			39.0	30.8	91.7	AFBC (7%)
305+53 rt			16.7	15.0	124.0	Untreated Layer
312+50 rt	8	25.7				AFBC (7%)
312+50 rt			38.3	17.7	118.9	Untreated Layer
SITE 35	14	25.6				AFBC (7%)
SITE 35			25.1	21.0	109.3	Untreated Layer
SITE 36	18	21.8				AFBC (7%)
552+00 rt				37.8	82.4	AFBC (7%)
552+00 rt				30.6	85.5	AFBC (7%)
552+05 rt	10	26.6				AFBC (7%)
554+50 rt	10	31.2				AFBC (7%)
Cement Modified Sections						
321+50 rt	21	23.6				Cement (10%)
325+50 rt	*	18.1				Cement (10%)
333+90 lt				22.8	97.3	Cement (10%)
333+90 lt			17.9	20.4	108.0	Untreated Layer
333+92 lt			47.1	19.8	119.6	Cement (10%)
334+00 rt	21	16.1				Cement (10%)
433+65 rt	7	18.2				Cement (7%)
433+75 rt			22.8	20.6	105.9	Cement (7%)
433+75 rt			43.7	18.1	130.6	Untreated Layer
433+75 rt			36.3	18.6	130.2	Untreated Layer
475+50 rt	11	18.7				Cement (7%)
475+50 rt			8.8	19.8	100.5	Cement (7%)
475+50 rt			18.0	17.6	118.2	Untreated Layer
515+00 rt	23	17.9				Cement (7%)
515+00 rt			66.6	15.8	135.3	Untreated Layer
515+00 rt			29.5	13.1	139.5	Untreated Layer
Hydrated Lime Modified Section						
371+47 rt	18	19.1				Lime (7%)
376+00 rt	26	17.8				Lime (7%)
376+00 rt			47.4	13.4	128.0	Untreated Layer
381+00 rt	16	19.0				Lime (7%)
381+00 rt			27.1	30.3	97.0	Lime (7%)
381+00 rt			28.4	19.8	115.6	Untreated Layer
Multicone Kiln Dust Modified Section						
409+50 rt	13	20.7				Kiln Dust (10%)
412+50 rt	11	17.6				Kiln Dust (10%)
412+50 rt			13.6	19.1	110.3	Untreated Layer
425+00 rt	*	16.3				Kiln Dust (10%)
Untreated Section						
523+00 rt	6	14.9				None
523+00 rt			59.3	15.7	135.9	None
524+50 rt	3	15.1				None
526+50 rt	3	18.8				None
526+50 rt			38.2	16.1	133.6	None
526+50 rt			24.1	18.5	127.8	None

* Indicates insufficient data for CBR computation.

TABLE 16. SOIL CLASSIFICATIONS OF SHELBY TUBE SPECIMENS; MARCH 1989

Station Number	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity	Grain-Size Analysis Percent Finer Than: 3/4 in. (%)	3/8 in. (%)	No.4 (%)	No.10 (%)	No.40 (%)	No.200 (%)	Classification AASHTO	Unified System	Type and Percent of Admixture
AFBC Spent Lime Modified Section												
SITE 5	37	4	2.76	100.0	99.5	97.7	92.9	68.7	44.0	A-4(0)	SM	AFBC (7%)
312+50 rt	35	6	2.73	99.3	99.0	98.1	92.5	66.4	45.6	A-4(0)	SM	AFBC (7%)
554+50 rt	NP	NP	2.84	100.0	95.5	93.9	90.0	68.7	43.5	A-4(0)	SM	AFBC (7%)
Cement Modified Sections												
321+50 rt	49	14	2.75	100.0	100.0	100.0	96.6	81.3	52.7	A-7-5(6)	CL	Cement (10%)
333+90 rt	36	7	2.68	100.0	100.0	96.3	90.9	73.3	51.5	A-4(2)	ML	Cement (10%)
334+12 rt	44	7	2.73	100.0	99.4	97.3	92.6	66.1	37.1	A-5(0)	SM	Cement (10%)
480+00 rt	NP	NP	2.75	100.0	98.0	97.1	95.9	82.6	48.6	A-4(0)	SM	Cement (7%)
Hydrated Lime Modified Section												
371+47 rt	--	--	2.73	100.00	96.8	94.6	91.4	79.7	50.0	--	--	Lime (7%)
376+00 rt	--	--	2.71	97.6	94.8	93.1	90.3	70.7	38.3	--	--	Lime (7%)
Multicone Kiln Dust Modified Section												
409+50 rt	44	10	2.71	100.0	98.8	96.1	91.7	71.9	47.3	A-5(2)	SM	Kiln Dust (10%)
Untreated Section												
523+00 rt	31	7	2.68	100.0	100.0	99.7	99.3	98.5	83.8	A-4(5)	ML-CL	None

Trends similar to those observed in the AFBC spent lime modified soil sections were evident in the cement modified soil sections with respect to bearing capacity values. The average CBR of the ten percent soil-cement section was 21 in 1989 but increased to a value greater than 100 during the 1991 investigation. The in-situ soil moisture remained relatively constant during the two testing periods however, averaging 19.3 percent and 18.9 percent during 1989 and 1991, respectively. The moisture content of extruded shelby tube specimens of the cement treated soil ranged from 19.8 to 22.8 percent and averaged 21.3 percent in 1989. Shelby tube samples of the ten percent cement treated subgrade layer could not be obtained during the 1991 investigation. The dry density of the chemically treated soil ranged from 97.3 pcf to 119.6 pcf and averaged 108.5 pcf in 1989. Bearing ratio tests performed in 1991 on the natural soil below the ten percent cement treated layer indicated an average CBR of five. The CBR values of the natural soil ranged from one to nine. The in-situ moisture content of the natural soil averaged 19.6 percent, ranging from 16.2 to 22.2 percent. The dry density of the untreated soil ranged from 113.9 to 127.9 pcf and averaged 120.0 pcf.

TABLE 17. RESULTS OF IN-SITU CBR TESTING; MARCH 1991

Station Number	In-Situ CBR	In-situ Moisture Content (%)	Shelby Tube Samples: Unconfined Compressive Strength (psi)	Shelby Tube Samples: Moisture Content (%)	Shelby Tube Samples: Dry Density (pcf)	Type and Percent of Admixture
AFBC Spent Lime Modified Sections						
302+10 rt	43	30.8	29.5	28.0	97.4	AFBC (7%)
302+10 rt	*	21.3	28.9	14.5	124.6	Untreated Layer
305+20 rt	18	28.9	10.9	33.2	90.2	AFBC (7%)
305+20 rt	3	21.8				Untreated Layer
312+50 rt	33	23.2				AFBC (7%)
312+50 rt	5	23.6	41.5	18.5	118.8	Untreated Layer
532+75 rt	19	31.4				AFBC (7%)
532+75 rt	4	23.1	33.9	17.5	119.3	Untreated Layer
546+10 rt	14	29.0			118.9	AFBC (7%)
546+10 rt	5	20.0	15.2	19.1	116.6	Untreated Layer
555+33 rt	35	25.8	17.7	25.5	97.2	AFBC (7%)
555+33 rt	3	25.0	36.3	18.5	119.3	Untreated Layer
Cement Modified Sections						
319+20 rt	248	15.6				Cement (10%)
319+20 rt	9	16.2				Untreated Layer
321+85 rt	133	21.4				Cement (10%)
321+85 rt	4	20.4	22.2	17.7	118.2	Untreated Layer
328+25 rt	137	19.7				Cement (10%)
328+25 rt	1	22.2	10.7	20.0	113.9	Untreated Layer A
328+25 rt			24.8	14.6	127.9	Untreated Layer B
433+75 rt	*	16.8				Cement (7%)
433+75 rt	8	18.4				Untreated Layer
463+00 rt	59	20.7	16.0	20.3	93.2	Cement (7%)
463+00 rt	12	19.3	13.2	18.8	101.0	Untreated Layer
495+80 rt	*	17.2				Cement (7%)
495+80 rt	8	17.5				Untreated Layer
Hydrated Lime Modified Section						
357+00 rt	127	24.3				Lime (7%)
357+00 rt	5	23.1	35.2	15.2	122.8	Untreated Layer A
357+00 rt			33.2	19.4	115.8	Untreated Layer B
SITE 8 rt	37	19.6	47.6	20.3	94.6	Lime (7%)
SITE 8 rt	4	21.4	12.3	17.7	107.0	Untreated Layer A
SITE 8 rt			11.4	19.8	104.6	Untreated Layer B
378+50 rt	*	17.6				Lime (7%)
378+50 rt	14	17.2	9.8	13.6	121.7	Untreated Layer A
378+50 rt			16.9	13.7	118.3	Untreated Layer B
Multicone Kiln Dust Modified Section						
410+80 rt	138	14.7				Kiln Dust (10%)
410+80 rt	5	20.0	19.3	15.2	140.4	Untreated Layer
419+30 rt	72	17.8				Kiln Dust (10%)
419+30 rt	6	21.8	16.0	19.4	106.6	Untreated Layer
428+50 rt	78	16.4				Kiln Dust (10%)
428+50 rt	8	15.8				Untreated Layer
Untreated Section						
522+50 rt	7	12.1	22.9	15.4	121.8	None
526+50 rt	7	13.4	48.0	17.8	119.4	None
531+50 rt	11	18.3				None

* Indicates insufficient data for CBR computation.

TABLE 18. SOIL CLASSIFICATIONS OF SHELBY TUBE SPECIMENS; MARCH 1991

Station Number	Liquid Limit (%)	Plasticity Index (%)	Specific Gravity	Grain-Size Analysis Percent Finer Than: 3/4 in. (%)	3/8 in. (%)	No.4 (%)	No.10 (%)	No.40 (%)	No.200	Classification	Unified	Type and Percent of
AFBC Spent Lime Modified Section												
302+10 rt	48	8	2.75	100.0	98.9	95.3	80.9	59.2	38.6	A-5(0)	SM	AFBC (7%)
305+20 rt	38	14	2.74	100.0	96.4	91.7	79.8	71.1	63.6	A-6(7)	CL	Untreated Layer
532+75 rt	49	11	2.74	100.0	98.0	95.4	81.2	59.8	39.7	A-7-5(1)	SM	AFBC (7%)
Cement Modified Sections												
319+20 rt	NP	NP	2.82	100.0	95.6	87.4	68.4	43.0	21.3	A-1-B(0)	SM	Cement (10%)
319+20 rt	38	16	2.77	100.0	91.7	82.6	68.7	59.0	52.8	A-6(5)	CL	Untreated Layer
433+75 rt	37	9	2.75	99.7	94.0	87.2	71.4	53.6	41.0	A-4(1)	SM	Cement (7%)
463+00 rt	42	19	2.77	100.0	99.2	95.4	83.4	77.4	72.8	A-7-6(13)	CL	Untreated Layer
Hydrated Lime Modified Section												
357+00 rt	NP	NP	2.77	99.4	94.7	84.9	62.9	42.4	28.7	A-2-4(0)	SM	Lime (7%)
357+00 rt	39	18	2.70	100.0	98.8	95.0	74.3	69.0	61.5	A-6(9)	CL	Untreated Layer
Multicone Kiln Dust Modified Section												
410+80 rt	NP	NP	2.73	100.0	96.5	88.9	72.4	50.1	30.7	A-2-4(0)	SM	Kiln Dust (10%)
419+30 rt	39	15	2.72	99.6	97.9	94.0	77.8	68.9	61.9	A-6(7)	CL	Untreated Layer
Untreated Section												
522+50 rt	39	17	2.76	99.6	98.6	94.5	74.2	68.5	60.4	A-6-(8)	CL	None

Similar trends were also observed in the seven percent soil-cement section. The average CBR value of the treated soil increased from 14 in 1989 to greater than 100 in 1991. The in-situ moisture content of the cement treated soil was practically the same for both years, averaging 18.3 percent in 1989 and 18.2 percent in 1991. Strength tests performed on the extruded Shelby tube samples of the treated soil were limited because of the difficulty in obtaining representative specimens. Results of the laboratory strength tests that were performed were inconclusive due to the limited number of test specimens. Moisture content of the cement treated specimens extruded from shelby tubes averaged 20.2 percent. Dry densities of the soil cement Shelby tube specimens averaged 103.2 pcf. Shelby tube specimens of the natural soil below the seven percent cement treated soil subgrade had an average moisture content of 16.6 percent and an average dry density of 130.8 pcf. Bearing ratio tests performed in 1991 on the natural soil below the cement treated layer resulted in an average CBR of nine. The CBR values of the natural soil ranged from eight to 12. The in-situ moisture content of the natural soil averaged 18.4 percent and ranged from 17.5 to 19.3 percent. The moisture content and dry density ~~values of one shelby tube specimen of the natural soil were 18.8 percent and 101.0 pcf,~~ respectively.

The hydrated lime section also exhibited increased bearing capacities from 1989 to 1991. The average CBR of the hydrated lime treated soil was 20 in 1989 and increased to 82 in 1991. The average in-situ moisture content of the treated soil increased from 18.6 percent in 1989 to 20.5 in 1991. The moisture content and dry density of the only shelby tube specimen of the hydrated lime treated soil evaluated in 1989 were 30.3 percent and 97.0 pcf, respectively. The moisture content and dry density of the only shelby tube specimen of the hydrated lime treated soil evaluated in 1991 were 20.3 percent and 94.6 pcf, respectively. The moisture content of tube samples of the untreated soil layer averaged 16.6 percent in 1989 and ranged from 13.4 to 19.8 percent. The dry density of the untreated soil averaged 121.8 pcf in 1989, ranging from 115.6 to 128.0 pcf. Bearing ratio tests performed in 1991 on the surface of the untreated soil layer below the hydrated lime treated subgrade resulted in an average in-place CBR of eight. In-place CBR values of the natural soil layer ranged from four to 14. The in-situ moisture content of the natural soil averaged 20.6 percent and ranged from 17.2 to 23.1 percent. Soil moisture content, determined from Shelby tube samples, averaged 16.6 percent in 1991, ranging from 13.6 to 19.8 percent. Dry density of the natural soil averaged 115.0 pcf in 1991, ranging from 104.6 to 122.8 pcf.

The experimental multicone kiln dust treated soil subgrade section exhibited significant increases in the magnitude of the bearing capacity of the subgrade in 1991 compared to 1989. The in-place CBR of the multicone kiln dust treated soil averaged 12 in 1989 and ranged from 11 to 13. The average in-place CBR value increased to 96 in 1991, ranging from 72 to more than 100. The in-situ moisture content of the treated soil averaged 18.2 percent in 1989, ranging from 16.3 to 20.7 percent. The in-situ moisture content decreased to 16.3 in 1991 and ranged from 14.7 to 17.8 percent. There were no Shelby tube samples of the multicone kiln dust treated soil evaluated in 1989 or 1991. The moisture content of the only tube sample of the untreated soil layer evaluated in 1989 was 19.1 percent. The dry density of that specimen was 110.3 pcf. Bearing ratio tests performed in 1991 on the surface of the untreated soil layer below the multicone kiln dust treated soil resulted in an average in-place CBR of six. In-place CBR values of the natural soil layer ranged from five to eight. The in-situ moisture content of the natural soil averaged 19.2 percent and ranged from 15.8 to 21.8 percent. Soil moisture contents, determined from Shelby tube samples of the natural soil, averaged 17.3 percent in 1991, ranging from 15.2 to 19.4 percent. Dry density of the natural soil averaged 123.5 pcf, ranging from 106.6 to 140.4 pcf.

The untreated soil subgrade section exhibited a slight increase in the bearing capacity of the subgrade in 1991 compared to 1989. The in-place CBR of the untreated soil averaged four in 1989 and ranged from three to six. The average in-place CBR value increased to eight in 1991, ranging from seven to 11. The corresponding in-situ moisture content of the soil subgrade averaged 16.3 percent in 1989, ranging from 14.9 to 18.8 percent. The in-situ moisture content decreased to 14.6 in 1991 and ranged from 12.1 to 18.3 percent. Information relative to moisture content and dry density derived from shelby tube samples of the soil subgrade taken in 1989 indicated an average 16.8 percent moisture and an average dry density of 132.4 pcf. The moisture content of the tube samples ranged from 15.7 to 18.5 percent. The dry density of the tube samples obtained in 1989 ranged from 127.8 to 135.9 pcf. Soil moisture contents, determined from shelby tube samples taken in 1991, averaged 16.6 percent. The soil moisture content values ranged from 15.4 to 17.8 percent. Dry density of the natural soil averaged 120.6 pcf, ranging from 119.4 to 121.8 pcf.

Pavement Swell Measurements

Placement of the bituminous surface course in all sections was delayed after the observance of differential pavement heaving in the two AFBC spent lime modified subgrade sections. Elevations on the uppermost base layer were monitored periodically at arbitrary locations selected within each chemically modified soil subgrade section to observed changes in the pavement surface profile. Initial measurements were obtained in early October, 1987, after the pavement surface on the two soil-AFBC sections exhibited noticeable signs of non-uniform swelling. Initially, survey points were established only within those sections having a chemically modified soil subgrade in order to monitor the vertical movement of the pavement surface. Subsequently, survey points had to be re-established in August of 1988 after the pavement within both AFBC spent lime modified soil subgrade sections had been milled and the surface course had been placed over the entire route. Survey points were established in the control, or untreated soil subgrade section as well after the placement of the bituminous surface course. Measurements were made in both transverse and longitudinal directions, and generally at two-foot intervals. Equipment used for this activity included a Topcon AT-F6 leveling instrument and a leveling rod having a level bubble. Readings were estimated to 0.001 foot.

Because of the considerable non-uniform swelling that was occurring in the soil-AFBC spent lime sections, the majority of the survey points were located there. A total of ten stations were monitored in the two distressed sections initially (three monitoring locations, Stations 564+00, 569+00 and 574+00 were eliminated from the study because the benchmarks were disturbed during the exercise). Survey points were established at stations within the other chemically modified soil subgrade sections so that vertical movements of all sections could be compared. Succeeding measurements were obtained in late October, November, and December of 1987, and in March 1988 prior to milling activities. Survey points were re-established in August 1988 after the pavement within the two AFBC spent lime modified soil sections had been milled and the entire length of the construction project received the final surface. Subsequent measurements were obtained in September 1988, January and July 1989, March 1990 and March 1991.

Table 19 lists results of the optical surveys conducted prior to placement of the bituminous surface. Elevation readings were obtained from October 8, 1987 through March 1, 1988 at locations in the two soil-AFBC spent lime sections, the hydrated lime section, the ten percent cement section, and the MKD section. Elevation changes for each monitoring location are shown graphically in Appendix D. The elevation differences shown in Table 19 are for the minimum, maximum, and average elevation change of the pavement surface. The upward movement of the pavement in both the cement and hydrated lime sections were insignificant during this observation period. The maximum elevation change observed in the ten percent cement section was 0.07 inch. In the soil-hydrated lime section, the maximum change in elevation was 0.19 inch. The two sections utilizing the by-product waste materials AFBC and MKD had elevation changes greater than those experienced in the cement and hydrated lime sections. The maximum elevation change in the MKD section was 0.49 inch (see Figure 31). The maximum elevation change in the AFBC sections exceeded three inches (see Figure 32). The AFBC modified soil subgrade section from Station 532+00 to 576+50 experienced greater expansions than the previously constructed AFBC modified soil section.

Table 20 contains results of the optical surveys conducted after placement of the bituminous riding surface. Initial survey points were established on August 9, 1988 and measurements were taken up through March 1991. The same stations were used as those used previously. There were seven stations located within the soil-AFBC spent lime subgrade sections, and one each in the hydrated lime section, ten percent cement section, and the MKD section. Additionally, a monitoring location was established at Station 530+00 in the untreated soil subgrade section. Elevation changes for each of these elevation monitoring locations are shown graphically in Appendix E. Similar to Table 19, the elevation differences are listed in terms of the minimum, maximum, and average elevation change observed in the pavement surface. Results of the continuous monitoring activities indicated that the initial subgrade swelling virtually ceased after placement of the bituminous surface course. The upward movements of the pavement in the cement, hydrated lime and MKD sections were typically less than one-quarter inch. Swelling of the soil-AFBC subgrade also diminished greatly, being about one-quarter inch, or less, at any one location during the three-year monitoring period. Based upon the model developed in the laboratory to estimate secondary swell of the soil-AFBC spent lime mixture, the subgrade swell during the three-year monitoring period would be about one-quarter inch. The field measurements of pavement heave provide a certain amount of validity to the predictive model of secondary pavement swell.

