Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
TX-99/2966-2		

4. Title and Subtitle	5. Report Date
FLY-ASH BASES IN THE ATLANTA DISTRICT: EVALUATION OF SURFACE TREATMENT BOND AND YEAR-TWO FIELD PERFORMANCE EVALUATIONS	December 1998
	6. Performing Organization Code
7. Author(s) Cindy Estakhri	8. Performing Organization Report No. Report 2966-2
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135	10. Work Unit No. (TRAIS) 11. Contract or Grant No. Project No. 7-2966
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Transfer Office P. O. Box 5080 Austin, Texas 78763-5080	13. Type of Report and Period Covered Research: September 1996 - August 1998 14. Sponsoring Agency Code

15. Supplementary Notes

Research performed in cooperation with the Texas Department of Transportation.
Research Project Title: Durability of Surface Treatments as the Wearing Course Placed on Crushed Fly Ash and Long-Term Performance of Crushed Fly Ash for Flexible Base

16. Abstract

Hydrated fly ash is produced by allowing powder fly ash (Class C) from coal power plants to cure with moisture. The hydrated (cured) fly ash becomes a stiff material that can be crushed to form a synthetic aggregate. When properly processed and compacted to optimum moisture content, the hydrated fly ash continues to gain strength after placement as a base material.

The Atlanta District has constructed six pavement sections since 1993 using hydrated fly ash as the flexible base material. This research project was initiated to evaluate and monitor performance and changes in material properties for these six pavements through the year 2001 and to evaluate a problem experienced during construction where the asphalt surface treatment did not bond well to the base.

A laboratory study was performed to investigate the bond strength of different types of prime materials to the fly-ash base. Curing extent of the base was also a variable in the experiment. Results of the laboratory study revealed that the type of prime material used during construction did not contribute to the inadequate bond achieved. It is more likely attributable to the extent of base cure prior to application of an asphalt membrane. Construction recommendations are provided in this report aimed at achieving adequate bond of the surface treatment to the fly-ash base.

Evaluation of pavement base performance was based on visual documentation, falling-weight deflectometer tests, ground penetrating radar, and compressive strengths of field cores. This report is an interim report documenting the performance evaluations conducted in the spring of 1998. This report covers the second annual evaluation in a series of five.

17. Key Words	18. Distribution Statement
Hydrated Fly Ash, Coal Combustion By-products, Fly Ash, Stabilized Bases, Highway Construction	No restrictions. This document is available to the public through NTIS: National Technical Information Service 5285 Port Royal Road Springfield, Virginia 22161

19. Security Classif.(of this report)	20. Security Classif.(of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	58	

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FLY-ASH BASES IN THE ATLANTA DISTRICT: EVALUATION OF SURFACE TREATMENT BOND AND YEAR-TWO FIELD PERFORMANCE EVALUATIONS

by

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Report 2966-2
Project Number 7-2966
Research Project:
Durability of Surface Treatments as the Wearing Course Placed on Crushed Fly Ash and Long-Term Performance of Crushed Fly Ash for Flexible Base

Sponsored by the
Texas Department of Transportation

December 1998

TEXAS TRANSPORTATION INSTITUTE
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College Station, Texas 77843-3135

IMPLEMENTATION STATEMENT

This report contains recommendations aimed at solving a problem experienced in the Atlanta District with hydrated fly ash used as a base material: asphalt surface treatments did not bond well to the fly-ash base. TxDOT personnel ascertained that potential causes for the lack of bond was tied to the type of prime used (MC-30), the degree of curing in the fly ash base and the high optimum moisture content. The laboratory effort in this study indicates that the MC-30 (in addition to other prime materials evaluated in this study) does not interfere in development of a bond between the asphalt surface treatment and the fly-ash base. Research points to the need for adequate curing of the base prior to application of an asphalt membrane. Specification recommendations are provided in this report which address this issue.

The six test pavements of fly-ash base which are being monitored in this study are performing well thus far. However, some of the nondestructive testing (FWD and GPR) show the need for continued monitoring. It is recommended that the pavements be monitored for the additional three years as scheduled in this study.

If any additional projects are constructed using hydrated fly ash as the base material (prior to completion of the research study), its use is recommended on pavements that do not have heavy truck traffic (until more is understood about this base material).

DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT) or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes.

ACKNOWLEDGEMENTS

The author would like to acknowledge the participation and assistance provided by the Atlanta District personnel. Thanks to Mr. Miles Garrison, Mr. Chappy Shelton and Mr. James Voss for providing the field cores, FWD data, and field assistance, and special thanks also to Mr. Miles Garrison for providing direction to the study as the Project Director. Thanks also go to the maintenance office personnel in the district who provided traffic control for the field evaluations.

