DRAFT

Research Report
KTC 93-16

PERFORMANCE OF EXPERIMENTAL HIGHWAY BASE AND SUBBASE LAYERS CONTAINING BY-PRODUCT MATERIALS FROM A COAL-FIRED POWER PLANT: KY ROUTE 3074, BLEICH ROAD

by

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in cooperation with
Kentucky Transportation Cabinet

and

Federal Highway Administration
US Department of Transportation

June 1993

Technical Report Documentation Page

1. Report No. KTC 93-16	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Performance of Experimental Highway Base and Subbase Layers Containing Coal-Fired Power Plant By-Product Materials: KY Route 3074, Bleich Road		5. Report Date June 1993 6. Performing Organization Code
7. Author(s) David Q. Hunsucker and R. Clark Graves		8. Performing Organization Report No. KTC 93-16
9. Performing Organization Name and Address Kentucky Transportation Center College of Engineering University of Kentucky Lexington, KY 40506-0043		10. Work Unit No. (TRAIS) 11. Contract or Grant No. KYHPR 87-117 13. Type of Report and Period Covered Interim
12. Sponsoring Agency Name and Address Kentucky Transportation Cabinet State Office Building Frankfort, KY 40622		14. Sponsoring Agency Code NCP 4E2B5042

15. Supplementary Notes Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration - Previous Reports: KTC 89-56, "Use of Ponded Fly Ash in a Highway Road Base," KTC 92-14, "Design and Performance of a Bituminous Surface Mixture Containing Bottom Ash Aggregate," KTC 92-17, "Atmospheric Fluidized Bed Combustion Residue in Highway Base and Subbase Construction."

16. Abstract

Three 750-foot test sections of a 22-foot wide roadway were constructed on newly constructed subgrade in May and June, 1988. The three experimental sections contained experimental base and subbase layers comprised of mixtures utilizing by-product materials from a coal-fired power plant. One test section included a base layer containing residue from an atmospheric fluidized bed combustion (AFBC) process, size No. 57 aggregate, and Class F fly ash. The second test section included a base layer constructed ponded fly ash, hydrated lime, and dense-graded aggregate. The third test section included a subbase layer constructed of pond ash stabilized with AFBC residue. A control section was included in the study for comparison purposes. Previous reports have documented preliminary engineering, construction details, and initial performance evaluations of the three experimental sections constructed on State Route 3074 in McCracken County, Kentucky. This report summarizes performance information derived from the field trials. Performance measurements included distress surveys, Road Rater deflections, pavement rutting measurements, and field cores of the experimental base and subbase layers were obtained for laboratory evaluations.

The two test sections containing residue from the AFBC process were regarded as failures due to the excessive expansion of the materials, although efforts were made to eliminate, or minimize, the inherent expansive properties of the residue by prehydrating the AFBC residue prior to its use in the mixtures. The two layers were eventually removed and replaced with conventional materials. The remaining experimental section has given good performance and service. No significant distresses have been noted. Rutting measurements indicated less rutting occurring in the ponded fly ash-hydrated lime-dense grade aggregate section than in the control section. Analyses of deflection data indicate superior structural characteristics compared to the control section. Unconfined compression tests indicate excellent strengths of the experimental mixture. The accomplishment of this application confirms that waste fly ash may be used successfully in stabilized road base construction.

17. Key Words Atmospheric Fluidized Bed Combustion residue, Ponded Ash, Fly Ash, Bottom Ash, Highway Base and Subbase, Performance, Pavement Deflection, Pavement Rutting, Unconfined Compressive Strength, Visual Distress		18. Distribution Statement Unlimited with approval of the Kentucky Transportation Cabinet	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 81	22. Price

Form DOT 1700.7 (8-72) Reproduction of completed page authorized

METRIC CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO METRIC UNITS

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

APPROXIMATE CONVERSIONS FROM METRIC UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.03937	inches	in.
m	metres	3.28084	feet	ft
m	metres	1.09361	yards	yd
km	kilometres	0.62137	miles	mi
		AREA		
mm^2	millimetres squared	0.00155	square inches	$in.^2$
m^2	metres squared	10.76392	square feet	ft^2
m^2	metres squared	1.19599	square yards	yd^2
ha	hectares	2.47103	acres	ac
km^2	kilometres squared	0.38610	square miles	mi^2
		VOLUME		
ml	millilitres	0.03381	fluid ounces	fl oz
l	litres	0.26417	gallons	gal.
m^3	metres cubed	35.31448	cubic feet	ft^3
m^3	metres cubed	1.30795	cubic yards	yd^3
		MASS		
g	grams	0.03527	ounces	oz
kg	kilograms	2.20462	pounds	lb
Mg	megagrams	1.10231	short tons (2000 lb)	T
		FORCE		
N	newtons	0.22481	pound-force	lbf
kPa	kilopascal	0.14504	pound-force per square inch	psi
		ILLUMINATION		
lx	lux	0.09290	foot-candles	fc
cd/m^2	candela/m^2	0.29190	foot-Lamberts	fl
		TEMPERATURE (exact)		
°C	Celsius temperature	1.8C + 32	Fahrenheit temperature	°F

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF FIGURES (continued)

ACKNOWLEDGEMENT

The authors wish to express their appreciation and gratitude to Mr. Charlie Dodson, Kentucky Department of Highways' Resident Engineer for McCracken County, and his staff for their invaluable assistance during construction and monitoring activities associated with the Bleich Road project. The authors also wish to acknowledge the support of Mr. Bryan Stewart, Mr. James Gray, and Mr. James Cain of the Kentucky Department of Highways' District One engineering staff.

EXECUTIVE SUMMARY

This report summarizes the performance of three experimental field trials wherein various by-products from a coal-fired generating plant were utilized to construct highway base and subbase layers. Three 750-foot test sections of a 22-foot wide roadway, containing the base and subbase mixtures, were constructed in May and June 1988 on newly constructed subgrade utilizing power plant wastes from the Tennessee Valley Authority's Shawnee Power Plant near Paducah, Kentucky. One test section included a base layer constructed of a mixture of residue from an atmospheric fluidized bed combustion (AFBC) process, size No. 57 aggregate, and Class F fly ash. Another test section included a base layer constructed of dense-graded aggregate combined with hydrated lime and ponded fly ash. The final test section included a subbase layer constructed of a mixture of AFBC residue and pond ash. A section constructed of conventional materials served as a control section. Previous reports have documented preliminary engineering, construction details, and initial performance of the three experimental sections constructed on State Route 3074 in McCracken County, Kentucky.

Performance evaluations and measurements performed as part of this research effort consisted of distress surveys, traffic counts, Road Rater deflections and analyses, pavement rutting characteristics, and strength and durability measurements of field cores obtained from the experimental base and subbase layers.

The base and subbase layers containing residue from the AFBC process were considered failures based upon very poor performance caused by the excessive expansion of the base and subbase mixtures and resulting pavement distresses. Although the AFBC residue was prehydrated prior to its use to eliminate, or control, the inherent expansive properties of the residue, the material continued to cause extensive pavement distress during the evaluation. The two sections wherein AFBC residue was used in the mixtures were eventually removed after being in service for about three years and replaced with conventional materials. Further use of the AFBC residue in highway base and subbase layers should not be permitted unless techniques can be used that will ensure elimination of the expansive nature of the AFBC residue. A pugmill set up near the construction would permit prehydration activities to transpire only a few days before the commencement of construction. A pugmill would also permit mixture components to be batched at a more suitable rate than achieved during this field trial.

The remaining experimental section containing a ponded fly ash-hydrated lime-dense grade aggregate base has exhibited superior performance during the evaluation period. One transverse and one longitudinal reflective crack have been observed. No significant distresses have been noted. Rutting measurements revealed less rutting in the experimental pozzolanic section than that observed in the control section. Analysis of the deflection data indicated exceptional structural characteristics. Unconfined compressive strength tests substantiated the excellent and consistent strengths of the mixture. The performance of this experimental pozzolanic base layer clearly demonstrates that a non-specification fly ash, a waste material, can be successfully substituted for specification fly ash in stabilized aggregate base construction.

INTRODUCTION AND BACKGROUND

Kentucky has traditionally been among the leading producers of coal. Coal-fired electric generating facilities are abundant in Kentucky and as a result, by-products in the form of fly ash, flue gas desulfurization sludge, boiler slag, and bottom ash are generated in large quantities. Approximately three million tons of fly ash are produced annually from Kentucky power plants. With the escalating costs of materials and construction for highways and streets, many agencies charged with the responsibility of designing and constructing highways are utilizing by-product materials. Development of procedures for utilizing by-products from coal-fired power plants serves three functions. Use of coal is enhanced since utilization of by-products from burning and pollution-control processes provides for the management of those by-product materials that otherwise would have required disposal; the economy of Kentucky is benefitted by the continued utilization of one of its most prized natural resources; and, utilization of by-product materials results in conservation of other valuable resources.

An experimental roadway was constructed on a construction access road at the Tennessee Valley Authority's Shawnee Power Plant near Paducah wherein residue from an atmospheric fluidized bed combustion (AFBC) process was combined with limestone aggregate and fly ash and was used as the base material for the roadway [3]. Although the section was very short, preliminary research indicated considerable promise for the use of this by-product waste material as a pavement component. Therefore, it was desirable to construct and evaluate a larger test section to more specifically identify and define the properties and long-term performance of base sections utilizing residue from the AFBC process. In January of 1987, representatives of the Kentucky Transportation Center (KTC) and the Tennessee Valley Authority (TVA) met with representatives of the Kentucky Transportation Cabinet (KyTC) to discuss the possibility of an experimental project wherein various waste materials from TVA's Shawnee Power Plant would be incorporated in the base and/or subbase layers of a roadway. The Kentucky Department of Highways (KDOH), had an upcoming project and agreed to the experimental use of TVA's by-product materials to construct base and subbase layers. Officials with TVA agreed to provide the by-product materials at no cost to the KDOH. By-product materials to be provided by TVA's Shawnee Power Plant were pond ash (bottom ash and fly ash) and residue from an Atmospheric Fluidized Bed Combustion (AFBC) process. There were no requirements that the waste materials meet any KDOH specifications. The AFBC residue was a by-product produced at a pilot plant wherein new clean-coal technology was being implemented in an attempt to lessen the amount of sulfur dioxide emissions produced by the coal-fired electric generating plant. The pond ash utilized during the study was a waste by-product produced at the conventional coal-fired electric generating plant located on the TVA reservation.

The objectives of this study were: conduct laboratory evaluations of mixtures containing residue from the AFBC process and pond ash materials; plan, design and monitor construction of the test road; and, to compare the performance of the experimental sections relative to a conventionally designed and constructed section. Previous reports have documented the laboratory characterizations of physical properties of the by-product materials and proposed mixtures, characteristics of the pavement designs incorporating the experimental base and subbase layers, and initial, short-term performance of the experimental layers [1,2]. This report contains a summary of information reported previously regarding preconstruction and construction activities associated with this project and documents the post-construction performance monitoring of the experimental and control sections. Specifically, this report documents distress surveys conducted to observe the performance of each section, dynamic deflection tests and associated analyses, pavement rutting measurements, and traffic histories. Laboratory evaluations performed on field cores obtained from the experimental base and subbase layers are also detailed herein.

SUMMARY OF INITIAL EVALUATIONS

This was the first full-scale project in Kentucky wherein AFBC residue was utilized in the construction of stabilized aggregate road base and subbase layers. This also was the first full-scale project wherein a non-specification fly ash was combined with hydrated lime and dense graded aggregate (DGA) to construct a stabilized base layer. Extensive laboratory evaluations of the by-product materials were performed prior to construction. These evaluations included, but were not limited to, particle-size analyses, specific gravities, moisture/density relationships, and development of unconfined compressive strength. The materials design of the AFBC concrete base mixture was based upon unconfined compressive strength tests and optimization of three mix variables: volume of water needed for prehydrating the AFBC residue; the fly ash to AFBC residue ratio; and, the amount of coarse aggregate. The AFBC residue used in the laboratory study was preconditioned, or prehydrated, by mixing thoroughly with 18 percent water, by weight, and stored in sealed 55-gallon drums until the time that it was combined with other components in the base and subbase mixtures.

Because of a materials handling and storage problem, prehydration of the AFBC residue at the site prior to its use was not feasible. Questions then arose to the effectiveness of preconditioning the AFBC residue in advance and storing it. Since similar methods had been employed during the laboratory evaluations, it was thought that storing the preconditioned AFBC residue in a warehouse would be acceptable. To check the condition of the AFBC residue KTC personnel visited the storage facility in March, 1988 and obtained samples for evaluation. Tests at that time indicated that the AFBC residue that apparently had been preconditioned and stored for several months prior to the visit by KTC personnel was reactive; that is, the material would generate heat when water was added to it. The AFBC residue had evidently regained some of its initial properties while being stored in the warehouse or was not properly prehydrated initially. Whatever the reason, results of impromptu tests performed on the residue prior to its use in construction indicated that some expansion of the mixtures containing the residue would occur. A decision was made to use the AFBC residue in its existing condition, as opposed to prehydrating the residue again, and to form gaps in the plastic base and subbase layers of sufficient width in an effort to accommodate the one or two percent expansion that was expected to occur.

Mixture Designs

Proportions for the optimum AFBC concrete base mixture had been developed previously by Dr. Jerry G. Rose, Professor of Civil Engineering, University of Kentucky, and were modified only slightly for this project. The optimum AFBC concrete base mixture evaluated in the laboratory initially contained 56 percent No. 57 limestone aggregate, 35 percent AFBC residue and nine percent ponded fly ash. At the request of TVA officials, a Class F fly ash was substituted for the ponded fly ash and the mixture proportions were adjusted based upon previous research conducted by Dr. Rose [4,5]. The altered mixture design for the AFBC concrete base was comprised of 64 percent No. 57 limestone aggregate, 25 percent AFBC residue, and 11 percent Class F fly ash. Unfortunately, there was not enough time to determine the 28-day compressive strength of the AFBC concrete base mixture prior to the commencement of construction activities. Laboratory evaluations could only assess the moisture-density relationship of the amended mixture. The revised AFBC concrete mixture had an optimum moisture content of 8.8 percent and a maximum dry density of 129.6 pcf.

The design of the ponded fly ash-hydrated lime-dense grade aggregate (pozzolanic) base mixture was based upon 28-day compressive strength development. Three mixtures were evaluated that contained various proportions of ponded fly ash, hydrated lime, and DGA. The amount of ponded fly ash varied from six to 11 percent. Hydrated lime in the mixtures varied from three to five percent and the amount of DGA varied from 84 to 91 percent. The optimum mixture design for the pozzolanic base contained 11 percent ponded fly ash, five percent hydrated lime, and 84 percent DGA. The pozzolanic mixture used had an optimum moisture content of 9.6 percent and a maximum dry density of 130.6 pcf. The 28-day compressive strength of the pozzolanic mixture was 780 psi.

Similar analyses were performed to develop the optimum design for the AFBC stabilized pond ash subbase mixture. Six separate mixtures were evaluated in the laboratory that contained various proportions of pond ash and AFBC residue. The pond ash was designated as coarse fractions (bottom ash) and fine fractions (fly ash). The amount of ponded fly ash in the mixtures was varied from five to nine percent. Ponded bottom ash varied from 46 to 90 percent of the total mixture. The amount of AFBC residue in the mixes varied from five to 45 percent. The mixture chosen for use as the subbase mixture exhibited uniform consistency and compressive strength development during the laboratory evaluations. The optimum AFBC stabilized pond ash mixture consisted of 60 percent ponded bottom ash, 32 percent AFBC residue, and eight percent ponded fly ash. The AFBC stabilized pond ash mixture had an optimum moisture content of 16.1 percent and a maximum dry density of 101.8 pcf. The 28-day compressive strength of the AFBC stabilized pond ash mixture was 960 psi.

Pavement Thickness Designs

Kentucky flexible pavement design procedures were used to determine thickness requirements of the AFBC concrete base, pozzolanic base, and AFBC stabilized pond ash subbase layers. AASHTO structural coefficients of 0.30, 0.28 and 0.10 were assumed for the AFBC concrete base layer, pozzolanic base layer, and AFBC stabilized pond ash subbase layer, respectively. The thickness design requirements were 8.0 inches, 8.0 inches, and 12.0 inches for the experimental AFBC concrete base layer, pozzolanic base layer, and AFBC stabilized pond ash subbase layer, respectively. The pavement design for the experimental AFBC stabilized pond ash subbase section included a bituminous curing seal above the subbase layer and a crushed aggregate base thickness of 8.0 inches. The pavement design of the experimental sections also specified 2.0 inches of compacted bituminous concrete base, 1.5 inches compacted bituminous binder, and 1.0 inch compacted bituminous concrete surface. Additionally, the pavement design for the AFBC concrete base section and pozzolanic section included a stress relief layer to minimize the occurrence of reflective cracking.

Construction

Prior to construction of the experimental sections, KTC personnel performed in-situ bearing capacity tests and Road Rater deflection tests on the prepared subgrade. During construction, KTC personnel monitored construction activities and prepared field compacted specimens for laboratory evaluations. Department of Highways' personnel were responsible for determining conformance to the standard construction specifications and special notes for the experimental project. The experimental base and subbase layers were placed using conventional equipment, typically an aggregate spreader box pushed by a small bulldozer. Difficulties documented during construction of the experimental base and subbase layers included consistency and moisture content of the mixtures and production of the materials at the concrete batch plant that was used to produce the mixtures. The lack of a uniform moisture content in materials delivered to the jobsite caused some delay in compacting the base and subbase materials and cutting the materials to proper grade. A compaction requirement of no less than 100 percent of the laboratory dry density was specified for the experimental base and subbase layers. Satisfactory densities were easily achieved during construction. Four-foot wide gaps were formed in the plastic AFBC concrete base material to accommodate the expected expansion. Gaps were also formed in the AFBC stabilized subbase layer, but not until two to three days after placement of the bituminous curing seal. Because the mixture had firmly set, formation of the gaps was quite difficult. The stress relief layer on the AFBC concrete base and pozzolanic base and the bituminous curing seal on the subbase was required to be placed no later than 24 hours after completion of compaction activities. The allotted time to place the stress relief layer and the curing seal was apparently disregarded because the contractor generally placed them immediately upon completion of compaction. This action resulted in the formation of several deep ruts in the still-plastic base and subbase layers.

Production of the AFBC concrete base mixture was satisfactory but less than ideal. Two days were required to complete placement of the 750-foot length of the AFBC concrete base. Because the AFBC concrete base mixture was placed in two lifts, the occurrence of delamination between lifts was thought possible. Production of the pozzolanic mixture and AFBC stabilized pond ash mixture was very sporadic and three days were required to complete placement of each section. Production was delayed due to various problems.

Failure of a motor on one conveyor belt caused a six-hour delay on the first day of production of the pozzolanic mixture. Also, delays were encountered because the batch plant operators continued producing concrete between batches of the base and subbase mixtures. The pozzolanic base mixture was placed in one lift to eliminate the possibility of delamination. However, because the AFBC stabilized pond ash subbase mixture was placed in two, six-inch lifts, there was a high probability that delamination would occur. Because of the delays at the concrete batch plant, it was concluded that a better set-up for production of the mixtures would have been a pugmill set up near the jobsite. Waste materials utilized constructing the three sections totaled approximately 1,138 tons. Approximately 460 tons of waste AFBC residue and 678 tons of pond ash were utilized constructing the three experimental sections incorporating AFBC residue and pond ash in the pavement structure.