There was, however, an isolated area that exhibited non-uniform swelling of such magnitude that milling was required to eliminate surface humps. Figure 33 shows the milled area in the through lane near Station 563+00. There also was a prominent crack in the pavement due to the upward pavement heave within the milled area.

Visual Surveys and Pavement Rutting Characteristics

The experimental and control sections were visually surveyed periodically for observable pavement distress since the completion of construction. Factors such as rutting and cracking were of principal concern. Overall, all sections are in good condition and no significant pavement distresses have been

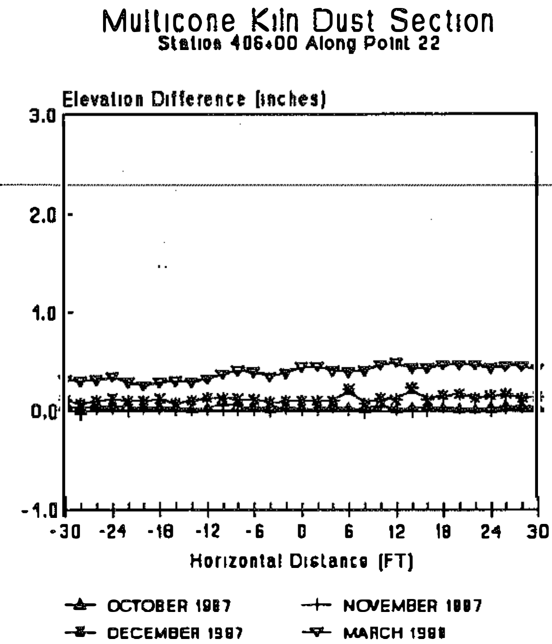

Figure 31. *Typical pavement swell characteristics of the multicone kiln dust modified subgrade section prior to final surfacing.*

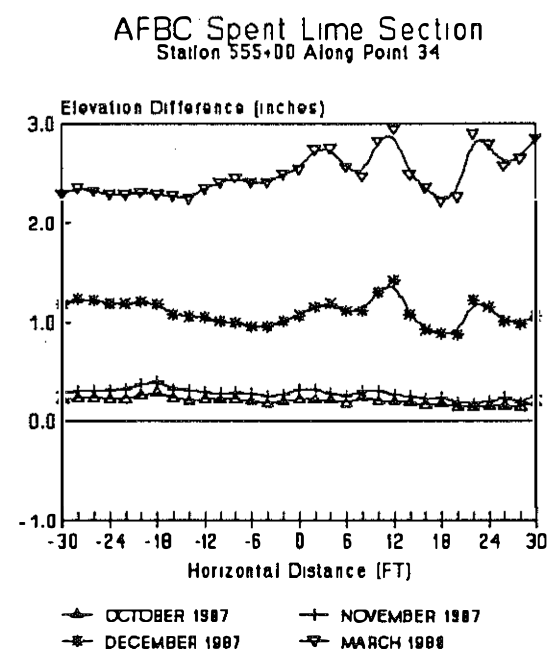

Figure 32. *Typical pavement swell characteristics of the AFBC spent lime modified subgrade section prior to final surfacing.*

TABLE 19. PAVEMENT SWELL RESULTS PRIOR TO FINAL SURFACING

Station\ Location	Chemical Admixture	Minimum Swell (in.)	Maximum Swell (in.)	Average Swell (in.)	Monitoring Station Located on:
270+00 T	AFBC	-0.120	0.708	0.216	Fill Section
270+00 L6	AFBC	-0.144	0.696	0.147	Fill Section
270+00 L22	AFBC	0.012	0.792	0.313	Fill Section
280+00 T	AFBC	0.384	2.316	1.372	Cut/Fill Section
280+00 L6	AFBC	0.612	2.088	1.306	Cut/Fill Section
280+00 L20	AFBC	0.612	1.824	1.297	Cut/Fill Section
285+00 T	AFBC	0.024	0.732	0.248	Cut Section
285+00 L8	AFBC	0.060	0.948	0.338	Cut Section
285+00 L22	AFBC	0.432	0.816	0.607	Cut Section
300+00 T	AFBC	0.600	1.668	1.008	Fill Section
300+00 L24	AFBC	0.756	1.932	1.154	Fill Section
334+00 T	10% Cement	-0.060	0.036	-0.005	Fill Section
334+00 L22	10% Cement	-0.048	0.072	0.014	Fill Section
379+00 T	Lime	-0.024	0.144	0.044	Cut Section
379+00 L22	Lime	0.002	0.192	0.016	Cut Section
406+00 T	MKD	0.192	0.456	0.272	Cut/Fill Section
406+00 L22	MKD	0.252	0.492	0.388	Cut/Fill Section
549+00 T	AFBC	0.312	2.004	0.886	Cut/Fill Section
549+00 L24	AFBC	0.672	1.248	0.946	Cut/Fill Section
555+00 T	AFBC	1.656	2.988	2.167	Fill Section
555+00 L24	AFBC	1.056	2.640	2.240	Fill Section
555+00 L34	AFBC	2.220	2.940	2.481	Fill Section
559+00 T	AFBC	0.204	3.456	1.487	Fill Section
559+00 L10	AFBC	0.336	2.028	1.289	Fill Section
559+00 L30	AFBC	1.896	3.372	2.564	Fill Section

observed to date. There were some isolated instances of non-uniform swelling in the two soil-AFBC sections. One area in the southbound through lane, in the vicinity of Station 563+00, was of such extent as to require milling of the pavement to eliminate surface humps. There also was some cracking of the bituminous pavement within the soil-AFBC sections. The cracking occurred predominately within shoulder areas and did not affect the travel lanes. The remaining modified soil sections did not exhibit observable pavement distresses during the monitoring period.

Measurements of rutting depth to the nearest one-sixteenth inch were obtained at 100-foot intervals, 300 feet on either side of each optical survey station, in both left and right wheel paths in the northbound and southbound directions during 1990 and 1991. On

TABLE 20. PAVEMENT SWELL RESULTS AFTER FINAL SURFACING

Station\ Location	Chemical Admixture	Minimum Swell (in.)	Maximum Swell (in.)	Average Swell (in.)	Monitoring Station Located on:
270+00 T	AFBC	0.096	0.300	0.180	Fill Section
270+00 L6	AFBC	-0.060	0.204	0.094	Fill Section
270+00 L22	AFBC	-0.036	0.192	0.098	Fill Section
285+00 T	AFBC	-0.192	0.012	-0.067	Cut Section
285+00 L8	AFBC	-0.156	-0.036	-0.093	Cut Section
285+00 L22	AFBC	-0.084	0.000	-0.039	Cut Section
300+00 T	AFBC	0.012	0.420	0.159	Fill Section
300+00 L6	AFBC	0.096	0.420	0.257	Fill Section
300+00 L24	AFBC	-0.072	0.144	0.071	Fill Section
334+00 T	10% Cement	-0.216	-0.012	-0.148	Fill Section
334+00 L20	10% Cement	-0.192	-0.120	-0.151	Fill Section
334+00 L36	10% Cement	-0.228	-0.096	-0.170	Fill Section
379+00 T	Lime	0.036	0.108	0.070	Cut Section
379+00 L12	Lime	0.048	0.168	0.089	Cut Section
406+00 T	MKD	0.084	0.252	0.150	Cut/Fill Section
406+00 L6	MKD	0.084	0.216	0.167	Cut/Fill Section
406+00 L22	MKD	0.060	0.204	0.130	Cut/Fill Section
530+00 T	None	-0.168	0.048	-0.092	Cut Section
530+00 L12	None	-0.132	-0.036	-0.076	Cut Section
549+00 T	AFBC	-0.408	0.204	0.045	Cut/Fill Section
549+00 L6	AFBC	0.144	0.348	0.221	Cut/Fill Section
549+00 L24	AFBC	0.084	0.252	0.166	Cut/Fill Section
549+00 L42	AFBC	-0.564	0.144	-0.218	Cut/Fill Section
555+00 T	AFBC	-0.804	-0.012	-0.191	Fill Section
555+00 L24	AFBC	-1.140	0.000	-0.319	Fill Section
555+00 L34	AFBC	-1.548	-0.024	-0.328	Fill Section
559+00 T	AFBC	-0.876	-0.168	-0.502	Fill Section
559+00 L10	AFBC	-1.236	0.204	-0.516	Fill Section
559+00 L30	AFBC	-1.008	0.156	-0.321	Fill Section

NOTE: Monitoring location, Station 280+00, was eliminated from the study after the benchmark had been disturbed.

average, the deepest rutting occurred in the control section. Some individual rutting measurements were greater in the soil-AFBC section from Station 532+00 to Station 576+50. Pavement rutting at Station 572+00 was 9/16-inch however, this was attributed to the upward slope of the pavement at that point and slower moving heavy trucks. The

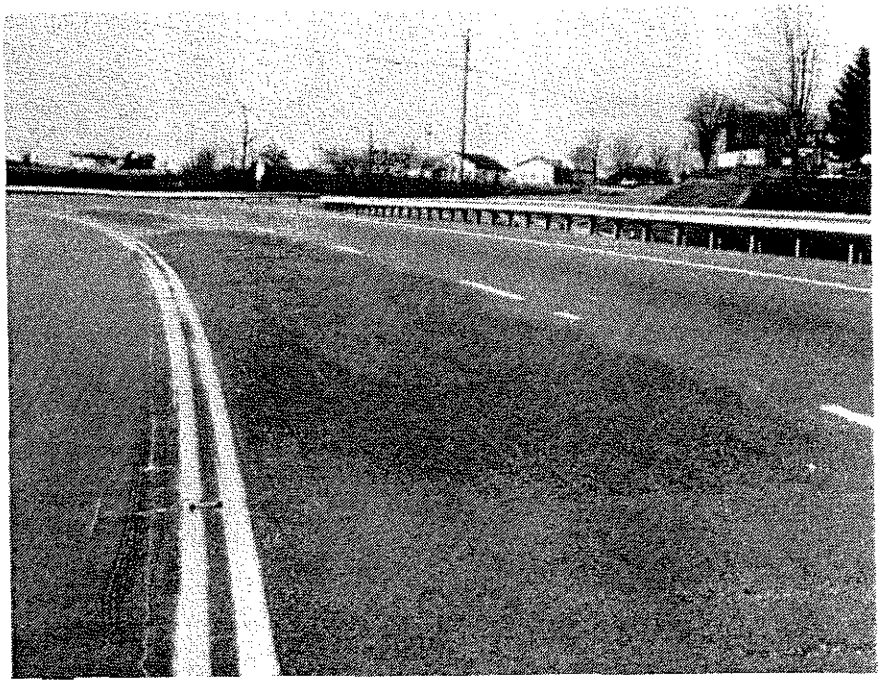

Figure 33. *Area near Station 563+00 that required milling after placement of final surface.*

rutting measurements indicated that the dimension of the rut depth decreased as the grade of the incline decreased. Results of the rutting surveys conducted in 1990 and 1991 are presented in Appendix F in tabular format. The 1990 data are presented first and then the 1991 data are presented for comparison purposes. A significant change in the overall rutting depth occurred in the control section and the soil-AFBC section from Station 532+00 to Station 576+50.

Road Rater Deflection Tests and Analyses

Deflection testing was performed to quantify the long-term structural characteristics of the various subgrade sections. The four-year structural performance of each section was evaluated using Road Rater deflection testing. All tests were conducted with the Model 400B Road Rater using a 1,200-pound dynamic load. Deflection tests were conducted on each section during 1988, 1989 and 1991. The deflection data were analyzed using a three-layer analysis (asphaltic concrete over dense graded aggregate over subgrade) which compares measured field deflections with theoretically calculated deflections. Because a three-layer solution was employed to analyze the deflection data, elastic moduli values of the various chemically modified layers were not determined. Rather, the analyses provide a means to evaluate the structural condition of the composite subgrade. Each chemically modified subgrade section exhibited higher strengths than the untreated control section. The cement modified subgrade sections had the largest increase in strength above the strength realized in the untreated control section. The cement modified subgrade sections were followed in order by the strength of the hydrated lime, MKD, and AFBC modified subgrade sections. The analyses also indicated an increase in subgrade strength with time for all sections including the untreated section. Based on these analyses, it appears that all sections are performing equally well. However, it should be noted that different thicknesses of asphaltic concrete were utilized on the various chemically modified subgrade sections.

SUMMARY AND CONCLUSIONS

Construction and short-term performance of highway field trials of admixture modification of several sections of subgrade have been described in this report. Four admixtures were used including type 1P cement, hydrated lime, and two waste by-products: atmospheric fluidized bed combustion residue and multicone kiln dust. A 1,000-foot section of the subgrade was constructed using conventional procedures. All admixture types, except type 1P cement, were used on an experimental basis.

An extensive laboratory testing program was used to determine the suitability of using the waste by-product materials as soil modifiers. The laboratory testing program consisted of determining select engineering properties of the soil in an untreated, or natural state, and in a state altered by the chemical admixtures. Index tests were performed, moisture-density relationships were determined, and bearing ratio tests and swell tests were performed. Laboratory procedures used to determine the optimum percentage of each admixture were described.

A laboratory procedure was developed to determine the optimum percentage of chemical admixture to add to a given soil type. When the optimum percentage of admixture is added to a given type of soil, the maximum unconfined compressive strength is obtained. An increase in the percentage of admixture above the optimum amount does not significantly increase the unconfined compressive strength properties of the modified soil. The optimum amount of type 1P cement necessary to achieve the maximum unconfined compressive strength was determined to be around ten to 12 percent. A value of ten percent was used for one subgrade modification section and a value of seven percent was used for another section. The laboratory strength of the soil-cement specimens was six to seven times greater and ten to 11 times greater than untreated soil specimens remolded from stockpile Stations 273+00 and 574+00, respectively. The unconfined compressive strength of soil-cement specimens ranged from about 265 psi to 470 psi and the unconfined compressive strength of the natural soil specimens was about 40 psi. The optimum amount of hydrated lime for soil modification, based on the maximum unconfined compressive strength, was around six or seven percent. A value of seven percent was used for construction. The laboratory strength of the soil-hydrated lime specimens was about 100 psi. The unconfined compressive strength of the natural soil specimens was about 40 psi. The optimum amount of AFBC residue for soil modification was around six percent. A value of seven percent was used constructing two experimental sections. The laboratory strength of the soil-AFBC spent lime specimens was about 160 psi. The unconfined compressive strength of the natural soil specimens was about 40 psi. The optimum amount of multicone kiln dust for soil modification was about eight to ten percent. A value of ten percent was used to construct the experimental section. The laboratory strength of the soil-MKD specimens was about 170 psi. There were no unconfined strength tests performed on the soil stockpiled at Station 334+00 with which to compare the laboratory strength of MKD modified soil specimens.

Index properties and soil classifications were generally improved when the natural soil was mixed with the chemical admixtures. The most significant changes in the index properties of the soils occurred when type 1P cement was added to the soil. Some improvement in the index properties was observed when hydrated lime was mixed with the soils. The AFBC spent lime produced mixed results with respect to soil index properties. A slight reduction was observed in the percentage of clay particles when the AFBC was mixed with the soils. However, the plasticity index showed little or no change. The index properties of soils modified with the waste by-product MKD were not investigated prior to construction.

Based on laboratory unconfined compression tests and CBR tests, all four admixtures significantly improved the shear strength and bearing strength of the soils at the study site. It was determined that as the percent hydrated lime and AFBC spent lime increased, the maximum dry density and optimum moisture content obtained from standard compaction procedures decreased and increased, respectively. During construction, the volume change that occurred when the natural soils were mixed with these admixtures required that the finished subgrades be trimmed significantly to obtain design grade elevation. Conversely, as the percent cement and MKD added to the soils increased, no significant changes were observed in the maximum dry density or optimum moisture content.

Construction requirements for AFBC residue roadbed modification, MKD roadbed modification and lime roadbed modification are contained in the appendices of this report. As with the initial use of any material, there were some difficulties encountered at the beginning of construction of the AFBC spent lime modified subgrade. Principally, there appeared to be little control in the amount of AFBC spent lime or water being placed. Because of the fine-grained nature of the AFBC spent lime, the material flowed much like a liquid and extra effort was devoted to constructing windrows along the edge of the shoulders to contain the AFBC residue. The mixing machines performed exceptionally well. The pulverization requirement was easily met after one pass with the pulverizing machine and the mellowing period required for the AFBC residue roadbed modification and the MKD roadbed modification was waived. Initially, inspectors had difficulties getting correct moisture readings using the nuclear density device. After the problem was identified, the inspectors determined the actual moisture content by applying a moisture-content correction factor. The incorporation of the AFBC spent lime into the soil caused significant volume change. Nearly four inches of the subgrade had to be trimmed in order to obtain proper grade elevation but trimming was easily accomplished 24 to 30 hours after final compaction. The subgrade subcontractor did not experience further troubles while constructing the remaining modified subgrade sections.

Investigations relative to the engineering properties of the modified soil subgrades continued during construction of the modified subgrade. Field testing consisted of moisture content / dry density tests for construction compliance, and in-place bearing capacity tests, moisture content tests and Road Rater deflection tests performed on the subgrade both before and after modification with chemical admixtures. Based on 84 nuclear density tests conducted on all sections of KY 11, the relative compaction averaged 98.2 percent with a standard deviation of +/- 2.6 percent. Since specifications required that all field dry densities be 95 percent of maximum dry density, all subgrade sections were compacted according to the dry density specification. With regard to the moisture content, compaction specifications required that the field moisture content be no less than the optimum moisture content nor more than five percent above optimum moisture. The average value of the differences between measured moisture contents in the field and specified optimum moisture contents was -0.2 percent. This means that generally the subgrades were compacted at moisture contents just slightly dry of the optimum moisture content.

In-place bearing ratio tests were performed throughout the test site on the natural soil prior to admixture modification. Attempts were made to repeat these tests after a period of seven days. Unfortunately, the attempt met with little success and tests were only performed after seven days within the AFBC sections. Results of the tests on the untreated soil indicated very high bearing capacities and correspondingly low moisture contents. In-place bearing ratio tests performed on the soil-AFBC spent lime modified section confirmed that the bearing capacity of the chemically modified subgrade had increased by about 36 percent above the value of the untreated subgrade. However, the moisture content determined in conjunction with the tests indicated a decrease in the moisture content of about 13 percent. The decrease in moisture content was attributed to incorrect moisture readings initially obtained with the nuclear device. Analysis of Road Rater deflection tests conducted before subgrade modification and seven days after modification provided initial indication of the benefits of chemical admixture modification. The mean moduli estimated from the Road Rater tests were 24,000 psi for the untreated subgrade. Modification with type 1P cement increased the estimated subgrade modulus to 137,000 psi. Modification of the soils with hydrated lime increased the estimated subgrade modulus to only 46,000 psi. The two waste by-products also proved beneficial with regard to increasing the shear strength of the natural soil. Modification of the soil with AFBC residue increased the estimated subgrade modulus to about 75,000 psi and modification with MKD increased the estimated subgrade modulus to 93,000 psi.

After construction, monitoring of the experimental and control sections continued. The initial post-construction analysis involved re-examining the expansive characteristics of the soil-AFBC spent lime mixture. Approximately two months after construction of the soil-AFBC spent lime subgrades, severe differential swell or heave of the pavement surface was noted. The preconstruction laboratory evaluation of the swell characteristics indicated swell of 3.1 percent when seven percent AFBC residue was combined with the natural soil. The natural soil exhibited 3.9 percent swell. Additional specimens of the soil-AFBC mixture were evaluated. A specimen remolded from a bag sample obtained from a trench that was opened to investigate the subgrade heave had less than one percent swell during the CBR test. Specimens were remolded using soils obtained from stockpiles and excessive quantities of the AFBC residue. It was determined that the volumetric swell of the remolded specimens containing 15 to 30 percent AFBC residue (by weight of the dry soil) ranged between 24 and 27 percent. Based on this investigation, it was concluded that the amount of AFBC residue mixed with the natural soil at locations where differential heave had occurred exceeded the specified seven percent necessary for soil modification. Primary and secondary swell characteristics were noted. A model was developed to estimate the time at which primary swell of the soil-AFBC spent lime mixture would be completed and the magnitude of swell that could be expected to occur during the secondary swell phase.

Performance monitoring of the chemically modified subgrade soils included performing in-place bearing capacity tests on the subgrades, obtaining undisturbed samples to analyze in the laboratory relative to unconfined compressive strength, moisture content, and soil classification, monitoring pavement elevations for swell attributes, and performing Road Rater deflection measurements on the completed pavement structure. Additional in-place CBR tests were performed during 1988, 1989 and 1991 to determine the benefits of subgrade modification with the waste by-products. The most extensive evaluation of the modified soil subgrade layers was performed in 1991. A comparison of the bearing capacity and moisture content of the treated layer and the underlying untreated layer was made. For reasons that cannot be explained, results of in-place CBR tests performed during 1989 were much lower for all soil-admixture types than results obtained during the other two test dates, though the moisture contents were relatively constant for each testing period.