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SUMMARY

A laboratory study was conducted to evaluate the bond strength of surface treatments to hydrated fly-ash base materials. Variables in the experiment included (1) type of prime material, and (2) curing condition for the base material. Tests used to evaluate the bond strength included a torsional shear test, a South African durability test, and a visual/subjective evaluation.

Based on the laboratory study, no obvious solution was identified as to the cause of the surface treatment not bonding to the base material. The laboratory study showed that it is possible to develop a good bond of the surface treatment to the hydrated fly- ash base using various types of prime materials, including MC-30. Inadequate bond of surface treatments to hydrated fly-ash base materials is probably not attributable to the type of prime material used.

Researchers believe that adequate curing of the base prior to application of the surface treatment may be the key to achieving a good bond. Since the hydrated fly ash base develops strength with time, it is important not to trap excess moisture in the base which could cause a strength reduction near the surface. Construction recommendations and specification changes are provided in the report.

Visual evaluations in 1998 showed that all six test pavements are still in very good condition. The 1998 falling weight deflectometer (FWD)D data were compared to that taken in 1997. There is no indication of any *weakening* of the base materials with time.

Ground penetrating radar (GPR) surveys of all six test pavements indicate a very high dielectric constant for the fly-ash bases. Values of this magnitude typically indicate the presence of excessive amounts of moisture and would generally warrant concern. However, optimum moisture content for these pavements was as high as 35%; therefore, these high dielectric constants may not necessarily be cause for alarm.

This document covers the second evaluation which occurred in the spring of 1998. Annual evaluations are scheduled for the next three years.

BACKGROUND

Hydrated fly ash is produced by allowing powder fly ash (Class C) from coal power plants to cure with moisture. The hydrated (cured) fly ash becomes a stiff material that can be crushed to form a synthetic aggregate. When properly processed and compacted to optimum moisture content, the hydrated fly ash continues to gain strength after placement as a base material (1).

The Atlanta District constructed six pavement sections in 1993 through 1995 using hydrated fly ash as the flexible base material. District personnel are pleased thus far with the performance of this industrial by-product as a base material; however, its long-term performance is in question. And while performance of the material as a base has been acceptable, problems were encountered with surface treatments separating from the base course. This research project was initiated to evaluate and monitor performance and changes in material properties for these six pavements through the year 2001. Evaluation of performance shall be based on the following types of data:

- visual evaluations of surface distress,
- nondestructive field testing (falling weight deflectometer, as a minimum), and
- compressive strength of field cores.

Also included in this study is a laboratory investigation into the cause and cure for the failure of the surface treatments on the hydrated fly-ash base courses.

History

The Atlanta District first began evaluating crushed fly ash in 1990. The district laboratory's initial investigation of the material found that the following material properties for the fly ash:

- Triaxial Classification - *Super* Class 1,
- Unconfined compressive strength: 220 psi,
- Dry loose unit weight: 68.0 lb/ft^3,

- Compacted dry density at optimum moisture of 28.6%: 85.5 lb/ft^3,
- Los Angeles Abrasion: 47, and
- 5 Cycles of freeze-thaw (15 hours freeze-thaw at room temperature for 9 hours) showed no damage and no volume change.

Based on promising test results from the laboratory investigation, the district worked with Southwestern Electric Power Company (SWEPCO) to construct a test section for the power plant haul road. This was a successful venture and performance of the pavement was promising, which led to the construction of six test pavements throughout the district and are the subject of this study.

A description of each of the six test sites, their locations, and typical cross sections are presented in Table 1. At the time these pavements were constructed, the final surface for all of the pavements (except the IH-20 frontage road which was designed for a surface treatment followed by an asphalt concrete surface course) was to have been a one/two course surface treatment directly over the primed fly-ash base. However, there were several problems that occurred soon after placement of surface treatments whereby the surface treatment delaminated from the underlying base material. It should be noted also that the projects on SH 154, FM 1326, and FM 1520 did not have these delamination problems except in some isolated spots. These problems eventually subsided.

Researchers interviewed contractors and district personnel in an attempt to identify potential construction practices/techniques which could have contributed to this phenomenon; however, no prominent solution could be identified. Therefore, researchers implemented a laboratory investigation aimed at identifying the cause of these types of failures. This laboratory investigation is described in the following chapter.