Post Construction Laboratory and Field Evaluations

Despite the observed construction difficulties, the initial effectiveness of the experimental base and subbase layers appeared favorable. Compressive strength evaluations of field compacted six-inch by 12-inch specimens indicated average strengths of 1,465 psi at seven days for the AFBC concrete base mixture. The compressive strength averaged 4,075 psi at 112 days. The 112-day strengths are comparable to a typical five bag per cubic yard concrete mix. Static chord elastic moduli values were lower than typical concrete. The static chord modulus of elasticity averaged 2.20 million psi at seven days and increased to 3.65 million at 112 days. There were no strength data obtained during the laboratory phase of the study from laboratory compacted specimens incorporating the Class F fly ash with which to compare the field data. The attempt to simulate proper compactive effort while preparing the field compacted specimens was successful. The wet densities of the field compacted AFBC concrete base specimens averaged 146.2 pcf, or 103.7 percent of the maximum density determined in the laboratory.

Average seven-day compressive strengths of the pozzolanic base mixture were only 65 psi. The seven-day specimens were so weak that no data were obtained for the static chord modulus of elasticity for fear of damaging the compressometer and dial indicators. After 14 days, the compressive strength of the field compacted specimens of the pozzolanic mixture had increased to 120 psi. The average 14-day static chord modulus of elasticity was 64,000 psi. The 28-day average compressive strength increased to 265 psi. The static chord elastic modulus increased almost tenfold over the 14-day modulus to 573,000 psi. Average values for compressive strength and static chord elastic modulus, at 56 and 112 days, had increased to 645 psi and 1,497,000 psi and 1,600 psi and 2,830,000 psi, respectively. Although an attempt was made to simulate proper compactive effort while preparing the field compacted specimens of the pozzolanic mixture, the average of the wet densities of the field compacted base specimens was 140.7 pcf, or 98.0 percent of the laboratory maximum wet density. While the average wet density of the field compacted specimens may appear close to the laboratory value, the wet densities of the field specimens ranged from 129.6 pcf, or about 90.5 percent of the maximum wet density to 147.9 pcf, or 103.4 percent of the laboratory maximum wet density. Of the 39 specimens prepared in the field, 19 equaled or exceeded the laboratory maximum wet density of 143.1 pcf.

Compressive strength evaluations of AFBC stabilized pond ash specimens indicated average strengths of 375 psi at seven days and increasing significantly to 2,345 psi at 112 days. These strengths were only slightly greater than the compressive strengths obtained for laboratory compacted specimens during the laboratory phase of the study. Static chord modulus of elasticity values of field compacted specimens averaged 0.40 million psi at seven days and increased to 1.55 million psi at 112 days. The attempt to simulate proper compactive effort while preparing the field compacted specimens of the AFBC stabilized pond ash mixture was not successful. The wet densities of the field compacted subbase specimens averaged 114.7 pcf, or 97.0 percent of the maximum wet density. None of the 42 compacted specimens of the subbase mixture equaled or exceeded the maximum wet density.

The base and subbase layers containing AFBC residue were optically monitored periodically for length change prior to the placement of the asphaltic concrete layers. Field compacted specimens of the AFBC concrete base mixture and the AFBC stabilized pond ash subbase mixture also were monitored in the laboratory for expansive characteristics. The magnitude of the expansion of the experimental mixtures was less in the field than that observed in the laboratory. The field expansion of the AFBC concrete base equaled 0.20 percent after 58 days. Expansion of field compacted AFBC concrete base specimens, cured in the laboratory, averaged 0.36 percent after 51 days and 0.59 percent after 112 days. Field expansion of the AFBC stabilized pond ash subbase also averaged 0.20 percent, but after only 34 days of monitoring. Compacted specimens of the AFBC stabilized pond ash, cured in the laboratory, expanded 0.43 percent after 24 days and averaged 0.62 percent expansion after 108 days.

Structural attributes were evaluated by obtaining deflection measurements on the experimental layers with a Model 400B Road Rater. Deflection measurements were obtained at various stages of construction. Deflection measurements were obtained on the compacted subgrade immediately before placement of the experimental materials and at various times after placement of the experimental AFBC concrete base, pozzolanic base, and AFBC stabilized pond ash subbase mixtures. Analysis of the deflection measurements generally indicated a significant increase in the overall stiffness of the pavement structure due to the addition of the experimental layers. Analysis of deflection tests conducted over an 82-day period on the experimental AFBC concrete base layer indicated the dynamic stiffness of the experimental layer appeared to peak after 14 days. There were some variations in the deflections after 14 days but the overall trend of the dynamic stiffness was to decrease. The variations were attributed to either temperature changes within the base and subgrade layers or changing moisture conditions within the subgrade. After 82 days, the average dynamic stiffness of the pavement structure had decreased 34 percent below the peak dynamic stiffness at 14 days. The results of compressive strength and static chord modulus of elasticity tests of field compacted specimens did not show a substantial decrease in strength. In fact, field compacted specimens of the AFBC concrete base mixture continued to gain strength throughout the 112-day laboratory evaluation period.

Deflection tests were conducted over a 68-day period on the experimental pozzolanic base layer. Analyses of this data indicated higher dynamic stiffnesses through 42 days. After 42 days, the rate of the strength gain appeared to decrease. There were some variations in the deflections after 42 days but the overall trend of the dynamic stiffness was to increase. These variations were attributed to either temperature changes within the pozzolanic layer or changing moisture conditions within the subgrade. The results of compressive strength and static chord modulus of elasticity tests of field compacted specimens of the pozzolanic base mixture also indicated a substantial increase in strength. Field compacted specimens of the pozzolanic base mixture continued to gain strength throughout the 112-day laboratory evaluation period.

Deflection tests were performed up through 45 days after final placement of the AFBC stabilized pond ash subbase mixture. Again, the experimental subbase layer substantially increased the dynamic stiffness of the pavement structure. The subgrade stiffness was estimated to be 230,000 pounds-force per inch just prior to the placement of the experimental subbase material. The subbase layer was tested after a seven-day curing period. The pavement structure had a dynamic stiffness of about 730,000 pounds-force per inch at that time. Deflections after 21 days were higher than the seven-day deflections, indicating a less rigid structure. However, deflection readings taken 35 and 45 days after placement indicated increasing dynamic stiffnesses. This indicates that the experimental AFBC stabilized pond ash subbase material continued to gain strength during the 45-day evaluation period. Laboratory strength tests also indicated continued strength gain throughout the 112-day laboratory evaluation period. Still, it must be cautioned that the apparent increase in the overall dynamic stiffness of the pavement structure could be as much the result of temperature changes within the experimental pozzolanic base and AFBC stabilized pond ash subbase layers, or changing moisture conditions within the subgrade as it could be an actual strengthening over time of the experimental layers.

Interim Conclusions

Previous research reported by others concluded that prehydrated AFBC residue, pulverized coal fly ash, and aggregate could be used to construct a stabilized base course, provided the AFBC residue was properly prehydrated prior to its use [6]. The AFBC residue used in this study was effectively prehydrated during the laboratory phase of the study. Mixtures containing AFBC residue that was prehydrated in the laboratory did not exhibit any expansive characteristics during the subsequent evaluations. However, that success was not duplicated during the field trial. Specimens made in the laboratory, just prior to construction of the experimental base and subbase layers, containing AFBC residue that was prehydrated at the concrete batch plant and stored for several months prior to the commencement of construction, revealed expansive characteristics of the mixtures. It was thought that either the AFBC residue was not properly prehydrated at the batch plant or the extended storage period significantly affected the properties of the residue. According to Dr. John Minnick, "the longer the storage period, the more detrimental effect carbonation is expected to have on the quality of the AFBC residue," [7]. Air from the atmosphere most likely reacted with the hydroxides in the AFBC residue and converted them to carbonates. This action had a detrimental result upon the mixtures containing AFBC residue.

Although tests performed on the AFBC residue that was prehydrated at the batch plant indicated a hydration reaction (temperature rise caused by the addition of water), construction of the experimental sections proceeded as planned. An attempt was made to accommodate the anticipated material expansion by forming gaps in the plastic base and subbase materials. Construction of the experimental layers using conventional equipment was generally acceptable when materials were available. Materials with the proper moisture content were placed and compacted with no apparent difficulties. The only readily apparent construction difficulty was cutting the materials to grade and the impatience displayed by the construction contractor in placing the stress relief layers and the bituminous curing seal. Production of the experimental mixtures was hindered because concrete batching operations were alternated with production of the experimental mixtures. Producing the mixture with a pugmill set up near the jobsite would have been preferable to the batch plant operation. This is true also of the AFBC prehydration process. It would have been more appropriate to prehydrate the AFBC residue just one or two days prior to mixing it with the other materials in the experimental mixtures.

Field preparation of specimens for compressive strength and elastic modulus determinations using modified procedures was moderately successful. Successful compaction of the AFBC concrete base mixture in the 6-inch by 12-inch molds was achieved. However, that was not the case with either the pozzolanic base mixture or the AFBC stabilized pond ash mixture. Compressive strength and static chord elastic moduli evaluations generally indicated low initial strengths but exceptional strength gain for all of the experimental mixtures. It appeared that both of the mixtures incorporating the AFBC residue possessed the potential for further expansion in the field trial application based upon expansions of field compacted specimens observed in the laboratory. The initial performance of the pozzolanic base was superior to the other experimental base and subbase layers that contained residue from the AFBC process.

It was uncertain as to the cause for the observed decreases in the apparent dynamic stiffnesses of the pavement structure of the AFBC concrete base section but was hypothesized to be the result of the continued expansion and fairly extensive cracking of the experimental base layer. Because the AFBC stabilized pond ash subbase contained 32 percent of the AFBC residue by weight, similar actions (a decrease in dynamic stiffness) from that section were expected as time passed. Deflection measurements obtained on the pozzolanic base indicated continued gains in the dynamic stiffness of the structure although the rate of gain decreased with the passage of time.

PERFORMANCE EVALUATIONS

Design and construction procedures for the use of an AFBC concrete base, pozzolanic base (ponded fly ash-hydrated lime-dense grade aggregate), and AFBC stabilized pond ash subbase have been documented in previous reports [1,2]. Also included in those reports were preliminary performance evaluations, principally conducted up through the time at which the asphaltic concrete base and binder layers were placed in August 1988.

The bituminous surface course was not placed at the same time that other bituminous layers were placed. Evaluations performed after placement of the base and binder courses are contained herein. These evaluations included periodic visual surveys, periodic detailed condition and distress surveys, obtaining pavement rutting measurements, obtaining cores of the experimental base and subbase layers for laboratory evaluations, performing in-situ tests of the underlying soil subgrade layer, and assessing the structural characteristics of the experimental layers by conducting periodic deflection tests.

The bituminous base and binder courses were placed beginning on August 15, 1988. Placement was completed on August 19, nearly two months after completion of the experimental base and subbase construction. The gaps formed in the plastic AFBC concrete material were backfilled with bituminous base materials. Gaps formed in the AFBC stabilized pond ash section were backfilled with dense graded aggregate. Humps appeared on the roadway surface above the expansion gaps shortly after the bituminous layers were placed. KTC personnel returned to the experimental site in October 1988 to observe the general condition of pavement surface in the experimental and control sections and to obtain elevation profiles of the pavement surface. Bench marks were established by KTC personnel and surface elevations were obtained. Unfortunately, continued monitoring of the pavement surface elevation was not practical because the expansion of the experimental layers was so instantaneous that milling of the humped areas was required to prevent the humped areas from being hazards to the traveling public.

Discernible humps were observed on the pavement surface during the October visit. In some areas, the humps were manifested as apparent waves in the pavement surface. Figure 1 is a photograph taken transversely across the pavement at STA 60+00, at the beginning of the AFBC stabilized pond ash section. KTC personnel also observed cracks in the bituminous layers above the experimental AFBC concrete base mixture. Figure 2 shows the observed cracking in a photograph of the bituminous binder taken near STA 51+55. Figure 3 depicts the surface of the AFBC concrete base material in the same general area shown in Figure 2. Note the deep wheel tracks made by the asphalt spreader truck. It is possible that the damage to the experimental base material caused by the asphalt spreader truck resulted in the reflective cracking seen in Figure 2.

KTC personnel returned to the experimental site in May 1989 to perform Road Rater deflection testing one year after construction. A member of the crew documented the

Figure 1. *Expansion of the experimental AFBC stabilized pond ash mixture resulted in the formation of humps on the pavement surface.*

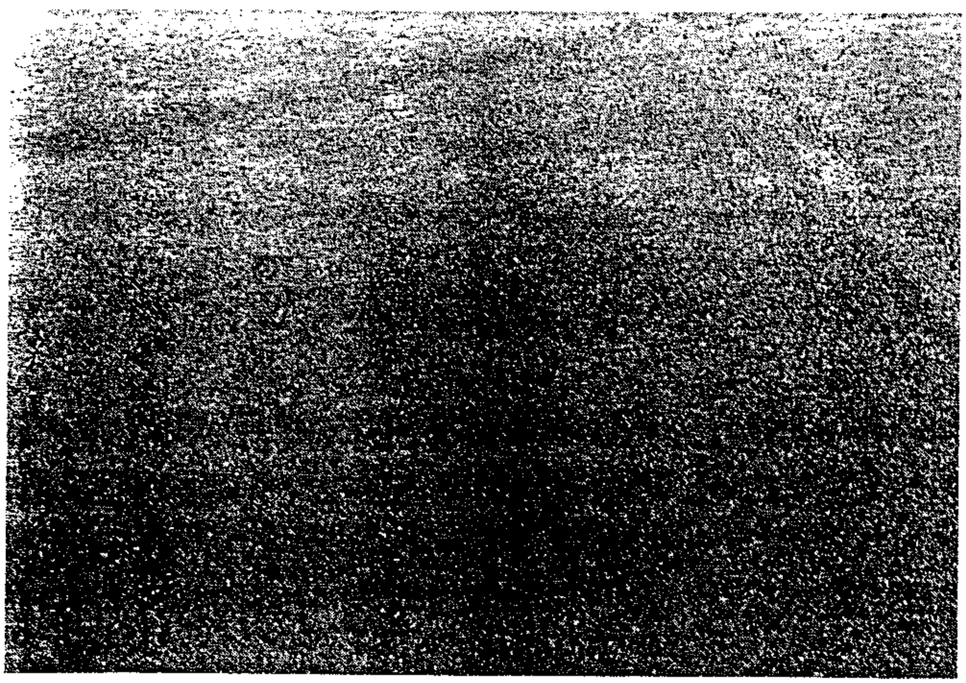


Figure 2. *Cracking of the experimental AFBC concrete base layer reflected through to the surface of the bituminous layer.*

Figure 3. *A view of the westbound lane of the experimental AFBC concrete base layer near STA 51+50 shortly after construction.*

milled areas on the pavement surface at the locations where the expansion gaps were formed. Apparently, the milling activity caused formation of potholes, especially in the AFBC stabilized pond ash section. In this section, the pavement milling had effectively eliminated the thickness of the bituminous material above the expansion gaps leaving the underlying aggregate exposed. Figures 4 and 5 illustrate the magnitude of the milling and spot patching activities at STA 45+00 and STA 67+50, respectively.

Visual Distress and Pavement Rutting Surveys

Detailed distress surveys were conducted by KTC personnel throughout the evaluation period. The initial detailed distress survey was conducted in October 1989, just over one year after the bituminous base and binder layers had been placed. Subsequent detailed surveys were conducted in February, 1990 and again in July, 1990, approximately two years after placement of the asphaltic concrete layers. The detailed survey was generally comprised of mapping observed pavement distresses and obtaining rutting measurements on the pavement surface at 50-foot intervals. The strip maps used to document observed

Figure 4. *Photograph of pavement surface at STA 45+00 taken during May 1989 investigation illustrates milling of the pavement over the gap made to accommodate expansion.*

Figure 5. *Photograph taken near STA 67+50 in May 1989 shows spot patching that was required after pavement milling activities.*

pavement distresses are reproduced in Appendix A. Information relative to the pavement rutting characteristics is presented in Appendix B.

During the initial detailed survey, it was observed that areas where the gaps had been placed to accommodate expected expansion in the two sections wherein AFBC residue had been incorporated in the mixtures had been milled and overlaid. Figure 6 depicts the overlay observed at STA 50+00. The experimental AFBC concrete base section was exhibiting significant reflective cracking of the bituminous pavement layers. A strip chart was created to identify and document progression of the pavement distresses. Nearly 120 linear feet of longitudinal centerline cracking was observed. Figure 7 shows longitudinal centerline cracking of the bituminous pavement near STA 45+50. Figure 8 is a photograph of this same general area and shows longitudinal centerline cracking on the surface of the experimental AFBC concrete base before placement of the bituminous layers. There also were 13 linear feet of random longitudinal cracking and 16 linear feet of random transverse cracking observed during the initial survey. Cracking was not significant in the remaining two experimental sections. Only a slight edge failure was observed in the pozzolanic base section. No cracking was observed in either the AFBC stabilized pond ash subbase section or the control section. Initial rutting measurements were obtained for each section. Average rutting depths ranged from 1/8 inch to 1/4 inch. The pozzolanic base section exhibited the least severe rutting when compared to the other experimental sections and the control section.

Figure 6. *The pavement near STA 50+00 had been milled and a full-width bituminous patch had been placed.*

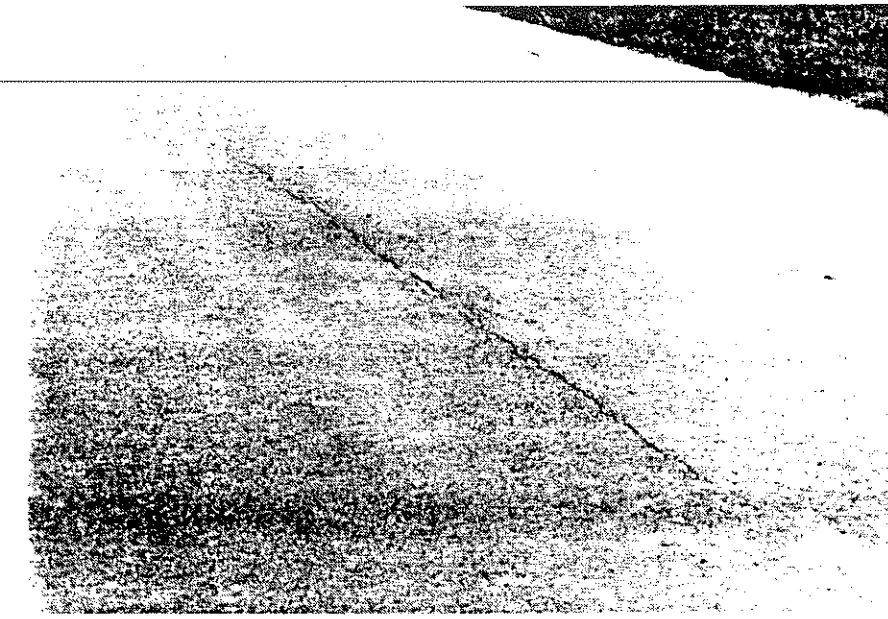


Figure 7. *Longitudinal reflective cracking observed during October 1989 survey on the centerline of the roadway near STA 45+50.*

Figure 8. *Longitudinal centerline cracking of the AFBC concrete base near STA 45+50 approximately two weeks after final compaction.*

Additional cracking of the asphaltic concrete was observed during the second visual distress survey conducted in February 1990. Also, areas observed to be milled and patched full width in October had once again been milled and in some places spot patching was required. Approximately 120 linear feet of additional longitudinal centerline cracking, 31 feet of random longitudinal cracking and 10 feet of random transverse cracking were documented within the AFBC concrete section during the survey. A fine hairline longitudinal crack about the centerline near STA 58+30 was observed in the pozzolanic base section. A full-width transverse crack was located at STA 58+67 (see Figure 9). Two very small (about one foot in length) longitudinal cracks were documented in the AFBC stabilized pond ash section.

