Research Report
KTC-89-32

DEMONSTRATION PROJECT NO. 59
THE USE OF FLY ASH IN HIGHWAY CONSTRUCTION

Fayette County, Kentucky

by

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in cooperation with
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16. Abstract

This report summarizes preliminary site investigations, design criteria and procedures, construction criteria and procedures, and results of initial and final evaluations of an experimental road base utilizing lime kiln dust, fly ash and aggregate. Observed performance has been satisfactory. Compressive strength evaluations indicate exceptional early strength and continued strength gain of the pozzolanic base. Back calculation of specific elastic layer moduli indicate that the moduli of the pozzolanic base material is most likely within a range of 500,000 to 1,000,000 psi.

Based upon observations to date, treated bases enhance overall pavement performance and their use should be continued. It is quite probable that pavement life may be extended at reduced costs.

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Fly Ash, Pozzolanic, Lime, Lime Kiln Dust, Base, Pavement, Highway, Transportation

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a.	fly ash	8.0 percent
b.	lime kiln dust	8.0 percent
c.	crushed limestone aggregate	84.0 percent

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INTRODUCTION AND BACKGROUND

The primary objective of this study was to demonstrate the use of a pozzolanic stabilized aggregate base as an alternate base construction wherein the added structural qualities of the stabilized base material would reduce thickness requirements for higher quality asphaltic concrete materials. Two secondary objectives of the study were generation of a data base for continued development of procedures for design and evaluation of pozzolanic base materials, and conservation of materials associated with construction of a highway pavement.

Kentucky has traditionally been among the leading producers of coal. Coal is used for the generation of electricity and, as such, an abundance of by-product materials are produced at coal-fired electric generating plants. A principle by-product material is fly ash. Fly ash has been used effectively in combination with other materials for various highway construction applications and is an alternative material source for use in highway construction. The availability of higher quality aggregates is becoming critical. Escalating costs associated with the production of high quality aggregates warrants the use of alternative materials. Pozzolanic stabilized bases apparently offer an economical alternative for pavement base construction.

For Kentucky's fly ash demonstration project, the pozzolanic stabilized base material consisted of 10.0 inches of a mixture of 8% fly ash, 8% lime kiln dust, and 84% limestone aggregate material. A bituminous curing membrane material was applied to the base course immediately after placement to facilitate curing by slowing moisture loss and facilitating the absorption of heat. A minimum compressive strength of 600 psi was required at 7 days when samples were prepared and cured according to Kentucky Department of Highways' Special Provision 70(79) "Fly Ash Stabilized Bases" (Appendix A). A bituminous stress-absorbing membrane interlayer consisting of 3/8-inch size aggregate, emulsified asphalt, and a polymer additive was applied with the expectation that reflective cracking in the asphaltic concrete surfacing would be minimized. A thickness of 4 inches asphaltic concrete was placed as the final phase of pavement construction.

The demonstration project is a 1.7-mile section of Man O'War Boulevard in southern Fayette County and extends from Nicholasville Road (US 27) eastward to Tates Creek Pike (KY 1974). Fayette County is located in the heart of the bluegrass region of Kentucky. Climate in the area is generally mild with average summer temperatures of 76 ^{O}F and average winter temperatures of 37^{O}F. Typical yearly rainfall in the region is 44 inches while yearly snowfall is typically about 14 inches.

HIGHWAY DESIGN SPECIFICS

The design average daily traffic is 14,000 vehicles per day with approximately 2 percent trucks. The highway consists of two 12-foot lanes with turning lanes where needed. Terrain over the 1.7-mile section is generally flat to slightly rolling. Concrete box curbs parallel the pavement for this urban section. The highway is crowned with curb and gutter drainage facilities located on either side of the roadway. Four alternate pavement designs were included in the bid proposal. Alternate pavement thickness designs were determined on the basis of 500,000 Equivalent Axleloads and a CBR 5 subgrade material. The alternate pavement thickness designs are summarized in Table 1.

TABLE 1: SUMMARY OF ALTERNATE PAVEMENT THICKNESS DESIGNS

	Pavement Layer Components			
Alternate	Asphaltic Concrete (in.)	Crushed Stone (in.)	Portland Cement Concrete (in.)	Pozzolanic Base Material (in.)
No. 1	----	3.00	8.00	-----
No. 2	6.50	12.50	----	-----
No. 3	9.25	-----	----	-----
No. 4	4.00	-----	----	10.00

Materials Information

Class "F" fly ash was obtained from Kentucky Utilities Company's E. W. Brown Station located in Burgin, Kentucky. The fly ash source was located within 40 miles of the project site. Typically, the loss on ignition ranges from 1.71 to 2.91. The fineness by the 325 sieve is generally within the range of 20 to 30 percent with a specific gravity of 2.44. Chemical composition of the Class "F" fly ash is presented in Table 2.

Lime kiln dust (baghouse lime) was supplied by Dravo Lime Company located near Maysville, Kentucky. The lime kiln dust source was located within 100 miles of the project site. Specifics describing the characteristics of the lime kiln dust are listed in Table 3.

Limestone aggregate materials were supplied by Lexington Quarry Company located in Nicholasville, Kentucky. The limestone aggregate source is located within 10 miles of the project site. Aggregate properties are summarized in Table 4.

TABLE 2: CHEMICAL ANALYSIS OF FLY ASH - CLASS "F"

		Sample A 9/23/83 (%)	Sample B 9/12/84 (%)	Sample C 11/28/84 (%)
Moisture		0.29	0.0	0.10
Loss on Ignition		1.71	1.96	2.91
Fineness		26.5	29.4	28.8
Specific Gravity		2.43	2.45	2.42
Aluminum Oxide	Al_2O_3	24.19	22.65	27.64
Iron Oxide	Fe_2O_3	17.73	18.46	17.19
Calcium Oxide	CaO	0.59	0.03	0.01
Magnesium Oxide	MgO	0.53	1.09	1.28
Sodium Oxide	Na_2O	0.58	3.16	7.57
Silicon Dioxide	SiO_2	51.36	47.09	50.86
Potassium Oxide	K_2O	2.33	2.70	2.77
Sulfur Trioxide	SO_3	0.28	0.52	0.54
Phosphorus Pentoxide	P_2O_5	0.10	0.35	0.40
Titanium Dioxide	TnO_2	0.98	1.25	1.21

TABLE 3: CHEMICAL ANALYSIS OF BAGHOUSE LIME KILN DUST

		(Percent)
Moisture		0.2
Loss on Ignition		N/A
Fineness		22.5
Calcium Carbonate	$CaCO_3$	51.9
Calcium Oxide	CaO	23.1
Magnesium Oxide	MgO	4.1
Sulfur	S	1.8
Silicon Dioxide	SiO_2	8.7
Iron Oxide	Fe_2O_3	0.5
Aluminum Oxide	Al_2O_3	2.2

TABLE 4: PROPERTIES OF LIMESTONE AGGREGATE

Gradation		Physical Characteristics	
Sieve	Percent Passing		
2"	---	Specific Gravity	2.69
1-1/2"	---	Absorption %	1.10
1"	100	L. A. Wear (500)	31
3/4"	98	Sand Equivalent Value	4.6
1/2"	---	Minus 200 Wash	9.8
3/8"	70		
No. 4	51		
No. 8	---		
No. 10	35		
No. 16	---		
No. 30	---		
No. 40	17		
No. 50	---		
No. 100	---		
No. 200	10		

Mixture Designs

Lime Kiln Dust-Fly Ash-Aggregate

Mixture design requirements are summarized in Kentucky Department of Highways' Special Provision 70(79) "Fly Ash Stabilized Bases" (Appendix A) (1). Preliminary mixture design analyses using procedures outlined in Appendix A indicated a mixture of 8% fly ash, 8% lime kiln dust, and 84% limestone aggregate (by weight) would result in a minimum compressive strength of 600 psi when cured for 7 days at 100°F in a sealed container. Specimens for evaluation of compressive strength and modulus of elasticity were prepared in general accordance with ASTM C 593(79) in 4-inch by 4.6-inch molds (2). Deviations from that method involve the use of a 5.5-lb. hammer and a 12-inch free fall instead of the specified 10-lb. hammer and 18-in. drop.

Materials submitted to this laboratory for analyses indicated a maximum dry density of 145.0 pcf and optimum moisture content of 6.6% for the proposed design when tested in accordance with ASTM D 698(79) (3). Maximum dry density and optimum moisture content were determined using a polynomial curve fitting procedure. A smoothing technique was used to eliminate localized changes in concavity. Additional specifics relating to preparation and testing of specimens are presented in Appendix A "Special Provision-Fly Ash Stabilized Bases". Unconfined compressive strengths were determined in accordance with ASTM C 39(72) (4). The static chord elastic modulus was determined by method ASTM C 469(83) (5). Information gained through initial laboratory study of base mixture materials is presented in Table 5.

TABLE 5: SUMMARY OF COMPRESSIVE STRENGTHS AND ELASTIC MODULI OF LABORATORY SAMPLES

Sample Number	Compressive Strength (psi)	Elastic Modulus (psi x 10^5)	Curing Conditions
1-2	2,565	2.74	7 days oven at 100°F
1-3	2,765	2.15	in sealed container, vacuum
1-4	2,625	1.83	saturated prior to testing
1-5	2,635	2.73	7 days oven at 100°F
1-6	2,760	3.01	in sealed container, soaked
1-7	1,630	1.86	prior to testing
1-8	2,775	2.07	14 days (7 oven at 100°F,
1-9	2,980	2.04	in sealed container, 7 days
1-10	2,725	4.78	air at ambient temperatures)
1-11	890	0.83	14 days at ambient temperature
1-12	940	0.68	in sealed bags, soaked prior
1-13	860	1.39	to testing
2-1	1,975	1.26	7 days oven at 100°F in
2-2	2,085	1.42	sealed container, soaked
2-3	1,970	1.21	prior to testing
2-4	2,075	1.29	7 days oven at 100°F in
2-5	2,125	1.47	sealed container, soaked
2-6	2,040	0.96	prior to testing
2-7	1,790	1.12	7 days oven at 100°F in
2-8	2,100	0.76	sealed container, soaked
2-9	2,040	1.35	prior to testing

Asphaltic Concrete

Asphalt cement for this project was supplied by the Ashland Petroleum Company, Louisville, Kentucky facility. All asphalt cement materials were AC-20 grade. The asphaltic concrete was placed in three courses: base, binder, and surfacing. The job mix formula and design asphalt contents for these courses are summarized in Table 6.

TABLE 6: ASPHALTIC CONCRETE PAVEMENT MIXTURE DESIGN INFORMATION

	Job Mix Formulas								
	Base			Binder			Surface		
Sieve Size	AC%	min	max	AC%	min	max	AC%	min	max
1-1/2"	5.0	100		5.2			5.6		
3/4"		86	98		100			100	
3/8"		53	63		65	75		87	97
No. 4		42	52		46	56		67	77
No. 8		28	38		35	45		50	60
No. 16		14	24		25	35		35	45
No. 50		5	13		5	13		5	13
No. 100		4	8		3	7		3	7
No. 200								25	55

Pavement Thickness Design Procedures

Thickness design procedures for flexible pavements in Kentucky have been developed on the basis of a limiting strain-repetitions criterion (6). The flexible pavement criterion limits the vertical compressive strain at the top of the subgrade and the tensile strain at the bottom of the asphaltic concrete. Preliminary analyses have indicated elastic layer concepts may also be applied for thickness design of pozzolanic bases (7). Thickness design requirements for the pozzolanic base alternate presented in Table 1 were determined by using the Kentucky flexible pavement design procedure to determine thickness requirements using conventional materials (asphaltic concrete and crushed limestone). AASHTO structural coefficients $a_1 = 0.44$ for asphaltic concrete and $a_2 = 0.14$ for crushed stone were used to determine the structural number for the conventional design determined from the Kentucky procedure (8). The structural number (SN) was then used in combination with the AASHTO design equation $SN = [a_1 \times d_1] + [a_2 \times d_2]$ and structural coefficients for asphaltic concrete $a_1 = 0.44$ and $a_2 = 0.28$ for pozzolanic base material to determine the thickness requirement for the pozzolanic base material. The results of this analysis are summarized in Table 1.