Results of in-situ bearing capacity tests on the soil-AFBC sections performed in 1988 and 1991 were comparable and indicated an average CBR of about 27 and an average moisture content of about 28 percent. The untreated soil layer below the treated layer had an average CBR and moisture content of four and 22.5 percent, respectively during 1991. Based on the results of the in-situ field tests, it appears that bearing capacity and moisture content of the soil-AFBC spent lime mixture has stabilized. The AFBC spent lime modified soil subgrade appears to be sustaining elevated shear strength values at very high moisture contents. The type 1P cement, hydrated lime and MKD modified soil subgrades appear to be continuing to gain strength with time. The multicone kiln dust modified soil subgrade had an average CBR of about 96 and an average moisture content of about 16 percent during 1991. The bearing capacity increased about 50 percent over the 1988 value. The moisture content also increased above the 1988 value but was similar to the 1989 value (18.2 percent). The untreated soil layer below the MKD-treated layer had an average CBR and moisture content of six and 19.2 percent, respectively during the 1991 test. The untreated section had an average CBR of eight and moisture content of about 15 percent in 1991.

Laboratory evaluations of the Shelby tube samples included performing unconfined compressive strength tests, determining moisture content, dry density and determining soil classifications of the extruded specimens. The Shelby tubes were difficult to push through the treated soil layer. Specimens that were obtained were difficult to extrude in the laboratory and many were disturbed, and even destroyed, during the extrusion process. Values obtained for the unconfined compressive strength during the laboratory evaluations were not considered to be representative of the true character of either the treated or untreated soils. Unconfined compressive strength results also were inconclusive because of the limited number of test specimens. Values of the dry density of the modified soils were largely obtainable for only the soil-AFBC spent lime modified subgrade section. The dry density of soil-AFBC spent lime specimens averaged 89.8 pcf and 94.9 pcf, respectively, during the 1989 and 1991 investigations. These values are substantially less than values recorded during construction and are a direct result of the volumetric swell that the soil-AFBC mixture underwent after construction. The untreated soils underlying the AFBC modified soils had a dry density of about 119 pcf during both years. Results of index tests performed on extruded shelby tube specimens indicated that the all chemically modified soils were generally classified as SM in the Unified Classification System and soil from the untreated layer was classified as CL.

Based on pavement elevations, no significant swell occurred in the subgrade sections stabilized with type 1P cement, hydrated lime or the waste material MKD. Laboratory swell tests also revealed that there were no swelling associated with type 1P cement or hydrated lime and that these two admixtures actually reduced the swelling of the natural soils. However, the soil-AFBC spent lime modified subgrade swelled significantly. Significant swell or heave, of the pavement placed on the two soil-AFBC sections occurred shortly after construction after a period of heavy rain in the region. The swelling nature of this material, when mixed with the natural soils, was not expected since a small quantity was to be mixed with the subgrade soils. It was concluded that the humps that formed on the pavement surface were caused by the combination of excessive amounts of the AFBC spent lime admixture and an insufficient amount of water being added in those areas where the subgrade heaved. The excessive amount of AFBC spent lime most likely occurred when the spreader trucks stopped and started while distributing the admixture. Similarly, the water trucks deposited more water in some areas than others because they often became bogged down or stuck. Typically, the width of the transverse humps on that occurred on the pavement surface were the same width as the spreaders.

The experimental and control sections were visually surveyed periodically for observable pavement distress since the completion of construction. Factors such as rutting and cracking were of principal concern. Overall, all of the chemically modified subgrade sections are in good condition and exhibiting excellent performance. With the exception of one area within the soil-AFBC section near Station 563+00, no significant pavement distresses have been observed to date. Pavement rutting characteristics were monitored during the study. On average, the deepest rutting occurred in the control section. The absence of significant pavement rutting in the chemically modified subgrade sections is illustrative of the benefits of chemical admixture subgrade modification.

Elastic moduli, as estimated from non-destructive Road Rater deflection tests, indicated substantial improvement after the fine-grained soils were modified with the all chemical admixture types. Based on the results of the Road Rater tests performed on the natural soil subgrade and the modified soil subgrade after seven days, the waste by-products AFBC spent lime and MKD improved the stiffness of the soil subgrade threefold and fourfold. Modification with type 1P cement provided appeared to provide the highest moduli values. Analyses of subsequent deflection tests performed with the Road Rater to quantify the long-term benefits of the admixture modification indicated that each chemically modified subgrade section continues to exhibit higher strengths than the untreated control section. The cement modified subgrade sections continued to exhibit the largest increase in strength above the strength realized in the untreated control section. However, the strength of the soil-hydrated lime subgrade section had surpassed that of the soil-MKD and soil-AFBC spent lime modified subgrade sections. The deflection analyses also indicated an increase in subgrade strength with time for all sections including the untreated section. Because a three-layer solution was employed to analyze the Road Rater deflection data collected during the evaluation period, elastic moduli values of the chemically modified subgrade layers were not specifically determined.

It may be concluded that the AFBC spent lime admixture enhanced the overall bearing capacity characteristics of the natural soil. However, the construction procedures employed by the subcontractor could not prevent excessive amounts of the AFBC residue from being mixed with the natural soil. Future use of the AFBC spent lime for soil subgrade modification is not recommended because of the extremely expansive nature of this waste by-product. Further research is needed to identify and control the mechanism that causes the swelling of the soil-AFBC spent lime mixtures.

It also may be concluded that multicone kiln dust waste material as a soil modifier provides increased the shear strength properties above those of the natural soil. The results of the in-situ field tests also indicates that the soil-MKD layer appears to be gaining strength over time. Because of the available calcium oxide in the waste material (about 23 percent), the strength gain over time was expected. The soil-MKD section has performed excellently and further use of this waste by-product is warranted. Future use of multicone kiln dust as a subgrade soil modifier is encouraged based on the results of this successful field trial.

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APPENDIX A

Kentucky Department of Highways'
Special Note for AFBC Residue
Roadbed Stabilization
(Experimental)

SPECIAL NOTE FOR
AFBC RESIDUE ROADBED STABILIZATION
~~(EXPERIMENTAL)~~

I. DESCRIPTION

This work shall consist of roadbed stabilization constructed by uniformly mixing atmospheric fluidized bed combustion (AFBC) residue with the roadbed soil, and the resulting mixture moistened and compacted to the lines, grades, thicknesses, and cross sections as specified in the contract. Section references herein are to the Department's Standard Specifications for Road and Bridge Construction.

II. MATERIALS

The atmospheric fluidized bed combustion (AFBC) residue shall be the lime by-product of the Ashland Petroleum Company's fluidized bed process.

Bituminous material for the curing seal shall be as specified in the contract for curing portland cement stabilized base.

Water shall be obtained from a source approved by the Engineer.

III. CONSTRUCTION REQUIREMENTS

A. General. Equipment and construction methods shall be as specified in Sections 304.03 through 304.13 for portland cement base stabilization, with exceptions and additions as specified herein.

The characteristics of the soils actually encountered in the subgrade may affect the quantity of AFBC residue necessary or desirable. The Department reserves the right to increase or decrease the quantity of AFBC residue used, if deemed necessary by the Engineer.

AFBC residue (dry) shall not be applied during periods of high winds which cause excessive loss of lime.

No traffic or equipment shall be permitted on the spread AFBC residue other than that required for spreading, watering, or mixing.

The AFBC residue shall be prepared, transported, distributed, and mixed with the soil in a manner that will not cause injury, damage, discomfort, or inconvenience to individuals or property.

B. Application of AFBC Residue. AFBC residue shall be spread at the required rate by equipment which will uniformly distribute the material without excessive loss. Due to the experimental nature of the use of the AFBC residue, the applicating rate may vary from 5% to 10% by volume as directed by the Engineer.

C. Mixing.

(1) Primary Mixing. Two-thirds of the specified quantity of AFBC residue shall be spread and immediately thoroughly mixed into the soil for the full depth of treatment. Water shall be added to the mixture so the moisture content is no less than optimum, nor more than optimum plus 5%. The primary mixing operation shall be completed within 4 hours after application of AFBC residue. At this time, the result shall be a homogeneous, friable mixture of soil and lime, free from clods or lumps exceeding 2 inches in size.

After primary mixing, the AFBC residue treated layer shall be shaped to the approximate section and lightly compacted to minimize evaporation loss. The surface shall be crowned so as to properly drain.

(2) Preliminary Curing (mellowing). Following primary mixing, the stabilized layer shall be allowed to cure for at least 48 hours, to permit the residue and water to break down or mellow the clay clods. The characteristics of the soil, temperature, and rainfall may influence the curing period necessary. The actual curing time shall be as determined by the Engineer, and final mixing and pulverizing shall not be performed until permitted by the Engineer. During preliminary curing, the surface of the material shall be kept moist to prevent drying and cracking.

(3) Final Mixing and Pulverizing. Immediately after completion of the preliminary curing, the remaining one-third of the AFBC residue shall be spread and the stabilized layer shall again be completely mixed and pulverized to the full depth of stabilization. Final mixing shall continue until all clods are broken down so that 100%, exclusive of rock particles, will pass a one-inch sieve and at least 60% will pass a No. 4 sieve. Additional water shall be added if necessary to raise the moisture content before compaction.

The stabilized roadbed shall be maintained as specified in Section 304.14.

(4) Exceptions. If the above pulverization requirement can be met during the primary mixing then the total quantity of AFBC can be added and the primary curing and final mixing steps can be eliminated.

Again, due to the experimental nature of the use of AFBC residue alternate construction procedures may be permitted when approved by the Engineer.

IV. METHOD OF MEASUREMENT

AFBC residue will be measured in tons for the residue actually incorporated into the completed work.

All water used will be considered incidental to the work and will not be measured for payment.

The stabilized subgrade will be measured in square yards in accordance with the requirements of Section 109.

Bituminous material for the curing seal will be weighed in accordance with Section 109.

V. BASIS OF PAYMENT

The accepted quantities of AFBC residue will be paid for at the contract unit price per ton, the accepted quantities of stabilized roadbed will be paid for at the contract unit price per square yard, and the accepted quantities of bituminous curing seal will be paid for at the contract unit price per ton, which payment shall be full compensation for all labor, equipment, materials, and incidentals necessary to complete the work as specified in the contract.

Payment will be made under:

PAY ITEM	PAY UNIT
AFBC Residue	Ton
AFBC Stabilized Roadbed	Square Yard
Bituminous Curing Seal	Ton

VI. TERMINATION

The experimental use of AFBC residne as a subgrade soil stabilizer shall be discontinued when requested either by the contractor or engineer.

If the experiment results in a value engineering proposal by the contractor the experimental section will not be included as a part of the value engineering proposal.

June 15, 1987

APPENDIX B

Kentucky Department of Highways'
Special Note for Lime
Roadbed Stabilization
(Experimental)

SPECIAL NOTE FOR LIME ROADBED STABILIZATION (EXPERIMENTAL)

I. DESCRIPTION

This work shall consist of roadbed stabilization constructed by uniformly mixing hydrated lime with the roadbed soil, and the resulting mixture moistened and compacted to the lines, grades, thicknesses, and cross sections as specified in the contract. Section references herein are to the Department's Standard Specifications for Road and Bridge Construction.

II. MATERIALS

Hydrated lime shall meet the requirements of AASHTO M 216 for Type I. Lime shall be handled and stored in moisture resistant containers until immediately before being transported to the site of the work. Storage bins shall be completely enclosed. Bagged lime shall be stored in weatherproof buildings with adequate protection from ground dampness.

Quantities have been calculated based on the use of Type I, Grade A hydrated lime. If Grade B or C lime is furnished, the quantity applied shall be increased as follows: Grade B 6%, Grade C 20%. These increased quantities shall be at no additional cost to the Department.

The Contractor shall advise the Engineer of the source of the hydrated lime sufficiently in advance for the material to be sampled and tested before stabilization work begins. The manufacturer shall advise which grade will be furnished. Once approved, the same grade material shall be furnished throughout the project unless a change in grade is approved in writing by the Engineer. The Engineer may take samples at the source or on the project during the course of the work. Any material not meeting specification requirements will be rejected.

Bituminous material for the curing seal shall be either RS-1, AE-60, SS-1, SS-1h, CRS-1, CSS-1, CSS-1h, or primer L, and shall meet the requirements of Section 806.

Water shall be obtained from a source approved by the Engineer.

III. CONSTRUCTION REQUIREMENTS

A. General. Equipment and construction methods shall be as specified in Sections 304.03 through 304.13 for portland cement base stabilization, with exceptions and additions as specified herein.

The characteristics of the soils actually encountered in the subgrade may affect the quantity of lime necessary or desirable. The Department reserves the right to increase or decrease the quantity of hydrated lime used, if deemed necessary by the Engineer.

Any lime that has been exposed to the open air for a period of 6 hours or more will not be accepted for payment.

Lime (dry) shall not be applied during periods of high winds which cause excessive loss of lime.

No traffic or equipment shall be permitted on the spread lime other than that required for spreading, watering, or mixing.

The hydrated lime shall be prepared, transported, distributed, and mixed with the soil in a manner that will not cause injury, damage, discomfort, or inconvenience to individuals or property.

B. Application of Lime. Unless otherwise specified, the lime may be spread dry or as slurry, at the Contractor's option.

(1) Dry Application. Hydrated lime shall be spread at the required rate by means of an approved spreader which will uniformly distribute the material without excessive loss, or by bag distribution.

(2) Slurry Application. Hydrated lime shall be mixed with water in approved agitating equipment and applied as a thin slurry, through approved distributing equipment. The distributor shall be equipped to provide continuous agitation of the mixture from the mixing site until applied to the roadbed. The proportion of lime shall be such that the dry solids content, by weight, will be at least 30%.

C. Mixing.

(1) Primary Mixing. Two-thirds of the specified quantity of lime shall be spread and immediately thoroughly mixed into the soil for the full depth of treatment. Water shall be added to the mixture so the moisture content is no less than optimum, nor more than optimum plus 5%. The primary mixing operation shall be completed within 4 hours after application of lime. At this time, the result shall be a homogeneous, friable mixture of soil and lime, free from clods or lumps exceeding 2 inches in size.

After primary mixing, the lime treated layer shall be shaped to the approximate section and lightly compacted to minimize evaporation loss. The surface shall be crowned so as to properly drain.

(2) Preliminary Curing (mellowing). Following primary mixing, the stabilized layer shall be allowed to cure for at least 48 hours, to permit lime and water to break down or mellow the clay clods. The characteristics of the soil, temperature, and rainfall may influence the curing period necessary. The actual curing time shall be as determined by the Engineer, and final mixing and pulverizing shall not be performed until permitted by the Engineer. During preliminary curing, the surface of the material shall be kept moist to prevent drying and cracking.

(3) Final Mixing and Pulverizing. Immediately after completion of the preliminary curing, the remaining one-third of the lime shall be spread and the stabilized layer shall again be completely mixed and pulverized to the full depth of stabilization. Final mixing shall continue until all clods are broken down so that 100%, exclusive of rock particles, will pass a one-inch sieve and at least 60% will pass a No. 4 sieve. Additional water shall be added if necessary to raise the moisture content before compaction.

The stabilized roadbed shall be maintained as specified in Section 304.14.

IV. METHOD OF MEASUREMENT

Hydrated Lime will be measured in tons for the lime actually incorporated into the completed work, if Grade A is used. If Grade B or C is furnished, the quantity applied will be adjusted using the percentages specified in Section II so the final pay quantity is the equivalent quantity of Grade A material.

All water used will be considered incidental to the work and will not be measured for payment.

The stabilized subgrade will be measured in square yards in accordance with the requirements of Section 109.

Bituminous material for the curing seal will be weighed in accordance with Section 109.

V. BASIS OF PAYMENT

The accepted quantities of hydrated lime will be paid for at the contract unit price per ton, the accepted quantities of stabilized roadbed will be paid for at the contract unit price per square yard, and the accepted quantities of bituminous curing seal will be paid for at the contract unit price per ton, which payment shall be full compensation for all labor, equipment, materials, and incidentals necessary to complete the work as specified in the contract.

Payment will be made under:

PAY ITEM	PAY UNIT
Hydrated Lime	Ton
Lime Stabilized Roadbed	Square Yard
Bituminous Curing Seal	Ton

June 18, 1986

APPENDIX C

Kentucky Department of Highways'
Special Note for Multicone Kiln Dust
Roadbed Stabilization
(Experimental)

SPECIAL NOTE FOR
MKD ROADBED STABILIZATION
(EXPERIMENTAL)

I. DESCRIPTION

This work shall consist of roadbed stabilization constructed by uniformly mixing multi-cone kiln dust (MKD) with the roadbed soil, and the resulting mixture moistened and compacted to the lines, grades, thicknesses, and cross sections as specified in the contract. Section references herein are to the Department's Standard Specifications for Road and Bridge Construction.

II. MATERIALS

The MKD shall be the by-product of Dravo Lime Company's Maysville, Kentucky plant.

Bituminous material for the curing seal shall be as either RS-1, AE-60, SS-1, SS-1h, CRS-1, CSS-1, CSS-1h, or primer L, and shall meet the requirements of Section 806.

Water shall be obtained from a source approved by the Engineer.

III. CONSTRUCTION REQUIREMENTS

A. Temperature and Weather Limitations.

During seasons of probable freezing temperatures, no MKD shall be applied unless the temperature is at least 40°F in the shade and rising, or between October 1 and March 31.

B. Equipment.

Hauling equipment shall be the same type equipment normally used for hauling portland cement. Any modification to the hauling equipment, or any additional equipment, that may be necessary to load the MKD at Dravo Lime Company's terminal without producing objectionable dust is the responsibility of the Contractor.

Any machine, combination of machines, or equipment, which will produce the completed stabilized roadbed meeting the requirements for pulverizing soil, distributing MKD, applying water, mixing materials, compacting, finishing, and providing protection and cover, as controlled by these specifications may be used upon approval by the Engineer. The machines and equipment used shall be maintained in a satisfactory operating condition at all times during use.

C. Job-Site Storage.

MKD may be stored on the project up to 3 days in approved hauling vehicles. Weatherproof storage facilities shall be provided if longer term storage is necessary. In no event shall the MKD be stored longer than 60 days.

D. Preparation of Existing Roadway.

Before proceeding with other construction operations, the roadway shall be graded and shaped to conform to the grades, lines, and cross section required for the completed roadway.

Before stabilization begins, the elevation of the subgrade shall be approved by the Engineer to allow for anticipated volume increase when the MKD is added. The subgrade shall conform to the ± 1/2 inch tolerance specified in Section 208.03 of the Standard Specifications both before and after stabilization. After stabilization, the Engineer may make such minor adjustments in plan grades as he deems necessary.

E. Application of MKD.

The characteristics of the soils actually encountered in the subgrade may affect the quantity of MKD necessary or desirable. The Department reserves the right to increase or decrease the quantity of MKD used, if deemed necessary by the Engineer.

MKD shall be spread at the required rate by equipment which will uniformly distribute the material without excessive loss. Due to the experimental nature of the use of the MKD, the application rate may vary from 5% to 10% by volume as directed by the Engineer.

MKD (dry) shall not be applied during periods of high winds which cause excessive loss of material.

No traffic or equipment shall be permitted on the spread MKD other than that required for spreading, watering, or mixing.

The MKD shall be prepared, transported, distributed, and mixed with the soil in a manner that will not cause injury, damage, discomfort, or inconvenience to individuals or property.

C. Mixing.

(1) Test Section. When mixing begins, the Contractor shall construct a test section at least 100 feet long and one traffic lane wide, to demonstrate the acceptability of his equipment and methods, and to provide a check on the resulting finish grade elevation. Changes in equipment or methods, or the initial grade elevations, shall be made as needed based on results of the test section. If changes in methods or equipment are made during the project, additional test sections may be required.

(2) *Primarg Mixing*. The optimum moisture content specified will be determined by the Engineer in accordance with KM 64-511 on mixtures of MKD and representative soil samples taken from the base material to be processed. Two-thirds of the specified quantity of MKD shall be spread and immediately thoroughly mixed into the soil for the full depth of treatment. Immediately after dry mixing, water shall be added to the mixture so the moisture content is no less than optimum, nor more than optimum plus 5%. The primary mixing operation shall be completed within 4 hours after application of MKD. At this time, the result shall be a homogeneous, friable mixture free from clods or lumps exceeding 2 inches in size.