There also was an area within the AFBC stabilized pond ash subbase section that experienced an incident comparable to a blowup in a cement concrete pavement. The bituminous pavement around the unexpected blowup had been milled and pot holes, resulting from the milling activity, had been patched. Figure 10 illustrates the blowup area near STA 63+85. The gaps placed by the contractor were much too narrow to accommodate the expansion of the mixture, being only about 18 inches to two feet in width as opposed to the suggested width of four feet. The narrow gaps probably contributed to the unexpected distress. Pavement rutting depths had increased slightly when compared to the October measurements. Average rutting depths ranged from 1/8 inch to 5/16 inch. Rutting depths were greatest in the AFBC stabilized pond ash section. The AFBC concrete section and control section exhibited similar overall rutting depths.

The last detailed visual survey was performed in July 1990, nearly two years after the pavement had been placed. Significant reflective cracking was observed in the AFBC concrete base section. An additional 122 feet of longitudinal centerline cracking were documented. These cracks were continuances of previously documented cracks. There were about 35 additional linear feet of random longitudinal cracking and six linear feet of added random transverse reflective cracking in the AFBC concrete base section since the February survey. In the pozzolanic base section, a longitudinal crack about 16 feet in length had developed. This crack was an extension of the longitudinal crack observed in February. The pavement within the AFBC stabilized pond ash section had begun to exhibit some minor cracking. There were three feet of longitudinal centerline cracking and an additional 31 feet of random longitudinal cracking. Typically, the longitudinal cracks were thin hairline cracks. No further distresses were noted. Individual pavement rut depths were slightly greater than those obtained during the February survey. However, average rutting depths still ranged from about 1/8 inch to 5/16 inch. Individual rutting depths remained greatest in the AFBC stabilized pond ash section. Some

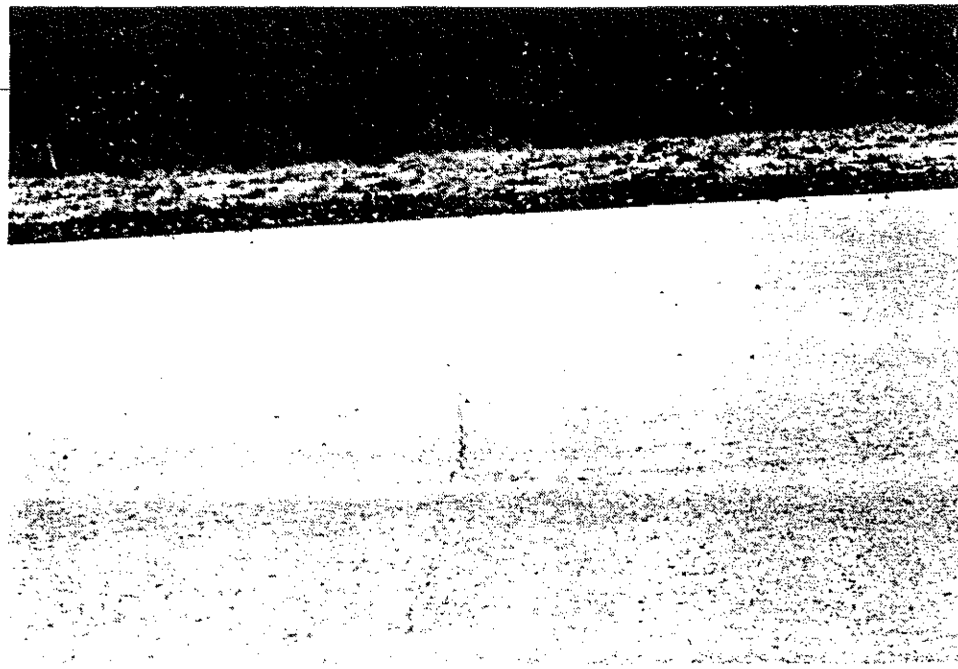


Figure 9. *A full-width transverse crack was observed near STA 58+67 in the experimental pozzolanic base section during the February 1990 survey.*

rutting depths were as great as 3/4 inch. The AFBC concrete section and control section exhibited similar overall rutting depths. The pozzolanic base section continued to exhibit less rutting than did the experimental and control sections. Large areas within the AFBC concrete base section and AFBC stabilized pond ash subbase section, particularly near the expansion gaps, were overlaid in July 1990 sometime after the distress survey was conducted. These sections were completely overlaid in July 1991.

Pavement rutting measurements were obtained in all sections during October 1990 when additional coring activities were conducted. During the August 1991 investigation, rutting measurements were obtained in the pozzolanic and control sections only. The rutting measurements obtained by KTC personnel during October 1990 indicated some variability in the data. The October 1990 data indicated a general increase in pavement rutting depths for the control section and pozzolanic base section. However, decreases in the average rut depths were apparent for the remaining two experimental sections. Close inspection of the data revealed that a number of the rutting measurements had been obtained in the general areas where the bituminous overlays were placed. During the August 1991 visit, pavement rutting depths were not obtained within the two sections which contained the AFBC residue because both sections had been overlaid throughout.

Figures 10, 11 and 12 graphically illustrate the rutting characteristics of the experimental sections compared to the control section as a function of time. As illustrated in the following figures, the experimental pozzolanic base section exhibited the best performance relative to pavement rutting.

By the summer of 1991, the two sections containing AFBC residue were exhibiting extreme signs of distress, particularly the AFBC concrete section. Officials with KDOH determined that the two sections should be replaced with conventional materials. Performance monitoring activities at the experimental sites continued through August. In September, KTC personnel witnessed removal of the experimental AFBC concrete and AFBC stabilized pond ash sections. Removal of the experimental materials was accomplished using a track-mounted Caterpillar 225BLC excavator. The materials were hauled to a spoil area near the jobsite. The AFBC stabilized pond ash section was rehabilitated first. The asphaltic concrete and DGA layers were removed and transported to the spoil pile. The removal of the AFBC stabilized pond ash mixture was moderately difficult as the materials were quite solid. Great effort was applied by the excavator to downsize the slabs of the AFBC stabilized pond ash mixture so they could be placed into the waiting dump trucks. The experimental subbase layer exhibited good integrity but was delaminated between the constructed lifts essentially throughout the entire section. The pavement structure was replaced with a six-inch layer of No. 3 gradation stone, 16-1/2 inches DGA, 5-1/2 inches bituminous base, 1-1/2 inches bituminous binder and a one-inch bituminous surface course.

Removal of the AFBC concrete base layer was entirely different from the AFBC stabilized pond ash subbase layer in terms of the integrity of the material removed. There were some areas where the base layer had basically disintegrated into just coarse aggregate particles. These areas were primarily located in the westbound lanes around Station 51+00. The geometry of the section at this location did not allow for drainage of any water. The trapped water undoubtedly had a detrimental affect on the experimental base layer. After removing the experimental base layer, several trenches had to be cut into the shoulder in an effort to bleed the ponded water off of the subgrade. In other areas, where water could migrate through the soil and turf shoulder materials, the base layer appeared to be more sound. Some areas of the experimental base layer exhibited delamination between lifts. In other areas, the two lifts of base material appeared to be well bonded. However, there was extensive cracking observed on the surface of the base layer throughout the section. After the pavement structure within the AFBC concrete base section was removed, the contractor replaced the structure with a six-inch layer of No. 3 gradation stone, 4-1/2 inches DGA, 5-1/2 inches bituminous base, 1-1/2 inches bituminous binder and a one-inch bituminous surface course.

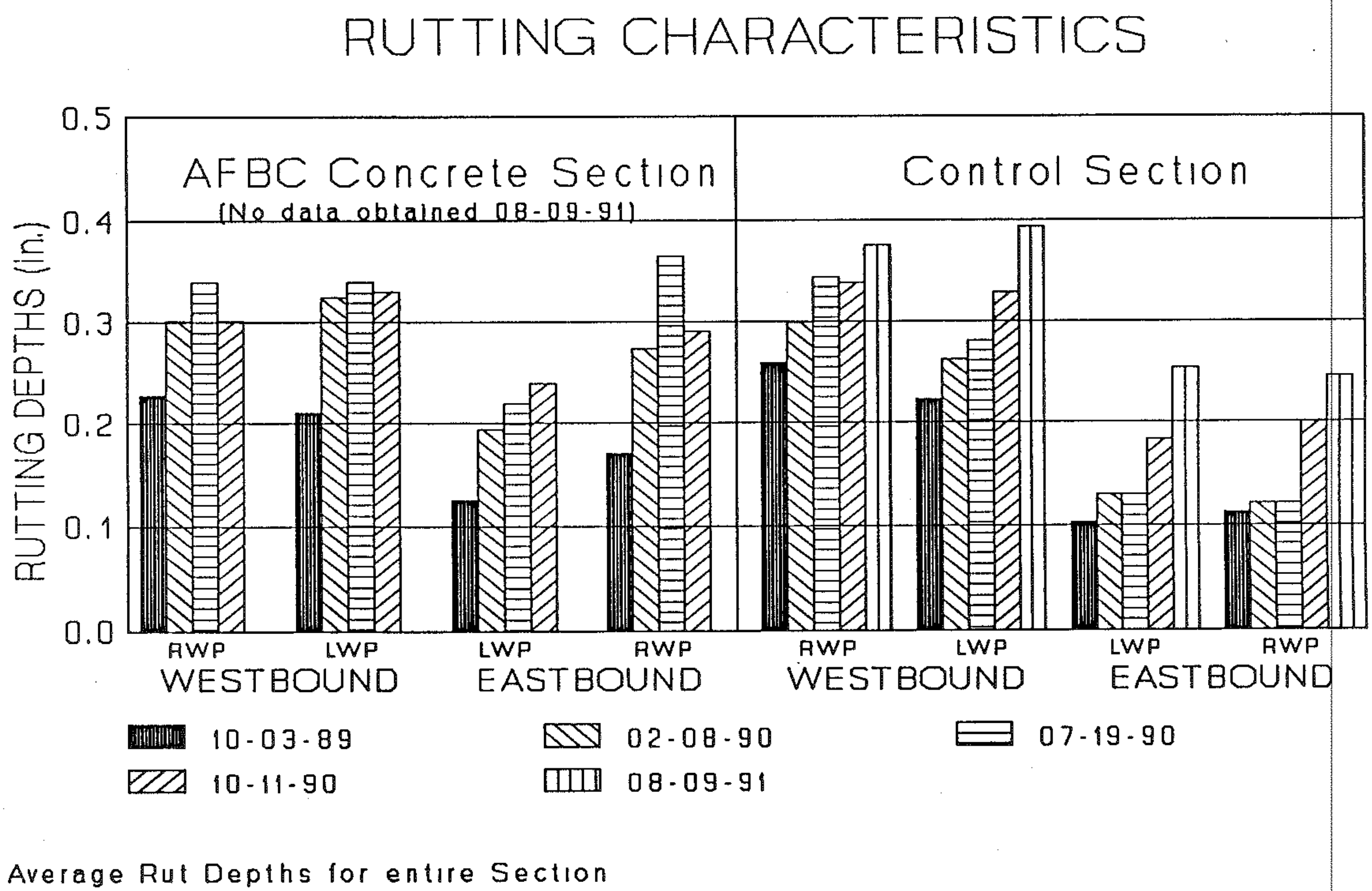


Figure 10. *Rutting characteristics of the experimental AFBC concrete base section and the control section as a function of time.*

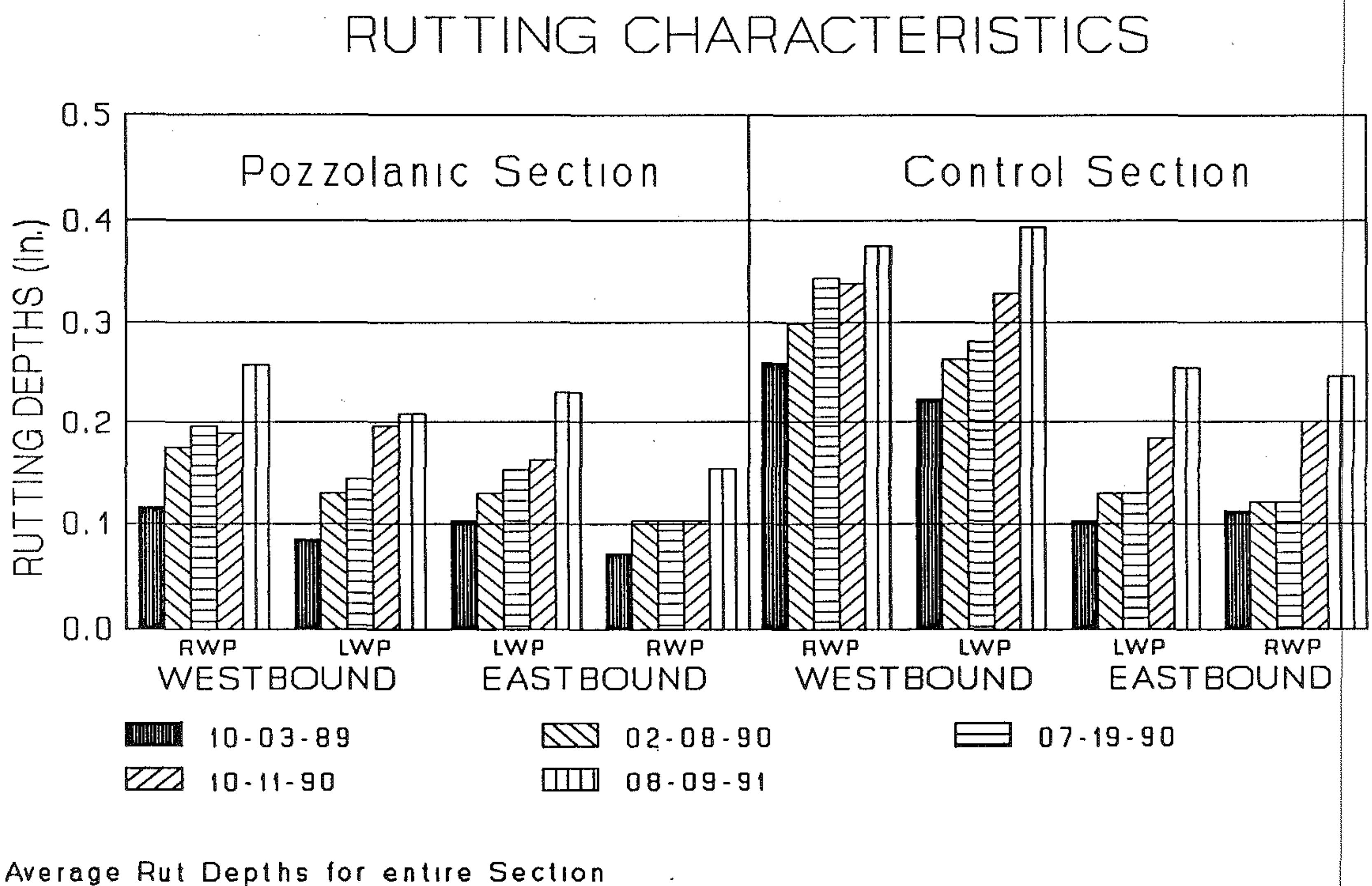
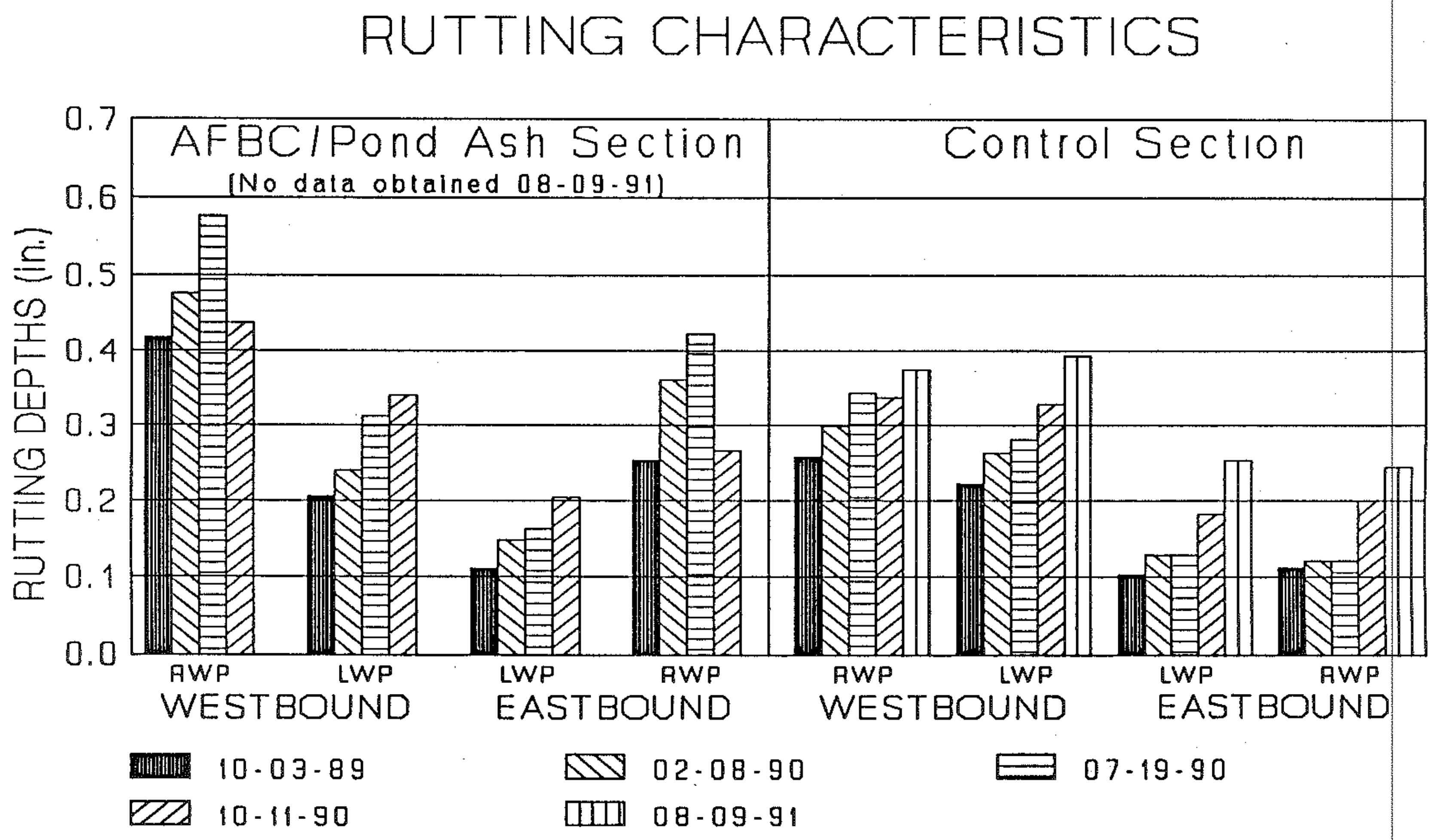


Figure 11. *Rutting characteristics of the experimental pozzolanic base section and the control section as a function of time.*



Average Rut Depths for entire Section

Figure 12. *Rutting characteristics of the experimental AFBC stabilized pond ash subbase section and the control section as a function of time.*

Results of Physical Tests Performed on Field Core Specimens

Field core specimens were obtained at various times after construction of the experimental pavement base and subbase layers and subjected to assorted laboratory tests including compression tests for strength, static chord modulus of elasticity determinations, and freeze and thaw durability tests. Densities of the field core specimens were also determined. Field cores of the experimental base and subbase layers were obtained and tested in general accordance with ASTM C-42, "Obtaining and Testing Drilled Cores and Sawed Beams of Concrete." When possible, field cores were evaluated for modulus of elasticity in general accordance with ASTM C 469, "Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." The static chord modulus test requires an eight-inch sample and the use of a compressometer device. Examinations of the freeze and thaw durability of the field cores were performed using equipment of the type specified in the standard test method for freeze and thaw testing of concrete, ASTM C-666, "Resistance of Concrete to Rapid Freezing and Thawing." The field cores were only monitored for weight loss or gain during the durability test and not for changes in the sonic modulus. Initial coring activities were conducted in July 1988, prior to placement of the bituminous layers. Subsequent field cores were obtained during the years 1989, 1990 and 1991.