Table 1. Test Site Descriptions

Roadway	County	Project Length	Location		Project Designation	Job Completion Date	Typical Pavement Cross Section
			From	To			
LP 390	Harrison	2.5 mi	US 59 in Marshall	0.3 mi S. of SH 43	1575-05-005 STP 92(7)UM	12/10/93	Grade 4 Seal Coat 2.0 in. Type C Hot Mix MC-30 Prime 10.0 in. Fly-Ash Base 8.0 in. Lime/FA Subgrade
IH 20 (FR)	Harrison	3000 ft	1.0 mi E. of Gregg Co. Line	0.6 mi W. of Loop 281	0495-08-056 CC 495-8-56	7/13/94	2.0 in. Type C Hot Mix One-Course Surface Trt. MC-30 Prime 11.0 in. Fly-Ash Base 8.0 in. Lime/FA Subgrade
SH 154	Upshur	2000 ft	0.1 mi E. of US 259	0.5 mi E. of US 259	0402-02-018 HES 000S(661)	6/8/93	Grade 4 Seal Coat One-Course Surface Trt. MC-30 Prime 6.5 - 13.0 in. FA Base
FM 1326	Bowie	400 ft	3.0 mi N. of US 82	3.0 mi N.	1570-02 Maint. Forces	9/93	CRS-2p Grade 5 CRS-2p Grade 4 5.5 in. Fly-Ash Base 2.0 in. Asphalt Concrete 5.0-7.0 in. Indeterminate (LRA or Black Base?)
FM 1520	Camp	7800 ft	0.1 mi E. of Picket Spring Branch	FM 1521	1232-03-09 A 1232-3-9	8/9/93	One-Course Surface Trt. MC-30 Prime 9.0 in. Fly-Ash Base 8.0 in. Lime/FA Subgrade
FM 560	Bowie	2300 ft	Barkman Creek and Relief	2300 ft N.	1021-01-007 BR 90(241)	4/28/95	1.8-2.5 in. Hot Mix MC-30 Prime One-Course Surface Trt. 6.0 - 12.0 in Fly Ash Base 0-6.0 in. Bank-Run RG

LABORATORY INVESTIGATION OF SURFACE TREATMENT BONDING TO FLY-ASH BASE

Descriptions of the problems encountered when asphalt surface treatments were placed on crushed hydrated fly-ash bases indicate the potential for at least two types of failure mechanisms. Either or both of these mechanisms could have detrimental effects on the interface between the base and the surface treatment. These are described below:

1. The high moisture content required for optimum compaction of the crushed fly ash may not have a chance to escape and moisture might accumulate in the upper portion of the base weakening the base material near the interface. As in concrete, where excess water creates a high water cement ratio (and lower strength), excess moisture in this type of stabilized base might also cause a strength reduction.
2. Another factor which might contribute to the surface treatment failure is the type of material used for a prime. Some have reported that oil (diesel or kerosene that is present in some prime materials) will prevent a cementitious bond (cement, lime, or fly ash) from occurring.

These two mechanisms working together could have had a detrimental effect on the interfacial bond between the base and the surface treatment. Decreased bond strength could result in complete failure (delamination) at the interface due to traffic (particularly braking or turning) or water vapor pressure.

Researchers designed a laboratory experiment aimed at measuring the effects of these mechanisms in the laboratory under controlled conditions that simulated field conditions as closely as possible.

Torsional Shear Test

The test procedure which was chosen to evaluate the bond strength between the prime material and the hydrated fly-ash base was a torsional shear test. This laboratory procedure was developed by Mantilla and Button (2) and was used to quantify interfacial strength at the prime coat interface. Cylindrical samples are molded in 6-inch diameter molds. The molds are fabricated in two sections to accommodate shear testing at the primed interface between the base and the pavement layer (Figure 1). An MTS torsional shear machine was used to test the samples. The torque-twist plots of each were recorded and a typical plot is shown in Figure 2.

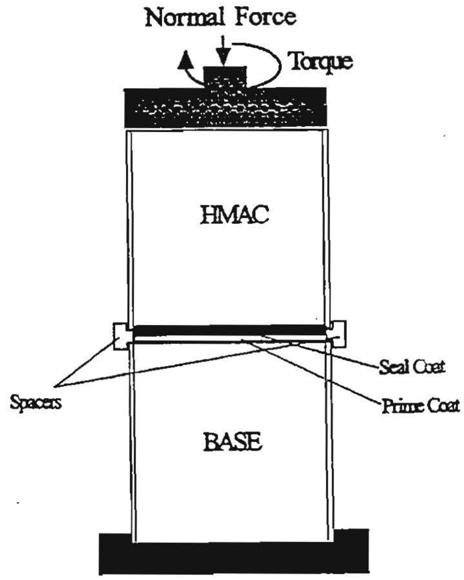

Figure 1. Diagram of Cylindrical Molds Fabricated to Accommodate Torsional Shear Testing at the Primed Interface Between the Asphalt Surface Treatment and the Hydrated Fly-Ash Base