After primary mixing, the MKD-treated layer shall be shaped to the approximate cross section and lightly compacted to minimize evaporation loss. The surface shall be crowned so as to properly drain.

(3) *Preliminary Curing (mellowing)*. Following primary mixing, the stabilized layer shall be allowed to cure for at least 48 hours, to permit the residue and water to break down or mellow the clay clods. The characteristics of the soil, temperature, and rainfall may influence the curing period necessary. The actual curing time shall be as determined by the Engineer, and final mixing and pulverizing shall not be performed until permitted by the Engineer. During preliminary curing, the surface of the material shall be kept moist by continuous sprinkling or other approved method to prevent drying and cracking.

(4) *Final Mixing and Pulverizing*. Immediately after completion of the preliminary curing, the remaining one-third of the MKD shall be spread and the stabilized layer shall again be completely mixed and pulverized to the full depth of stabilization. Final mixing shall continue until all clods are broken down so that 100%, exclusive of rock particles, will pass a one-inch sieve and at least 60% will pass a No. 4 sieve. Additional water shall be added if necessary so the moisture content of the completed and compacted roadbed is between optimum and optimum plus 5.0%.

(5) *Exceptions*. Upon approval by the Engineer, the contractor may construct a test section to demonstrate that the entire quantity of MKD can be added, acceptably mixed, and the pulverization requirement in paragraph (3) above can be met in one operation. If the demonstration is successful, the primary curing and final mixing steps can be eliminated.

D. Compaction and Surface Finish.

Prior to the beginning of compaction, and as a continuation of the mixing operations, the mixture shall be thoroughly loosened to its full depth. The mixture shall then be uniformly compacted for its full depth, to the specified density. Sheep's foot rollers will be required if the depth of treatment exceeds 8 inches. During compaction, the surface of the stabilized roadbed shall be reshaped to the approximate crown and grade.

The mixture shall be compacted to at least 95 percent of the maximum density obtained by KM 64-511. Density determinations will be made in the field by KM 64-512 or by nuclear gages.

After the mixture is compacted, the surface of the roadbed shall be reshaped, at optimum moisture, to the required lines, grades, and cross section.

The moisture content of the material must be maintained at no less than its specified optimum during all finishing operations. The surface compaction and finishing for the specified width of stabilized roadbed shall be done in a manner to produce, a smooth, closely-knit surface, free of cracks, ridges, or loose material; and the finished surface shall conform to the required crown, grade, and line.

The density of all the stabilized roadbed will be determined by the Engineer each day. Any portion of the roadway which does not meet the specified density shall be reconstructed to meet these specifications.

The average thickness of roadway construction during one day shall be within 1/2 inch of the thickness shown on the plans, except that the thickness at any one place may be within 3/4 inch of that shown on the plans. Where the average thickness shown by the measurements in that day's construction is not within the specified tolerances, the Contractor will be required to reconstruct that day's work or portion of day's work at his sole expense.

After curing is completed, Department representatives will take samples from the stabilized roadbed. The Contractor shall cooperate with the Department's representatives, and shall not place succeeding pavement courses until the samples have been taken.

E. Curing and Protection.

After the roadbed has been finished as specified herein, it shall be protected against drying for 7 calendar days by applying a bituminous curing seal.

The curing seal shall be applied as soon as possible, but no later than 24 hours after completion of finishing operations. The finished roadbed shall be kept moist, by continuous sprinkling if necessary, until the curing seal is applied. When the bituminous material is applied, the surface of the roadbed shall be dense, free from loose extraneous material, and shall contain sufficient moisture to prevent penetration of the bituminous material.

The curing seal shall consist of the bituminous material specified and shall be uniformly applied at the rate of approximately 1.6 pounds per square yard with approved distributing equipment. The actual rate and application temperature of bituminous material will be determined by the Engineer. The curing seal shall be applied in sufficient quantity to provide a continuous membrane over the roadbed.

No traffic or equipment other than curing equipment will be permitted on the finished surface until completion of 7 satisfactory curing days, unless permitted by the Engineer. A satisfactory curing day shall be any day when the temperature of the completed base does not fall below 50°F. If any damage occurs to the curing seal prior to completion of curing, the damaged area shall be immediately resealed at the Contractor's expense.

completed, which work shall be done by the Contractor at his own expense and repeated as often as may be necessary to keep the roadway continuously intact. Repairs shall completely restore the uniformity of the surface and durability of the repaired portion.

IV. METHOD OF MEASUREMENT

Multi-cone kiln dust (MKD) will be measured in tons for the quantity actually incorporated into the completed work.

All water used will be considered incidental to the work and will not be measured for payment.

The stabilized roadbed will be measured in square yards of stabilized base actually constructed and accepted.

Bituminous material for the curing seal will be weighed in accordance with Section 109.

V. BASIS OF PAYMENT

The accepted quantities of MKD will be paid for at the contract unit price per ton, the accepted quantities of stabilized roadbed will be paid for at the contract unit price per square yard, and the accepted quantities of bituminous curing seal will be paid for at the contract unit price per ton, which payment shall be full compensation for all labor, equipment, materials, and incidentals necessary to complete the work as specified in the contract.

Payment will be made under:

PAY ITEM	PAY UNIT
MKD	Ton
AFBC Stabilized Roadbed	Square Yard
Bituminous Curing Seal	Ton

August 27, 1987

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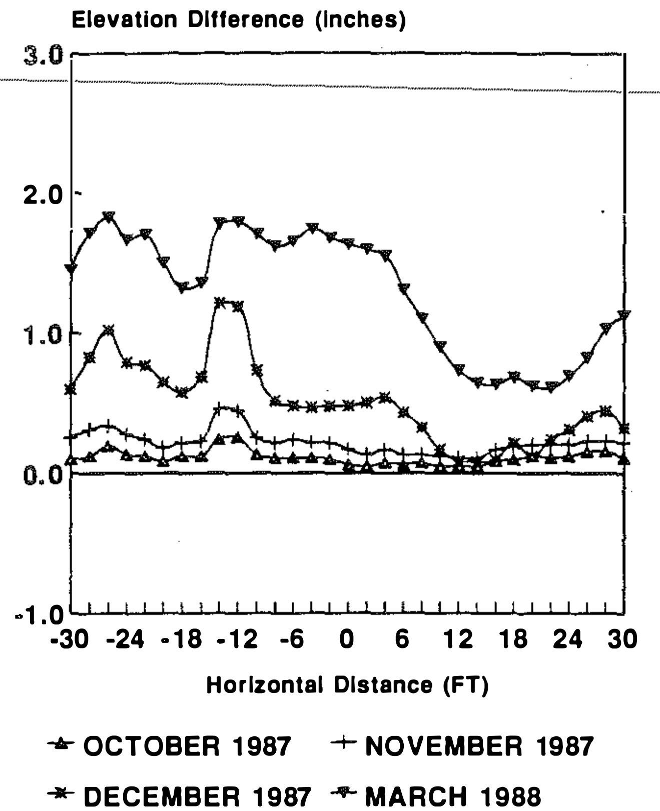

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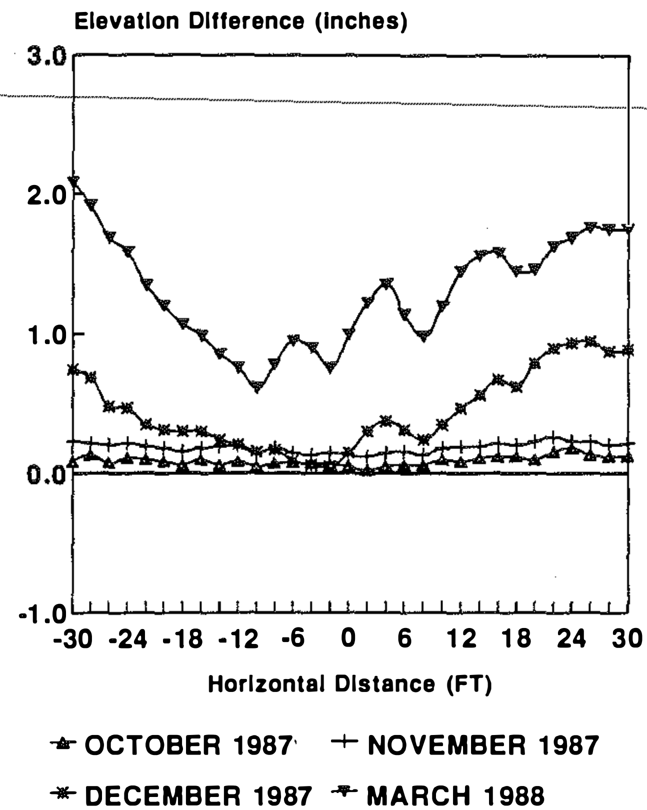

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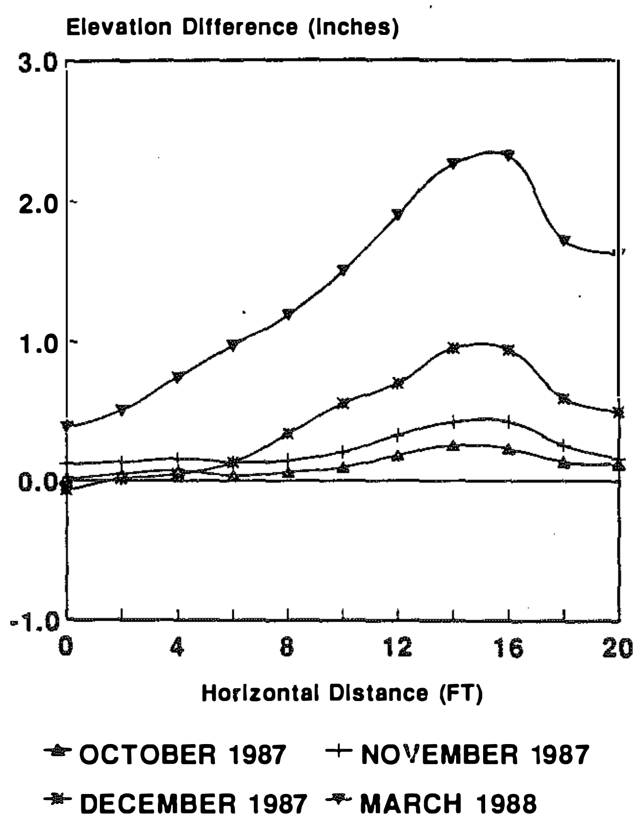

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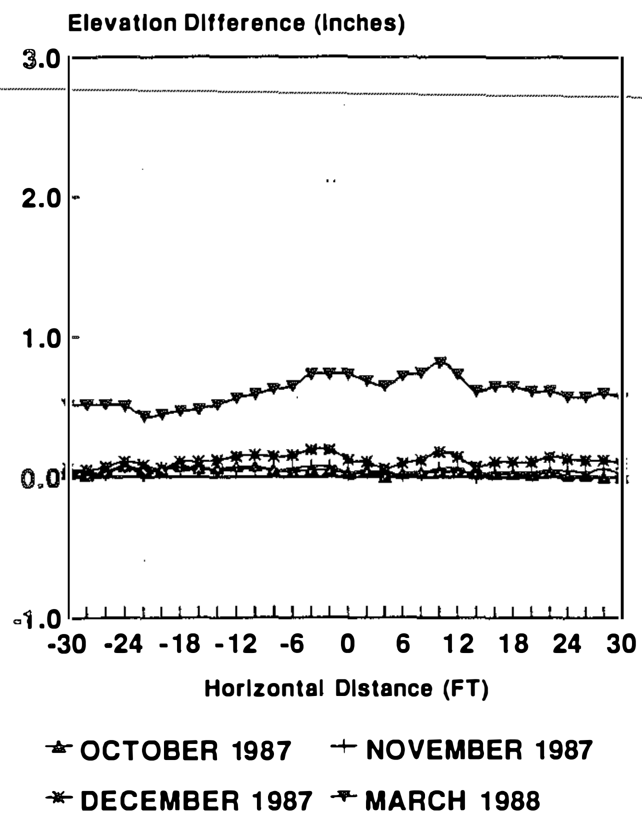

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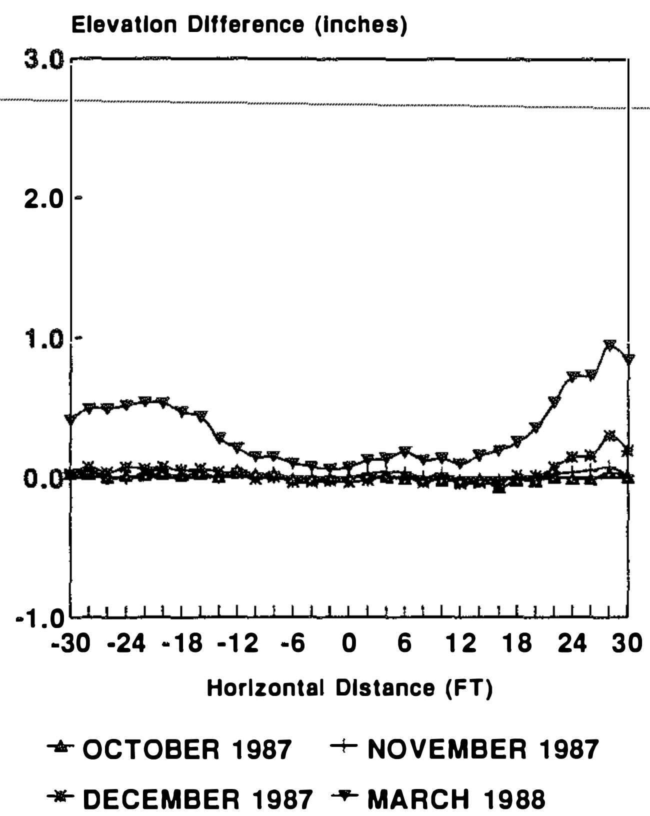

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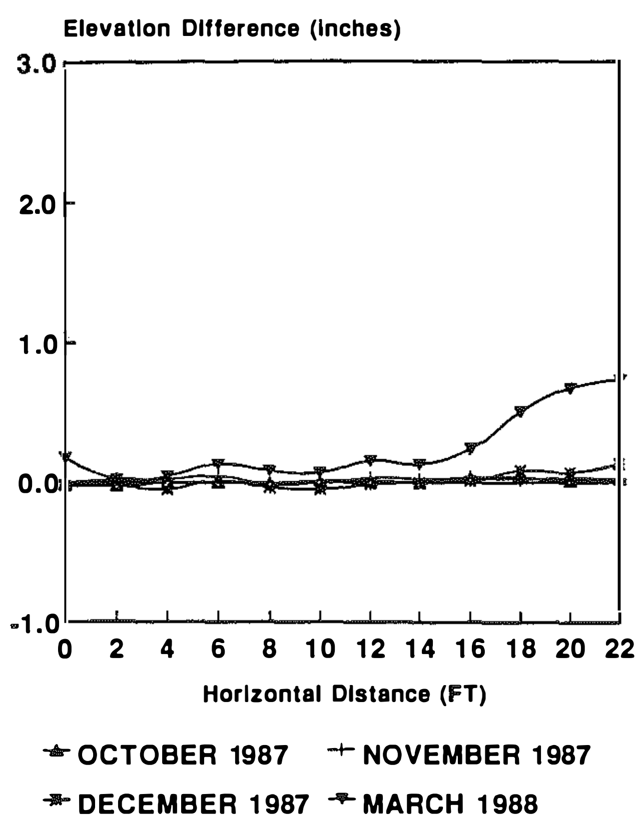