Core specimens were obtained in July 1988 from within each experimental section at ages of 67, 51 and 44 days for the AFBC concrete base materials, pozzolanic base materials, and the AFBC stabilized pond ash subbase materials, respectively. The first four-inch core obtained in the AFBC concrete section, at Station 45+50 in the eastbound lane, revealed delamination between the two, four-inch lifts of the base material. During construction of the experimental base, equipment malfunction dictated the second lift of the AFBC concrete base be placed on the following day. Although the contractor wetted the surface of the first course prior to placing the second course, some delamination between lifts was expected. Five other cores obtained from the AFBC concrete base were found to be intact. Recorded core lengths ranged from 7-1/4 inches to eight inches. The designed thickness was eight inches. One specimen contained a large pop-out on the side and was not evaluated. Compressive strengths of the four specimens that were evaluated ranged from 2,200 psi to 4,070 psi and averaged 3,310 psi. There were no modulus tests or durability tests performed on the field specimens. The average density of the four specimens was 141.5 pcf. During construction, the designed wet density of the mixture was 141.0 pcf. Three, four-inch cores were obtained from the pozzolanic base section. Recorded lengths of the cored specimens ranged from eight to 8-1/2 inches. The designed thickness was eight inches. Compressive strengths of these specimens ranged from 1,250 psi to 1,790 psi and averaged 1,540 psi. Modulus tests and durability tests were not performed on the field specimens. The average density of the three field cores was 144.3 pcf. The designed wet density of the pozzolanic base mixture was 143.1 pcf during construction. Three six-inch cores were obtained from the AFBC stabilized pond ash subbase section. Each core location revealed delamination between the two, six-inch lifts. Two-inch core specimens were obtained from the six-inch cores in order to evaluate the compressive strength of the material. A total of eight specimens were evaluated for compressive strength. The compressive strengths of these field specimens ranged from 1,910 psi to 3,860 psi and averaged 2,370 psi. Modulus tests and durability tests were not performed on the AFBC stabilized pond ash specimens. The average density of the eight core specimens was 114.5 pcf. Densities of the field specimens ranged from 111.7 pcf to 116.2 pcf. The designed wet density of the AFBC stabilized pond ash mixture was 118.2 pcf during construction.

A second series of core specimens was obtained from within each experimental section in October 1989. The respective ages of the experimental layers were 525, 509 and 502 days for the AFBC concrete base materials, pozzolanic base materials, and the AFBC stabilized pond ash subbase materials. The first specimen obtained in the AFBC concrete section, at Station 45+75 in the eastbound lane, was delaminated between the two lifts. Five additional cores obtained from the AFBC concrete base were found to be intact. Recorded core lengths of these specimens ranged from about seven to ten inches. The two specimens from core location STA 45+75 were evaluated for freeze/thaw durability over a period of 15 cycles. The average weight gain of the two specimens was 1.9 percent. The specimen from core location 49+50 was not evaluated. The four remaining specimen had an average compressive strengths of 3,480 psi, ranging from 3,080 psi to 3,840 psi. Modulus tests were performed on two of the core specimens. The static chord elastic modulus of the two specimens was 2.25 million psi and 1.95 million psi. The average density of the four specimens was 142.9 pcf. Six core specimens were obtained from the pozzolanic base section. Recorded lengths of the these specimens ranged from about seven to nine inches. Two specimens from core locations STA 53+50 and STA 54+00 were evaluated for freeze/thaw durability over a period of 15 cycles. The average weight gain of the two specimens was 0.5 percent. Compressive strengths of the four remaining specimens ranged from 1,140 psi to 2,380 psi and averaged 1,655 psi. Modulus tests could not be performed on the field specimens because the specimens were too short to test. The average density of the six field cores was 142.6 pcf. Six, four-inch cores were obtained from the AFBC stabilized pond ash subbase section. Delamination between the two constructed lifts was observed at all but one core location. Four of the field specimens were evaluated for freeze/thaw durability over a period of 15 cycles. The average weight gain of the specimens was 1.0 percent. A total of five specimens were evaluated for compressive strength. The compressive strengths of these specimens were very high and ranged from 3,060 psi to 4,190 psi and averaged 3,780 psi. The static chord elastic modulus test was performed on one field specimen. The static chord modulus for this specimen was 1.30 million psi. Densities of the AFBC stabilized pond ash specimens ranged from 109.6 pcf to 118.9 pcf and averaged 113.6 pcf.

Researchers returned to the experimental site in July 1990 to assess the condition of the two sections wherein residue from the AFBC process was incorporated into the experimental mixtures. Extensive expansion of the AFBC concrete base mixture was of great concern with KDOH officials. Consideration was given to replacing the experimental mixtures with conventional materials. KTC was requested to provide a recommendation to this effect. Nine core specimens were obtained from the experimental AFBC concrete base section. The age of the experimental base layer was estimated to be about 791 days at the time of the coring activity. Of the nine cores obtained, four exhibited delamination between the first and second lifts and one core was destroyed during removal. Four of the core specimens were found to be intact. Recorded core lengths of these specimens ranged from about six to eight inches. The specimen obtained at STA 46+75 was obtained in an area that exhibited cracking. The retrieved core specimen also displayed a vertical crack in the core. Compression tests were performed on the four specimens that were removed intact. The remaining samples were evaluated only for densities. The four specimens that were tested in compression had an average compressive strength of 2,795 psi, and ranged from 1,300 psi to 3,630 psi. The lowest strength value was from the core obtained at STA 46+75. Densities of the core specimens ranged from 136.1 pcf to 143.3 pcf and averaged 139.0 pcf. During the visual inspections of the cores from the AFBC concrete base section, it was noted that the paste appeared to be shrinking away from the coarse aggregate. A photograph, shown in Figure 13, taken shortly after obtaining the core specimen at STA 46+50 illustrates this phenomenon. The cause of this action was unclear. A memorandum prepared by the Chief Research Engineer recommended replacement of the two sections. However, KDOH officials believed that the expansion had ceased and took a wait-and-see position with regard to replacing the two sections.

The two remaining sections were investigated in October 1990. Field core specimens were obtained from the pozzolanic and AFBC stabilized pond ash sections. The respective ages of the experimental layers were 859 and 852 days for the pozzolanic base materials, and the AFBC stabilized pond ash subbase materials at the time of the coring activities. Five

Figure 13. *A field core obtained in the AFBC concrete section during the 1990 investigation exhibited shrinkage of the paste from the coarse aggregate.*

core specimens were obtained from the pozzolanic base section. Recorded lengths of the these specimens ranged from about six to eight inches. One specimen from core location STA 59+20 was evaluated for freeze/thaw durability over a period of 52 cycles. The percent weight loss for the specimen was 19.2 percent. Compressive strengths of the four remaining specimens ranged from 1,540 psi to 2,430 psi and averaged 1,950 psi. Modulus tests could not be performed on the pozzolanic specimens. Densities of the five field cores ranged from 143.2 pcf to 145.5 pcf and averaged 144.6 pcf. Four, four-inch cores were obtained from the AFBC stabilized pond ash subbase section. Delamination between the two constructed lifts was observed at all but one core location. Two pieces of two core specimens were evaluated for freeze/thaw durability over a period of 25 cycles. The average weight loss of the specimens was 56.0 percent. Figure 14 is a photograph of a deteriorated sample after 25 freeze and thaw cycles. Only one core specimen was able to be evaluated for compressive strength. The compressive strength of this specimen was determined to be 1,950 psi. Densities of the AFBC stabilized pond ash specimens that were evaluated ranged from 111.7 pcf to 118.3 pcf and averaged 115.2 pcf.

By the summer of 1991, the two sections containing AFBC residue were showing extreme signs of distress, particularly the AFBC concrete section. Officials with KDOH

Figure 14. *A field core obtained in the AFBC stabilized pond ash section after 25 cycles of freeze and thaw testing.*

determined that the two sections should be replaced with conventional materials. Researchers visited the experimental site in August 1991 to assess the condition of the two sections wherein residue from the AFBC process was incorporated into the experimental mixtures and obtain a final set of field core specimens, measurements of the subgrade strength and deflection measurements. There were no cores obtained from the pozzolanic base section during this investigation.

Twelve locations were identified as coring sites within the AFBC concrete base section. The age of the experimental AFBC concrete base layer was estimated to be about 1,176 days at the time of the coring activity. Most of the specimens were so deteriorated that they could not be recovered and only one core was retrieved that could be evaluated for compressive strength. This specimen was delaminated between the first and second lift. Two-inch cores were obtained from the four-inch cores to perform the compression test. The two specimens tested in compression had an average compressive strength of only 1,070 psi, ranging from 1,215 psi to 920 psi. Densities of the two specimens ranged from 136.7 pcf to 138.6 pcf and averaged 137.6 pcf. The visual inspections of the core specimens from the AFBC concrete base revealed extensive deterioration of the aggregate-to-paste bond. The AFBC concrete base essentially crumbled into an aggregate base material with little stability. Coring activities were conducted at Station 47+50 and

Station 50+00 where expansion gaps had been placed during construction. The expansion gaps had been backfilled with bituminous materials prior to placement of the bituminous base layers. Although there were no solid cores obtained in these areas, the coring activities provided evidence that the AFBC concrete base layer had expanded to the extent that there was no longer any bituminous material within the expansion gaps. Apparently the AFBC concrete base layers had expanded through the four-foot expansion gaps.

Sixteen locations were identified as coring sites within the AFBC stabilized pond ash subbase section. The age of the experimental subbase layer was estimated to be about 1,153 days at the time of the coring activity. Delamination of the two constructed lifts of the AFBC stabilized pond ash mixture was observed at each coring location. Two-inch cores were obtained from the four-inch cores to perform compression tests. Ten specimens were tested in compression. The average compressive strength of the specimens was 2,210 psi, and ranged from 1,500 psi to 3,480 psi. Densities of the AFBC stabilized pond ash subbase specimens ranged from 115.1 pcf to 120.4 pcf and averaged 117.0 pcf. Visual inspections of core specimens from the AFBC stabilized pond ash subbase indicated that the materials were largely intact. This fact was substantiated by the compression tests. Coring activities were conducted at Station 62+50 and Station 65+00 where expansion gaps had been placed during construction. The expansion gaps located within the AFBC stabilized pond ash subbase section had been backfilled with aggregate prior to placement of the bituminous base layers. There were no solid cores of the experimental subbase materials obtained in these areas. However, results of the coring activities conducted at the expansion gaps indicated that the AFBC stabilized pond ash subbase layer had expanded through the approximately two-foot expansion gaps as no aggregate materials were found.

In-situ California Bearing Ratio (CBR) tests were performed at five locations within the AFBC concrete base section during the August investigation. The subgrade of the AFBC concrete base section had an average in-situ CBR of six and varied from three to 16. The moisture content of the subgrade material during this time averaged 14.2 percent and ranged from 10.5 percent to 17.4 percent. In-situ CBR values and moisture contents obtained just before the placement of the experimental base layer averaged 24 and 8.3 percent, respectively. Data obtained from this testing activity indicates a substantial reduction in the bearing capacity and a significant increase in the in-situ moisture content of the underlying subgrade materials over the three year period since the time of construction.

From the results of coring activities, visual inspections of the cored specimens and physical tests performed on the cores, it was concluded that the experimental AFBC concrete base material underwent significant changes as it aged. The expansive nature of the AFBC residue caused complete destruction of the structural integrity of the materials. The apparent shrinkage of the paste material from the coarse aggregate was thought to be a function of this phenomenon. The AFBC concrete base layer appeared to be weakest in those areas where water was trapped in the section, particularly in the westbound lane near Station 51+00. Figure 15 is a photograph of a core taken in the westbound lane at Station 51+50 during the 1990 investigation and illustrates the deterioration that the experimental base layer underwent after construction. The lower portion of the core is quite deteriorated and delamination occurred between the first and second lifts. There is visible evidence that the experimental layer underwent extreme stress during placement of the stress absorbing membrane interlayer (SAMI). The asphalt emulsion permeated through cracks in the AFBC concrete base layer that were caused by the asphalt spreader. The deterioration of the experimental materials is further demonstrated by the decrease in the compressive strength of the materials obtained by testing field core specimens. Figure 16 graphically depicts the average compressive strength of field core specimens obtained from the experimental sections as a function of time. The general deterioration of the experimental AFBC concrete base mixture with time is quite evident although the data were limited. Figure 16 also depicts

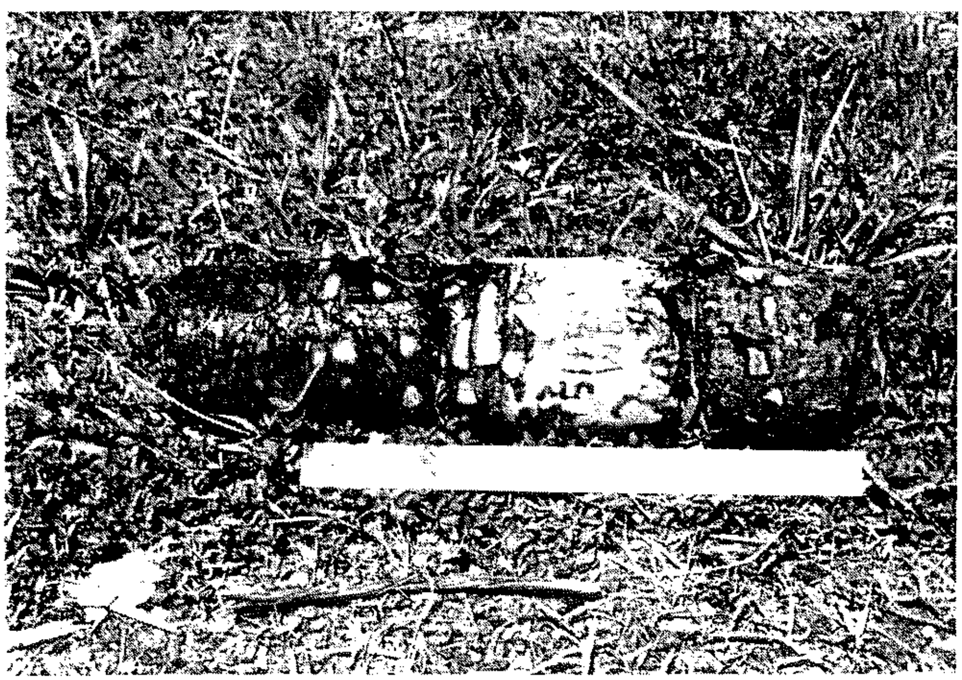


Figure 15. *A field core specimen obtained at Station 51+50 illustrates the deterioration of the AFBC concrete base material.*

apparent increases and decreases in the average compressive strength of specimens obtained from the AFBC stabilized pond ash subbase. The average compressive strength of core specimens obtained from the pozzolanic base section however, appears to be increasing slightly with the passage of time. Appendix C contains data obtained from the coring and testing activities.

Combining the compressive strengths of the field specimens and the compressive strengths of specimens compacted in the field during construction of the experimental sections provides a complete history of the compressive strength development and subsequent deterioration of the of the mixtures containing residue from the AFBC process. Figure 17 illustrates the initial strength gain of the mixtures containing the

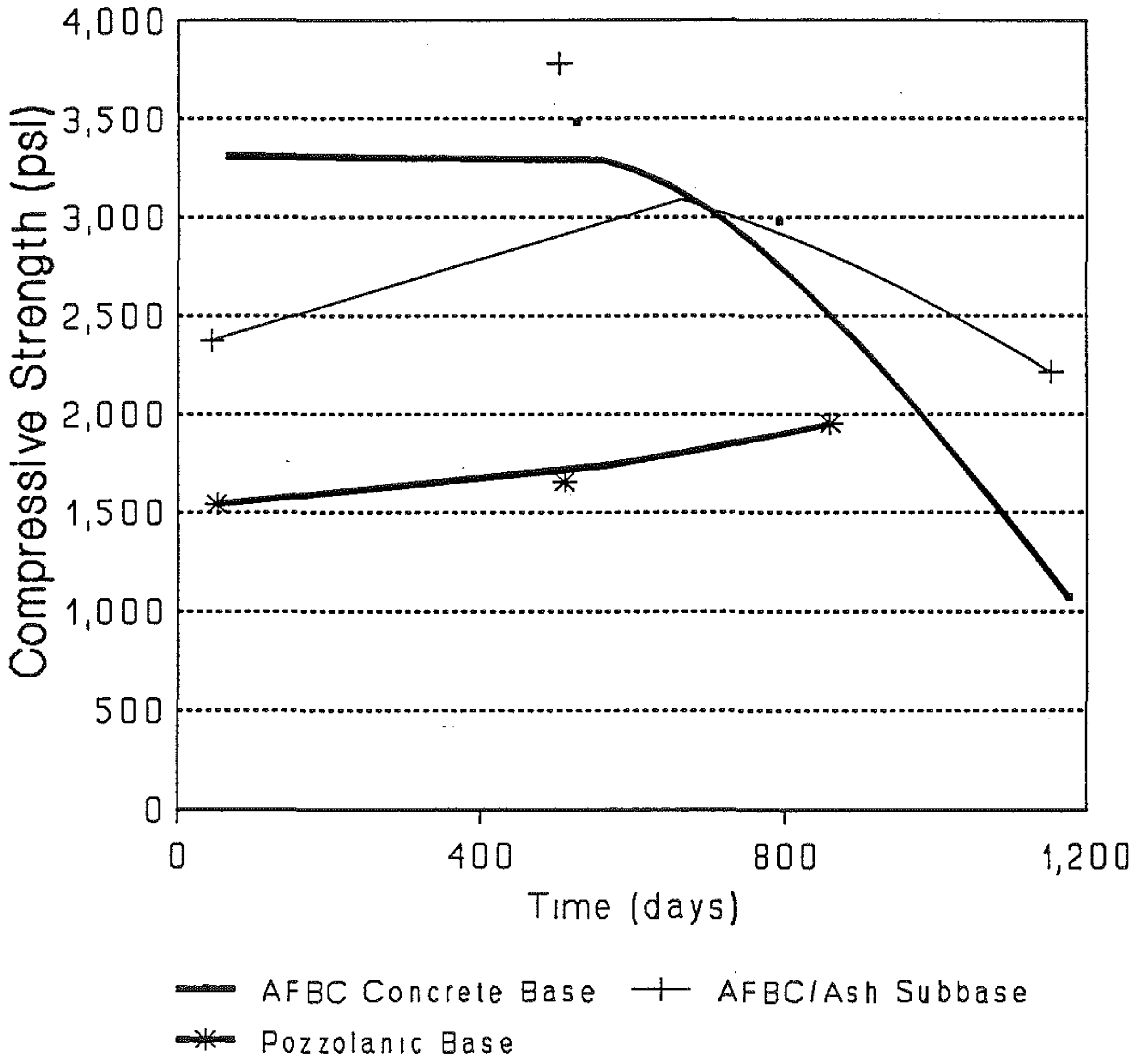


Figure 16. *Average compressive strength of field core specimens as a function of time.*

AFBC residue and following decrease in compressive strength. The deterioration of the AFBC concrete layer is most evident whereas the AFBC stabilized pond ash mixture still displays adequate strengths. Figure 17 also depicts the constancy of the pozzolanic base mixture that contains approximately 11 percent ponded fly ash.