CONSTRUCTION PROCEDURES

The project, Fayette County SSP 034-7229-60-C, is a combined grade and drain, and surfacing project. The contract was awarded to Carey Construction Inc. and CHW Construction Inc., both of Lexington, as a joint venture on September 20, 1983. Grading and drainage work were initiated during September 30, 1983.

Preparation of the soil subgrade was completed in September 1984. Subgrade moisture, density and laboratory and in-situ California Bearing Ratio (CBR) information are contained in Table 7. Subgrade density measurements and moisture determinations were made using nuclear instruments. Values of CBR's of remolded field samples were obtained in accordance with Kentucky Method 64-501-80, "Kentucky Soaked CBR" (9). Measurement of in-situ subgrade bearing strength was in general accordance with ASTM D 1883(73) (10), except that the tests were performed on the soil in its actual in-situ condition. Moisture content of the soil for this test was determined in accordance with ASTM D 2216(80) (11), "Laboratory Determinations of Water (Moisture) Content of Soil, Rock and Soil-Aggregate Mixtures."

The fly ash stabilized base material was blended in a continuous volumetric-proportion pugmill. Lime kiln dust was fed dry from a silo onto an aggregate belt. Conditioned fly ash was stockpiled without protection and loaded into the feeder bin at the prevailing moisture content. Occasional, problems were encountered with fly ash clumping which sometimes prevented uniform flow through the bin opening. Dense graded limestone aggregate materials were also loaded into the feeder bin at prevailing moisture contents. The amount of mixing water required for optimum conditions was computed and the proper amount of water required for blending was added accordingly. The blended base material was transported approximately 10 miles to the paving site by dump truck.

The pozzolanic base materials were end dumped into and spread by a conventional aggregate spreader box pushed by a small bulldozer. A motor grader was used to distribute the base material around preformed concrete curb and gutter at intersecting roads. The 10.0 inches of base material was placed in two equal lifts and compacted using steel-wheeled vibratory rollers having a minimum weight of 10 tons (see Figure 1). Although it could not be verified, reportedly some portion of the pozzolanic base material was placed in one 10-inch lift. Calculated material quantities used in the production of the lime kiln dust-fly ash-aggregate mixture are listed in Table 8.

TABLE 7: SUMMARY OF SUBGRADE CONSTRUCTION INFORMATION

Station Number	Dry Density (pcf)	Moisture Content (%)	California Bearing Ratio Field (%)	California Bearing Ratio Laboratory (%)
404+50	99.4	22.1		
405+00	95.0	21.9	10.0	29.0
409+00	97.0		7.0	
409+50	97.3	25.8		
414+50	103.5	23.5		
415+00	100.0	15.0	6.5	27.0
417+50	105.5	23.0		
419+50	99.0	24.0		
422+50	106.7	20.7		
423+00	102.1	23.4		
425+00	106.0	14.0	9.0	41.0
427+50	101.0	24.4		
428+00	96.7	23.4		
430+00	99.4	23.7		
435+00	100.0	12.7	10.0	41.0
437+50	107.2	17.4		
438+00	95.7	23.4		
445+00	106.0		8.0	
447+50	105.0	22.7		
448+00	107.5	15.0		
451+00	98.0	21.0	7.0	
453+00a	101.0	18.8		
453+00b	101.6	20.9		
455+00	101.0		9.0	
458+00a	91.4	21.5		
458+00b	101.8	23.1		
463+00	95.5	22.0		
465+00	93.0	16.3	7.0	33.0
468+00	95.0	22.2		
472+00	113.0	16.0	2.0	
473+00	103.5	15.4		
474+50		10.5		92.0
475+00	107.0	18.0	3.0	
477+50	108.0	18.0	3.5	
478+00	100.4	20.7		
481+00	99.0	24.0	5.0	
482+50	104.0	17.0	2.5	
483+00	101.7	17.8		
483+90	99.6	17.1		
485+80	97.2	24.5		
486+00	105.0	20.1	10.0	21.0
489+00	99.0	22.0	4.0	
489+50	108.4	17.3		

TABLE 8: BASE MATERIAL QUANTITIES

Material	Quantity (tons)	Percent of Total (%)
Lime	1,484.50	7.95
Fly ash	1,486.17	7.96
Aggregate	15,704.69	84.09
Totals	18,675.36	100.00

A compaction requirement of 102% of laboratory dry density (AASHTO T-99)(12) was specified for the pozzolanic base. During construction, fewer than 30% of tests indicated this requirement was met. A mean average density of 100.3% of the 145.0 pcf with a standard deviation of 1.6% for the mean was obtained. A more reasonable standard may be 100% of laboratory density. A summary of construction density measurements is presented in Table 9. A motor grader was used to trim the base material to grade prior to placing the curing seal.

The consistency of the mixes appeared to be very good. Placement and compaction operations proceeded smoothly without any apparent difficulties. The base material was placed over a two week period.

An initial curing membrane of bituminous material was required within 24 hours after final compaction. Primer L was applied with an asphalt distributor at a rate of about 1.2 pounds per square yard (see Figure 2). Properties of Primer L are summarized in Table 10. A 2-1/2-hour time limit between mixing and completion of compaction was initially specified. This limit generally proved impractical. In some situations, the base remained plastic for two to three days after placement. This sometimes resulted in difficulties in placing curing membrane within the specified 24-hour period since the asphalt distributor would sometimes rut the plastic base.

After the demonstration project was under contract, reflective cracking was observed for a section of an earlier project where a similar base material had been used. That project involved a ramp and mainline segment of a local city street with construction being completed in October 1983. The earlier project also utilized a similar design (4.0 inches asphaltic concrete and 10.0 inches fly ash stabilized base). Cracking was first noticed in the spring of 1984 on the mainline pavement segment. Cracking was subsequently observed on the ramp section as well. The observed cracking occurred at somewhat regular intervals of 80 to 90 feet on the mainline segment. Cracking on the ramp was also observed on a somewhat regular interval in the order of 40 to 60 feet.

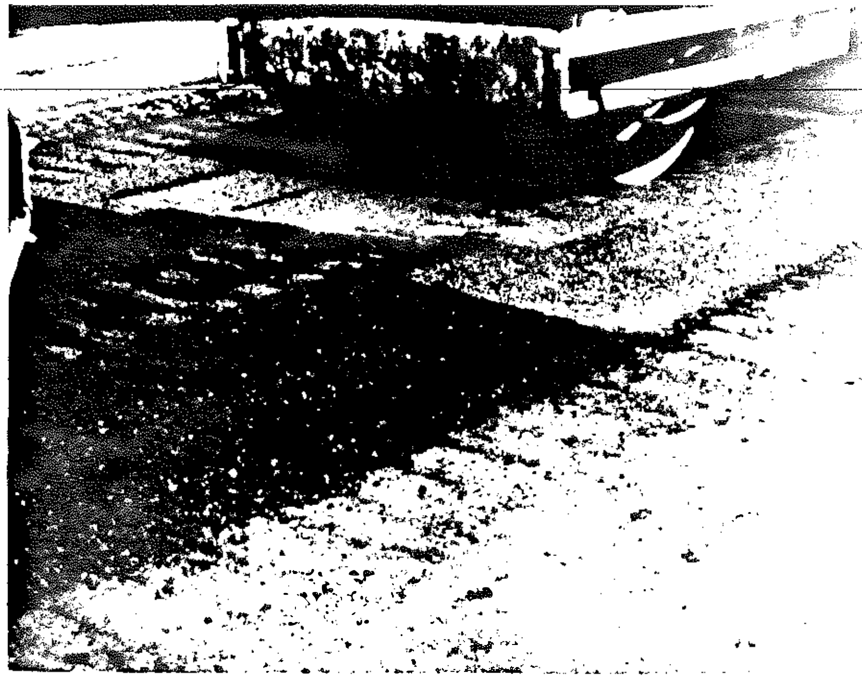

Figure 1. Compacting the Pozzolanic Base Material

Figure 2. Applying the Curing Seal

TABLE 9: SUMMARY OF DENSITIES FOR POZZOLANIC BASE CONSTRUCTION

Date	Station Number	Offset	Moisture Content (%)	Field Density (% of optimum)	In/ Out	Remarks
9/11/84	448+00	LT	-	101.8	In	
	440+00	RT	-	102.1	In	
	435+00	RT	-	100.5	Out	
	432+00	LT	-	101.7	In	
9/13/84	422+00	-	6.2	99.6	Out	all tests on
	416+00	-	6.7	100.1	Out	first course
	448+00	-	7.1	101.8	In	
	443+00	-	6.3	102.1	In	
	438+00	-	6.1	100.5	Out	
	433+00	-	6.4	101.7	In	
9/14/84	411+00	-	6.5	101.6	In	all tests on
	407+00	-	6.9	97.4	Out	first course
	403+00	-	7.4	100.6	Out	
9/19/84	488+50	LT	6.1	97.6	Out	first course
	486+00	LT	6.4	100.1	Out	first course
	489+00	RT	6.4	100.9	Out	second course
	489+00	LT	6.8	100.7	Out	second course
	486+50	LT	7.0	99.9	Out	first course
9/20/84	476+00	LT	6.5	101.1	Out	all tests on
	476+00	RT	7.0	100.1	Out	first course
	471+00	LT	6.4	100.3	Out	
	471+00	RT	6.4	101.6	In	
	466+00	RT	6.9	98.1	Out	
	466+00	LT	7.6	98.6	Out	
9/20/84	482+00	LT	6.5	103.0	In	all tests on
	482+00	RT	6.1	101.6	In	second course
	476+00	LT	6.6	101.0	Out	
	476+00	RT	6.8	99.9	Out	
	471+00	LT	6.4	99.7	Out	
	471+00	RT	6.5	100.0	Out	
	466+00	RT	6.9	98.1	Out	
	466+00	LT	6.6	101.0	Out	
9/21/84	461+00	RT	6.0	100.1	Out	
	461+00	LT	7.1	99.8	Out	
	457+00	LT	6.2	100.2	Out	
	457+00	RT	6.2	97.8	Out	
	453+00	RT	6.4	99.0	Out	
	453+00	LT	6.7	100.3	Out	
9/25/84	401+25	RT	6.2	104.6	In	first course
	400+50	LT	6.9	99.9	Out	first course
	402+00	RT	6.1	101.8	In	second course
	401+00	LT	6.2	100.6	Out	second course

TABLE 10: SUMMARY OF CHARACTERISTICS FOR BITUMINOUS CURING MEMBRANE

Primer L (Cut Back Asphalt Emulsion)

TEST	REQUIREMENTS Min.	Max.
Viscosity, Saybolt-Furol, AASHTO T 59 at 77°F, sec.	30	100
Water content,(AASHTO T 55 using Xylene), percent	3	8
Asphalt content (AASHTO T 78 using residue from water content determination and results of water content tests), percent	45	
Tests on residue from Distillation-Float Test (AASHTO T 50) at 122°F, sec.	80	
Wet Stone Coating (KM-64-414)	SHALL PASS	
Solubility in Trichloroethylene (AASHTO T 44), percent	97.5	
Recommended application temperature, degrees F	60	120

Because of the occurrence of reflective cracking at the earlier project, a change order was executed for application of a rubber-asphalt stress-absorbing membrane interlayer (SAMI). Scheduling difficulties prevented its use and, instead, a CRS-2S polymer emulsion was used with No. 9m limestone chips (3/8 inch maximum) to construct a stress relief layer approximately 1/2 inch thick. Application rate of the limestone chips was about 27 pounds per square yard. The stress relief layer was applied to the 10.0-inch pozzolanic base material from Station 400+06.50 to Station 437+78.25 and from Station 452+78.25 to Station 490+50.00. A 1,500-foot section in the center of the demonstration project did not receive the stress relief layer. Specifics relating to the polymer-asphalt emulsion CRS-2S are presented in Table 11.