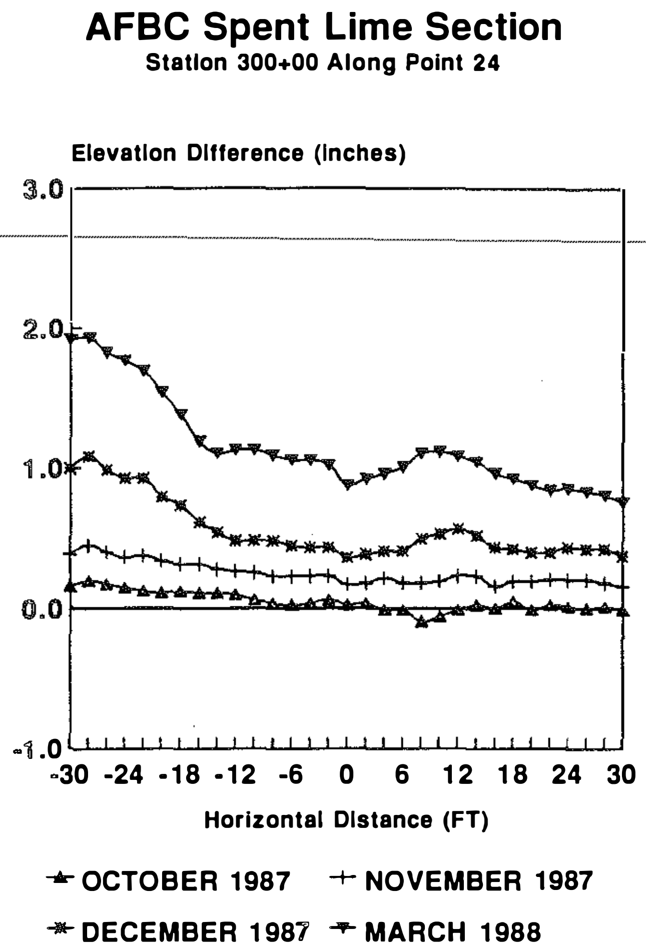
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Horizontal Distance (FT)
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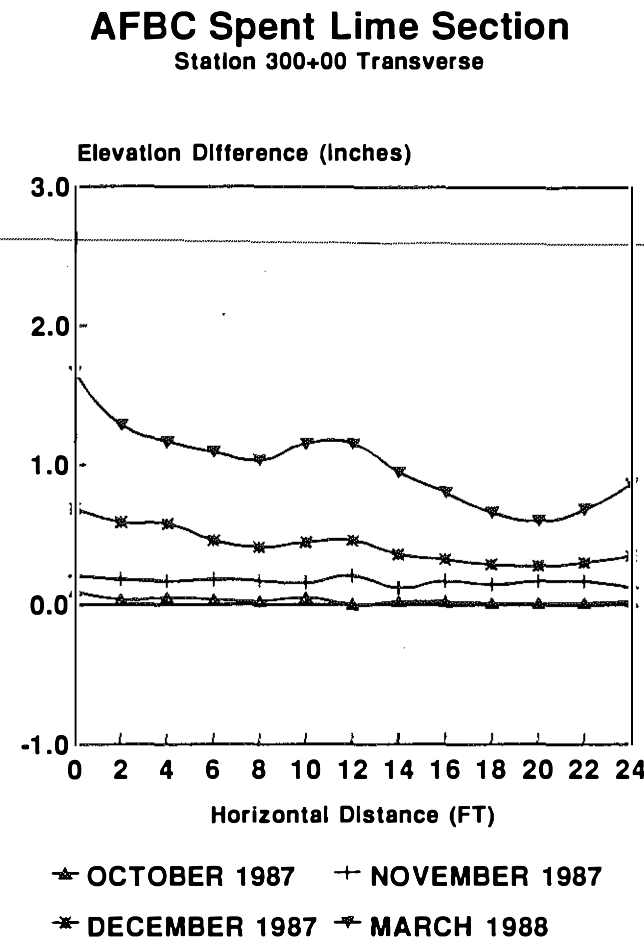
NOVEMBER 1987
DECEMBER 1987
MARCH 1988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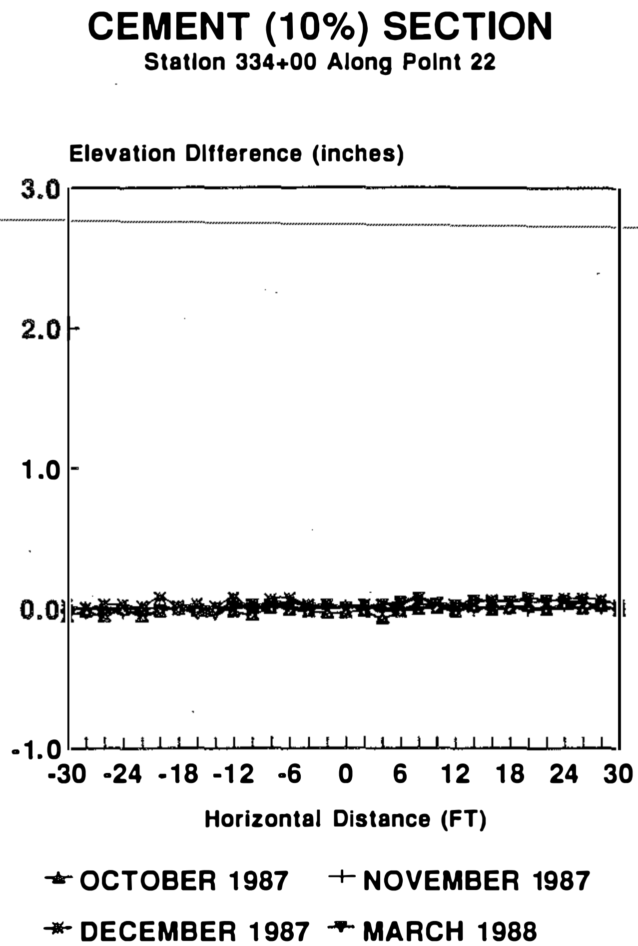
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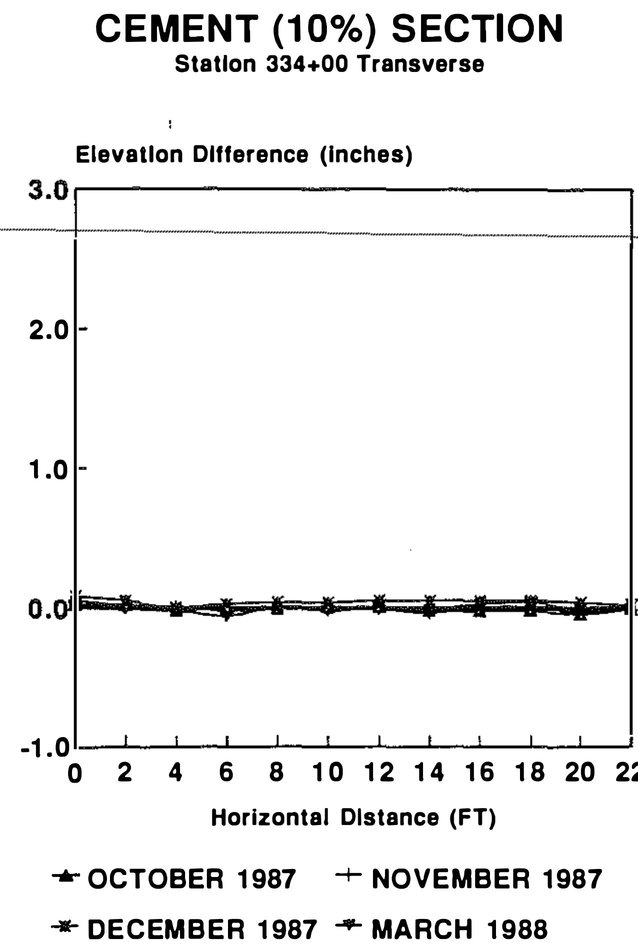
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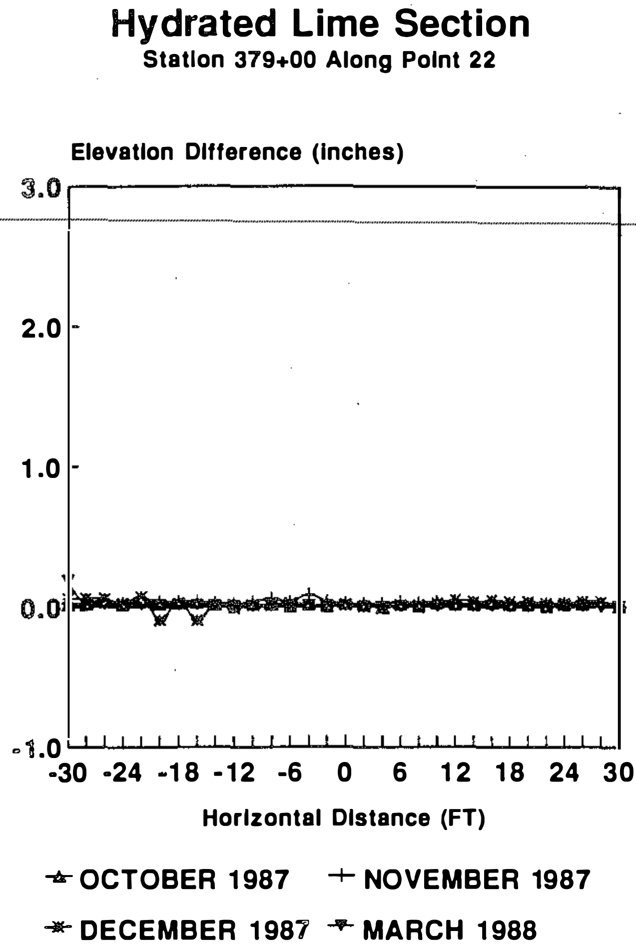
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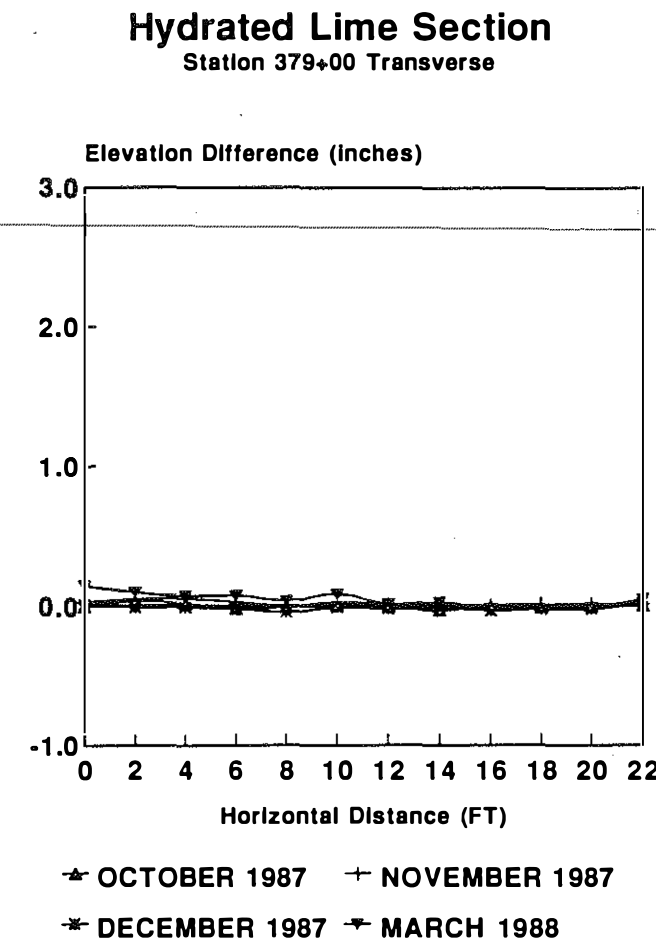
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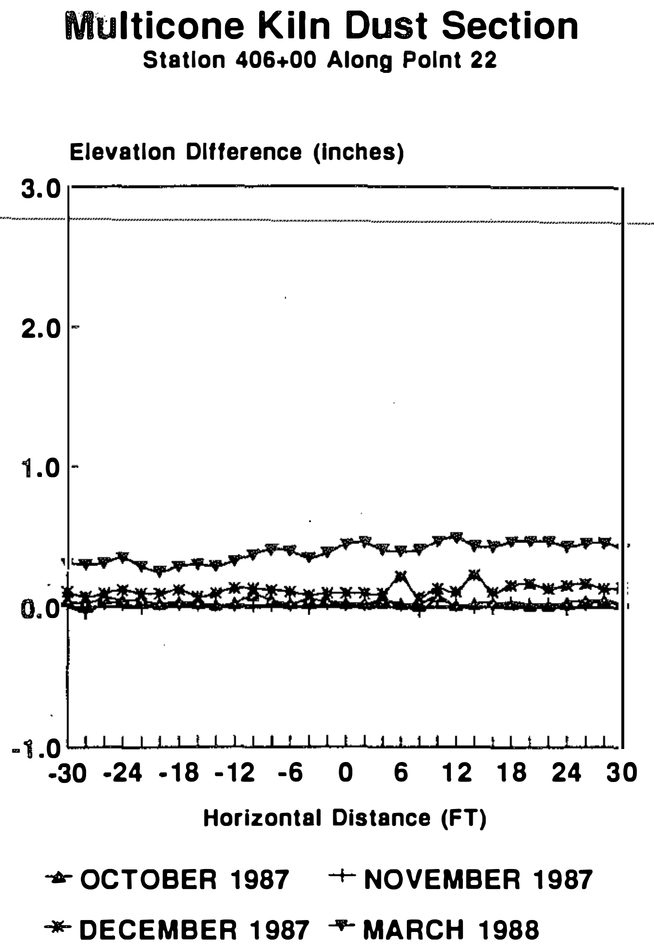
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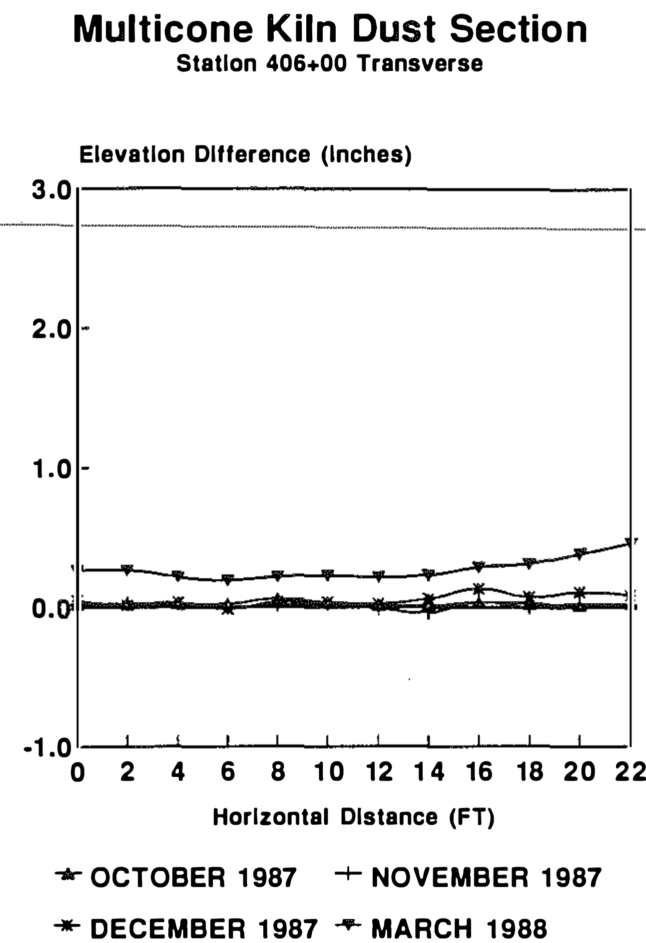
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DECEMBER 1987
MARCH 1988

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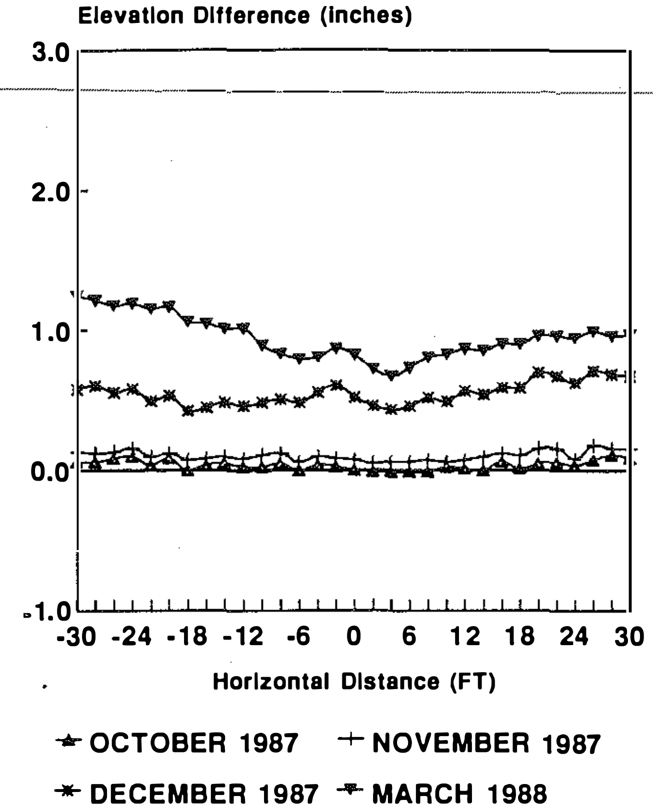

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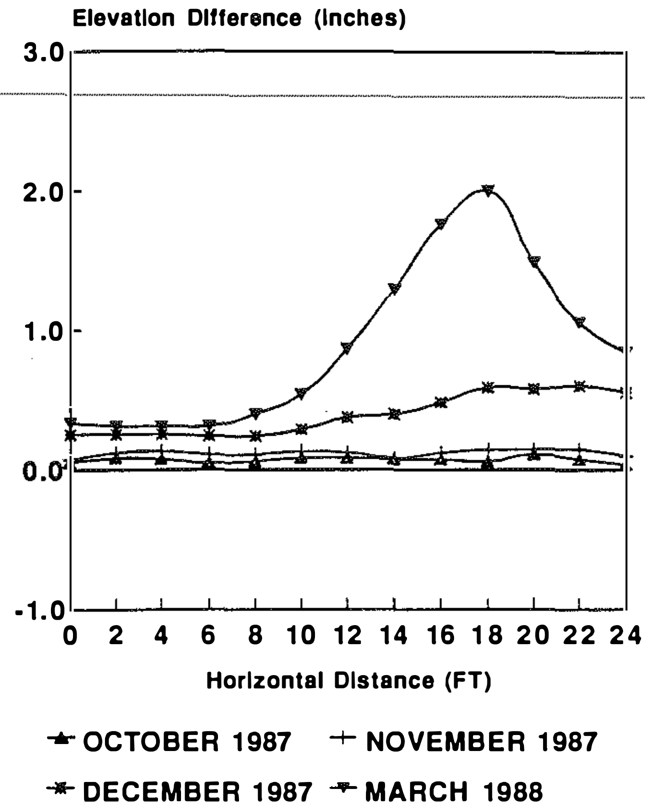

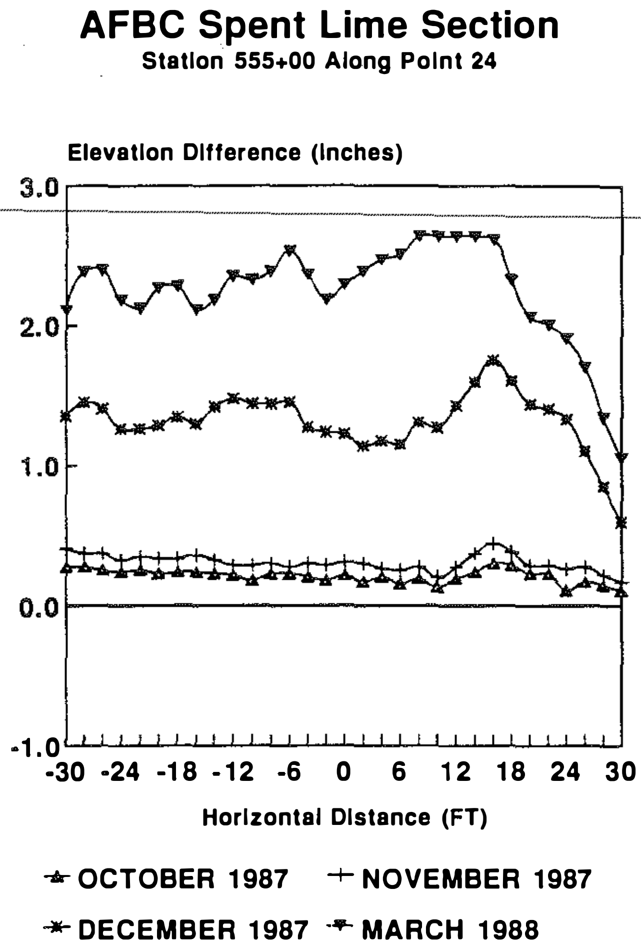

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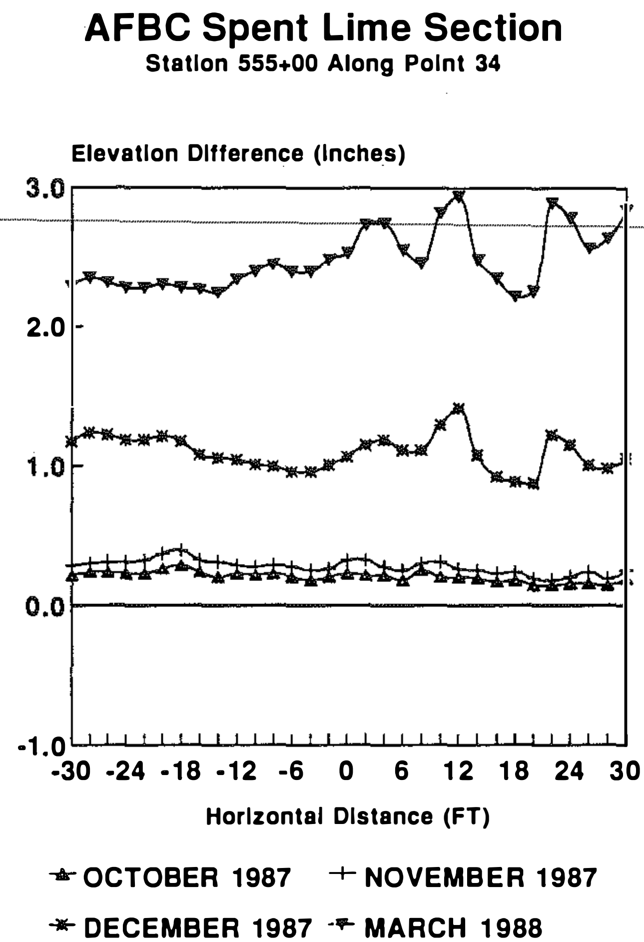

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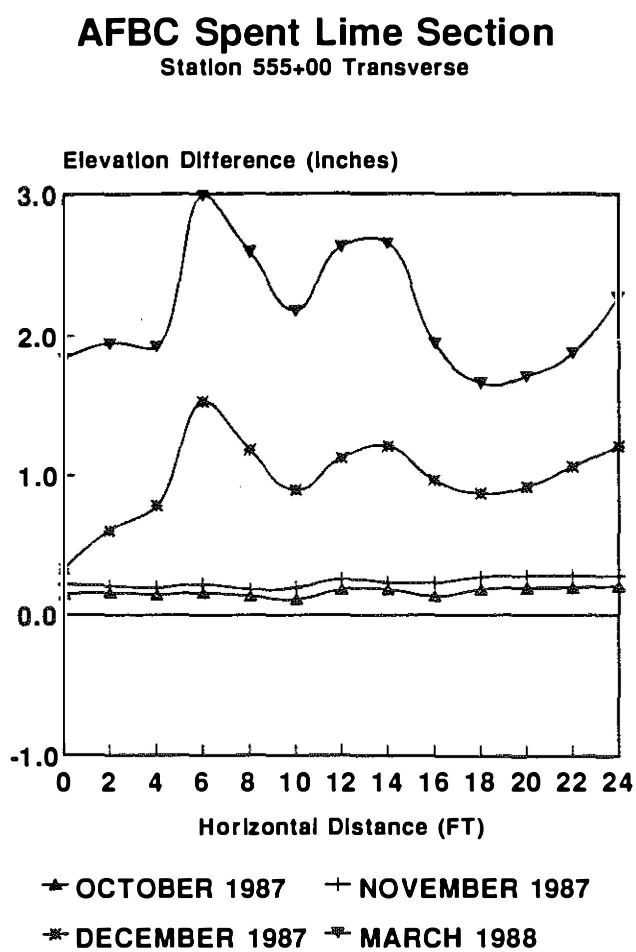