Dynamic Deflection Tests and Analyses

Pavement deflections were obtained on each test section shortly after placement of the bituminous base and binder layers and at various times thereafter. Deflections were

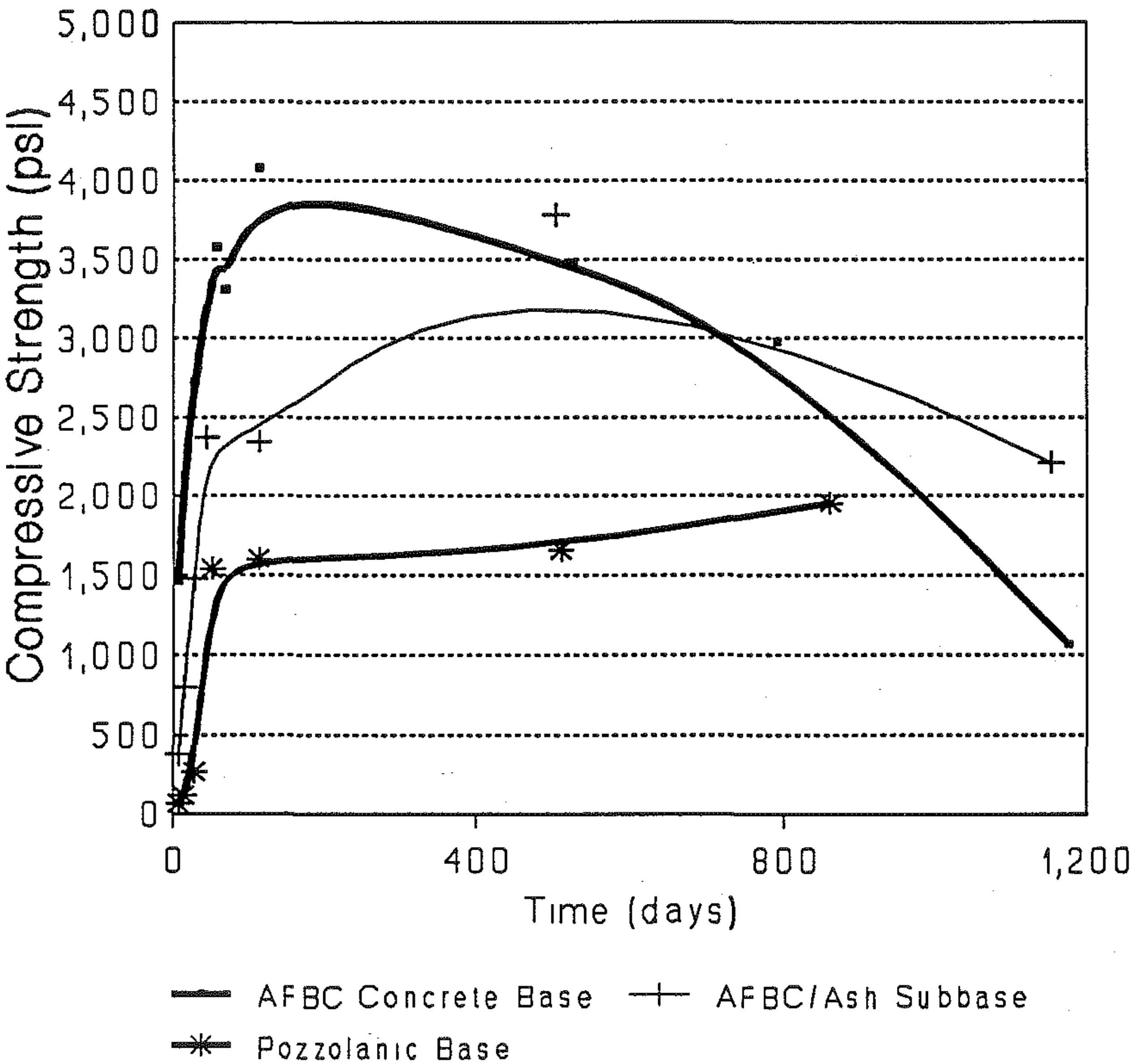


Figure 17. *Average compressive strength of field compacted specimens and field core specimens as a function of time.*

obtained using a Model 400B Road Rater, which applies a steady state dynamic load to the pavement. Deflections were measured at dynamic loads of 600 and 1,200 pounds. Pavement deflection measurements were also obtained in 1991 prior to the reconstruction of portions of the project. These deflections were measured using the JILS-20 Falling Weight Deflectometer (FWD). The FWD applies an impulse dynamic load to the pavement and measures the corresponding pavement deflections at seven radial distances from the center of the load.

The measured pavement deflections from each testing device were analyzed using the WESDEF modulus backcalculation program. Through an iterative process, this program matches the measured deflection bowls with theoretically calculated deflection bowls based upon the error between the deflection bowls. Layer moduli associated with the theoretically calculated deflection bowls are assumed to be representative of the in-situ moduli of the pavement structure.

In comparing the data from 1988 and 1991 in general the subgrade and asphaltic concrete moduli remained relatively constant. There were some small variations in the subgrade moduli, which is most probably attributed to changes in the in-situ moisture content. Variations of the backcalculated layer moduli for the experimental base and subbase layers were generally greater than variations for the subgrade or asphaltic concrete layers. The coefficient of variation of the subgrade for all sections ranged from 15 to 40 percent. The coefficient of variation of ranged from 25 to 60 percent for the asphaltic concrete and 48 to 120 percent for the experimental base and subbase layers. The coefficient of variation of the experimental layers of the two sections containing AFBC material ranged from 60 to 120 percent. The coefficient of variation for the pozzolanic base section was 49 percent. The average backcalculated moduli of the experimental layers for 1988 and 1991 were as follows. The coefficient of variation for each section is shown in parentheses.

	Backcalculated Modulus (psi)	
Section	1988	1991
AFBC Concrete Base	1,792,108	684,458
	(64.9)	(120.8)
Pozzolanic Base	1,752,993	2,961,993
	(47.9)	(50.9)
AFBC/Pond Ash Subbase	313,686	398,393
	(60.4)	(82.2)

It may be seen from the results of the deflection testing performed throughout the evaluation period that the pozzolanic base material increased in strength approximately 70 percent while the coefficient of variation remained relatively constant. However, the AFBC concrete base material decreased in strength by approximately 60 percent and became very variable throughout the section as illustrated by the considerable increase in the coefficient of variation. The AFBC stabilized pond ash subbase materials remained relatively constant, increasing only 37 percent while increasing also in variability.

Based on the results of the deflection testing and analyses, the pozzolanic base materials performed very well while being less variable than the remaining two sections. The increase in variability of the two sections containing residue from the AFBC process supports the findings of other investigations that indicated the sections deteriorated with time. The large decrease in modulus of the AFBC concrete base section and the small increase in modulus of the AFBC stabilized pond ash subbase section would also support these observations.

SUMMARY

Previous reports have described preliminary engineering and design, construction details, and initial performance of three experimental sections constructed on State Route 3074 in McCracken County, Kentucky wherein various by-products from a coal-fired power generation plant were utilized to construct highway base and subbase layers. Waste products utilized during the study were residue from an atmospheric fluidized bed combustion process and ponded ash. Subsequent performance surveys and measurements performed as part of this research effort and reported herein consisted of distress surveys, obtaining dynamic deflections and performing data analyses, pavement rutting characteristics, and strength and durability measurements of field cores obtained from the experimental base and subbase layers.

The three, 750-foot test sections of a 22-foot wide roadway, contained base and subbase layers comprised of by-product waste materials combined in some instances with conventional materials. Mixture proportions for the base and subbase layers were developed during an extensive laboratory testing program for use in the field trial application. The field trials were constructed near Paducah, Kentucky in May and June, 1988 utilizing power plant wastes from the Tennessee Valley Authority's Shawnee Power Plant. One experimental section incorporated a base layer that was a mixture of residue from an atmospheric fluidized bed combustion (AFBC) process, coarse limestone aggregate (Size No. 57), and specification grade, Class F fly ash. Another test section included a base layer comprised of dense-graded aggregate, hydrated lime and non-specification waste fly ash. A third test section included a subbase layer containing a mixture of residue from the AFBC process and pond ash. An adjacent section constructed of conventional materials served as the control section for the project. Because expansion of the base and subbase layers containing residue from the AFBC process was expected, gaps were constructed in these layers in an attempt to accommodate the expansion. The gaps were backfilled with bituminous materials and DGA for the AFBC concrete base and AFBC stabilized pond ash subbase sections, respectively.

Shortly after the bituminous base and binder layers were placed at the experimental site, humps were observed on the pavement surface within the sections that had incorporated residue from the AFBC process in the mixtures. During the three-year evaluation period, the humps required approximately ten millings. Expansion of the base and subbase mixtures that contained the AFBC residue was so immense that the backfilled materials were eventually squeezed from the gaps and milled away. The areas near the expansion gaps required repeated maintenance to prevent the sections from being a traveling hazard to the public. Dynamic deflection tests performed on the experimental sections indicated gradual deterioration of the pavement structure during the evaluation period. This was especially true of the AFBC concrete base layer. These two sections also exhibited the greatest depth of pavement rutting. Compression tests of cored field specimens also indicated a gradual deterioration of the base and subbase materials during the three-year period. After continual milling and patching operations and visible deterioration of the pavement structure, the base and subbase sections incorporating residue from the AFBC process were judged to be failed experiments. The sections were removed and replaced with conventional construction materials.

The remaining experimental section containing the pozzolanic (ponded fly ash-hydrated lime-dense grade aggregate) base demonstrated superior performance during the evaluation period. One transverse and one longitudinal reflective crack were observed. Other than the reflective cracks caused by shrinkage known to be associated with pozzolanic bases, no significant distresses were noted. Analysis of the deflection data obtained within this section indicated exceptional structural integrity for the pozzolanic base layer. Rutting measurements indicated less rutting occurred in the bituminous layers of the experimental pozzolanic section than that observed in the control section. Unconfined compressive strength tests substantiated the excellent and consistent strengths of the experimental pozzolanic mixture.

CONCLUSIONS AND RECOMMENDATIONS

The base and subbase layers containing residue from the AFBC process were classified as failures because of the poor performance demonstrated by the sections. The poor performance was due to a number of things but the excessive expansion that the base and subbase mixtures underwent was the principal reason for the resulting pavement distresses and ultimately, the failure of the sections. The Special Note for Construction, developed for the waste materials utilized in this project, required that the AFBC residue be prehydrated prior to its use. Prehydration of the residue from the AFBC process is necessary to eliminate, or minimize the inherent expansive properties of the residue. Because of materials handling and storage problems, the materials handler prehydrated the AFBC residue several months prior to construction and stored it in a warehouse. KTC representatives obtained a sample of the prehydrated AFBC residue from the storage warehouse prior to the construction date. That sample had a hydration reaction when water was added to it and thus, it was known that the material would swell when placed in the mixtures. It is believed that either the AFBC residue was not prehydrated at the batch plant according to specifications or the extended storage period somehow adversely affected the properties of the residue. Nevertheless, officials involved with the study were determined to proceed with the construction and the AFBC residue was used. It was thought that the expected expansion could be accommodated by placing gaps in the plastic base and subbase layers. This response to the expansive properties of the materials was not successful and the two experimental sections subsequently failed. The experimental base and subbase layers containing the expansive AFBC residue likely should not have been constructed without prehydrating the AFBC residue again. The AFBC residue continued to cause extensive pavement distress throughout the evaluation. Public clamor over the apparent waste of taxpayer monies on the failed experiment was quite harsh. The KDOH suffered through a public relations nightmare due to the failed experiment. However, the experiment was a learning experience.

Use of the AFBC residue from the Shawnee Power Plant in highway base and subbase layers, or as a subgrade modifier, may be feasible at some point in the future but should not be permitted unless techniques can be developed and used that will absolutely guarantee elimination or, at the very least, minimization of the expansive nature of the AFBC residue. The successes obtained during the laboratory studies must be transmittable to field trial applications. One possibility could be prehydration of the AFBC residue within one or two days prior to its use. A pugmill set up near the construction site would permit prehydration activities to take place only a few days before the commencement of construction activities. The pugmill would also permit mixture components to be batched at a more suitable rate than achieved during this experiment. Future experiments should be conducted in an effort to effectively utilize the by-product materials from the atmospheric fluidized bed combustion process in highway construction applications.

The excellent performance of the experimental pozzolanic base layer clearly demonstrates that a non-specification fly ash, a waste material, was successfully substituted in stabilized aggregate base construction for the specification-grade fly ash that meets the requirements of ASTM C 593 and ASTM C 618. It is recommended that KDOH officials allow the substitution of non-specification fly ashes for specification grade fly ash in stabilized aggregate bases on a limited basis and to perform additional detailed performance and cost comparisons to validate the any advantages of this practice. Current KDOH Special Provision No. 70D (91) for stabilized aggregate bases governs probable compositions for hydrated lime-fly ash-DGA mixtures.

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4. Rose, J.G.; Bland, A.E.; Jones, C.E.; and Szwilski, A., "Laboratory Testing of Potential Utilization of AFBC Wastes and Other Fossil Fuel Waste in the Production of Low-Strength Concretes for Mining Applications," TVA/PUB-84-4:NTIS DE86-9000-98, Institute for Mining and Minerals Research, University of Kentucky, March, 1985.

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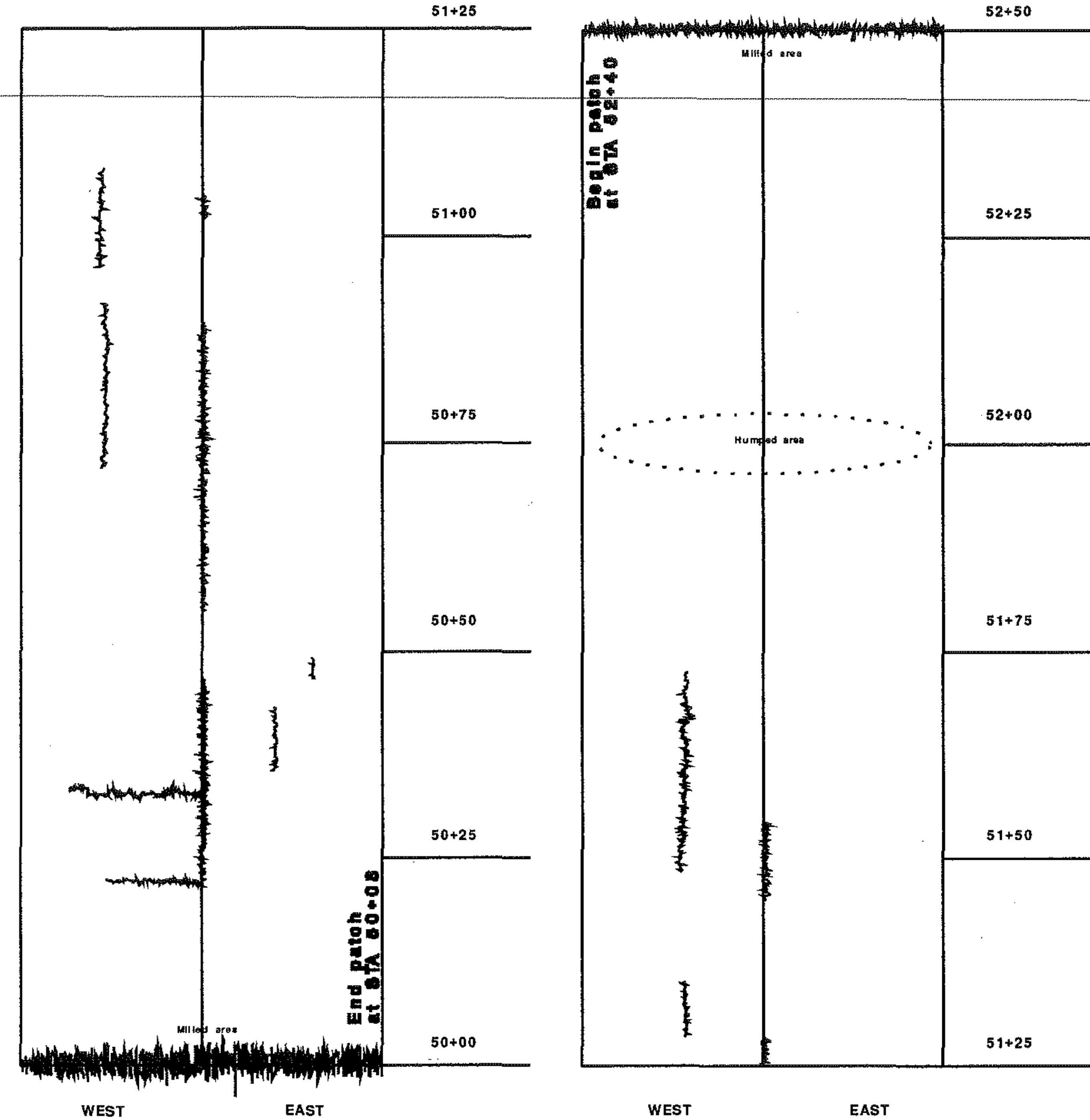
6. Minnick, L.J., "Development of Potential Uses for the Residue from Fluidized Bed Combustion Processes," DOE/ET/10415-T6, December, 1982.