TABLE 11: SUMMARY OF CHARACTERISTICS OF POLYMER-ASPHALT EMULSION CRS-2S

TEST	Min.	Max.
Viscosity, Saybolt-Furol, (AASHTO T 59) at 122°F, sec.	100	400
Storage Stability, 24 hrs., percent (AASHTO T 59)		1
Sieve test, 20 mesh, percent (AASHTO T 59)		0.1
Distillation: (AASHTO T 59) oil distillate by Vol. of Emulsion, percent		3.0
Residue from distillation, percent	65	
Penetration, 77°F, 100, 5 sec., mm. (AASHTO T 49)	125	200
Ductility, 77°F, cm. (AASHTO T 51)	50+	
Ductility, 34°F, cm. (AASHTO T 51)	15+	
Softening Point (Ring and Ball), °F (AASHTO T 102)	90	130
Solubility in Trichloroethylene, percent (AASHTO T 44)	97.5	
Demulsibility, 0.02 $CaCl_2$, percent (AASHTO T 59)	40+	
Demulsibility, 35 ml, 0.8% Sodium dioctyl Sulfosuccinate, percent (AASHTO T 59)	20+	

Asphalt pavement construction was begun approximately three weeks after placement of the pozzolanic base material. Asphaltic concrete was placed in three lifts: base, binder, and surface. Specifics relating to the mixture design for the various layers are presented in Table 6. Nominal layer thicknesses for construction were 2-inches asphaltic concrete base, 1-inch asphaltic concrete binder, and 1-inch asphaltic concrete surfacing. Placement of the lime kiln dust-fly ash-limestone aggregate base was completed by October 1984. Placement of asphaltic concrete layers was completed by mid November, 1984. Final inspection and acceptance of construction was not completed until January 16, 1985.

EVALUATIONS

During and after construction, investigations relative to the engineering properties of the fly ash stabilized aggregate base continued. Laboratory compacted field materials were cured under various conditions and subjected to destructive testing. Cores of the pozzolanic base were obtained and tested for compressive strength at varying ages. Road Rater deflection surveys were performed on compacted subgrade, cured base material, and asphaltic concrete layers. Visual distress surveys were performed to assess the condition of the pavement.

Compressive Strength and Modulus of Elasticity

Specimens made from field materials for evaluation of compressive strength and modulus of elasticity were prepared in general accordance with ASTM C 593(79) in 4-in. by 4.6-in. molds. Deviations from that method involve the use of a 5.5-lb. hammer and a 12-in. free fall instead of the specified 10-lb. hammer and 18-in. drop. Moisture-density relationships were determined in accordance with ASTM D 698(79) instead of ASTM D 1557(79). Maximum dry density and optimum moisture content were determined using a polynomial curve fitting procedure. A smoothing technique was used to eliminate localized changes in concavity. Additional specifics relating to preparation and testing of specimens are presented in Appendix A "Special Provision-Fly Ash Stabilized Bases". Unconfined compressive strengths were determined in accordance with ASTM C 39(72). The static chord elastic modulus was determined by method ASTM C 469(83).

Compressive strength and elastic modulus data are summarized in Table 12. Table 12a summarizes compressive strengths and elastic moduli of materials collected at the jobsite and compacted and tested in the laboratory by Kentucky Transportation Center (KTC) personnel. A summary of compressive strengths for core samples taken at various ages and tested in KTC laboratories is given in Tables 12b and 12c. Results of compressive strength determinations by the Division of Materials of laboratory compacted field materials are given in Table 12d. Table 12e lists compressive strengths of field cores obtained and tested by the Kentucky Department of Highways' Division of Materials. The differences associated with compressive strengths and elastic moduli of laboratory compacted samples and field cores may be directly attributable to the degree of compaction. An inspection of densities listed in Table 9 provides supporting evidence. The In/Out column of Table 9 indicates whether or not density of the compacted pozzolanic base was within required specifications. Field densities with a decimal value of 0.5 or greater are normally rounded to the nearest whole number. Only 29% of the field densities obtained during construction met the required compaction of 102% of the laboratory dry density (AASHTO T-99). Because compaction requirements were not always met, differences in the compressive strengths and elastic moduli of laboratory compacted specimens and field cores can be expected.

TABLE 12a: SUMMARY OF COMPRESSIVE STRENGTHS AND ELASTIC MODULI FOR FIELD MATERIALS COMPACTED IN THE LABORATORY

Station	Age at Test (days)	Compressive Strength (psi)	Elastic Modulus (psi x 10^5)
426+50	7	1,715	3.78
436+00	7	1,355	2.01
436+00	10	1,360	4.31
436+00	28*	1,410	4.90
443+50	28	2,385	6.97
444+00	8	1,650	4.50
444+00	28	2,265	6.97
459+00	7	1,330	3.08
459+00	28	2,085	5.85
461+00	7	970	2.54
461+00	28	960	2.38
476+00	7	1,440	3.93
476+00	28*	2,050	6.82
478+00	7	1,600	5.11
478+00	28*	2,045	7.05
488+25	7	1,665	4.51
488+25	28*	2,150	7.56
488+75	7	1,475	3.52
488+75	28*	1,575	4.77

Notes: * Indicates ambient cure, all others cured at 100°F in a sealed container.

TABLE 12b: SUMMARY OF COMPRESSIVE STRENGTHS AND ELASTIC MODULI FOR FIELD CORE SAMPLES

Station	Age at Test (days)	Compressive Strength (psi)	Elastic Modulus (psi x 10^5)
405+00	4	225	0.48
405+00	7	440	1.32
415+00	5	250	0.83
415+00	8	330	1.17
425+10	6	470	1.02
425+12	6	540	1.02
465+00	14	690	1.02
474+50	14	760	1.02
485+00	14	725	1.02

TABLE 12c: SUMMARY OF COMPRESSIVE STRENGTHS FOR FIELD CORE SAMPLES

Station	Offset from Curb (ft)	Compressive Strength (psi)
423+00	6	2,315
423+00	9	3,095
423+50	6	2,635
423+50	3	1,935
424+00	9	1,685

Note: All cores obtained and tested at the Kentucky Transportation Center. Age of cores was approximately 525 days.

TABLE 12d: SUMMARY OF COMPRESSIVE STRENGTHS FOR FIELD MATERIALS COMPACTED IN THE LABORATORY

Date Sampled	Compressive Strength (psi)	Curing Conditions
9/11/84	2,465 2,405 2,190	7 days oven cured at 100°F
9/12/84	1,530 1,730 1,695	7 days oven cured at 100°F
9/14/84	1,730 1,770 1,770	2 days air, 5 days oven, 2 days air
9/19/84	1,035 935 980	1 day air, 6 days oven
9/29/84	2,110 2,070 2,090	7 days oven

Note: All materials obtained, compacted, and tested by the Kentucky Department of Highways' Division of Materials.

TABLE 12e: SUMMARY OF COMPRESSIVE STRENGTHS FOR FIELD CORE SAMPLES

Station	Offset Left of Curb (ft)	Compressive Strength (psi)
401+70	6	1,960
411+75	18	1,590
421+70	32	1,670
431+60	18	2,545
441+80	7	2,375
452+00	22	1,390
461+84	5	1,350
482+00	25	995

Note: All cores obtained and tested by the Kentucky Department of Highways' Division of Materials. Age of cores was approximately 65 days.

An inspection of the compressive strengths of field specimens contained in Table 12 illustrates the long-term strength gain characteristics of a lime kiln dust-fly ash-aggregate base. Average 14-day compressive strengths were 725 psi. The compressive strength increased to about 1,735 psi at 65 days and to 2,335 psi at about 525 days. Again, differences associated with compressive strengths of laboratory compacted samples and field cores may be related to the degree of compaction.

Road Rater Deflection Measurements

Deflection measurements were obtained at various stages of construction. More specifically, deflection measurements were obtained after compaction of the subgrade but before placement of the pozzolanic base material, and at various intervals after placement of the asphaltic concrete material. Results of deflection measurements are presented in Table 13.

An inspection of deflection data presented in Table 13 indicates a considerable reduction of deflection after placement of the pozzolanic base material. One series of deflection measurements after completion of all construction activities was obtained. Unfortunately, the device used for measurement of pavement temperatures malfunctioned and invalidated the analysis.

Continued monitoring of deflections over a period of time will provide valuable information relative to the long-term structural behavior of lime kiln dust-fly ash-limestone aggregate bases. The historical deflection and compressive strength information will provide specific information relative to the long-term strength gain characteristic of these pozzolanic base sections.

Visual Surveys

The experimental section has been surveyed periodically for observable pavement distress since completion of construction. Factors such as rutting and cracking were of principle concern. During August 1986 and again during October 1987, KTC personnel conducted a detailed visual crack survey of the roadway surface. Results of those surveys are contained in Appendix B. The 1.7-mile experimental section was canvassed by technicians using rola-tape measuring devices. All visible cracks were charted on data sheets with station numbers. For convenience, Station 0+00 was established at the intersection of the subject road with Nicholasville Road (US 27). The survey progressed in an easterly direction and culminated at Station 90+43, just short of the intersection of the subject road with Tates Creek Pike (KY 1974). Photographs were obtained to document the condition of the road.

TABLE 13a: SUMMARY OF DEFLECTION MEASUREMENTS OBTAINED DIRECTLY ON THE COMPACTED SUBGRADE

			Road Rater Model 400 Deflections (in. x 10^{-5})			
Station	Direction	Date	Sen1	Sen2	Sen3	Sen4
401+00	East	9-06-84	144	88	44	28
401+00	East	9-11-84	140	74	72	22
405+00	East	9-06-84	88	25	20	8
405+00	East	9-11-84	116	46	20	12
410+00	East	9-06-84	810	410	140	20
410+00	East	9-06-84	410	195	75	5
410+00	East	9-11-84	680	270	80	20
415+00	East	9-06-84	350	165	105	10
415+00	East	9-11-84	138	64	30	20
420+00	East	9-06-84	132	42	16	2
420+00	East	9-11-84	150	48	26	18
424+95	East	9-06-84	78	34	15	8
425+00	East	9-06-84	410	155	100	125
425+00	East	9-11-84	148	64	36	20
430+00	East	9-06-84	98	30	22	12
430+00	East	9-11-84	100	40	14	8
435+00	East	9-06-84	96	24	6	10
435+00	East	9-11-84	130	22	8	8
439+50	East	9-11-84	152	28	4	4
440+00	East	9-06-84	120	28	12	20
440+05	East	9-06-84	102	32	12	8
445+00	East	9-06-84	300	96	30	6
445+00	East	9-11-84	154	52	18	14
449+00	East	9-06-84	124	28	8	36
449+00	East	9-11-84	136	34	12	10
450+00	East	9-07-84	470	90	--	--
451+00	East	9-18-84	234	66	9	--
455+00	East	9-07-84	180	42	44	24
455+00	East	9-18-84	162	70	30	12
460+00	East	9-07-84	114	74	20	--
460+00	East	9-18-84	154	62	12	8
465+00	East	9-07-84	291	75	21	15
465+00	East	9-18-84	85	51	12	10
470+00	East	9-07-84	166	78	40	32
470+00	East	9-18-84	91	35	20	16
475+00	East	9-07-84	204	105	45	--
475+00	East	9-18-84	176	78	26	20
480+00	East	9-07-84	138	64	34	30
480+00	East	9-18-84	83	29	13	7
483+50	East	9-07-84	148	34	--	--
486+00	East	9-07-84	144	128	16	8
490+00	East	9-07-84	207	60	27	24

TABLE 13a: SUMMARY OF DEFLECTION MEASUREMENTS OBTAINED DIRECTLY ON COMPACTED SUBGRADE (continued)