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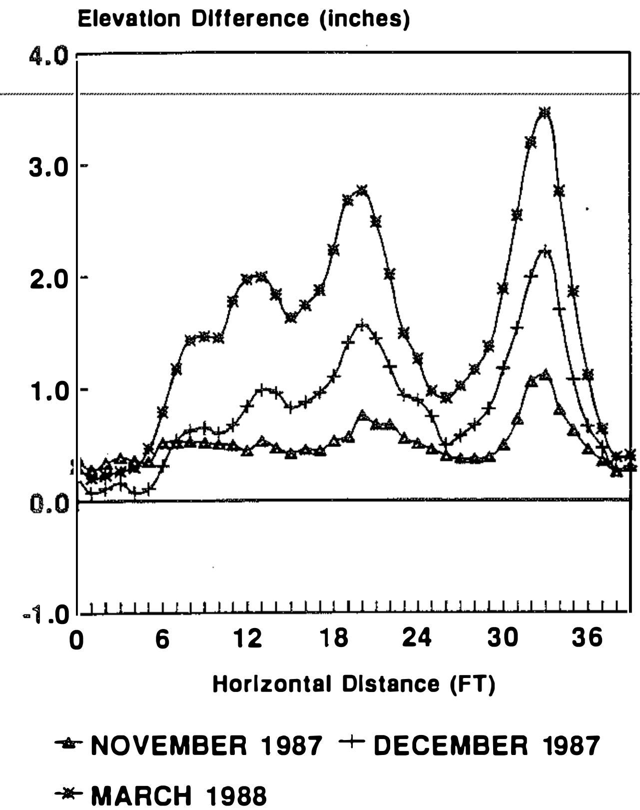

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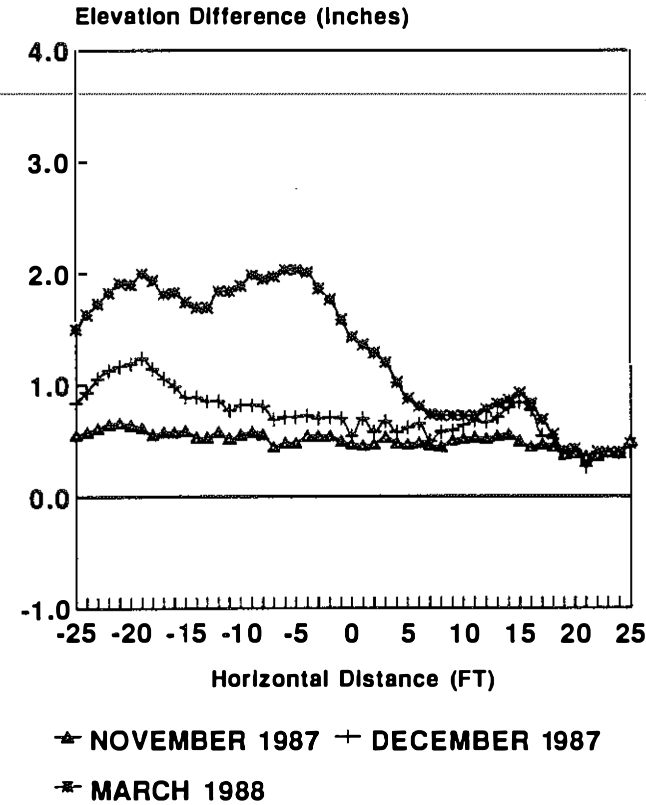

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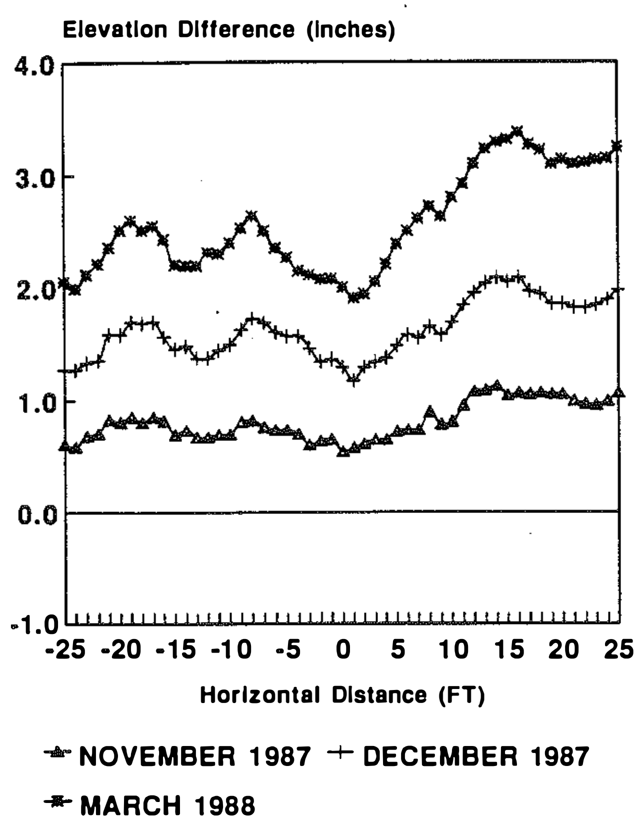

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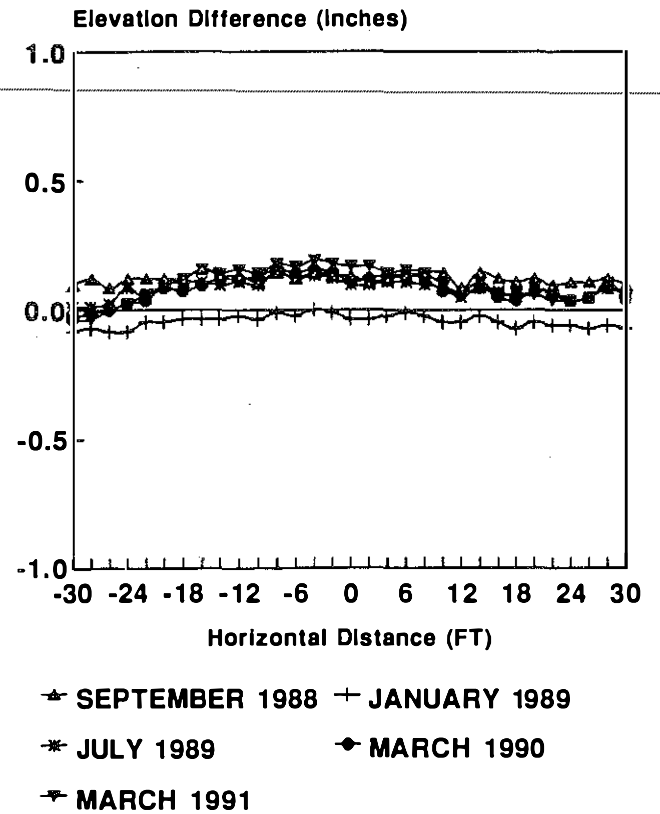

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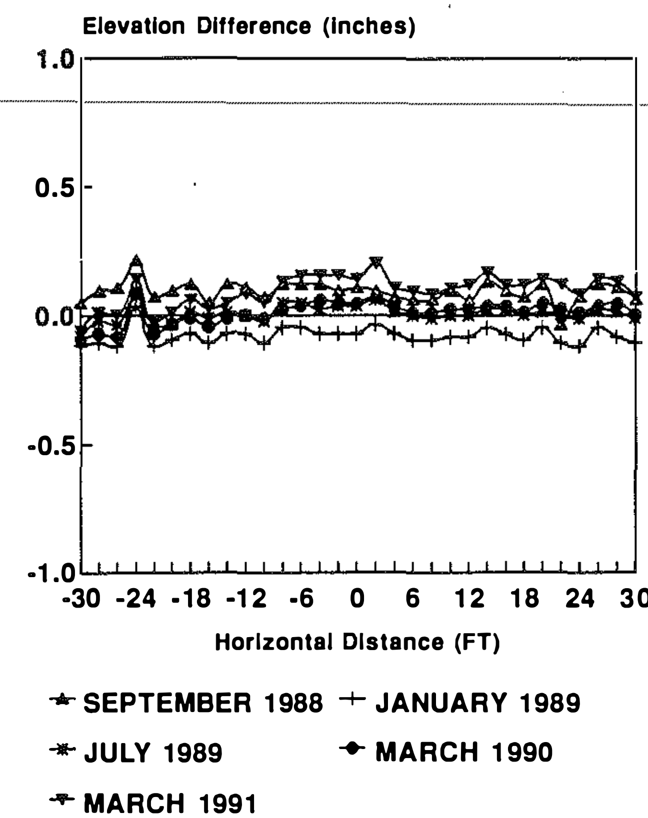

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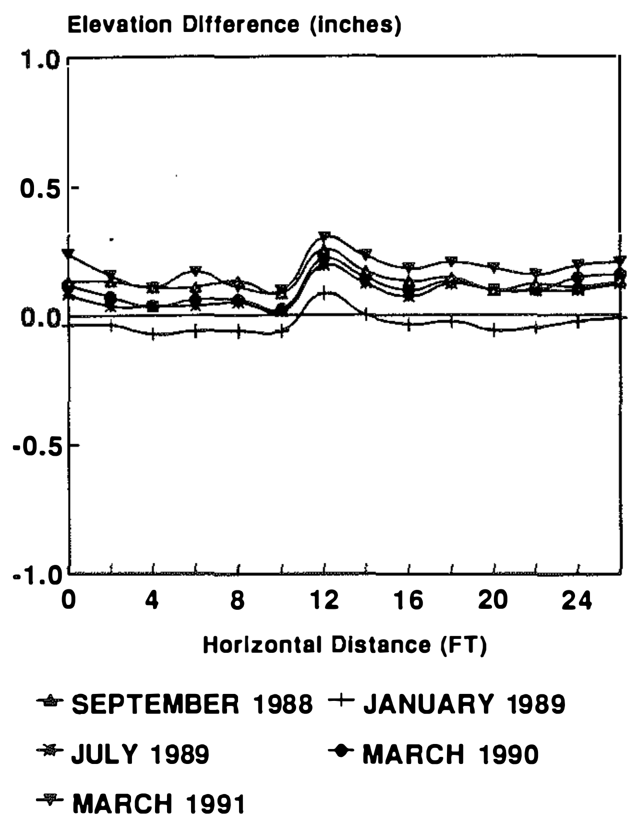

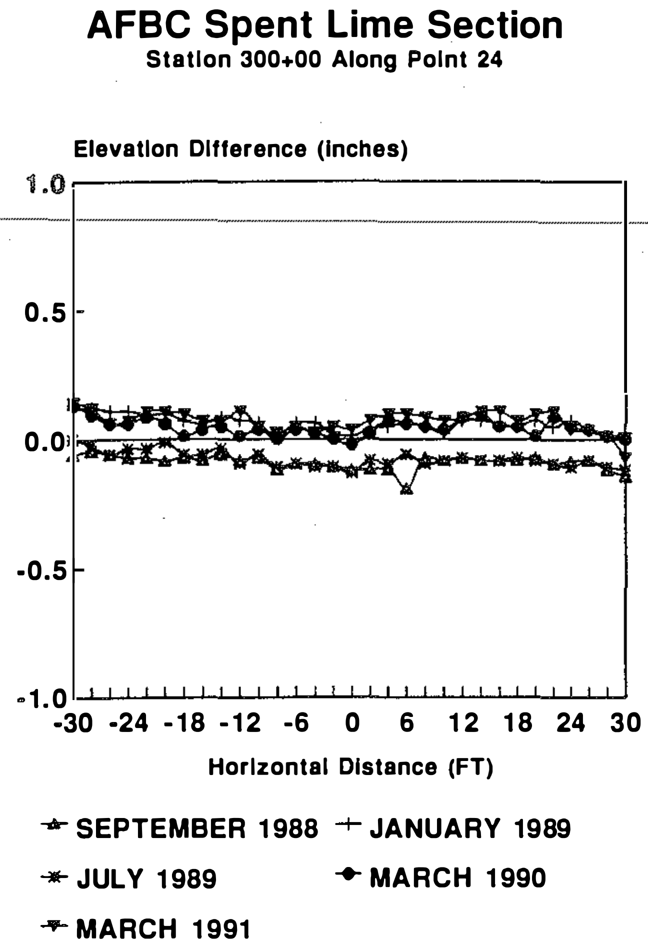
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Horizontal Distance (FT)
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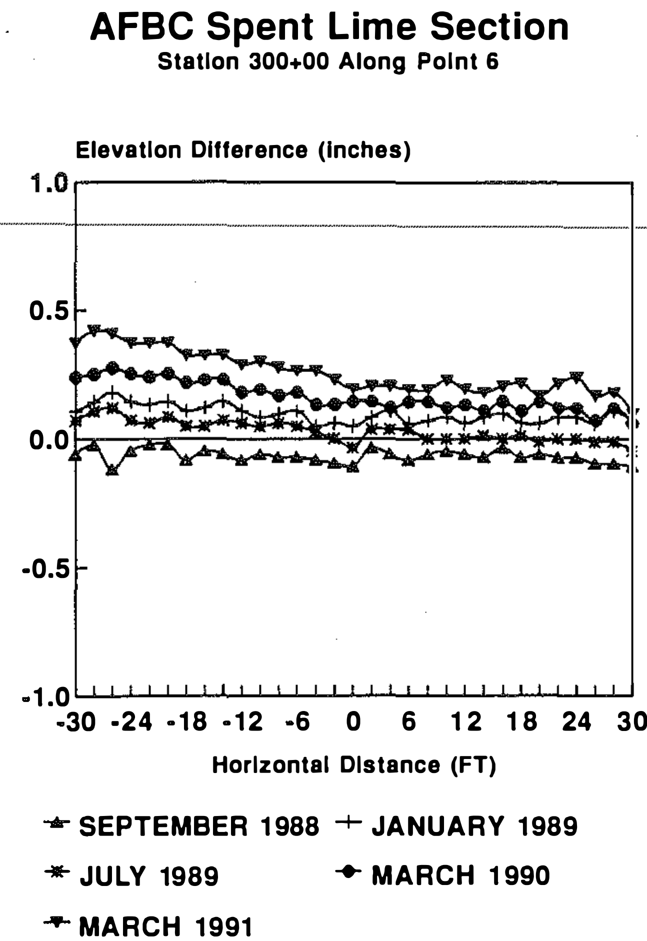
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991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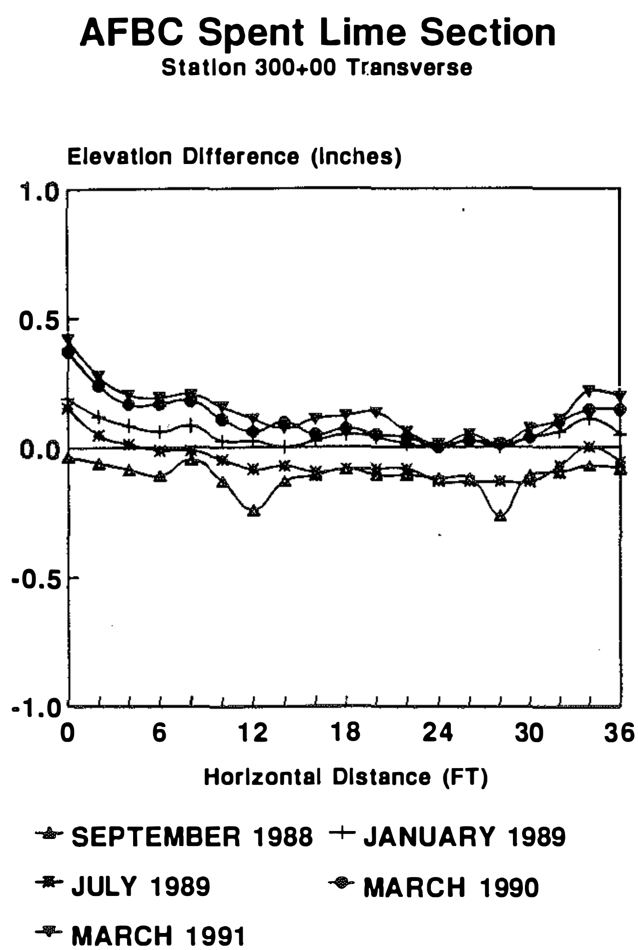
AFBC Spent Lime Section
Station 300+00 Transverse
Elevation Difference (inches)
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MARCH 1990
MARCH 1991

Cement (10%) Section
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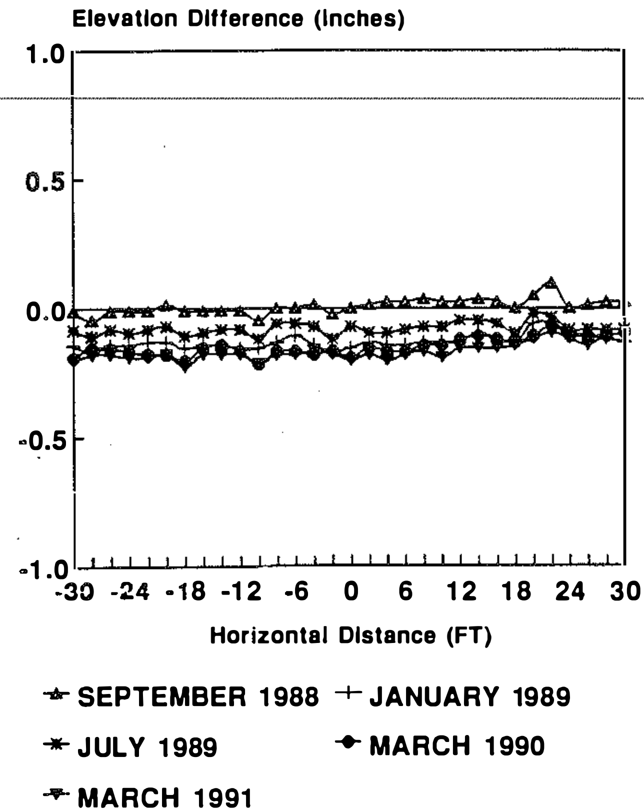
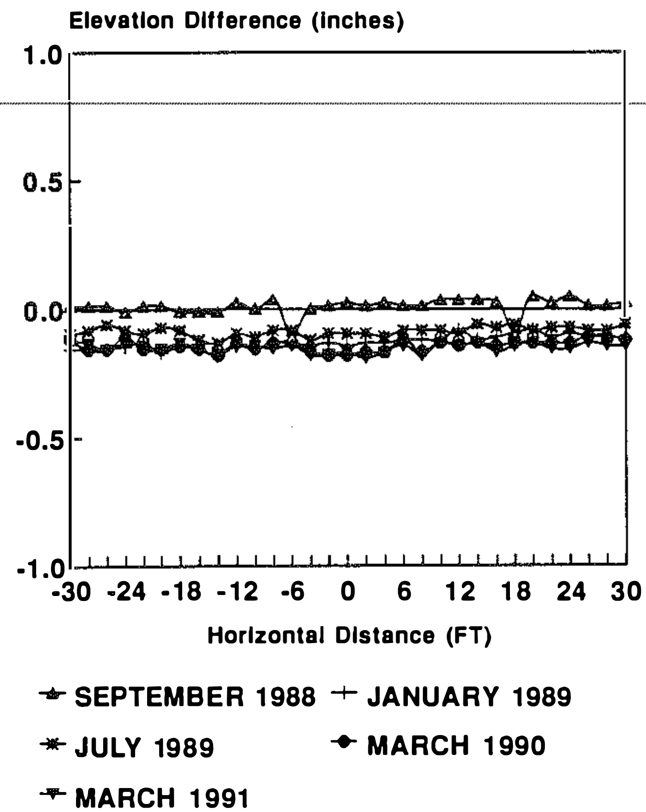
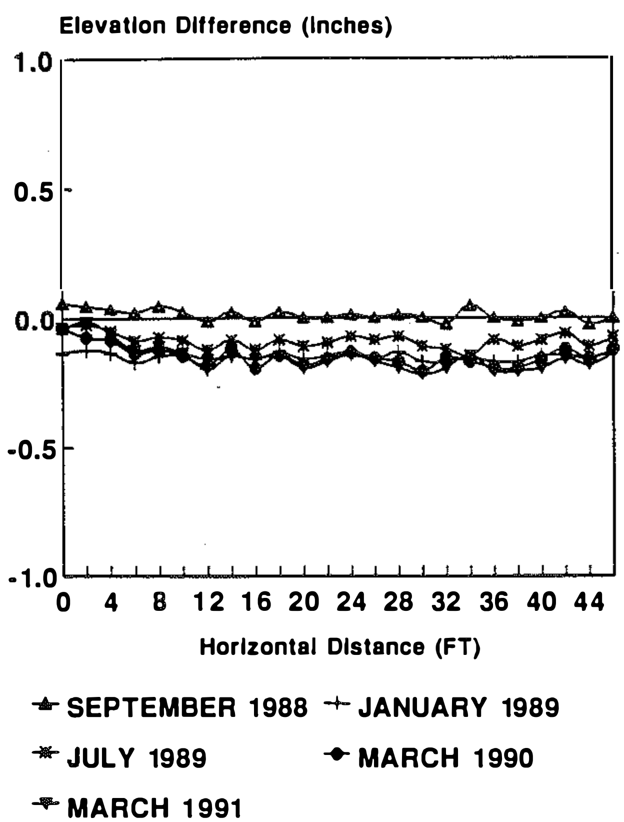
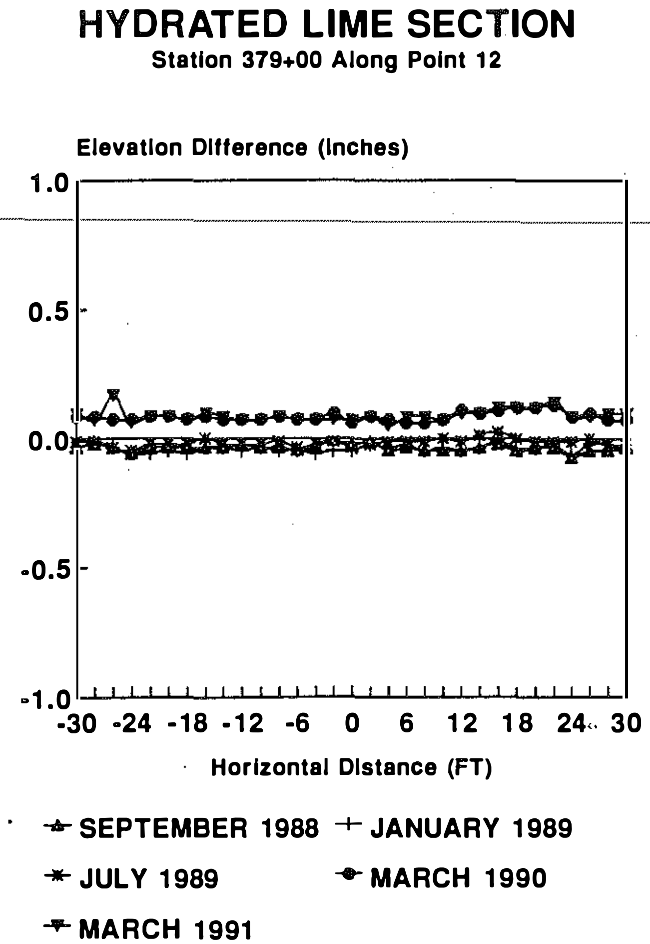