APPENDIX A

Detailed Pavement Distress Survey

KY 3074, Bleich Road



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----- Data obtained 07-12-90

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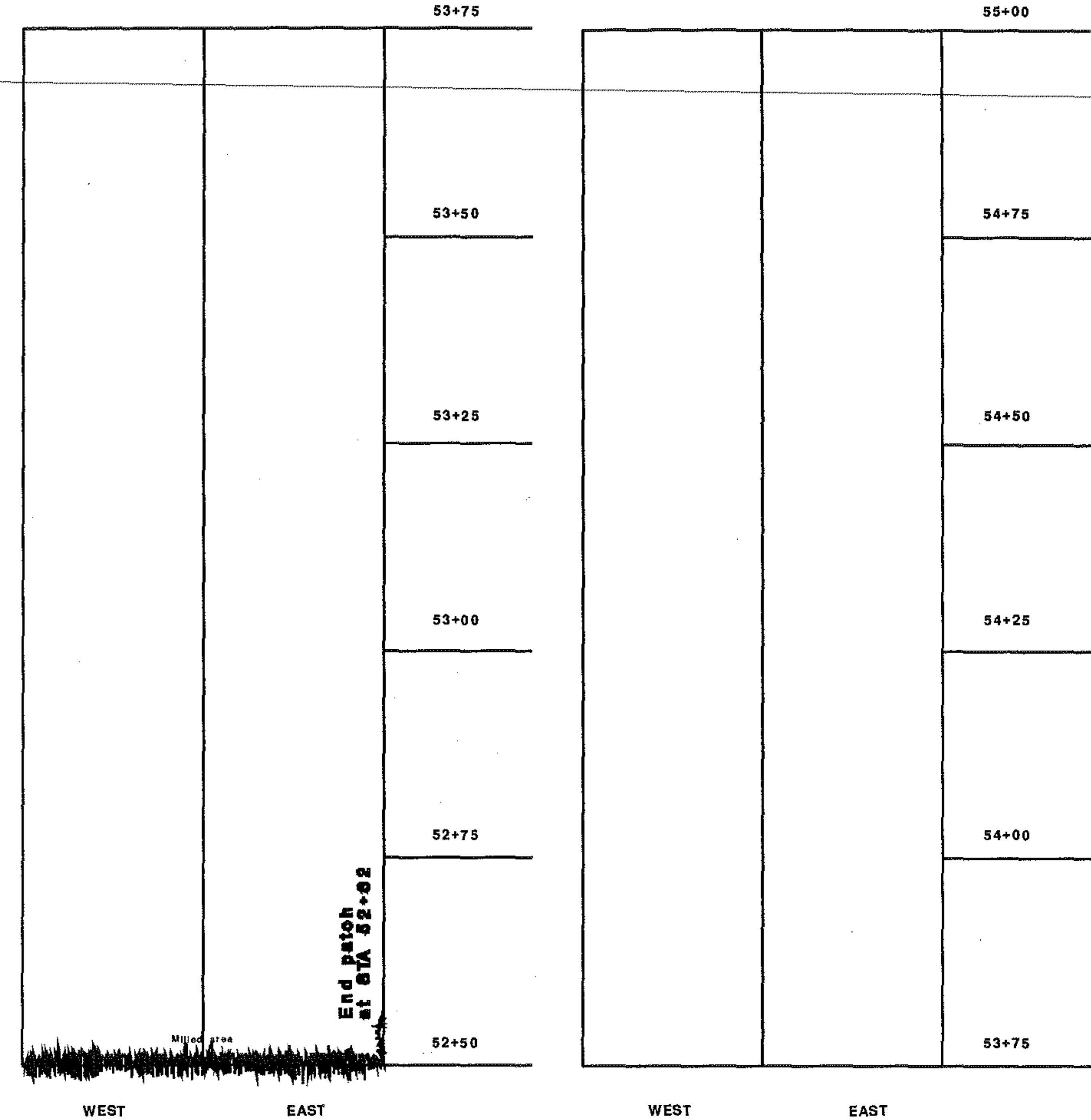

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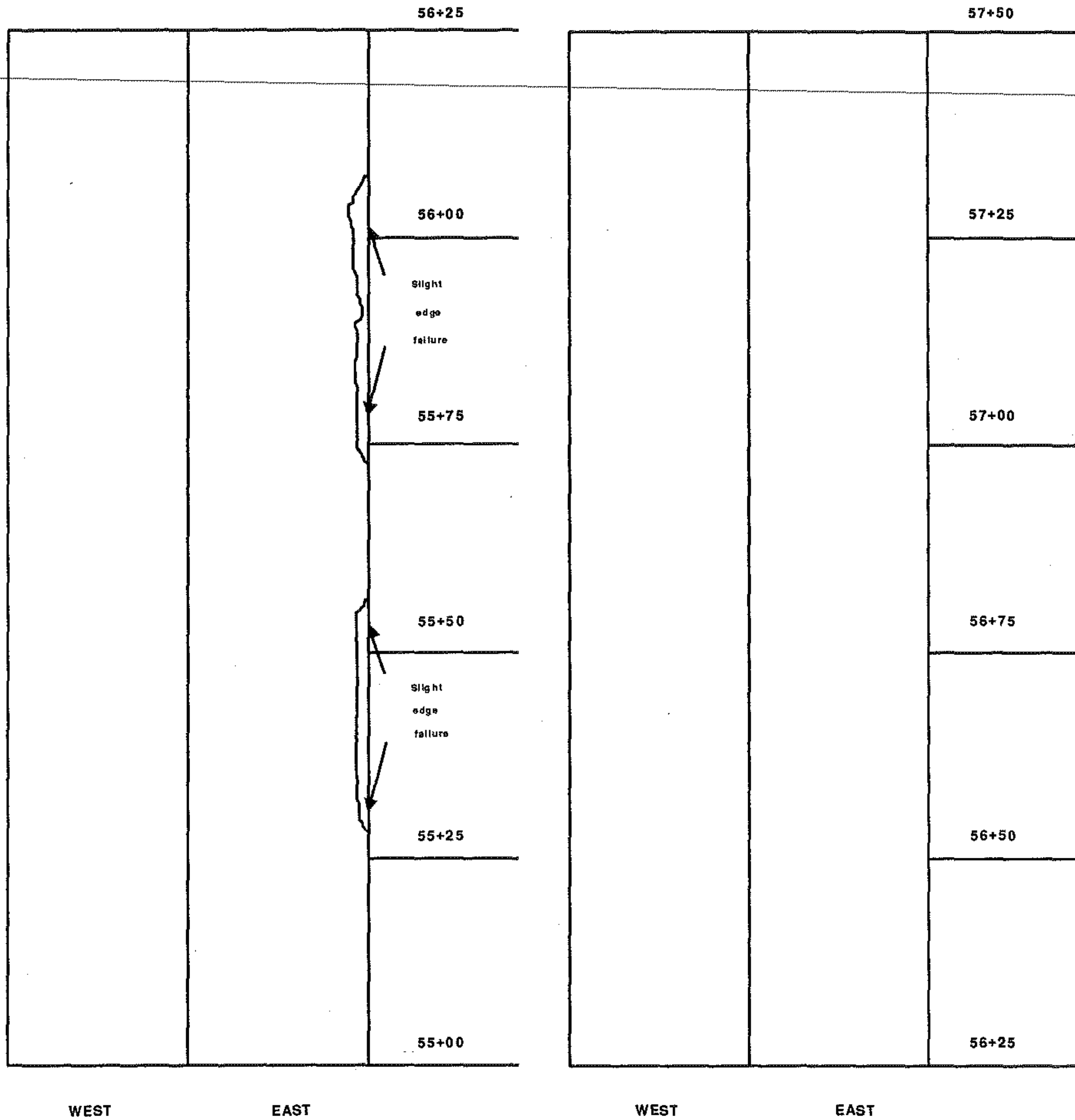

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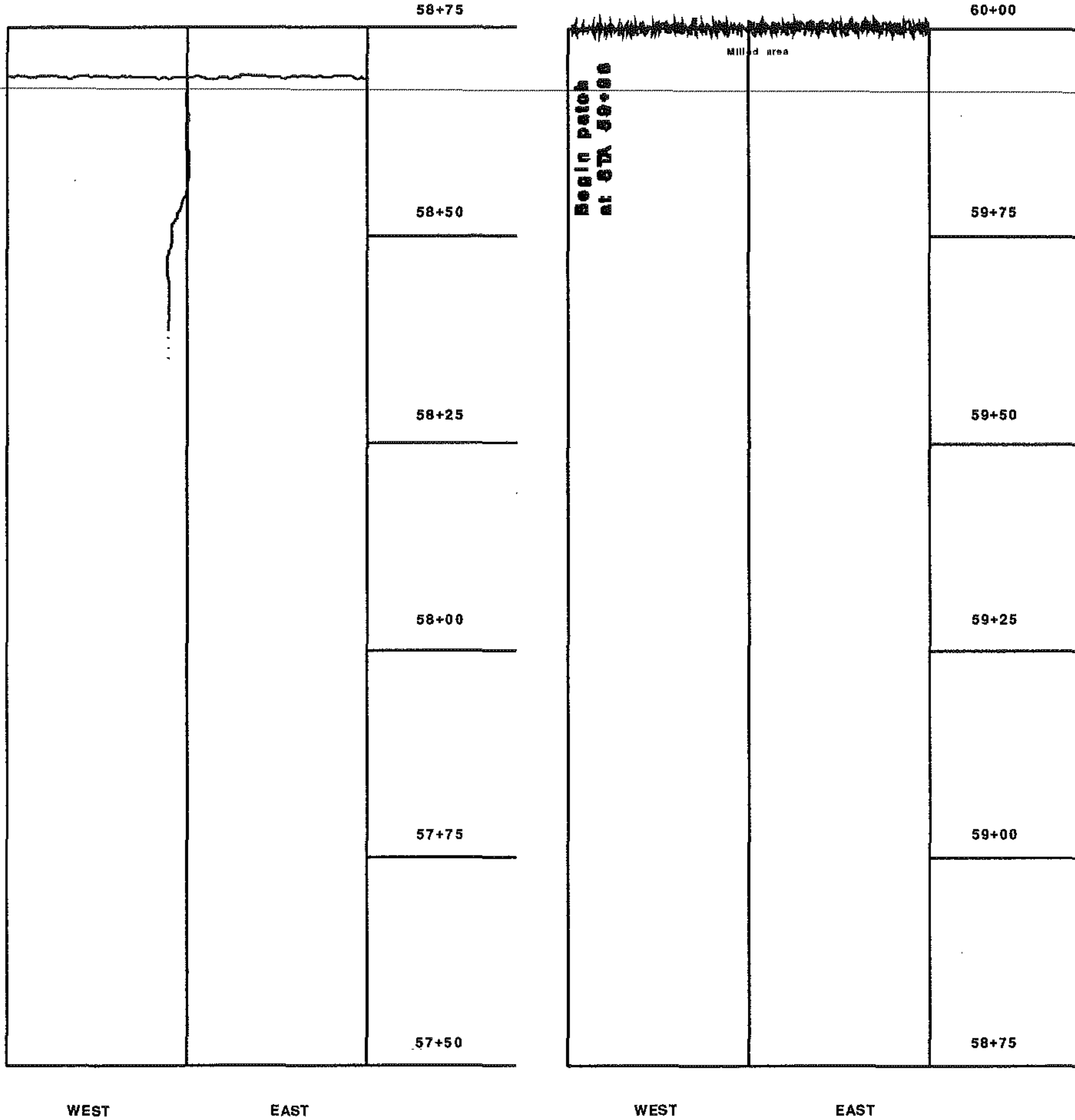

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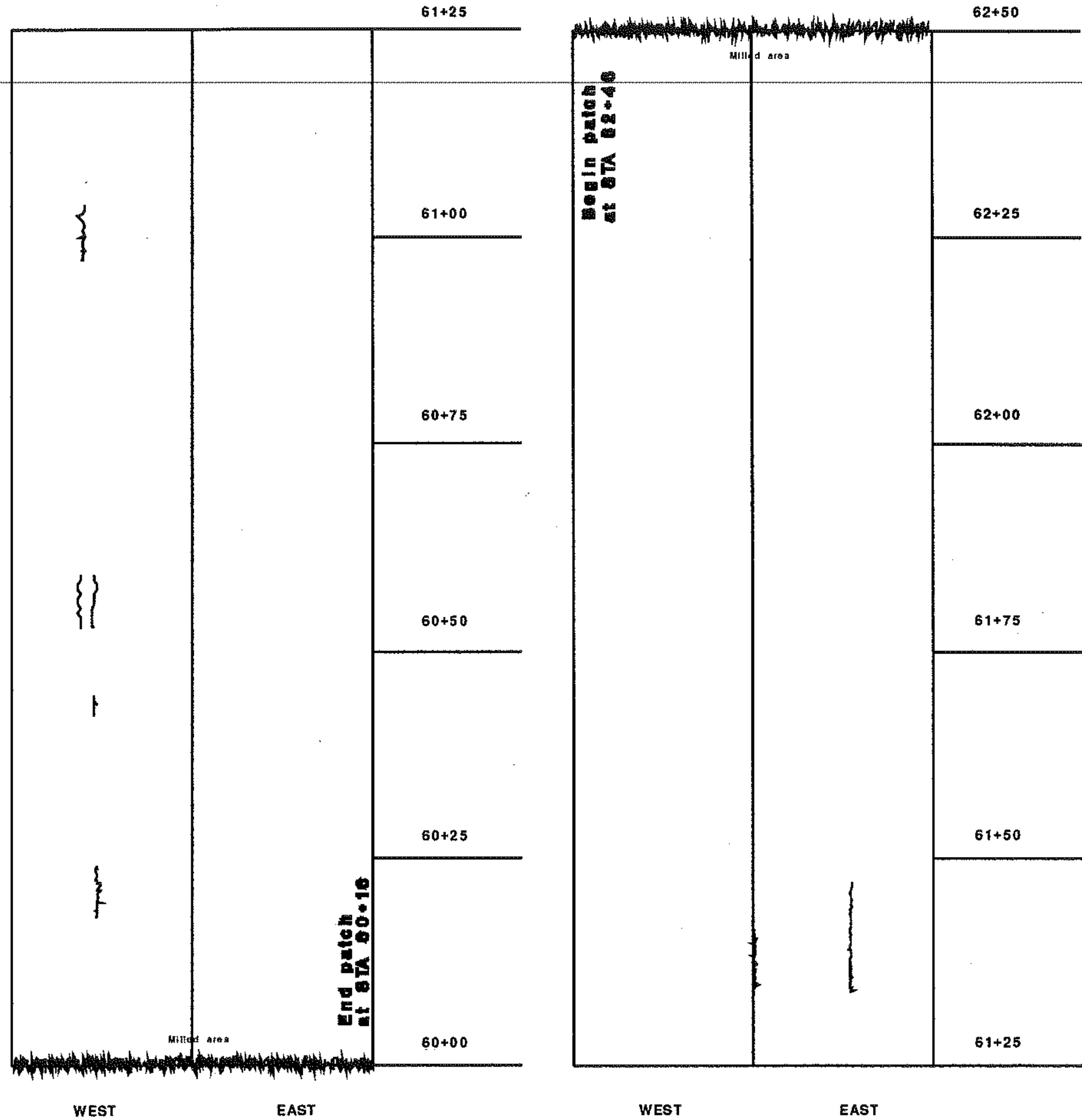

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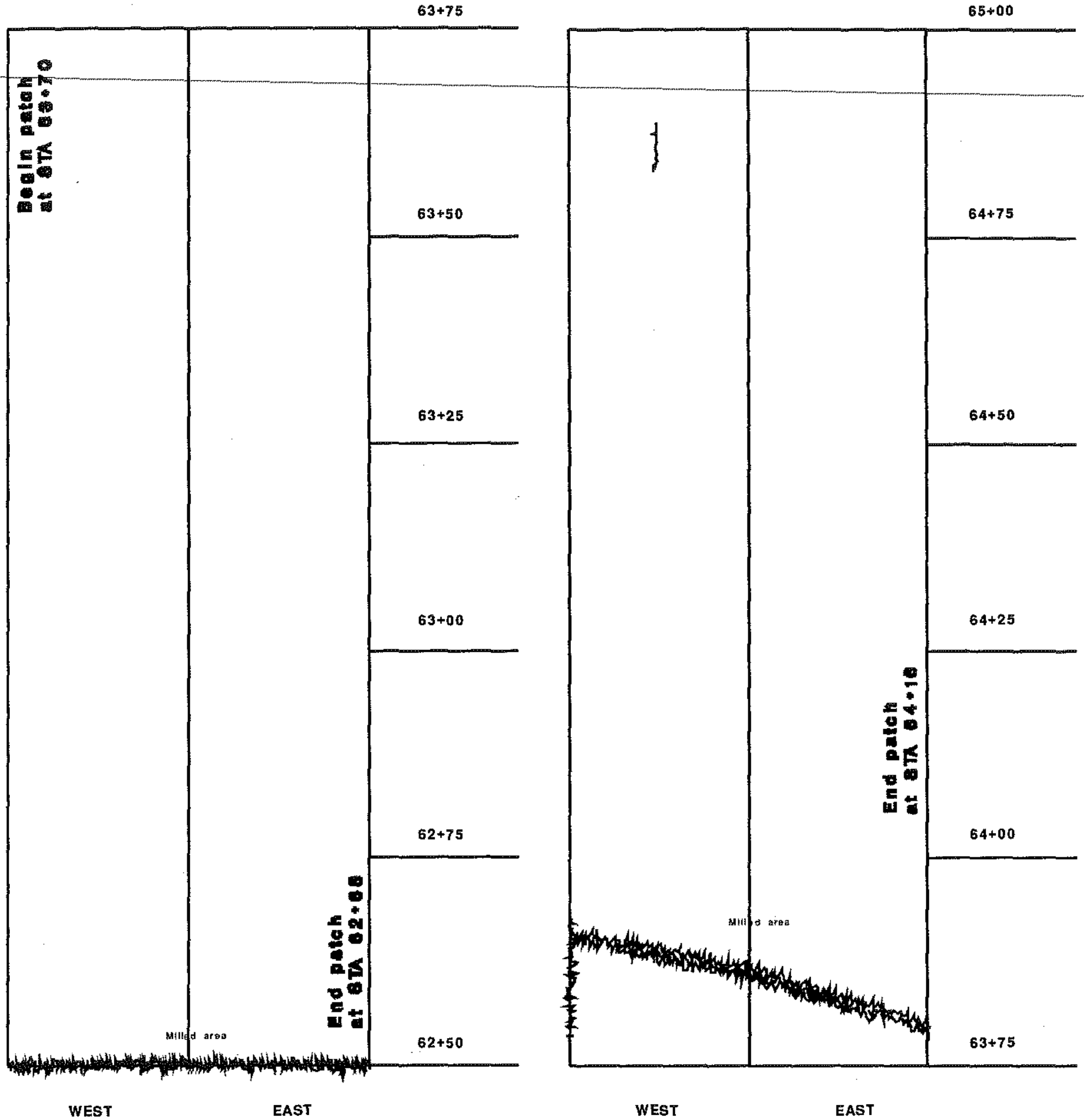

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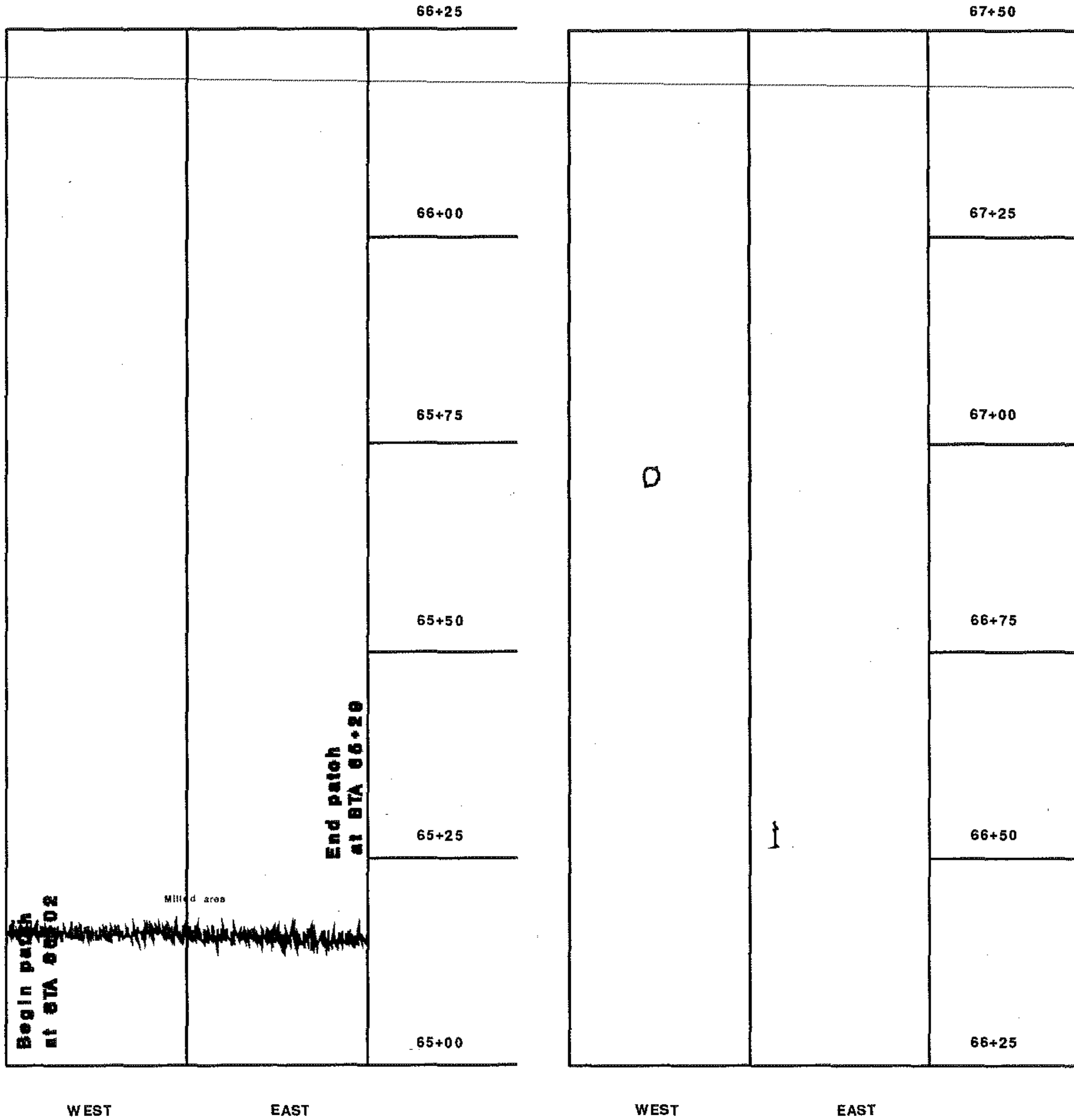