			Road Rater Model 400 Deflections (in. x 10^{-5})			
Station	Direction	Date	Sen1	Sen2	Sen3	Sen4
401+00	West	9-06-84	207	123	78	48
401+00	West	9-11-84	186	58	20	18
405+00	West	9-06-84	114	56	32	14
405+00	West	9-11-84	96	32	12	10
410+00	West	9-06-84	134	54	28	24
410+00	West	9-11-84	176	84	48	36
415+00	West	9-06-84	370	180	60	25
415+00	West	9-11-84	495	300	150	50
420+00	West	9-06-84	128	76	14	10
420+00	West	9-11-84	178	66	34	20
425+00	West	9-06-84	225	78	27	3
425+00	West	9-06-84	179	58	26	18
425+00	West	9-11-84	340	135	60	40
430+00	West	9-06-84	106	44	12	2
430+00	West	9-11-84	110	30	6	4
435+00	West	9-06-84	72	27	20	2
435+00	West	9-11-84	104	30	12	8
439+50	West	9-06-84	144	30	4	4
439+50	West	9-11-84	80	27	10	9
445+00	West	9-06-84	120	32	16	20
445+00	West	9-11-84	112	32	4	6
449+00	West	9-06-84	160	46	12	8
449+00	west	9-11-84	156	34	12	14
450+00	West	9-07-84	390	40	--	--
451+50	West	9-18-84	164	48	8	8
455+00	West	9-07-84	180	30	14	--
455+00	West	9-18-84	132	32	8	2
460+00	West	9-07-84	162	38	28	14
460+00	West	9-18-84	164	100	40	6
465+00	West	9-07-84	385	205	85	35
470+00	West	9-07-84	192	78	36	22
470+00	West	9-18-84	156	56	16	6
475+00	West	9-07-84	198	96	51	36
475+00	West	9-18-84	154	60	36	8
480+00	West	9-07-84	178	114	20	12
480+00	West	9-18-84	83	29	13	7
483+50	West	9-07-84	57	25	5	4
486+00	West	9-07-84	88	38	22	12
490+00	West	9-07-84	345	160	75	50

TABLE 13b: SUMMARY OF DEFLECTION MEASUREMENTS OBTAINED DIRECTLY ON COMPACTED POZZOLANIC BASE MATERIAL

Station	Direction	Date	Road Rater Model 400 Deflections (in. x 10^{-5}) Sen1	Sen2	Sen3	Sen4
402+50	East	9-20-84	23	19	13	11
405+00	East	9-20-84	18	14	6	6
410+00	East	9-20-84	24	24	11	9
415+00	East	9-20-84	31	31	21	19
420+00	East	9-20-84	20	16	10	8
425+00	East	9-20-84	88	77	48	36
430+00	East	9-20-84	18	16	6	6
435+00	East	9-20-84	14	9	2	3
440+00	East	9-20-84	20	14	6	6
445+00	East	9-20-84	31	20	12	8
449+00	East	9-20-84	24	14	7	5
451+50	East	9-27-84	45	36	24	--
455+00	East	9-27-84	36	27	20	--
460+00	East	9-27-84	36	30	16	--
465+00	East	9-27-84	52	36	20	--
470+00	East	9-27-84	31	24	15	--
475+00	East	9-27-84	31	20	12	--
480+00	East	9-27-84	29	20	8	--
483+75	East	9-27-84	16	12	6	--
485+00	East	9-27-84	24	15	6	--
490+00	East	9-27-84	32	26	20	
402+00	West	9-20-84	31	25	19	14
405+00	West	9-20-84	12	8	4	4
410+00	West	9-20-84	26	24	14	14
415+00	West	9-20-84	36	37	33	27
420+00	West	9-20-84	12	12	6	6
425+00	West	9-20-84	38	27	18	12
430+00	West	9-20-84	22	17	10	8
435+00	West	9-20-84	20	12	6	4
440+00	West	9-20-84	20	14	9	7
445+00	West	9-20-84	20	14	6	6
449+00	West	9-20-84	22	16	6	4
451+50	West	9-27-84	32	21	12	--
455+00	West	9-27-84	34	26	16	--
460+00	West	9-27-84	49	41	31	--
465+00	West	9-27-84	32	24	15	--
470+00	West	9-27-84	31	26	18	--
475+00	West	9-27-84	16	13	6	--
480+00	West	9-27-84	16	10	4	--
483+75	West	9-27-84	14	14	8	--
485+00	West	9-27-84	18	12	6	--
490+00	West	9-27-84	38	32	20	--

Overall, the pavement was generally in good condition. The most notable distress observed was shrinkage cracks appearing around most curb inlet drains. From the survey data sheets in Appendix B, it may be noted that 49 drain inlets were charted, 11 of those being located in the section not receiving the stress relieving interlayer. There was a distinct cracking pattern of the asphaltic concrete around those drains that exhibited cracking. Relative to the percentage of drain inlets exhibiting cracking in the two distinct sections there was no difference. Approximately 63 % of the drain inlets exhibited various amounts of shrinkage cracking in both sections. Figures 3, 4, and 5 show this cracking pattern. Figure 3 illustrates how the concrete gutter of the curb inlet drain juts out into the paving lane. This design was observed to have contributed to problems relative to compaction of the experimental base material during construction. Field observations during the construction phase did not indicate any effort on the part of the contractor to rectify this situation (ie. use of a pneumatic tamping device). The drain in Figure 3 was located near Station 16+72. Note the fine debris that has built up in the outer area.

Figure 4 shows the curb inlet drain near Station 17+88. Note the similarities of the cracking pattern when compared to Figure 3. Again, the geometry of the curb inlet drain apparently made compaction of the experimental base material difficult. Note the fine debris that has built up in the gutter area and the vegetation growing there. This obstruction in the gutter area will probably cause water to flow out onto the traffic lane. The water will most likely infiltrate the experimental base by seeping through the shrinkage cracks and the longitudinal joint at the curb gutter pavement interface. This may lead to an accelerated failure of the base material in the area of the drain inlet and result in failure of the asphaltic concrete layers. Data contained in the survey data sheets are supportive of this contention. The asphaltic concrete pavement adjacent to the concrete gutter drain located near Station 57+15 was exhibiting light alligator cracking, which is indicative of impending base failure, during the October 1987 visual survey.

Figure 5 shows the curb inlet drain located near Station 19+54. The similarities of the crack pattern and fine debris deposited in the gutter area are easily recognized. These figures are typical of those curb drain inlets exhibiting cracking. There were seven curb inlet drains in the section receiving the stress relief interlayer that exhibited this cracking pattern.

Figure 6 is a photograph of a transverse crack located near Station 6+72 which is within a section receiving the interlayer. The crack extended from curb to curb, across both lanes and the two-way left turn lane. Another transverse crack extending from curb to curb is shown near Station 5+85 in the crack survey data sheets. Six additional transverse

Figure 3. Shrinkage Cracking around Concrete Inlet Drain

Figure 4. Shrinkage Cracking around Concrete Inlet Drain

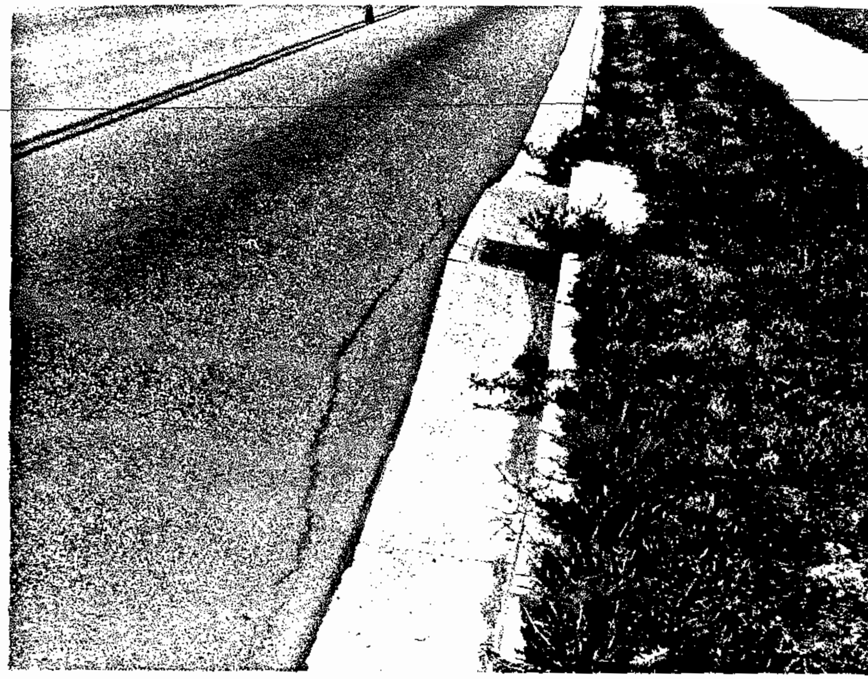

Figure 5. Shrinkage Cracking around Concrete Inlet Drain

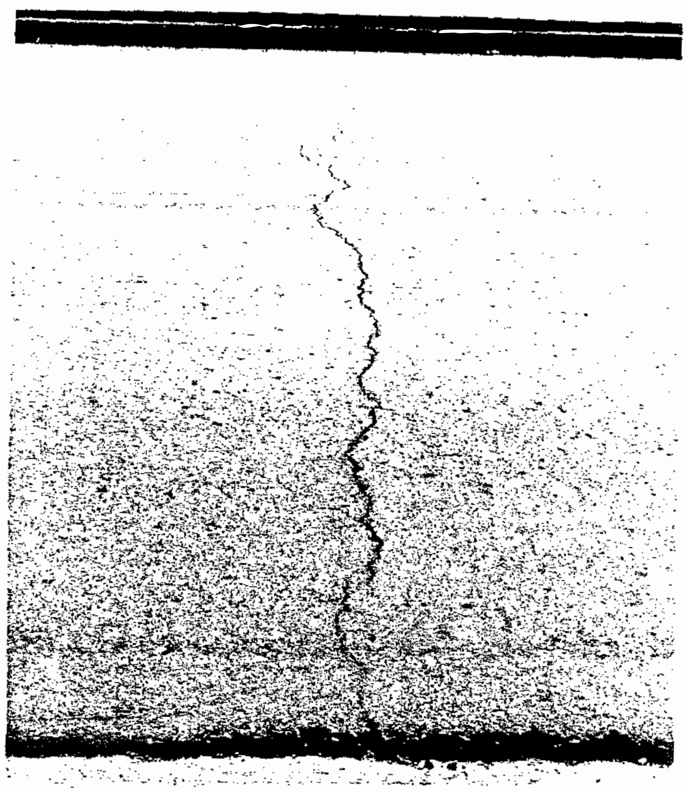

Figure 6. Transverse Reflective Cracking of Pozzolanic Base

cracks were located within the treated sections and were near Stations 10+38, 18+38, 19+48, 20+45, 22+03, and 60+47. These cracks were present in the eastbound travel lane only.

Of particular interest was the amount of reflective cracking in the section not receiving the stress relief interlayer. The 1,500-foot section contained three transverse cracks which principally extended from curb to curb. These cracks were located near Stations 45+41, 46+36 and 49+40. Other forms of distress noted on the data sheets included small areas of surface raveling, a number of small potholes and some random longitudinal cracking.

The August 1986 survey indicated 465 lineal feet of longitudinal cracking in the section receiving the stress relief interlayer and 34 lineal feet of longitudinal cracking in the section not receiving the interlayer. Longitudinal cracking was located primarily near the drain inlets. Transverse cracking was observed in both sections. The 1,500-foot section not receiving the interlayer had 54 lineal feet of transverse cracking while the remaining 1.4 miles of the experimental section had 112 lineal feet of transverse cracking. The October 1987 visual survey revealed additional amounts of cracking in both sections. An additional 413 and 119 lineal feet of longitudinal cracking was observed for the treated sections and untreated section, respectively. Only 20 lineal feet of additional transverse cracking was observed in the section receiving the interlayer while an additional 57 lineal feet of transverse cracking was detected in the section not receiving the interlayer. Due to the relative increase in the amount of reflective cracking, it may be concluded that the stress relief interlayer at least slows the development of reflective cracking.

Measurements of rut depth were obtained every 1000 feet. These results have been tabulated and are listed in Table 14. The average was higher for the left wheel path in both travel lanes, averaging nearly 3/16 inch. The average for the right wheel path was about 1/8 inch.

TABLE 14: RUTTING DEPTHS -- MAN O'WAR BOULEVARD

Station Number	DIRECTION East LWP (in.)	East RWP (in.)	West LWP (in.)	West RWP (in.)
10+00	3/16	4/16	6/16	2/16
20+00	4/16	2/16	4/16	4/16
30+00	2/16	2/16	3/16	1/16
40+00	1/16	1/16	2/16	3/16
50+00	3/16	2/16	1/16	2/16
60+00	3/16	1/16	3/16	3/16
70+00	4/16	2/16	3/16	2/16
80+00	4/16	2/16	3/16	1/16
90+00	1/16	1/16	1/16	1/16
Average	3/16	2/16	3/16	2/16

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Design and construction procedures for utilization of a fly ash-lime kiln dust-limestone aggregate base have been demonstrated. No major construction problems were encountered. The development of additional information relative to design procedures would be of significant benefit.