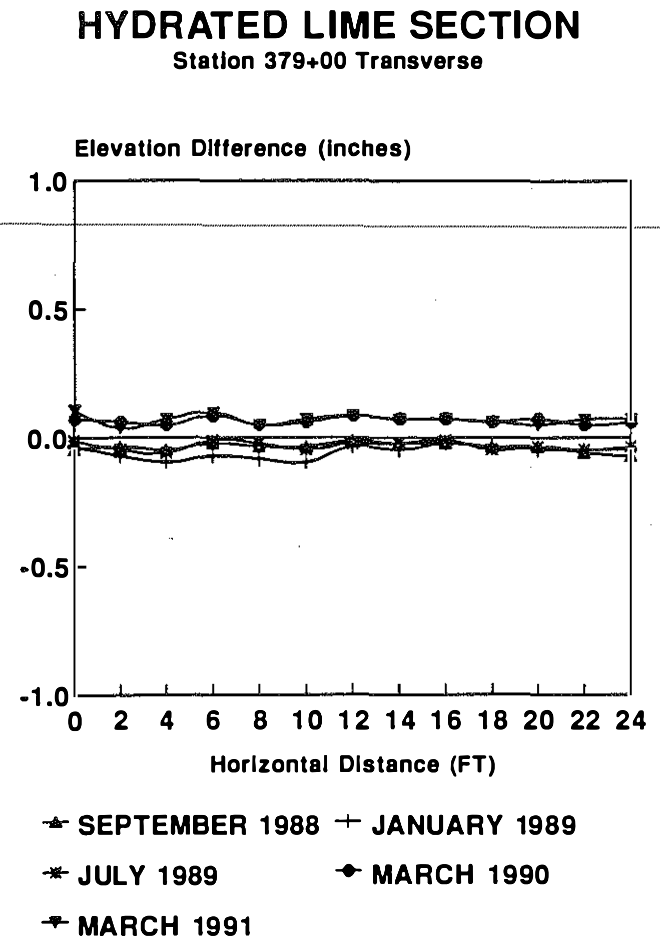
Cement (10%) Section
Station 334+00 Along Point 20



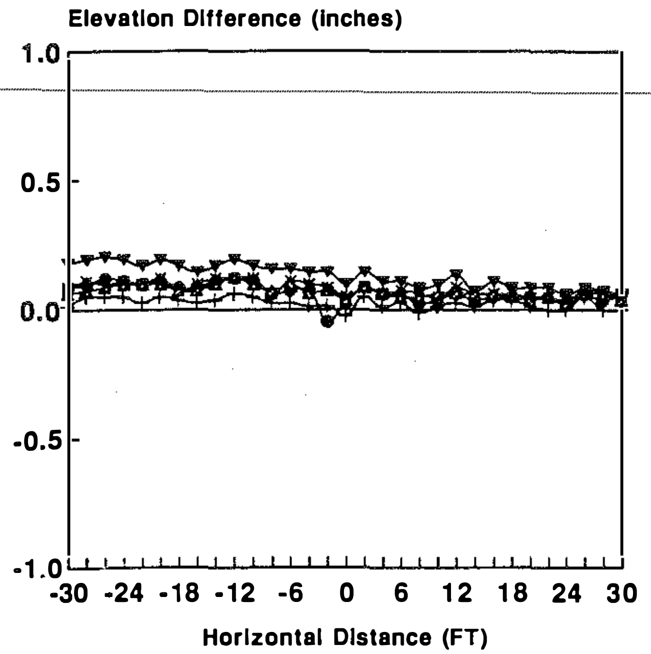
Cement (10%) Section
Station 334+00 Transverse




HYDRATED LIME SECTION
Station 379+00 Along Point 12
Elevation Difference (inches)
1.0
0.5
0.0
-0.5
-1.0
-30 -24 -18 -12 -6 0 6 12 18 24 30
Horizontal Distance (FT)
SEPTEMBER 1988
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991

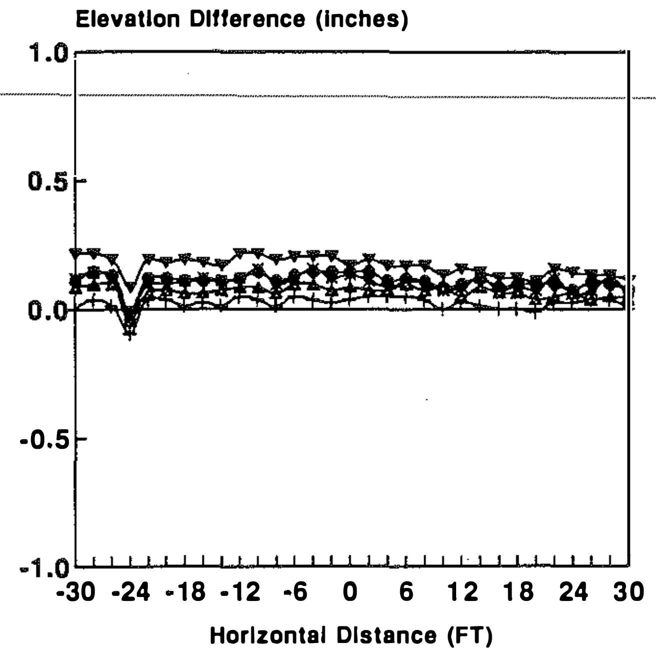

HYDRATED LIME SECTION
Station 379+00 Transverse
Elevation Difference (inches)
1.0
0.5
0.0
-0.5
-1.0
0 2 4 6 8 10 12 14 16 18 20 22 24
Horizontal Distance (FT)
SEPTEMBER 1988
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991

MULTICONE KILN DUST SECTION
Station 406+00 Along Point 22



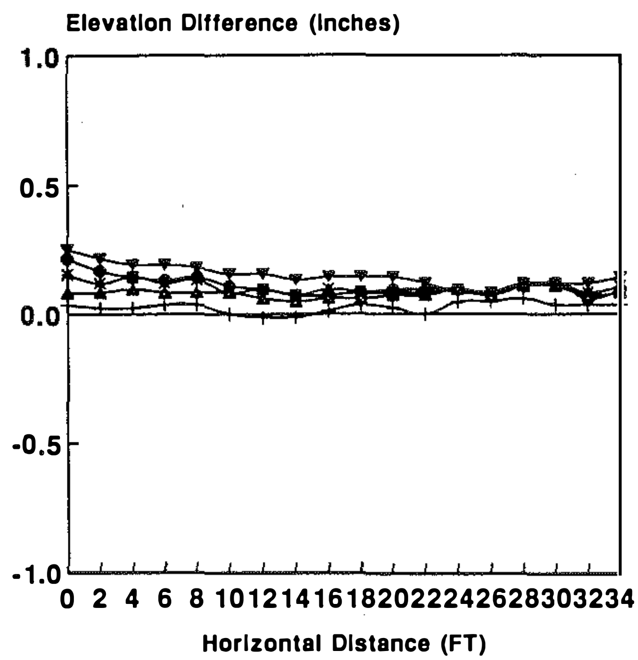
SEPTEMBER 1988 JANUARY 1989
JULY 1989 MARCH 1990
MARCH 1991

MULTICONE KILN DUST SECTION
Station 406+00 Along Point 6



SEPTEMBER 1988 JANUARY 1989
JULY 1989 MARCH 1990
MARCH 1991

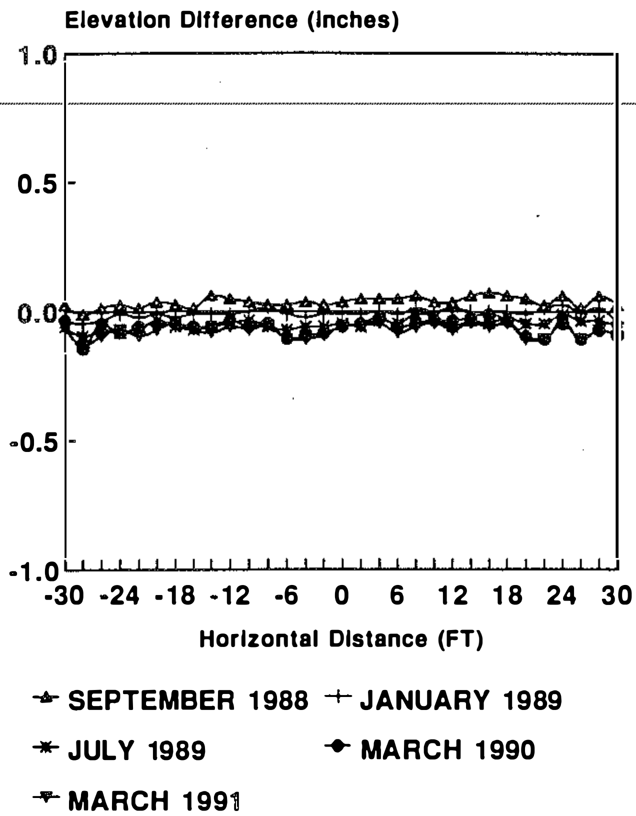
MULTICONE KILN DUST SECTION
Station 406+00 Transverse



SEPTEMBER 1988 JANUARY 1989
JULY 1989 MARCH 1990
MARCH 1991

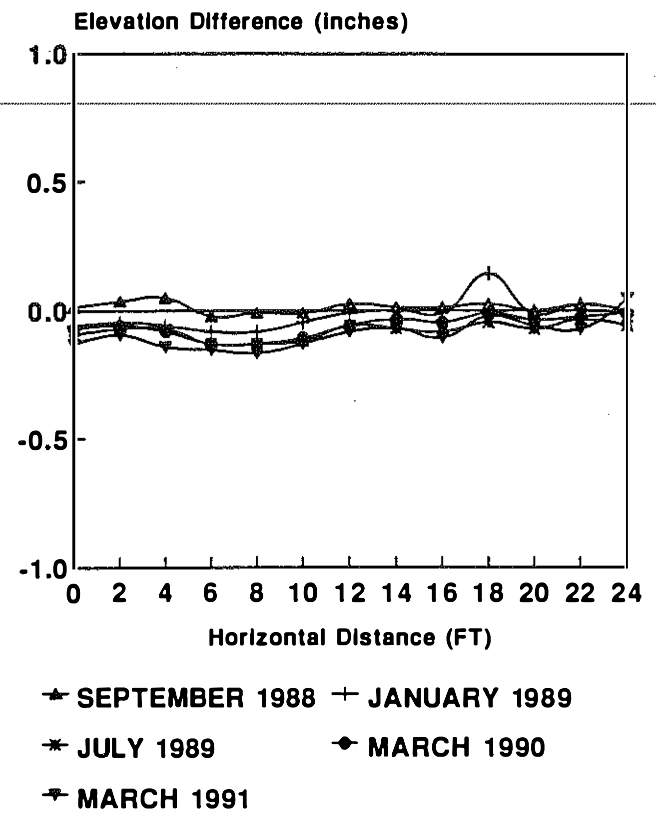
UNTREATED SECTION

Station 530+00 Along Point 12

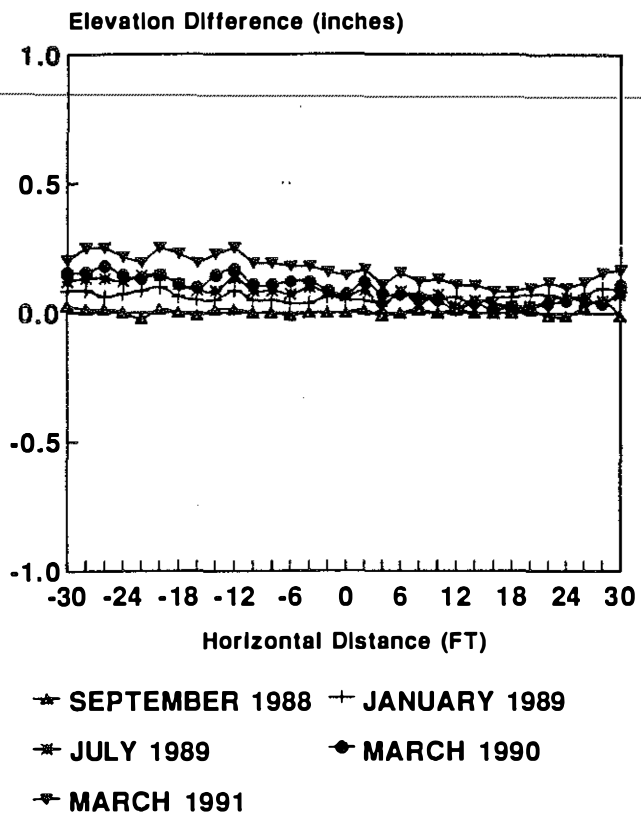


UNTREATED SECTION

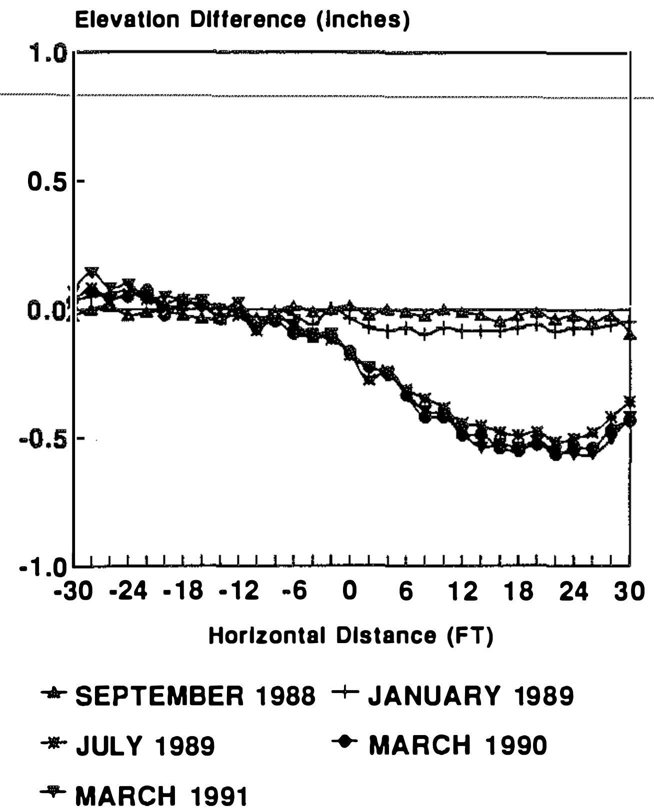
Station 530+00 Transverse



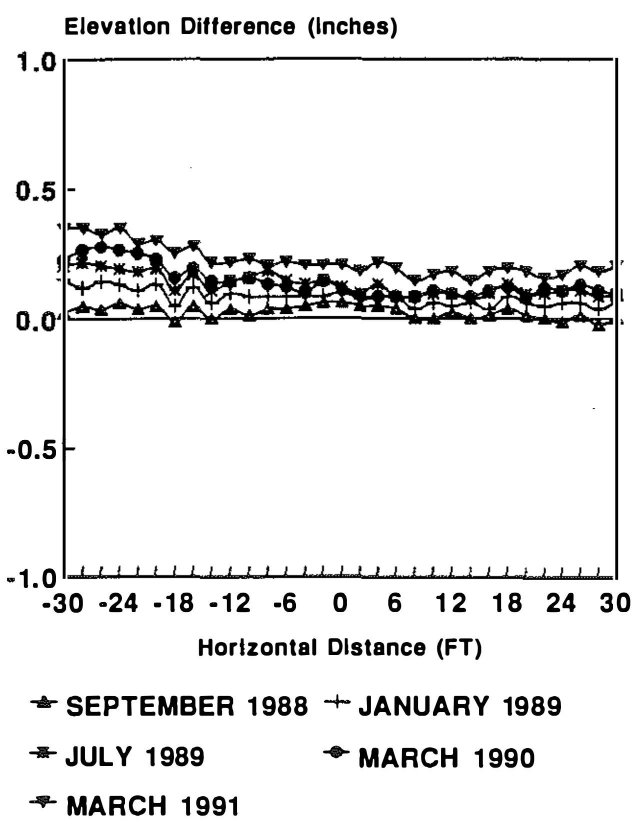
AFBC Spent Lime Section
Station 549+00 Along Point 24



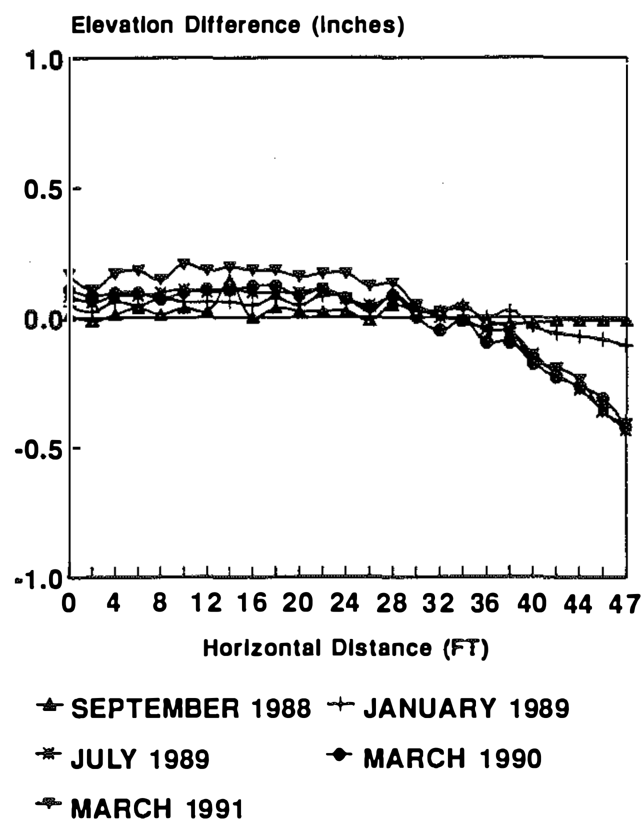
AFBC Spent Lime Section
Station 549+00 Along Point 42