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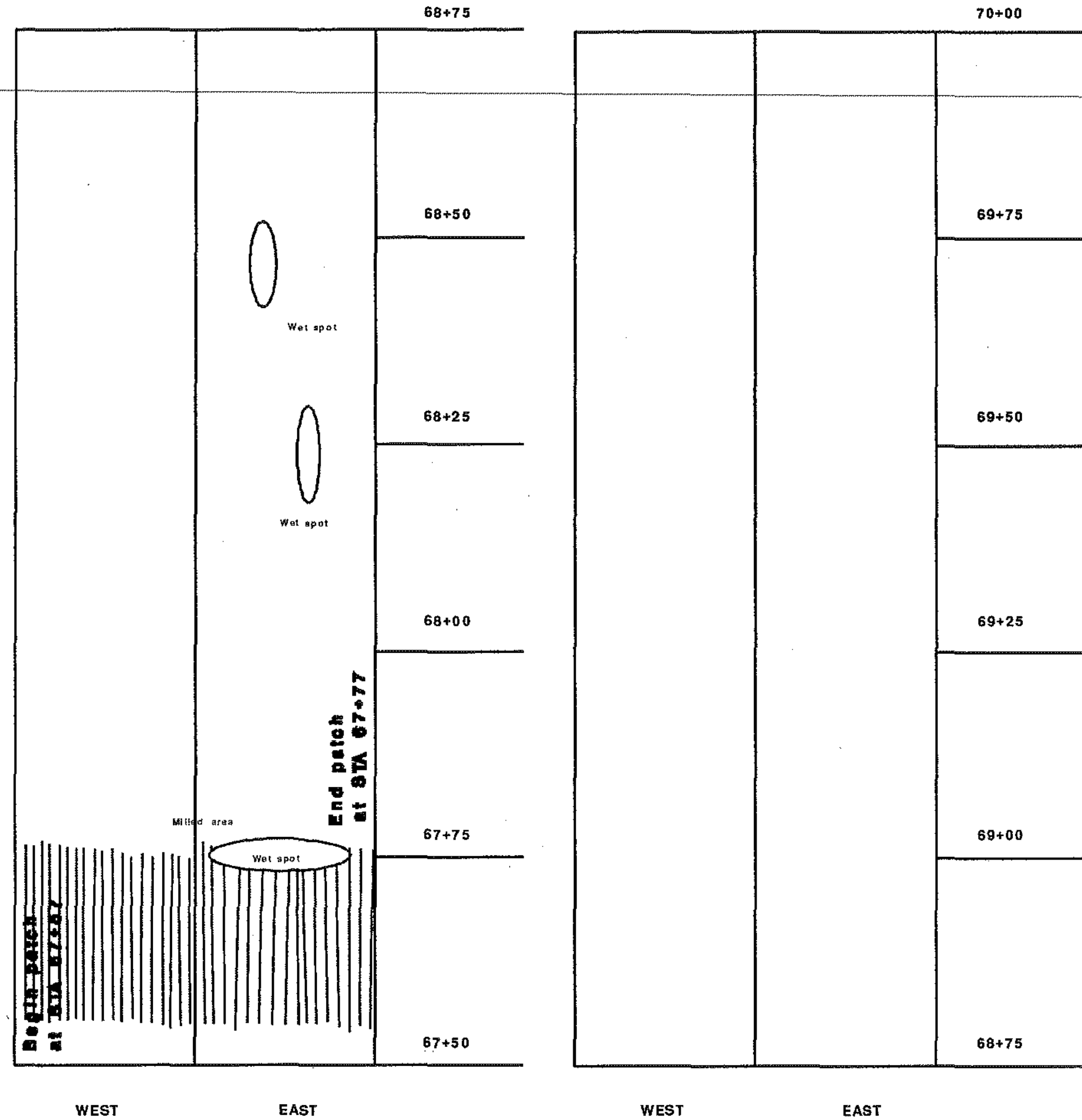

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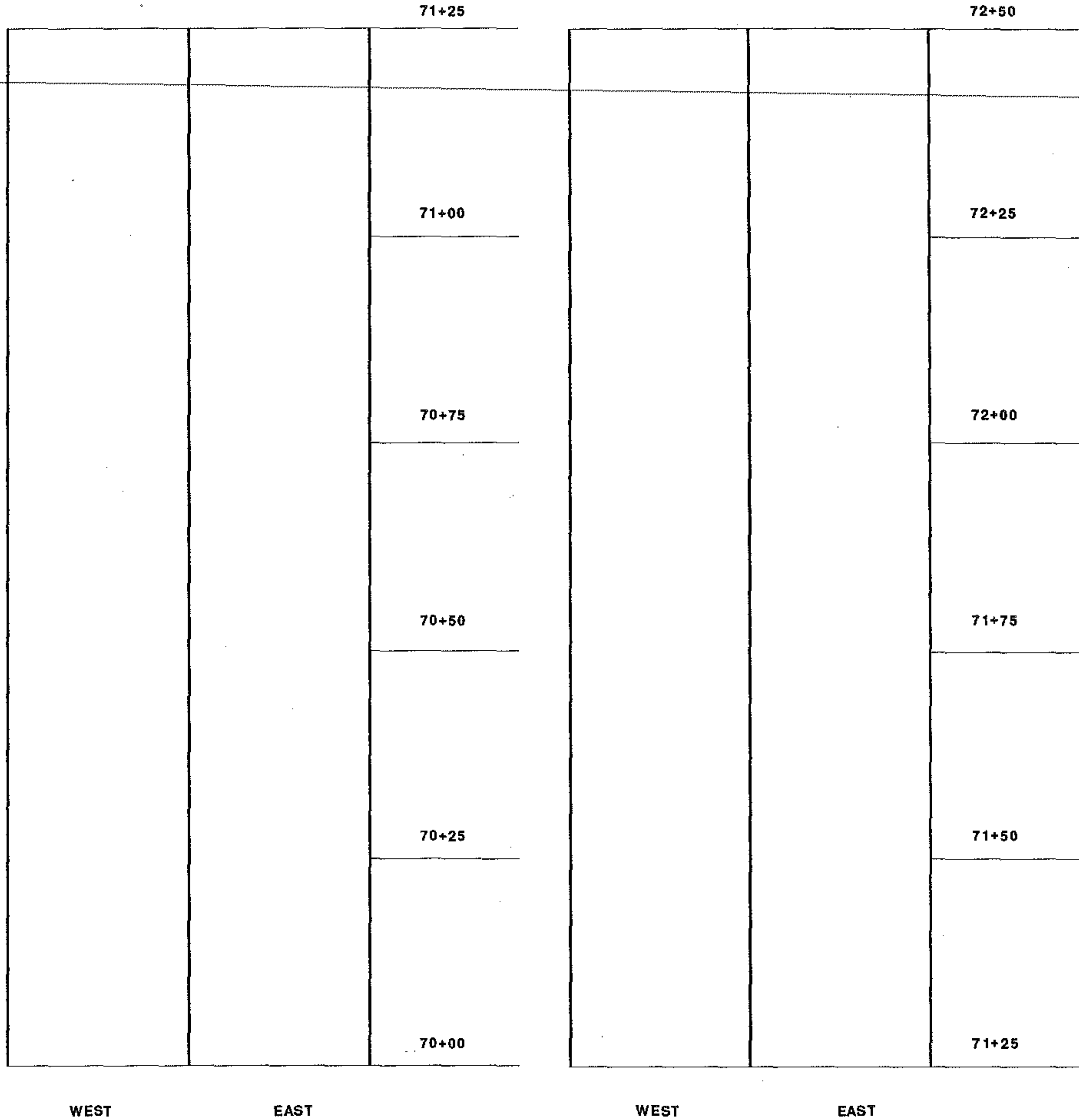

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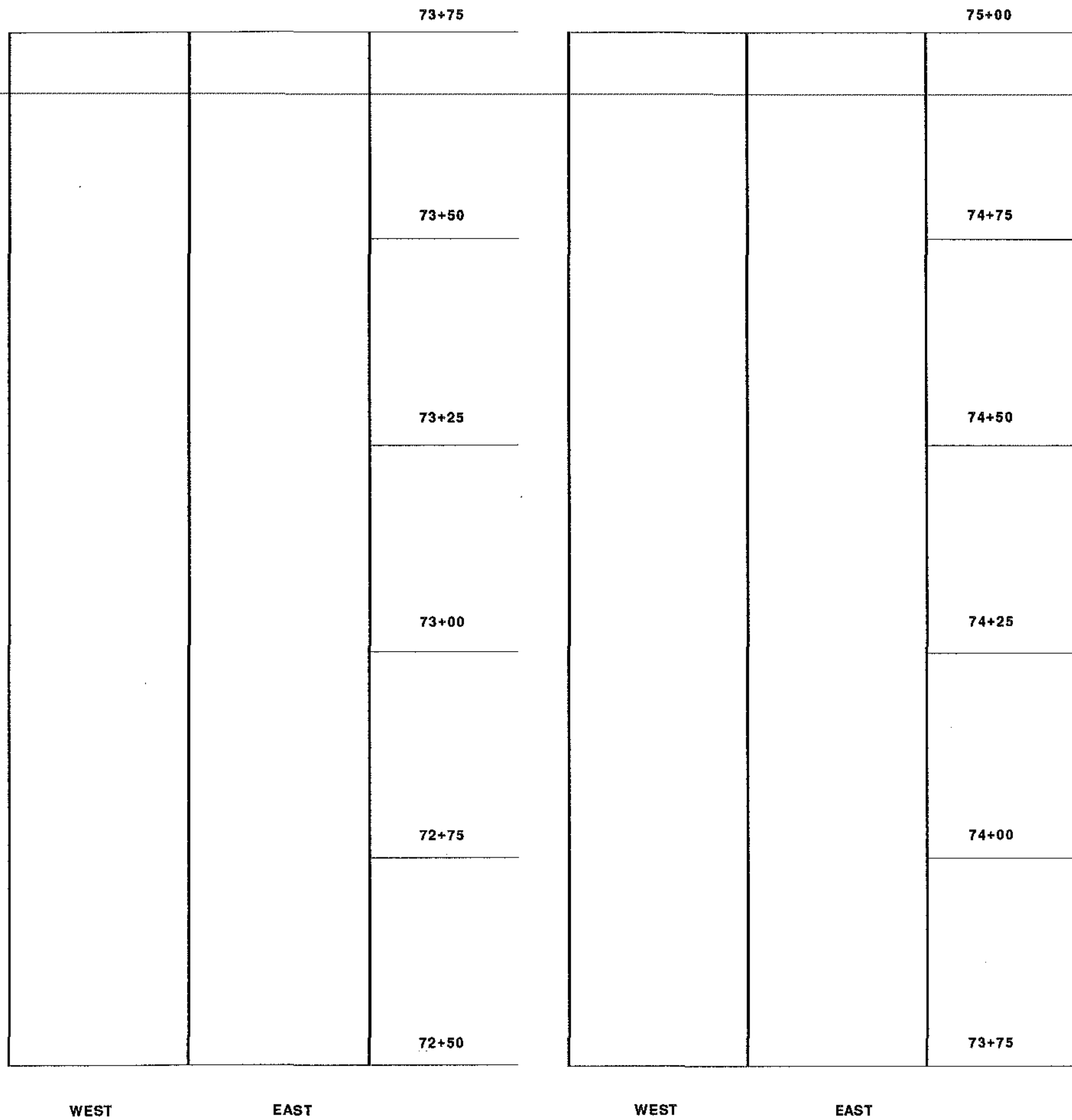

LEGEND:

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----- Data obtained 02-08-90

----- Data obtained 07-12-90

NOT TO SCALE

APPENDIX B

Pavement Rutting Characteristics

KY 3074, Bleich Road

TABLE B1. RUTTING DEPTHS -- AFBC CONCRETE SECTION

DATE: 10-03-89

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
45+50	3/16	4/16	5/16	1/16
46+00	2/16	3/16	6/16	1/16
46+50	5/16	2/16	3/16	3/16
47+00	5/16	3/16	2/16	2/16
48+00	3/16	7/16	1/16	3/16
48+50	4/16	2/16	1/16	4/16
49+00	2/16	5/16	1/16	5/16
49+50	7/16	5/16	2/16	4/16
50+50	4/16	2/16	2/16	4/16
51+00	4/16	2/16	2/16	1/16
51+50	1/16	2/16	1/16	2/16
52+00	4/16	6/16	2/16	7/16

TABLE B2. RUTTING DEPTHS -- AFBC CONCRETE SECTION

DATE: 02-08-90

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
45+50	3/16	5/16	6/16	3/16
46+00	3/16	5/16	3/16	3/16
46+50	8/16	3/16	4/16	3/16
47+00	7/16	6/16	3/16	5/16
48+00	3/16	9/16	3/16	4/16
48+50	6/16	3/16	1/16	7/16
49+00	4/16	7/16	4/16	7/16
49+50	9/16	10/16	1/16	9/16
50+50	5/16	3/16	4/16	2/16
51+00	3/16	2/16	3/16	1/16
51+50	2/16	4/16	2/16	4/16

TABLE B3. RUTTING DEPTHS -- AFBC CONCRETE SECTION

DATE: 07-19-90

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
45+50	5/16	7/16	8/16	5/16
46+00	4/16	6/16	7/16	5/16
46+50	10/16	5/16	5/16	5/16
47+00	8/16	6/16	4/16	6/16
48+00	6/16	9/16	3/16	7/16
48+50	6/16	3/16	1/16	9/16
49+00	3/16	9/16	4/16	10/16
49+50	9/16	10/16	2/16	8/16
50+50	5/16	4/16	4/16	8/16
51+00	6/16	2/16	2/16	1/16
51+50	3/16	4/16	2/16	6/16

TABLE B4. RUTTING DEPTHS -- AFBC CONCRETE SECTION

DATE: 10-11-90

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
45+50	4/16	6/16	6/16	4/16
46+00	4/16	6/16	4/16	4/16
46+50	7/16	5/16	3/16	4/16
47+00	5/16	5/16	5/16	5/16
48+00	5/16	5/16	5/16	4/16
48+50	4/16	4/16	2/16	5/16
49+00	3/16	7/16	2/16	7/16
49+50	9/16	8/16	3/16	4/16
50+50	4/16	4/16	4/16	8/16
51+00	5/16	4/16	5/16	3/16
51+50	3/16	4/16	3/16	3/16

TABLE B5. RUTTING DEPTHS -- POZZOLANIC SECTION

DATE: 10-03-89

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
53+00	2/16	2/16	2/16	2/16
53+50	1/16	2/16	2/16	1/16
54+00	1/16	1/16	1/16	1/16
54+50	3/16	1/16	1/16	1/16
55+00	2/16	2/16	2/16	1/16
55+50	2/16	1/16	2/16	2/16
56+00	1/16	1/16	2/16	1/16
56+50	2/16	1/16	1/16	1/16
57+00	2/16	2/16	1/16	1/16
57+50	1/16	1/16	1/16	1/16
58+00	2/16	1/16	2/16	1/16
58+50	3/16	2/16	3/16	1/16
59+00	2/16	1/16	1/16	1/16
59+50	2/16	1/16	2/16	1/16

TABLE B6. RUTTING DEPTHS -- POZZOLANIC SECTION

DATE: 02-08-90

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
53+00	3/16	2/16	2/16	2/16
53+50	2/16	3/16	2/16	2/16
54+00	3/16	2/16	2/16	2/16
54+50	3/16	2/16	1/16	1/16
55+00	4/16	2/16	2/16	1/16
55+50	3/16	1/16	3/16	2/16
56+00	2/16	2/16	2/16	2/16
56+50	3/16	3/16	2/16	1/16
57+00	3/16	2/16	1/16	2/16
57+50	3/16	2/16	2/16	1/16
58+00	1/16	2/16	2/16	1/16
58+50	3/16	2/16	3/16	2/16
59+00	3/16	3/16	2/16	2/16
59+50	3/16	1/16	3/16	2/16

TABLE B7. RUTTING DEPTHS -- POZZOLANIC SECTION

DATE: 07-19-90

STATION	WESTBOUND LANE RWP (in.)	WESTBOUND LANE LWP (in.)	EASTBOUND LANE LWP (in.)	EASTBOUND LANE RWP (in.)
53+00	3/16	3/16	3/16	3/16
53+50	3/16	3/16	4/16	2/16
54+00	3/16	1/16	3/16	4/16
54+50	3/16	2/16	2/16	1/16
55+00	3/16	2/16	2/16	2/16
55+50	2/16	1/16	2/16	2/16
56+00	3/16	2/16	2/16	1/16
56+50	3/16	3/16	1/16	1/16
57+00	3/16	2/16	2/16	1/16
57+50	3/16	2/16	2/16	1/16
58+00	3/16	3/16	3/16	1/16
58+50	5/16	3/16	3/16	1/16
59+00	4/16	3/16	3/16	2/16
59+50	3/16	2/16	2/16	1/16

TABLE B7. RUTTING DEPTHS -- POZZOLANIC SECTION

DATE: 10-11-90

STATION	WESTBOUND LANE RWP (in.)	WESTBOUND LANE LWP (in.)	EASTBOUND LANE LWP (in.)	EASTBOUND LANE RWP (in.)
53+00	2/16	2/16	2/16	3/16
53+50	3/16	3/16	2/16	1/16
54+00	3/16	2/16	2/16	3/16
54+50	2/16	2/16	3/16	2/16
55+00	2/16	3/16	3/16	2/16
55+50	3/16	2/16	2/16	2/16
56+00	2/16	3/16	2/16	1/16
56+50	4/16	4/16	2/16	1/16
57+00	3/16	3/16	2/16	2/16
57+50	3/16	3/16	2/16	0/16
58+00	4/16	5/16	3/16	1/16
58+50	4/16	4/16	4/16	1/16
59+00	3/16	4/16	4/16	1/16
59+50	4/16	4/16	3/16	3/16