Observed performance has been favorable thus far. Reflective cracking has been minimal with the exception of shrinkage cracking observed at drain inlets. Compressive strength evaluations indicate satisfactory to exceptional initial compressive strengths and also continued strength gain over a period of time. Deflection measurements indicate a significant reduction in deflection for before placement of the base relative to after placement. Placement of the pozzolanic base also provided a considerably more uniform base for placement of the asphaltic concrete material. The magnitudes of deflection measurements for the pozzolanic base material indicate a very strong pavement structure even without the benefit of the asphaltic concrete layers. Back-calculation of specific elastic layer moduli indicate that the moduli of the pozzolanic base material is most likely within a range of 5×10^5 to 1×10^6 psi.

The relatively small amount of reflective cracking present in the section that received the stress relief interlayer indicates the apparent success of the application to date. Major reflective cracking has been observed for a similarly designed pavement section elsewhere which did not receive a stress relieving interlayer. A far greater amount of reflective cracking could be expected if the pozzolanic base had not received the stress relief interlayer. The observation of increased reflective cracking in the 1,500-foot untreated section supports this conclusion.

Based upon observations to date, treated bases apparently enhance overall pavement performance and their continued use should be evaluated as long-term performance data (durability, freeze/thaw resistance, etc.,) becomes available. It is quite probable that pavement life may be extended at reduced costs by the use of treated bases.

REFERENCES

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3. ASTM D 698(78), Standard Test Method for "Moisture-Unit Weight Relations of Soils and Soil-Aggregate Mixtures, Using 5.5-lb Rammer and 12-in. Drop": Natural Building Stones; Soil and Rock; Peats, Mosses and Humus, 1978 Annual Book of ASTM Standards, Part 19(1978), American Society for Testing and Materials, Philadelphia, U.S.A.

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APPENDIX A

KENTUCKY TRANSPORTATION CABINET
DEPARTMENT OF HIGHWAYS
SPECIAL PROVISION NO. 70 (79)

FLY ASH STABILIZED BASE
(EXPERIMENTAL)
(DGA, Fly Ash, and Lime or Kiln Dust)

This Special Provision shall apply when indicated on the plans or in the proposal. Section references herein are to the Department's 1979 Standard Specifications for Road and Bridge Construction.

I. DESCRIPTION

This work shall consist of furnishing all materials, and the construction of a plant-mixed base material consisting of DGA, fly ash, water, and either lime or kiln dust placed on a prepared subgrade to the lines, grades, and thickness specified in the contract or directed by the Engineer.

II. MATERIALS

A. Aggregate. The aggregate shall be Dense Graded Aggregate (DGA) conforming to the requirements of subsection 805.04.03, Part A.

B. Fly Ash. Fly ash shall conform to the requirements of ASTM C 593.

C. Lime. Lime shall conform to the requirements of ASTM C 207, Type N, Sections 3, 6, 7.1.1, 10, and 11.

D. Cement Kiln Dust or Lime Kiln Dust. The cement kiln dust or lime kiln dust should be basically lime in the form of CaO or CaOH or percentages of each. The Contractor shall submit certified test data on the chemical composition of the kiln dust (if used). The kiln dust shall be reasonably uniform. The supplier may be required to furnish past test data to show uniform properties. The Engineer reserves the right to perform plant inspection at the source of the kiln dust.

E. Water. Water mixed with the base material shall conform to the requirements of Section 803.

F. Bituminous Curing Seal. The bituminous material for the curing seal shall be either RS-1, RS-2, SS-1, SS-1h, CRS-1, CSS-1, CSS-1h or Primer L, conforming to the requirements of Section 806.

G. Natural Sand. Natural sand added to the base material shall conform to the requirements of Section 804.04.02.01, Part A, and shall have a minimum sand equivalent of 60.

III. CONSTRUCTION REQUIREMENTS

A. General. The subgrade shall be prepared in accordance with Section 208 and shall be maintained free from irregularities. Where the required thickness of the base is 6 inches or less, the mixture may be spread and compacted in one layer. Where the required thickness is more than 6 inches, the mixture shall be spread and compacted in 2 or more layers of approximately equal thickness, and the maximum compacted thickness of any one layer shall not exceed 6 inches. Work on each layer shall be performed in a similar manner and the surface of the compacted material shall be kept moist or prevented from drying, by a method approved by the Engineer, until covered with the next layer. The second layer may be applied immediately after obtaining satisfactory compaction of the first layer.

When a base course extends under the shoulders, the section under the pavement shall be constructed first and the Contractor may defer the placing of the remaining portion of the base course under the shoulders until after construction of the paving lane. In such a case, the minimum width of initial base construction shall extend 2 feet beyond the paving lane edges. In no case shall construction joints of the base lie underneath the proposed joints of the pavement to be superimposed.

B. Seasonal Limitations. The fly ash stabilized base shall not be placed between the period of October 1 and April 30.

C. Composition of Mixture. The composition of the mixture will depend upon materials and sources of materials selected for use by the Contractor. Therefore, the Contractor shall be responsible for submitting a recommended mixture composition together with samples of each proposed ingredient to the Division of Materials at least 30 calendar days prior to placement on the job site.

The ingredient proportions of the mixture of DGA, fly ash, water, and either lime or kiln dust shall be determined in accordance with procedures outlined in ASTM C 593, except Method 3 of KM 64-511 shall be used in place of Method C of ASTM D 1557 as referenced under 8.3 Proportioning of ASTM C 593. The resultant recommended mixture shall produce a 7-day compressive strength of no less than 600 psi.

Probable compositions may be within the following ranges:

Lime - Fly Ash - DGA

Ingredient*	Percent Range
Lime	2 - 6
Fly Ash	6 - 20
DGA	74 - 89

Kiln Dust - Fly Ash - DGA

Ingredient*	Percent Range
Kiln Dust	6 - 12
Fly Ash	6 - 20
DGA	74 - 89

*the free water content may be approximately 7 percent of SSD weights of all other materials.

A portion of the DGA may be replaced with natural sand at the Contractor's option, providing the specified gradation requirements for DGA are met.

The recommended composition, as submitted by the Contractor, will be checked by the Division of Materials. Base materials shall not be placed on the job site prior to receipt of approval of materials and composition from the Division of Materials.

Material proportions, as approved for use on the job, shall be within the following tolerances in percent by weight of the total mixture:

Lime or Kiln Dust	± 0.5
Fly Ash	± 1.5
Water (free)	± 1.0
Aggregate Components	± 2.0

D. Plant and Equipment. Plant mixing may be accomplished in either a separate weigh batch increment type plant or a continuous volumetric proportioning type plant as the Contractor may elect.

Fly ash and either lime or kiln dust to be weighed at batch type plants shall each be weighed on scales separate ~~from the aggregate batching scales, except that if a~~ compartment for pre-proportioned fly ash and either lime or kiln dust is contained within the aggregate hopper and the pre-mixed fly ash and lime or kiln dust for each batch is weighed prior to the weighing of the aggregate, the pre-proportioned material may be weighed on the aggregate scale.

If fly ash and lime or kiln dust are pre-proportioned, both ingredients shall be dry, or the pre-proportioning shall not be performed until immediately before batching.

Continuous volumetric plants shall be equipped with feeding and metering devices which will add the aggregate, fly ash, water, and lime or kiln dust into the plant in the specified quantities. Feeding equipment or procedures that do not consistently maintain the contents of the mixture within the specified tolerances shall be modified or replaced. The water supply system shall be equipped with a positive cut-off control which will stop the flow of water simultaneously with any stoppage in the flow of aggregate into the pugmill.

E. Mixing. Water shall be added to the aggregate, fly ash, and lime or kiln dust mixture in sufficient quantity, and mixing shall continue until all component materials are evenly distributed through the mass and a uniform unchanging appearance is obtained. Mixer components shall be regularly checked for wear and efficiency.

F. Transporting and Spreading. Materials shall be transported to the roadbed by means of suitable vehicles. The vehicles shall be equipped with protective covers when the time between loading the vehicle and spreading the mixture exceeds 30 minutes. Material shall be deposited on a moist subgrade by approved spreading equipment. Depositing and spreading the mixed materials on the roadbed shall commence at the point farthest from the point of loading and shall progress continuously as far as practical without breaks. No hauling shall be done over the completed base course except as necessary to place the succeeding layer of base or pavement. Dumping in piles upon the subgrade will not be permitted except when special equipment which distributes the material uniformly is used and is approved by the Engineer.

The mixture shall be spread to such width and thickness that, after compacting, the finished base will conform to the required grade and cross section.

Fly ash stabilized base to be placed on areas inaccessible to mechanical spreading equipment may be spread by other methods approved by the Engineer.

G. Compaction and Finishing. Immediately upon completion of each portion of spreading operations, the material shall be thoroughly compacted. Moisture shall be maintained at a level sufficient to facilitate compaction. Initial and final rolling shall be performed by compaction equipment which will produce the required density and surface finish within the time limit specified below.

All high spots on the finished surface of the final layer outside of the specified tolerance shall be trimmed off to within the specified tolerance prior to placing any material thereon. The excess material shall be removed and disposed of as directed by the Engineer immediately after trimming and before any further rolling. Trimmed areas shall be rolled. Rolling shall be performed in such a manner as to avoid the formation of irregularities, and the finished surface shall be true to the required grade and cross section.

Areas inaccessible to rollers shall be compacted by means of pneumatic tampers or other compacting equipment which produces the required density.

The finished base shall be compacted to at least 102 percent of the maximum density determined by KM 64-511.

The in-place density of each course will be determined by nuclear gages or by KM 64-512.

Not more than 2 1/2 hours shall elapse between the time water is added to the combined aggregate, fly ash, and lime or kiln dust and the time of completion of final compaction of the base or layer, as the case may be. Any mixture that has not been compacted and finished shall not remain undisturbed for more than 30 minutes.

It is intended that all trimming and fine grading be accomplished during the 2 1/2 hours mentioned above, and that trimming of the completed and cured base be limited to occasional minor irregularities.

H. Joints. At the end of each day's work and when base operations are delayed or stopped for more than 2 hours, a construction joint shall be made by trimming the end of the compacted material to a vertical face. The same procedure shall be followed in trimming longitudinal edges where the abutting course is to be placed.

I. Tolerances.

(1) Surface Tolerance. The surface of the top of the base shall be smooth and uniform and shall not deviate more than 1/2 inch from the specified cross section at any point and shall not deviate from the specified longitudinal grade more than 3/8 inch in 10 feet at any location. When final grading is to be performed by an automatic grading machine, the base shall be trimmed to such accuracy that the succeeding base and/or surface courses will meet their respective specified surface and thickness tolerances.

The Contractor shall furnish all devices necessary to check the surface, such as stringlines, straightedges, etc., and the labor necessary to handle the devices.

When the completed base is found to deviate from the tolerances, the deviations shall be corrected after the 7-calendar-day curing period, by leveling and wedging with an approved bituminous concrete mixture. This corrective work shall be performed at no cost to the Department.

(2) Thickness Tolerance. The base course will be checked for proper thickness after compaction. The Contractor shall refill all test holes with approved mixture and adequately compact the material.

No base with a deficiency in thickness greater than 1/2 inch will be accepted.

J. Curing. The completed fly ash stabilized base shall be protected against drying by covering with a bituminous curing seal. The curing seal will be required only for the top layer.

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

The curing seal shall consist of the bituminous material specified and shall be uniformly applied to the surface of the completed base at the rate of approximately 1.2 pounds per square yard with approved distributing equipment. The actual rate and application temperature of bituminous material will be determined by the Engineer. The curing seal shall be applied in sufficient quantity to provide a continuous membrane over the base.

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

K. Maintenance and Protection. Traffic on the completed base should be held to the minimum necessary to complete the work. Areas subjected to traffic shall be rechecked for grade and cross section and necessary corrections made, and any damaged areas repaired as directed, before the succeeding course is constructed.