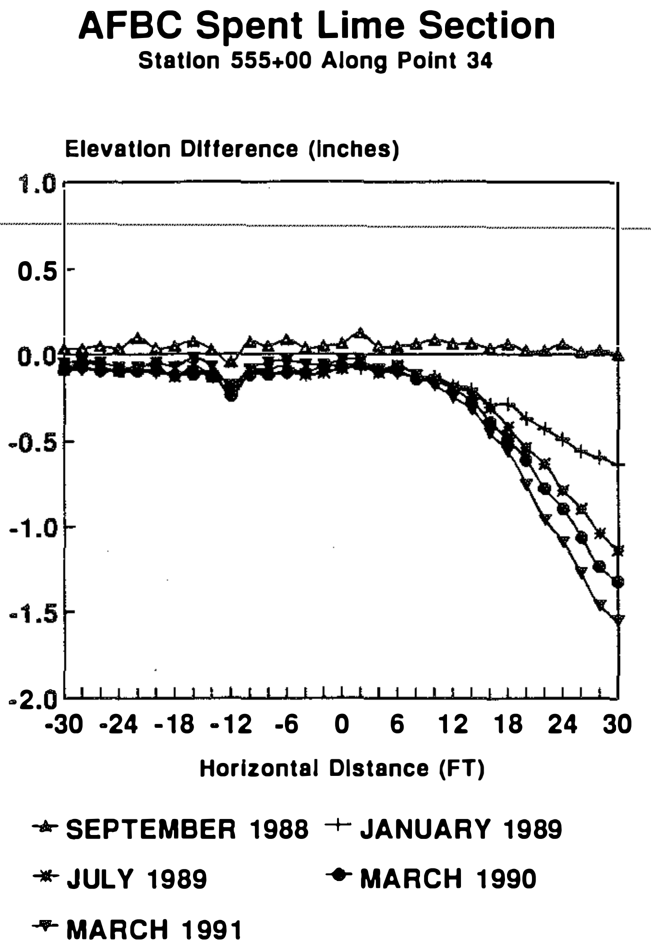


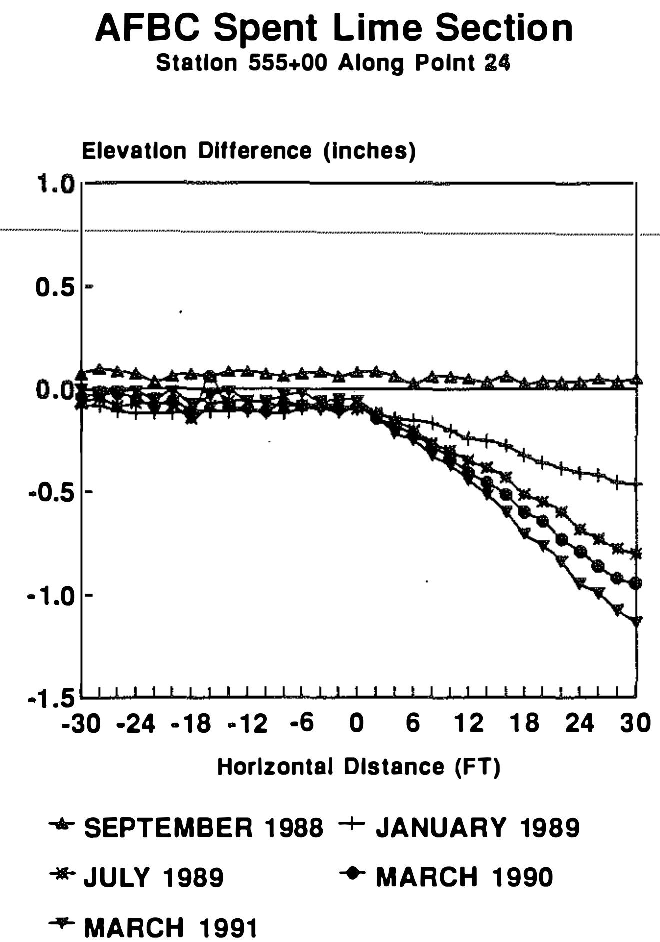
AFBC Spent Lime Section
Station 549+00 Along Point 6

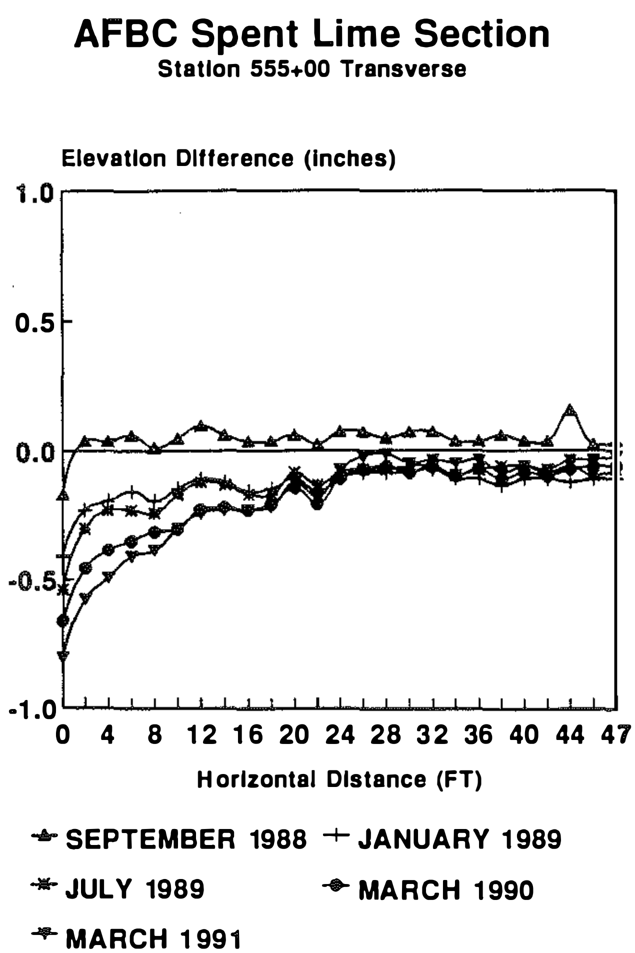


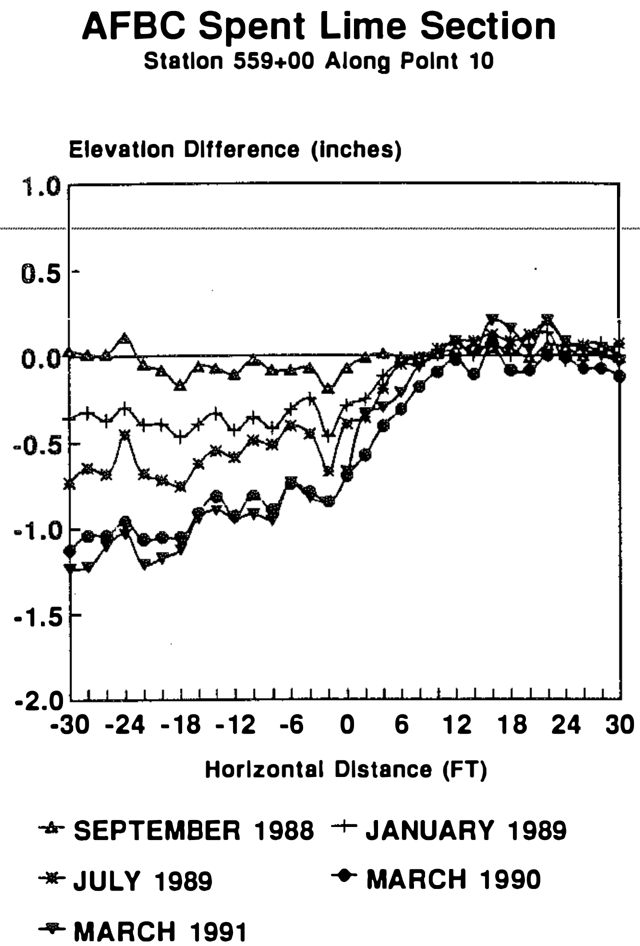
AFBC Spent Lime Section
Station 549+00 Transverse

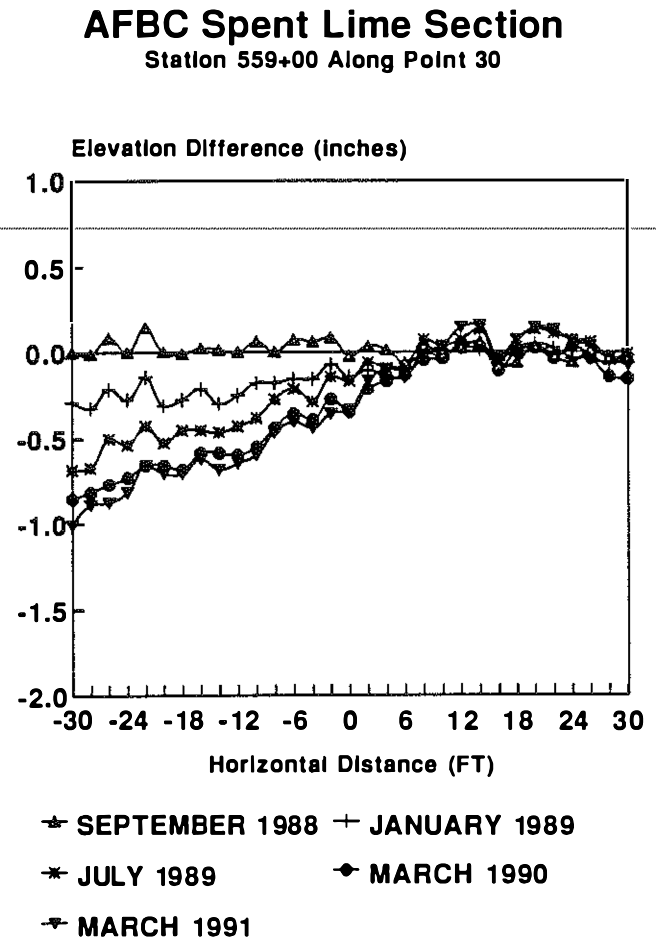


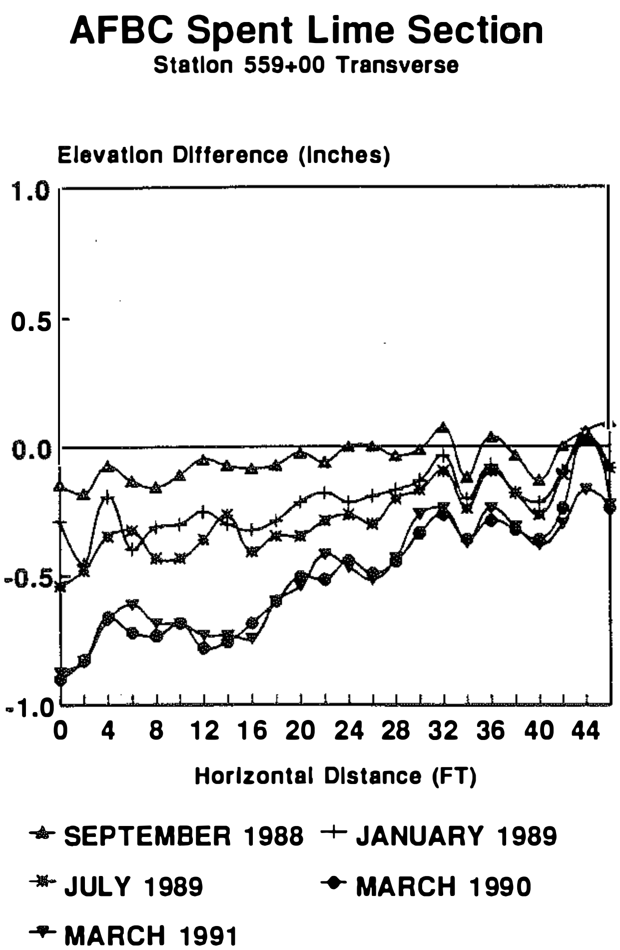

AFBC Spent Lime Section
Station 555+00 Along Point 34
Elevation Difference (inches)
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-30 -24 -18 -12 -6 0 6 12 18 24 30
Horizontal Distance (FT)
SEPTEMBER 1988
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991


AFBC Spent Lime Section
Station 555+00 Along Point 24
Elevation Difference (inches)
1.0
0.5
0.0
-0.5
-1.0
-1.5
-30 -24 -18 -12 -6 0 6 12 18 24 30
Horizontal Distance (FT)
SEPTEMBER 1988
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991


AFBC Spent Lime Section
Station 555+00 Transverse
Elevation Difference (inches)
1.0
0.5
0.0
-0.5
-1.0
0 4 8 12 16 20 24 28 32 36 40 44 47
Horizontal Distance (FT)
SEPTEMBER 1988
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991


AFBC Spent Lime Section
Station 559+00 Along Point 10
Elevation Difference (inches)
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-30 -24 -18 -12 -6 0 6 12 18 24 30
Horizontal Distance (FT)
SEPTEMBER 1988
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991


AFBC Spent Lime Section
Station 559+00 Along Point 30
Elevation Difference (inches)
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
-30 -24 -18 -12 -6 0 6 12 18 24 30
Horizontal Distance (FT)
SEPTEMBER 1988
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991


AFBC Spent Lime Section
Station 559+00 Transverse
Elevation Difference (inches)
1.0
0.5
0.0
-0.5
-1.0
0 4 8 12 16 20 24 28 32 36 40 44
Horizontal Distance (FT)
SEPTEMBER 1988
JANUARY 1989
JULY 1989
MARCH 1990
MARCH 1991

APPENDIX F

Pavement Rutting Characteristics

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1991

SOIL-AFBC SECTION STATION 260+00 TO STATION 317+50

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
267+00			1/16	1/16	2/16	1/16		
268+00			2/16	0	2/16	0		
269+00			3/16	0	1/16	0		
270+00			3/16	0	2/16	1/16		
271+00			4/16	1/16	2/16	1/16		
272+00			3/16	1/16	1/16	2/16		
273+00			2/16	0	0	0		
277+00			2/16	0	0	1/16		
278+00			3/16	0	0	1/16		
279+00			2/16	1/16	0	1/16		
280+00			4/16	0	0	1/16		
281+00			2/16	0	0	0		
282+00			2/16	3/16	2/16	1/16		
283+00			3/16	0	1/16	2/16		
284+00			1/16	0	1/16	1/16		
285+00			3/16	1/16	0	1/16		
286+00			3/16	2/16	2/16	1/16		
287+00			2/16	2/16	2/16	1/16		
288+00			2/16	1/16	1/16	1/16		
297+00			0	1/16	1/16	1/16		
298+00	0	0	0	0	2/16	2/16		
299+00	1/16	0	0	0	0	1/16		
300+00	3/16	1/16	0	0	1/16	0		
301+00	1/16	1/16	0	0	2/16	1/16		
302+00	3/16	1/16	0	0	0	0		
303+00	2/16	1/16	0	0	6/16	1/16		

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1990

SOIL-CEMENT SECTION STATION 317+50 TO STATION 348+00

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
331+00	1/16	0	0	0	0	0	0	0
332+00	0	0	0	0	0	0	0	0
333+00	0	0	0	0	0	0	0	0
334+00			0	2/16	0	0	0	0
335+00			0	5/16	0	0	0	0
336+00			0	2/16	0	0	1/16	0
337+00			0	0	0	0	0	0

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1991

SOIL-CEMENT SECTION STATION 317+50 TO STATION 348+00

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
331+00	2/16	1/16	0	0	0	0	0	0
332+00	1/16	0	0	0	0	0	0	0
333+00	1/16	0	0	0	0	0	0	0
334+00	3/16	0	0	0	0	0	0	0
335+00			0	3/16	0	0	0	0
336+00			0	4/16	0	0	0	0
337+00			0	0	0	0	0	0

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1990

SOIL-HYDRATED LIME SECTION STATION 348+00 TO STATION 402+50

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
376+00			0	0	0	0		
377+00			0	0	0	0		
378+00			0	0	1/16	0		
379+00			1/16	2/16	0	0		
380+00			0	2/16	0	0		
381+00			0	2/16	0	0		
382+00			2/16	2/16	1/16	0		

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1991

SOIL-HYDRATED LIME SECTION STATION 348+00 TO STATION 402+50

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
376+00			1/16	0	0	0		
377+00			1/16	0	0	0		
378+00			1/16	0	0	0		
379+00			2/16	2/16	0	1/16		
380+00			2/16	2/16	1/16	0		
381+00			2/16	0	0	0		
382+00			3/16	3/16	1/16	0		

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1990

UNTREATED SOIL SECTION STATION 522+00 TO STATION 532+00

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
527+00			3/16	2/16	0	2/16		
528+00			3/16	1/16	1/16	2/16		
529+00			1/16	1/16	1/16	1/16		
530+00			2/16	0	0	1/16		
531+00			1/16	0	0	0		
532+00			2/16	0	0	0		
533+00			4/16	1/16	0	1/16		

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1991

UNTREATED SOIL SECTION STATION 522+00 TO STATION 532+00

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
527+00			3/16	1/16	0	1/16		
528+00			2/16	0	0	2/16		
529+00			3/16	0	0	2/16		
530+00			4/16	1/16	0	2/16		
531+00			3/16	3/16	0	1/16		
532+00			4/16	1/16	0	2/16		
533+00			7/16	2/16	0	1/16		

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1990

SOIL-AFBC SECTION STATION 532+00 TO STATION 576+50

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
546+00	2/16	1/16	1/16	0	0	0		
547+00	2/16	2/16	0	0	0	2/16		
548+00	2/16	2/16	0	0	0	1/16	0	0
549+00	2/16	1/16	0	0	0	0	0	0
550+00	3/16	0	HUMP	1/16	1/16	0	0	0
551+00	2/16	1/16	0	0	0	0	0	0
552+00	2/16	2/16	0	0	0	1/16	0	0
553+00	2/16	1/16	0	0	0	0	0	1/16
554+00	1/16	1/16	1/16	0	0	0	0	0
555+00	2/16	1/16	1/16	0	0	1/16	0	1/16
556+00	2/16	2/16	1/16	0	1/16	0	1/16	0
557+00	1/16	0	0	0	1/16	0	1/16	0
558+00	1/16	1/16	0	0	1/16	0	3/16	0
559+00	2/16	1/16	0	0	0	0	1/16	3/16
560+00	1/16	0	0	0	1/16	0	1/16	0
561+00	1/16	0	0	0	1/16	0	1/16	0
562+00	0	0	1/16	0	1/16	0	3/16	0
563+00	1/16	0	4/16	0	HUMP	HUMP	1/16	0
564+00	0	0	0	0	1/16	0	3/16	1/16
565+00	0	0	0	0	1/16	0	2/16	1/16
566+00	1/16	0	0	0	2/16	0	1/16	1/16
567+00	7/16	1/16	0	12/16	2/16	0	3/16	1/16
568+00	5/16	2/16	2/16	7/16	1/16	0	2/16	0
569+00			0	0	3/16	0	2/16	0
570+00			2/16	0	0	0	0	1/16
571+00			1/16	0	0	1/16	0	0
572+00			2/16	0	0	0	9/16	1/16

KENTUCKY ROUTE 11 RUTTING DATA; MARCH 1991

SOIL-AFBC SECTION STATION 532+00 TO STATION 576+50

	NORTHBOUND DIRECTION				SOUTHBOUND DIRECTION			
	TRUCK LANE		THROUGH LANE		THROUGH LANE		TRUCK LANE	
STATION	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)	LWP (in.)	RWP (in.)
546+00	4/16	3/16	2/16	0	1/16	0		
547+00	3/16	3/16	0	0	2/16	4/16		
548+00	4/16	5/16	1/16	0	1/16	3/16	0	0
549+00	3/16	4/16	1/16	0	0	0	0	0
550+00	7/16	2/16	7/16	4/16	1/16	1/16	0	0
551+00	4/16	3/16	1/16	0	1/16	1/16	0	0
552+00	4/16	4/16	1/16	4/16	0	2/16	0	0
553+00	4/16	2/16	1/16	0	0	0	0	2/16
554+00	2/16	3/16	2/16	0	0	0	0	2/16
555+00	5/16	4/16	1/16	0	0	1/16	1/16	2/16
556+00	4/16	5/16	1/16	0	0	0	2/16	3/16
557+00	3/16	2/16	0	0	1/16	0	3/16	3/16
558+00	3/16	3/16	0	0	2/16	0	5/16	0
559+00	3/16	4/16	2/16	0	2/16	0	4/16	7/16
560+00	2/16	1/16	1/16	1/16	4/16	0	5/16	0
561+00	1/16	1/16	1/16	1/16	3/16	0	4/16	3/16
562+00	1/16	0	2/16	0	3/16	2/16	7/16	1/16
563+00	1/16	0	9/16	0	1/16	2/16	9/16	1/16
564+00	0	1/16	0	1/16	4/16	0	4/16	1/16
565+00	0	0	1/16	0	2/16	0	5/16	1/16
566+00			3/16	0	6/16	0	8/16	1/16
567+00			2/16	0	0	0	6/16	2/16
568+00			10/16	0	Dip due to cross drain			
569+00			0	0	7/16	0	3/16	1/16
570+00			2/16	0	1/16	0	1/16	1/16
571+00			1/16	0	3/16	1/16	1/16	0
572+00			2/16	0	0	0	9/16	2/16