TABLE B8. RUTTING DEPTHS -- POZZOLANIC SECTION

DATE: 08-09-91

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
53+00	3/16	3/16	4/16	3/16
53+50	4/16	4/16	5/16	2/16
54+00	3/16	3/16	3/16	3/16
54+50	5/16	3/16	3/16	2/16
55+00	4/16	3/16	3/16	2/16
55+50	4/16	2/16	4/16	3/16
56+00	4/16	3/16	4/16	2/16
56+50	5/16	5/16	4/16	3/16
57+00	5/16	4/16	3/16	2/16
57+50				
58+00				
58+50				
59+00				
59+50				

TABLE B9. RUTTING DEPTHS -- AFBC STABILIZED POND ASH SECTION

DATE: 10-03-89

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
60+50	6/16	2/16	2/16	6/16
61+00	4/16	2/16	1/16	3/16
61+50	4/16	3/16	2/16	4/16
62+00	7/16	5/16	1/16	5/16
63+00	7/16	4/16	2/16	4/16
63+50	9/16	4/16	2/16	4/16
64+50	8/16	2/16	2/16	2/16
65+00	8/16	3/16	1/16	4/16
65+50	6/16	4/16	2/16	5/16
66+00	5/16	2/16	2/16	3/16
66+50	5/16	5/16	2/16	3/16
67+00	8/16	4/16	1/16	2/16
67+50	10/16	3/16	3/16	8/16

TABLE B10. RUTTING DEPTHS -- AFBC STABILIZED POND ASH SECTION

DATE: 02-08-90

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
60+50	8/16	4/16	3/16	9/16
61+00	6/16	2/16	3/16	5/16
61+50	5/16	4/16	2/16	5/16
62+00	7/16	6/16	2/16	5/16
63+00	6/16	5/16	2/16	5/16
63+50	12/16	4/16	3/16	7/16
64+50	10/16	2/16	3/16	2/16
65+00	8/16	3/16	2/16	6/16
65+50	6/16	5/16	2/16	7/16
66+00	5/16	3/16	4/16	4/16
66+50	5/16	4/16	2/16	4/16
67+00	9/16	4/16	2/16	4/16
67+50	12/16	4/16	1/16	12/16

TABLE B11. RUTTING DEPTHS -- AFBC STABILIZED POND ASH SECTION

DATE: 07-19-90

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
60+50	12/16	6/16	4/16	13/16
61+00	6/16	4/16	3/16	6/16
61+50	6/16	4/16	3/16	6/16
62+00	10/16	7/16	2/16	9/16
63+00	8/16	6/16	3/16	5/16
63+50	8/16	4/16	4/16	8/16
64+50	12/16	4/16	3/16	4/16
65+00	11/16	4/16	2/16	10/16
65+50	9/16	6/16	1/16	6/16
66+00	6/16	4/16	3/16	5/16
66+50	8/16	6/16	2/16	4/16
67+00	12/16	5/16	3/16	5/16
67+50	12/16	5/16	1/16	7/16

TABLE B12. RUTTING DEPTHS -- AFBC STABILIZED POND ASH SECTION

DATE: 10-11-90

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
60+50	8/16	6/16	4/16	4/16
61+00	5/16	4/16	3/16	4/16
61+50	6/16	6/16	3/16	4/16
62+00	6/16	8/16	2/16	6/16
63+00	6/16	5/16	4/16	4/16
63+50	10/16	5/16	4/16	5/16
64+50	8/16	4/16	3/16	4/16
65+50	6/16	6/16	3/16	6/16
66+00	6/16	5/16	4/16	3/16
66+50	7/16	6/16	3/16	4/16
67+00	9/16	5/16	3/16	3/16

TABLE B13. RUTTING DEPTHS -- CONTROL SECTION

	WESTBOUND LANE		EASTBOUND LANE	
	RWP	LWP	LWP	RWP
STATION	(in.)	(in.)	(in.)	(in.)
68+00	3/16	2/16	2/16	2/16
68+50	5/16	4/16	2/16	2/16
69+00	6/16	4/16	2/16	2/16
69+50	5/16	5/16	2/16	2/16
70+00	6/16	4/16	2/16	1/16
70+50	7/16	5/16	2/16	1/16
71+00	5/16	5/16	2/16	1/16
71+50	5/16	2/16	1/16	1/16
72+00	3/16	2/16	1/16	1/16
72+50	3/16	2/16	2/16	3/16
73+00	2/16	4/16	1/16	3/16
73+50	3/16	4/16	2/16	2/16
74+00	3/16	4/16	1/16	2/16
74+50	2/16	3/16	1/16	2/16

TABLE B14. RUTTING DEPTHS -- CONTROL SECTION

DATE: 02-08-90

STATION	WESTBOUND LANE RWP (in.)	WESTBOUND LANE LWP (in.)	EASTBOUND LANE LWP (in.)	EASTBOUND LANE RWP (in.)
68+00	4/16	3/16	2/16	2/16
68+50	6/16	5/16	2/16	2/16
69+00	7/16	5/16	2/16	1/16
69+50	5/16	5/16	2/16	2/16
70+00	7/16	5/16	2/16	1/16
70+50	7/16	5/16	3/16	2/16
71+00	6/16	6/16	2/16	2/16
71+50	5/16	2/16	1/16	1/16
72+00	3/16	3/16	1/16	2/16
72+50	3/16	2/16	2/16	3/16
73+00	3/16	5/16	2/16	4/16
73+50	4/16	5/16	4/16	2/16
74+00	4/16	5/16	2/16	2/16
74+50	3/16	3/16	2/16	1/16

TABLE B15. RUTTING DEPTHS -- CONTROL SECTION

DATE: 07-19-90

STATION	WESTBOUND LANE RWP (in.)	WESTBOUND LANE LWP (in.)	EASTBOUND LANE LWP (in.)	EASTBOUND LANE RWP (in.)
68+00	6/16	3/16	3/16	4/16
68+50	7/16	6/16	2/16	2/16
69+00	6/16	5/16	2/16	1/16
69+50	6/16	6/16	2/16	1/16
70+00	8/16	6/16	2/16	1/16
70+50	8/16	5/16	2/16	2/16
71+00	7/16	7/16	1/16	1/16
71+50	6/16	2/16	1/16	1/16
72+00	3/16	2/16	1/16	1/16
72+50	4/16	3/16	2/16	3/16
73+00	4/16	4/16	3/16	4/16
73+50	4/16	6/16	3/16	2/16
74+00	5/16	5/16	2/16	3/16
74+50	3/16	3/16	3/16	1/16

TABLE B16. RUTTING DEPTHS -- CONTROL SECTION

DATE: 10-11-90

	WESTBOUND LANE		EASTBOUND LANE	
STATION	RWP (in.)	LWP (in.)	LWP (in.)	RWP (in.)
68+00	6/16	5/16	3/16	4/16
68+50	6/16	7/16	3/16	3/16
69+00	6/16	5/16	3/16	3/16
69+50	7/16	7/16	3/16	3/16
70+00	6/16	5/16	2/16	2/16
70+50	10/16	6/16	3/16	3/16
71+00	7/16	7/16	3/16	4/16
71+50	6/16	4/16	2/16	2/16
72+00	4/16	5/16	2/16	3/16
72+50	4/16	4/16	3/16	5/16
73+00	4/16	6/16	4/16	5/16
73+50	4/16	5/16	5/16	4/16
74+00	4/16	5/16	2/16	3/16
74+50	3/16	4/16	3/16	3/16
75+00	4/16	4/16	3/16	1/16

TABLE B17. RUTTING DEPTHS -- CONTROL SECTION

DATE: 08-09-91

	WESTBOUND LANE		EASTBOUND LANE	
STATION	RWP (in.)	LWP (in.)	LWP (in.)	RWP (in.)
68+50	7/16	9/16	4/16	5/16
69+00	7/16	7/16	4/16	3/16
69+50	6/16	6/16	4/16	3/16
70+00	8/16	7/16	3/16	3/16
70+50	10/16	8/16	4/16	4/16
71+00	9/16	9/16	3/16	4/16
71+50	7/16	5/16	3/16	2/16
72+00	4/16	5/16	3/16	4/16
72+50	4/16	4/16	3/16	5/16
73+00	4/16	7/16	5/16	7/16
73+50	5/16	7/16	5/16	4/16
74+00	5/16	6/16	4/16	4/16
74+50	3/16	4/16	6/16	4/16
75+00	5/16	4/16	6/16	3/16

APPENDIX C

Results of Physical Tests of Field Specimens

KY 3074, Bleich Road

AFBC CONCRETE BASE SECTION

STATION 45+00 TO STATION 52+50

TABLE C1. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN AFBC CONCRETE SECTION, JULY 1988.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
45+50 RT (2nd lift)	3.99	4.50	-	-	-	-	NOT EVALUATED		
45+50 RT (1st lift)	3.99	2.75	-	-	-	-	NOT EVALUATED		
46+50 LT	4.01	8.00	-	-	-	-	NOT EVALUATED		
48+00 RT	3.97	7.50	3.97	6.80	6.8	139.6	2,200	-	-
48+00 LT	3.97	7.50	3.97	6.24	6.4	143.2	3,710	-	-
49+50 CL	3.98	8.00	3.98	6.62	6.8	142.7	4,070	-	-
50+50 LT	3.99	7.50	3.99	6.39	6.5	140.6	3,250	-	-

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratio. Specimens from core location 45+50 RT were delaminated and were not evaluated. The specimen obtained for STA 46+50 LT contained a large pop-out, or void on the side and was not evaluated.

TABLE C2. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN AFBC CONCRETE SECTION, OCTOBER 1989.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
45+75 RT (2nd lift)	3.99	3.25	3.99	2.30	2.4	144.2			(+) 2.4
45+75 RT (1st lift)	3.99	4.58	3.99	3.67	3.9	144.5			(+) 1.5
46+00 LT	4.01	8.32	4.01	7.25	7.7	142.8	3,080		
47+00 RT	3.99	8.80	3.99	8.18	8.6	143.1	3,840	1.96	
48+25 LT	4.00	10.10	4.00	8.90	9.4	142.8	3,680	2.25	
49+50 LT	4.01	8.59	4.01	7.50	7.6	139.4	NOT EVALUATED		
51+00 RT	3.99	6.94	3.99	5.25	5.5	143.3	3,330		

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratios. Specimens from core location 45+75 RT were evaluated for freeze/thaw durability for a period of 15 cycles. The freeze thaw durability test was performed in general accordance with ASTM C-666, Method B, which requires freezing the specimen in air and thawing in water. The weight loss or gain shown in the table above was determined after 15 cycles.

TABLE C3. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN AFBC CONCRETE SECTION, JULY 1990.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
45+50 LT	-	-	-	-	-	-	NOT EVALUATED		
46+00 RT	6.00	7.75	4.01	6.68	7.0	143.3	3,630	-	-
46+50 LT	4.00	7.05	4.00	5.65	5.7	139.1	2,910	-	-
46+75 RT	5.99	6.25	4.02	5.20	5.2	136.1	1,300	-	-
48+00 RT (2nd lift)	3.99	3.50	-	-	-	-	NOT EVALUATED		
48+00 RT (1st lift)	3.99	4.75	-	-	-	-	NOT EVALUATED		
48+50 RT (2nd lift)	4.00	3.75	-	-	-	-	NOT EVALUATED		
48+50 RT (1st lift)	4.00	4.75	-	-	-	-	NOT EVALUATED		
49+00 LT (2nd lift)	3.99	4.00	-	-	-	-	NOT EVALUATED		
49+00 LT (1st lift)	3.99	4.50	-	-	-	-	NOT EVALUATED		
51+00 LT	4.01	6.75	4.01	5.63	5.7	137.7	3,340	-	-
51+50 LT (2nd lift)	4.00	5.00	-	-	-	-	NOT EVALUATED		
51+50 LT (1st lift)	4.00	3.00	-	-	-	-	NOT EVALUATED		

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratios. Specimens that separated between lifts were not evaluated. The specimen obtained at STA 46+75 contained a vertical crack in the specimen. It was concluded that the specimen was in a weakened condition prior to testing for compressive strength.

TABLE C4. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN AFBC CONCRETE SECTION, AUGUST 1991.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
46+75 LT (2nd lift)	4.00	4.00	1.98	3.86	1.0	136.7	920	-	-
46+75 LT (1st lift)	4.00	4.50	1.98	3.91	1.0	138.6	1,220	-	-

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratios.

POZZOLANIC BASE SECTION

STATION 52+50 TO STATION 60+00

TABLE C5. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN POZZOLANIC BASE SECTION, JULY 1988.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
53+00 RT	4.00	8.01	4.00	5.53	5.8	144.2	1,790	-	-
55+50 CL	4.00	8.35	4.00	7.81	8.2	144.4	1,580	-	-
59+00 LT	4.00	8.48	4.00	5.34	5.6	144.2	1,250	-	-

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratio.

TABLE C7. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN POZZOLANIC BASE SECTION, OCTOBER 1990.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
56+50 LT	4.00	8.31	4.00	7.13	7.4	143.2	1,570	-	-
57+00 RT	4.00	6.92	4.00	5.01	5.2	144.0	2,430	-	-
57+50 RT	4.00	8.26	4.00	6.81	7.2	145.2	2,040	-	-
58+50 LT	4.00	7.72	4.00	6.59	7.0	145.3	1,750	-	-
59+50 LT	4.00	5.98	4.00	4.26	4.5	145.5	-	-	(-) 19.2

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratios. The specimen from core location STA 59+50 LT was evaluated for freeze/thaw durability over a period of 52 cycles. The freeze and thaw durability test was performed in general accordance with ASTM C-666, Method B, which requires freezing the specimen in air and thawing water. The weight loss or gain shown in the table above was determined after 52 cycles.

AFBC STABILIZED POND ASH SUBBASE SECTION

STATION 60+00 TO STATION 67+50

TABLE C8. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN AFBC STABILIZED POND ASH SUBBASE SECTION, JULY 1988.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
62+00 LT (2nd lift, core II)	6.00	6.32	1.99	3.96	0.8	115.8	3,860	-	-
62+00 LT (1st lift, core I)	6.00	5.87	1.98	3.98	0.8	115.4	3,200	-	-
63+50 CL (2nd lift, core I)	6.00	6.18	1.98	4.02	0.8	112.1	2,080	-	-
63+50 CL (2nd lift, core II)	6.00	6.18	1.98	3.88	0.8	111.7	2,040	-	-
66+75 RT (2nd lift, core I)	6.00	6.00	1.99	4.01	0.8	114.7	1,960	-	-
66+75 RT (2nd lift, core II)	6.00	6.00	1.99	4.06	0.8	114.3	2,040	-	-
66+75 RT (1st lift, core I)	6.00	5.76	1.99	3.93	0.8	116.2	1,910	-	-
66+75 RT (1st lift, core II)	6.00	5.76	1.99	3.98	0.8	115.9	1,920	-	-

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratios. Six-inch diameter field core specimens were obtained from the experimental subbase at three stations. All of the specimens exhibited delamination between the 1st and 2nd lifts. In order to evaluate the compressive strength of the field specimens, it was necessary to extract two-inch diameter cores in the laboratory from the six-inch diameter cores.

TABLE C9. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN AFBC STABILIZED POND ASH SUBBASE SECTION, OCTOBER 1989.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
60+50 LT (2^{nd} lift)	3.98	3.27	3.98	2.91	2.4	114.5	-	-	(+) 1.7
60+50 LT (1^{st} lift)	3.99	10.61	3.99	9.86	8.4	118.4	3,060	1.30	-
61+00 RT (2^{nd} lift)	4.00	7.27	4.00	6.40	5.5	118.9	3,920	-	-
61+00 RT (1^{st} lift)	3.87	2.70	3.87	1.80	1.4	115.1	-	-	OUT
63+00 RT (2^{nd} lift)	4.00	4.20	4.00	3.75	3.0	109.6	NOT EVALUATED		
63+00 RT (1^{st} lift)	3.99	7.26	3.99	6.84	5.6	112.7	3,840	-	-
63+50 LT (2^{nd} lift)	4.00	2.60	4.00	2.16	1.7	110.8	-	-	(+) 0.4
63+50 LT (1^{st} lift)	4.00	7.18	4.00	6.69	5.5	112.5	3,910	-	-
65+50 LT	4.00	9.11	4.00	8.63	7.1	113.8	NOT EVALUATED		
66+00 RT (2^{nd} lift)	4.00	5.25	4.00	4.65	3.8	112.2	4,190	-	-
66+00 RT (1^{st} lift)	4.00	4.48	4.00	3.88	3.1	111.1	-	-	(+) 0.9

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratios. Specimens from core locations STA 60+50 LT (2^{nd} lift), STA 61+00 RT (1^{st} lift), 63+50 LT (2^{nd} lift), and STA 66+00 RT (1^{st} lift) were monitored for freeze/thaw durability for a period of 15 cycles. The freeze and thaw durability test was performed in general accordance with ASTM C-666, Method B, which requires freezing the specimen in air and thawing in water. The weight loss or gain shown in the table above was determined after 15 cycles.

TABLE C10. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN AFBC STABILIZED POND ASH SUBBASE SECTION, OCTOBER 1990.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
64+00 LT	4.00	9.88	-	-	-	-	-	-	-
64+50 RT	4.00	10.01	4.00	6.31	5.4	118.3	1,950	-	-
66+50 RT	4.00	10.49	4.00	4.44	3.7	115.7	-	-	(-) 35.6
67+00 LT (1" lift)	4.00	4.57	4.00	3.80	3.1	111.7	-	-	(-) 76.5

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratios. The specimen from core location STA 64+00 LT was obtained in a milled area and was retrieved in three very deteriorated pieces. KTC personnel did not identify the pieces relative to their position in the pavement. The specimen from STA 66+50 RT separated between lifts and the separate pieces were not identified. The listed core lengths for these two cores is the total length of the recovered material. Specimens 66+50 RT and 67+00 LT were evaluated for freeze/thaw durability over a period of 25 cycles. The freeze and thaw durability test was performed in general accordance with ASTM C-666, Method B which requires freezing in air and thawing in water. The weight loss or gain shown in the table above was determined after 25 cycles.

TABLE C11. RESULTS OF TESTS PERFORMED ON FIELD CORES OBTAINED IN AFBC STABILIZED POND ASH SUBBASE SECTION, AUGUST 1991.

Sample Location	Core Diameter (in.)	Core Length (in.)	Test Diameter (in.)	Test Length (in.)	Test Weight (lbs)	Density (pcf)	Compressive Strength (psi)	Elastic Modulus (psi x 10^6)	Freeze Thaw Weight Gain/Loss (%)
60+25 RT (1st lift)	4.00	7.50	1.99	3.57	0.7	116.5	2,920	-	-
60+25 CL (1st lift)	4.00	7.50	1.99	3.37	0.7	117.7	1,760	-	-
61+50 RT (1st lift)	4.00	6.44	1.99	3.87	0.8	120.4	2,700	-	-
62+05 RT (1st lift)	4.00	5.75	1.98	4.06	0.8	115.4	1,500	-	-
62+50 RT (1st lift)	4.00	5.56	1.98	4.06	0.8	115.1	1,220	-	-
62+75 LT (1st lift)	4.00	6.25	1.99	3.89	0.8	116.1	3,480	-	-
63+50 RT (2nd lift)	4.00	5.50	1.98	3.98	0.9	117.6	2,460	-	-
65+50 RT (2nd lift)	4.00	5.50	1.99	3.88	0.8	116.2	1,880	-	-
66+00 RT (2nd lift)	4.00	6.00	1.98	4.11	0.9	118.6	2,250	-	-
66+00 RT (1st lift)	4.00	5.00	1.99	4.00	0.8	116.8	1,940	-	-

NOTE: Field core specimens were obtained and tested in general accordance with ASTM C 42. Compressive strengths were corrected for improper length to diameter ratios.