Any damage to the base by hauling or other means at any time shall be repaired with an approved bituminous concrete mixture at no cost to the Department.

It is intended that the base shall be completely covered with the specified pavement courses before the work is suspended for the winter months. The Contractor shall make every reasonable effort to accomplish this objective. When the base is not completely covered with the specified pavement courses, the Engineer will then determine the extent of any further work necessary to protect and maintain the uncompleted work during the winter months and until the beginning of spring paving operations. When extra materials, methods, and construction are determined to be necessary to protect, maintain, and repair any portion of the uncompleted work, the cost of such extra materials, methods, and techniques shall be borne by the Contractor.

IV. METHOD OF MEASUREMENT

Water used for dampening the subgrade, mixing with the mixture, or for maintaining moisture in the base during shaping and compacting will not be measured for payment, but will be considered incidental to the base.

Fly ash stabilized base materials will be weighed in accordance with Section 109; no deduction will be made for water in the mixture.

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

V. BASIS OF PAYMENT

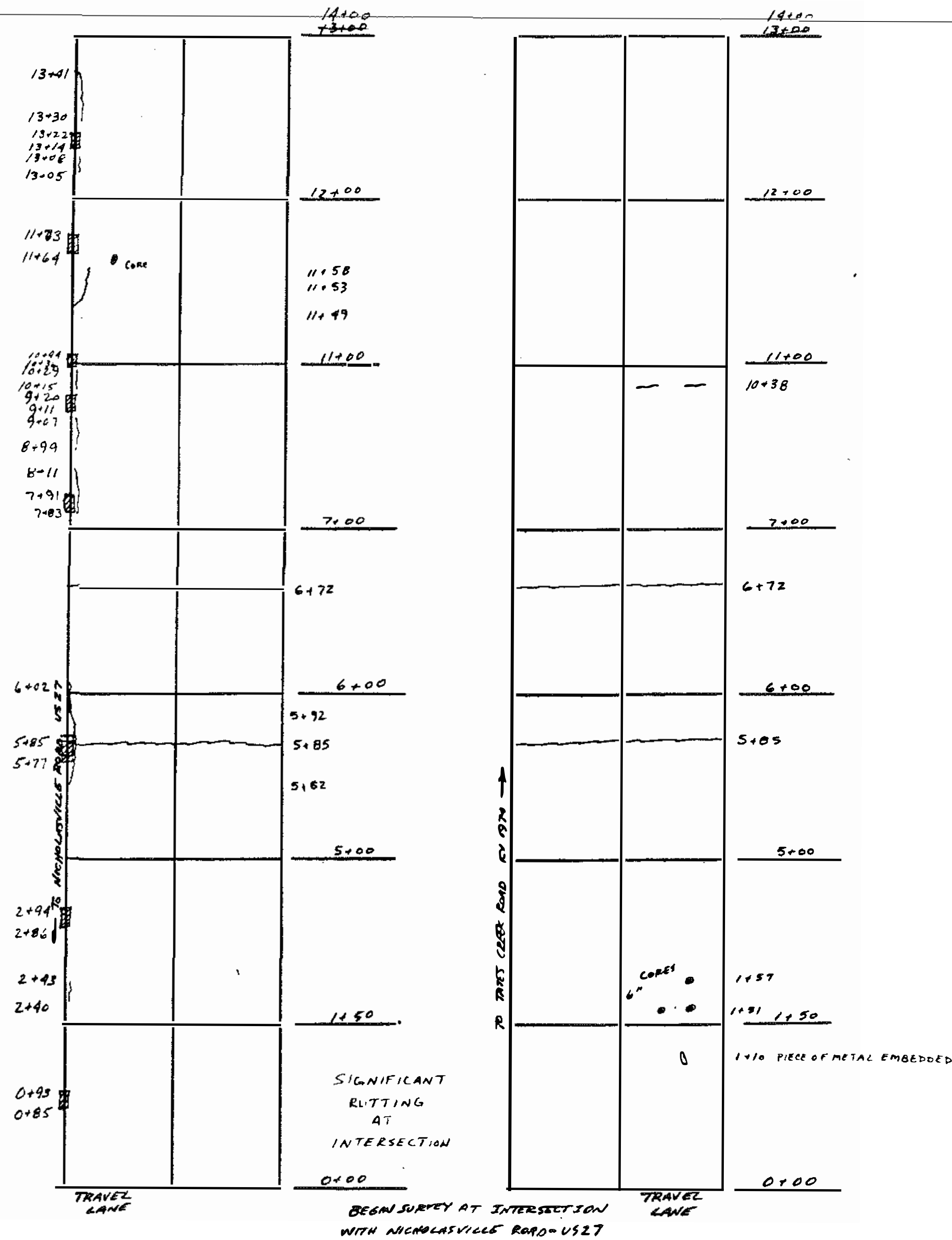
Payment for the accepted quantities at the contract unit prices shall be full compensation for all labor, equipment, materials, hauling, and incidentals necessary to complete the work specified herein.

Payment will be made under:

Pay Item	Pay Unit
Fly Ash Stabilized Base	Ton
Bituminous Curing Seal	Ton

APPROVED James W. Fehr 7/29/83
JAMES W. FEHR, P.E. DATE
STATE HIGHWAY ENGINEER

APPENDIX B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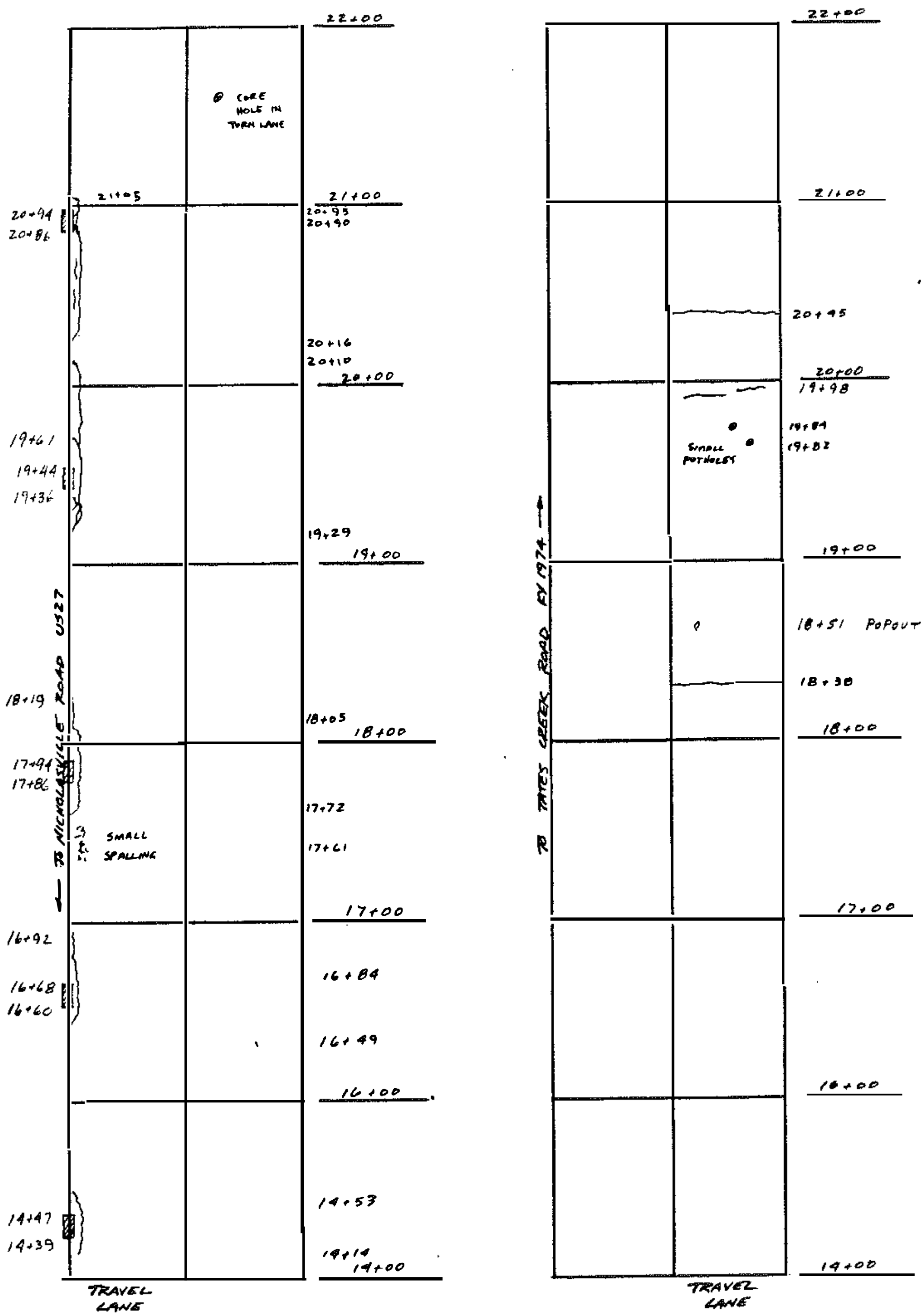
11+58
11+53
11+49
11+00
10+38
8+99
8+11
7+91
7+83
7+00
6+72
6+02
6+00
5+92
5+85
5+77
5+62
5+00
NICHOLASVILLE ROAD US27
TO TATES CREEK ROAD KY 1974
2+94
2+86
2+43
2+40
CORES
6"
1+57
1+51
1+50
1+10 PIECE OF METAL EMBEDDED
SIGNIFICANT RUTTING AT INTERSECTION
0+93
0+85
0+00
TRAVEL LANE
BEGIN SURVEY AT INTERSECTION WITH NICHOLASVILLE ROAD - US27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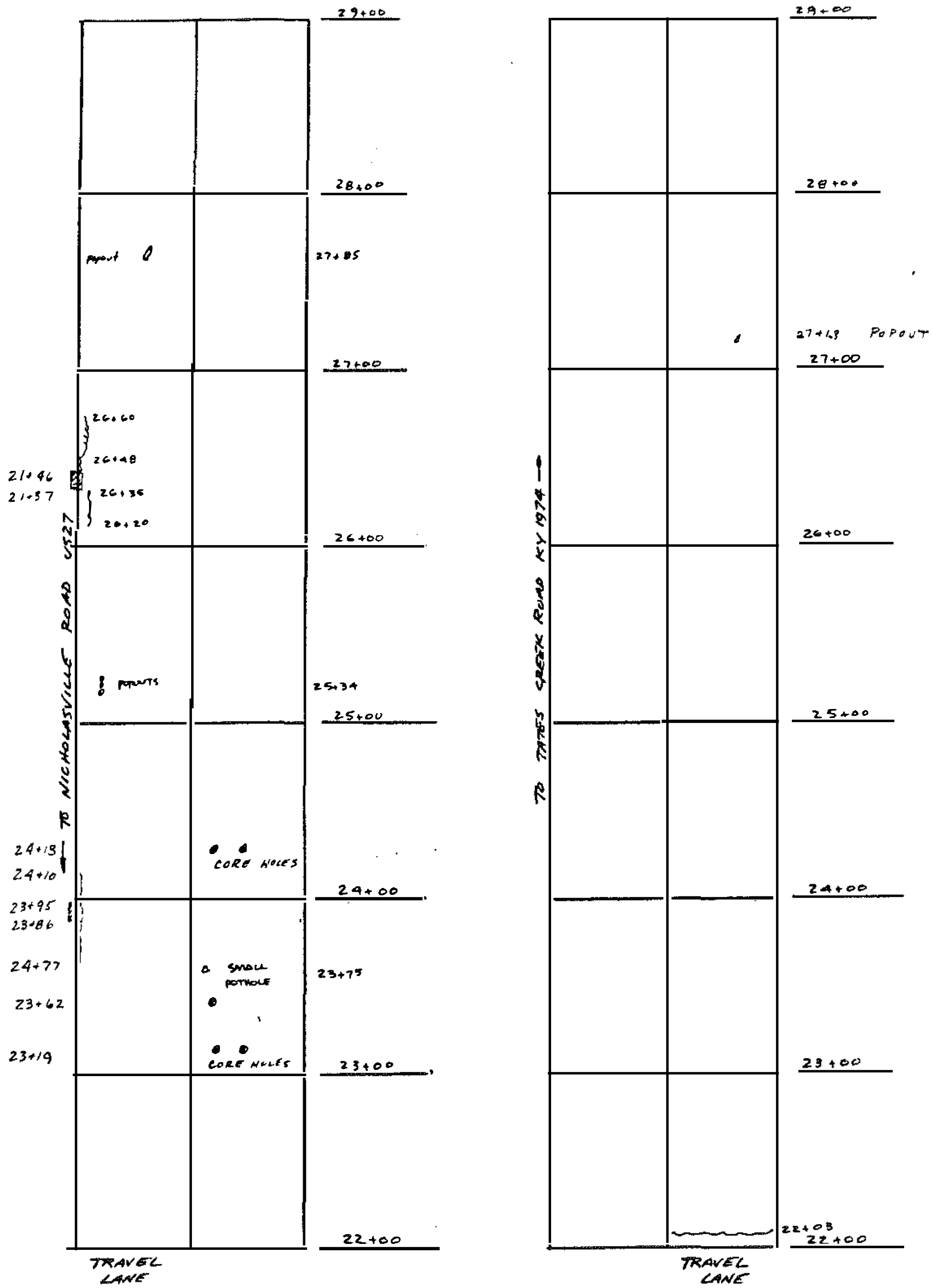
CORE HOLE IN TURN LANE
21+05
21+00
20+94
20+86
20+95
20+90
20+16
20+10
20+00
19+61
19+44
19+36
19+29
19+00
TO NICHOLASVILLE ROAD US27
18+19
18+05
18+00
17+94
17+86
17+72
SMALL SPALLING
17+61
17+00
16+92
16+84
16+68
16+60
16+49
16+00
14+53
14+47
14+39
14+14
14+00
TRAVEL LANE
22+00
21+00
20+45
20+00
19+98
19+84
19+82
SMALL POTHOLES
19+00
TO TATES CREEK ROAD KY 1974
18+51 POPOUT
18+38
18+00
17+00
16+00
14+00
TRAVEL LANE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28+00
Popout
27+85
27+00
26+60
26+48
21+46
21+37
26+35
26+20
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TO NICHOLASVILLE ROAD US27
POPOUTS
25+34
25+00
24+13
24+10
CORE HOLES
24+00
23+95
23+86
24+77
SMALL POTHOLE
23+75
23+62
23+19
CORE HOLES
23+00
22+00
TRAVEL LANE
29+00
28+00
27+48 POPOUT
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TO TATES CREEK ROAD KY 1974
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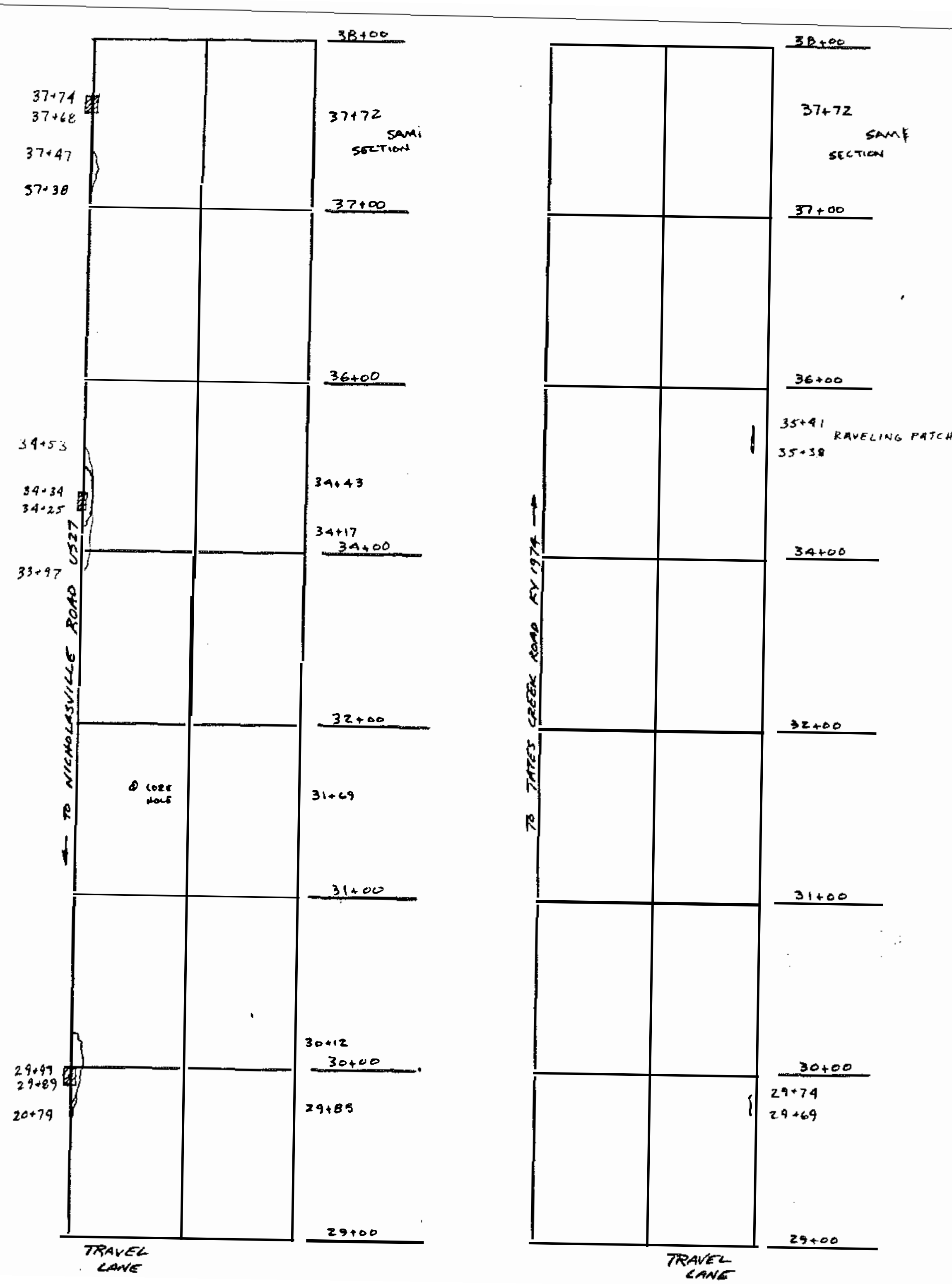
22+03
22+00
TRAVEL LANE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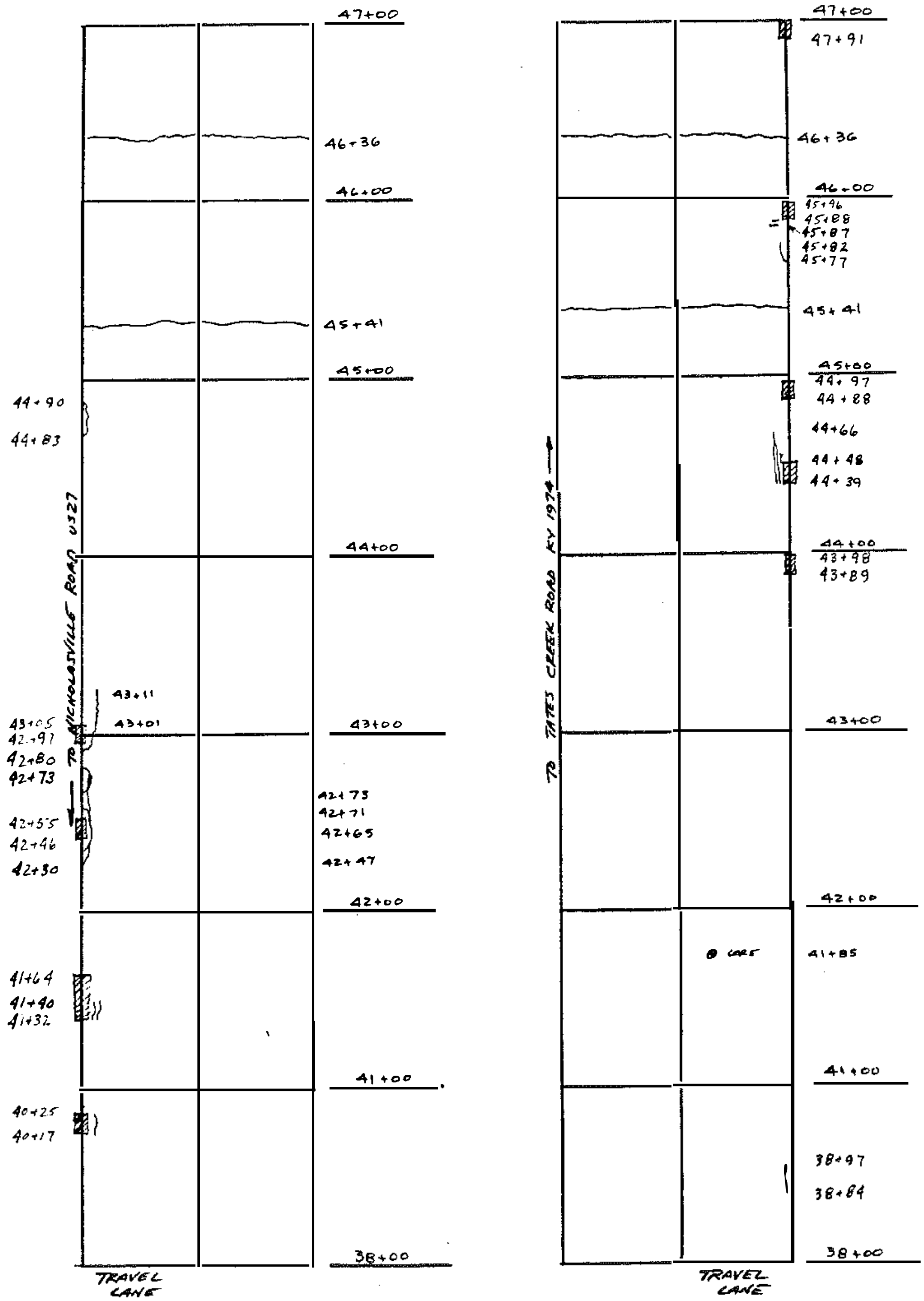
37+74
37+68
37+72
SAME SECTION
37+47
37+38
37+00
36+00
34+53
34+43
34+34
34+25
34+17
34+00
33+97
TO NICHOLASVILLE ROAD US27
32+00
CORE HOLE
31+69
31+00
30+12
30+00
29+99
29+89
29+85
20+79
29+00
TRAVEL LANE
38+00
37+72
SAME SECTION
37+00
36+00
35+41
RAVELING PATCH
35+38
34+00
TO TATES CREEK ROAD FY 1974
32+00
31+00
30+00
29+74
29+69
29+00
TRAVEL LANE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43+05
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42+80
42+73
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42+73
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38+00
TRAVEL LANE
47+00
47+91
46+36
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45+87
45+82
45+77
45+41
45+00
44+97
44+88
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44+48
44+39
44+00
43+98
43+89
TO TATES CREEK ROAD KY 1974
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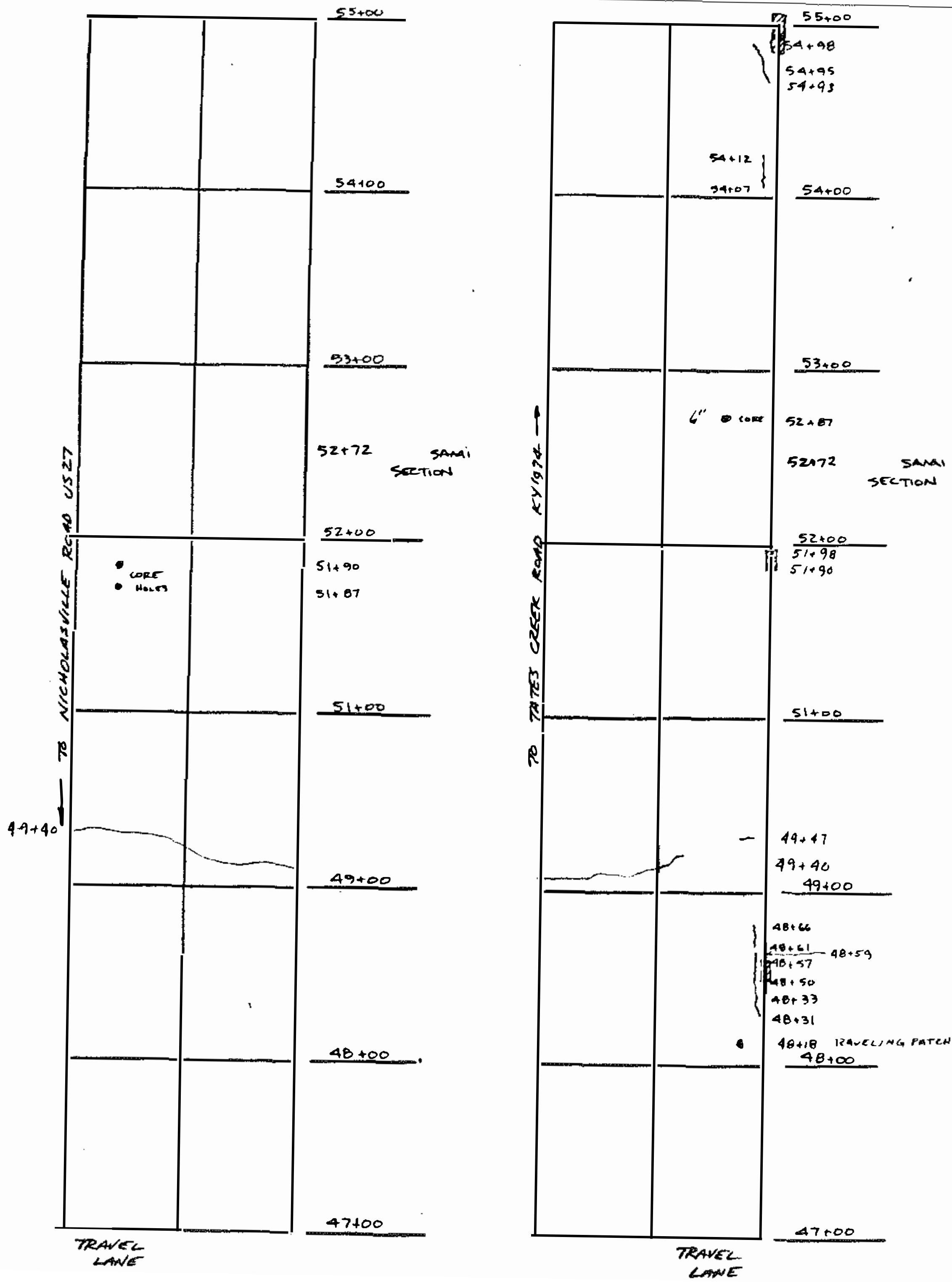
42+00
CORE
41+85
41+00
38+97
38+84
38+00
TRAVEL LANE



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53+00
52+72
SAMI SECTION
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CORE HOLES
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51+00
TO NICHOLASVILLE ROAD US27
49+40
49+00
48+00
47+00
TRAVEL LANE
55+00
54+98
54+95
54+93
54+12
54+07
54+00
53+00
6" CORE
52+87
52+72
SAMI SECTION
52+00
51+98
51+90
TO TATES CREEK ROAD KY1974
51+00
49+47
49+40
49+00
48+66
48+61
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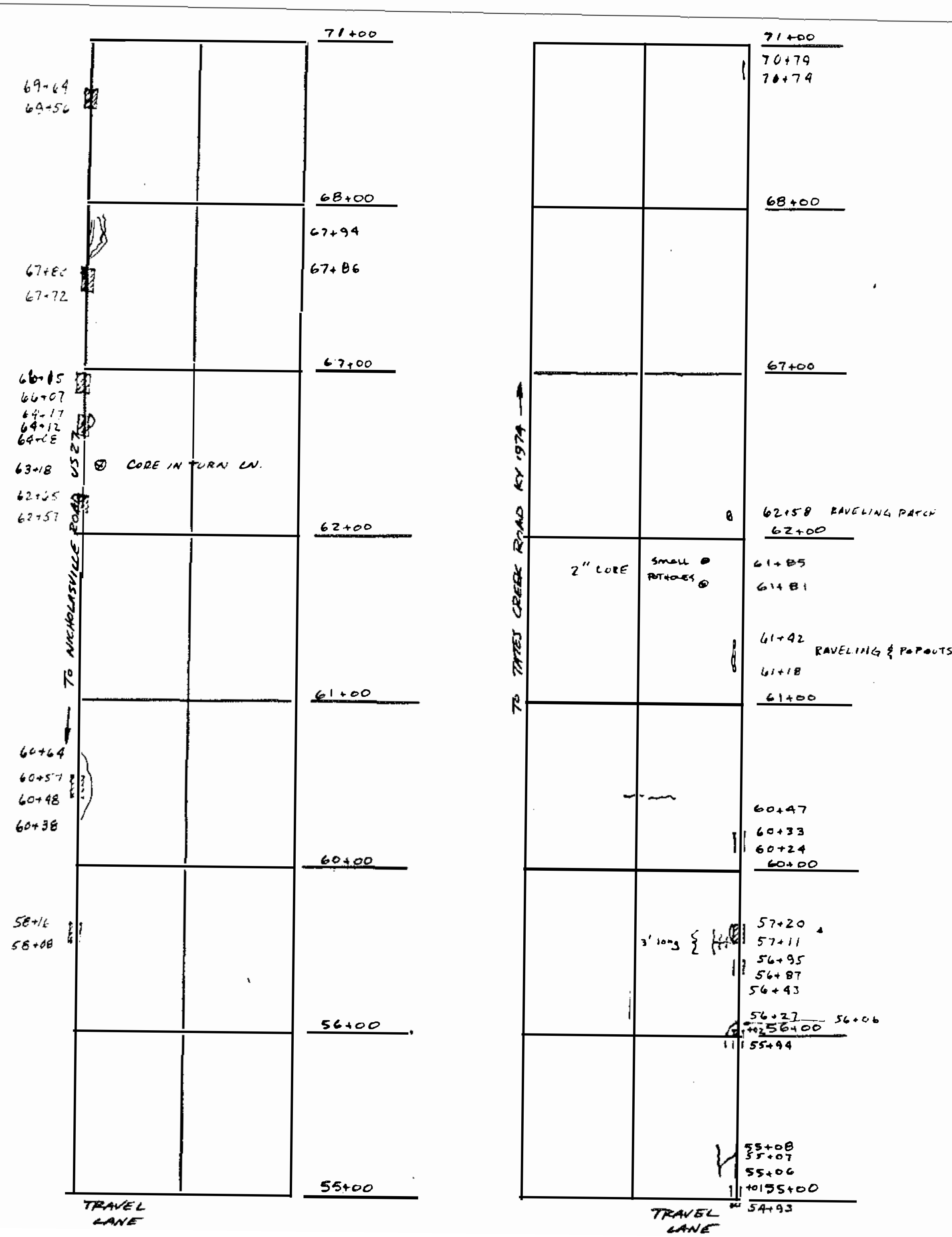
48+50
48+33
48+31
48+18
RAVELING PATCH
48+00
47+00
TRAVEL LANE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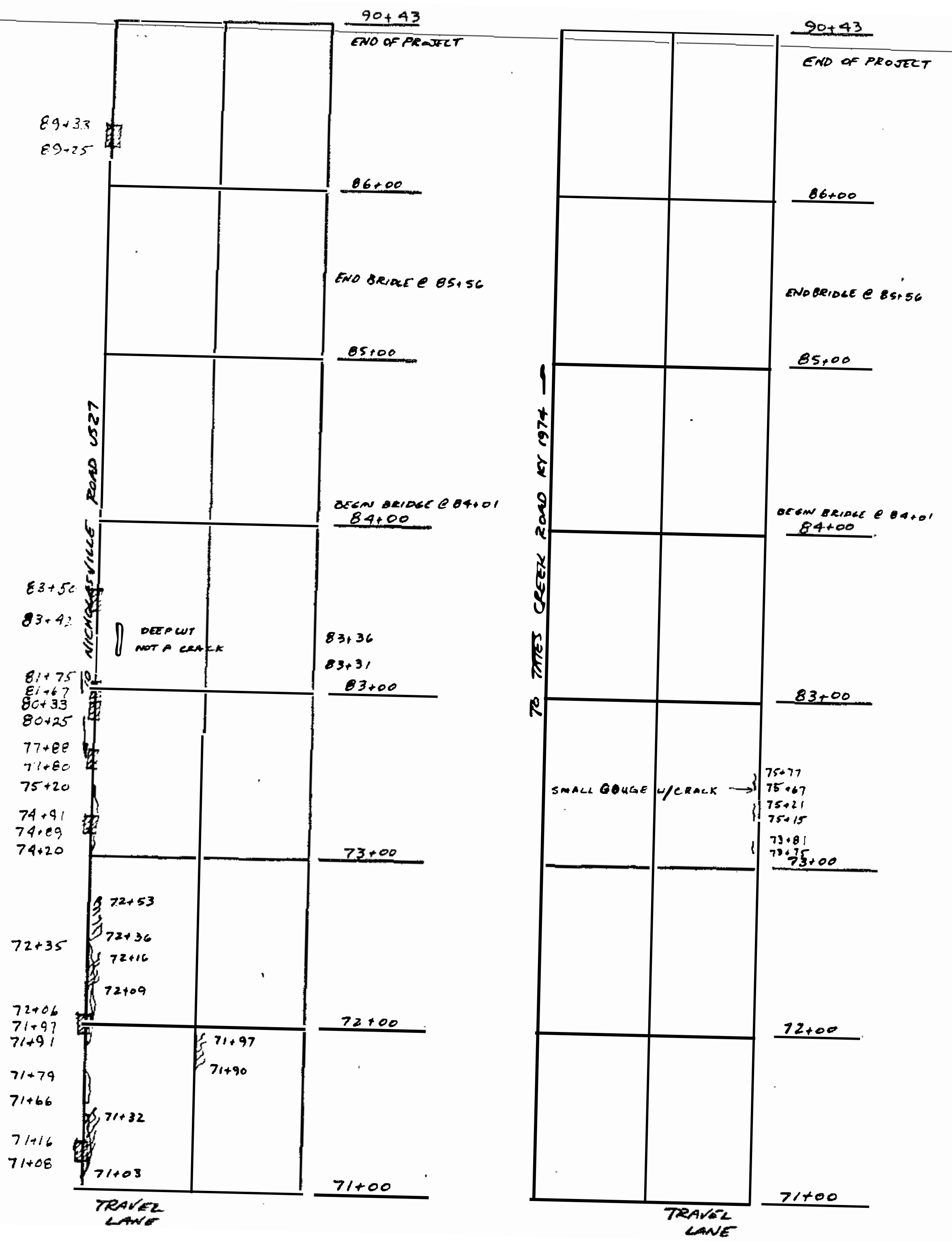
68+00
67+00
62+00
61+00
60+00
56+00
55+00
CORE IN TURN LN.
TO NICHOLASVILLE ROAD US 27
TO TATES CREEK ROAD KY 1974
TRAVEL LANE
2" CORE
SMALL POTHOLES
62+58 RAVELING PATCH
RAVELING & POPOUTS
3' long
TRAVEL LANE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86+00
END BRIDGE @ 85+56
85+00
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BEGIN BRIDGE @ 84+01
84+00
83+50
83+42
DEEP CUT NOT A CRACK
83+36
83+31
83+00
81+75
81+67
80+33
80+25
77+88
77+80
75+20
74+91
74+89
74+20
73+00
72+53
72+36
72+35
72+16
72+09
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72+00
71+97
71+90
71+79
71+66
71+32
71+16
71+08
71+03
71+00
TRAVEL LANE
90+43
END OF PROJECT
86+00
END BRIDGE @ 85+56
85+00
TO TATES CREEK ROAD KY 1974
BEGIN BRIDGE @ 84+01
84+00
83+00
SMALL GOUGE w/CRACK
75+77
75+67
75+21
75+15
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73+75
73+00
72+00
71+00
TRAVEL LANE