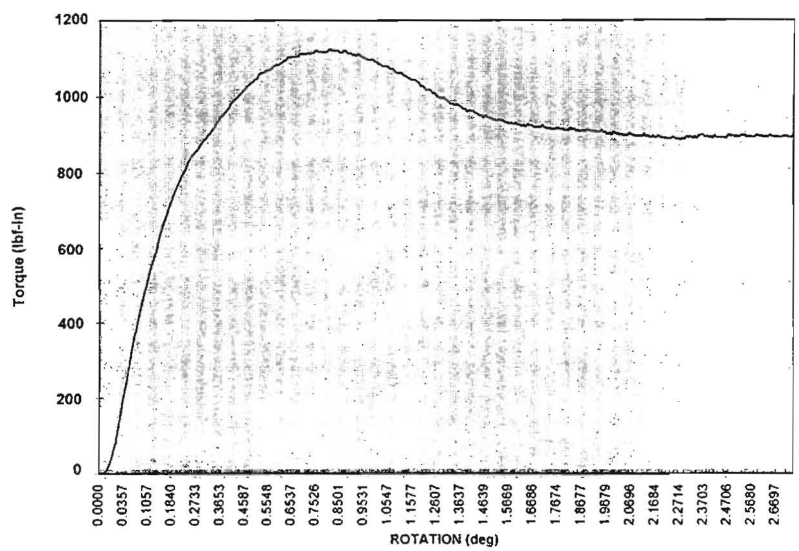

Figure 2. Typical Data Plot from the Torsional Shear Test

Materials and Sample Preparation

Samples of hydrated fly-ash base material were obtained from the Welsh Power Plant in Cason, Texas, and brought back to Texas Transportation Institute's, TTI, laboratory for experimentation. An optimum moisture-density curve as shown in Figure 3 yielded an optimum moisture content of 28.5% with a dry density of 82.0 lb/ft^3.

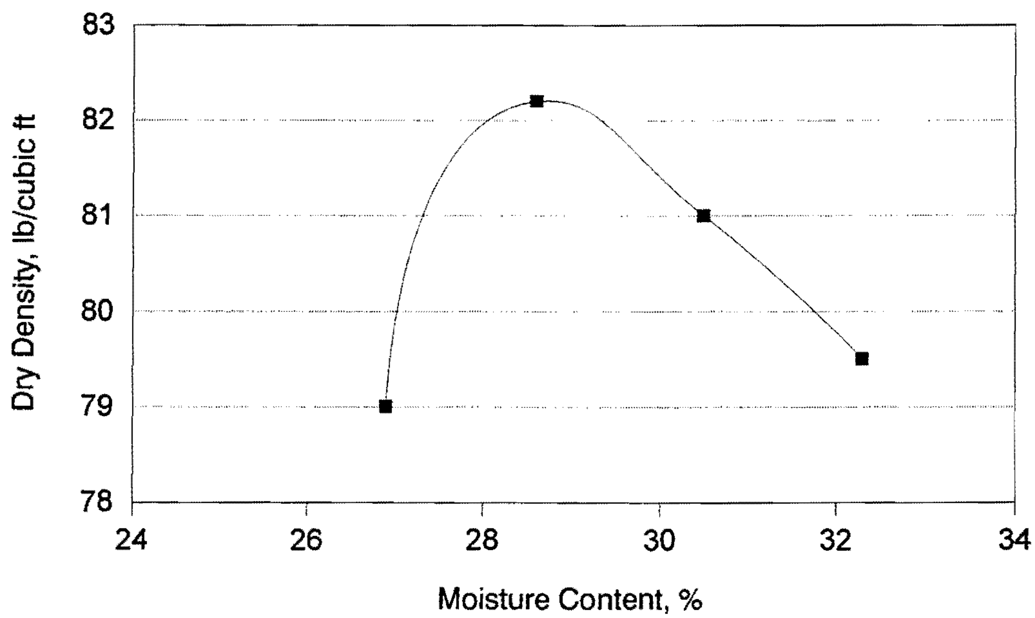

Figure 3. Optimum Moisture-Density Curve for Hydrated Fly-Ash Base

Samples of the hydrated fly-ash base material were compacted at optimum moisture content in 6-inch diameter molds. The samples were cured according to the conditions described in the experiment design below. The samples were then primed using one of the prime materials listed below. Prime application rates were 0.18 gal/yd^2 for the MC-30 and 0.22 gal/yd^2 for the emulsions. The emulsion samples were diluted one part emulsion to three parts water. Base samples were cured again according to the conditions described in the next section. An AC-10/Grade 4 surface treatment was then placed on top of the samples to simulate field conditions. For those samples where seal coat grade emulsions were used as the prime (CRS-2 and HFRS-2p), the same emulsions (not diluted) were also used to construct the surface treatment.

The specimens were again allowed to cure according to various conditions described below. The upper half of the mold was attached to the lower half with the base material. Spacers were placed between the two halves to create a 0.1 inch space at the point of shear. This was designed to apply a shear force at the primed interface between the base and the asphalt seal. After curing, hot-mix asphalt was compacted in the top portion of the mold. The hot-mix asphalt layer in the upper half of the mold was bonded to the surface treatment and provides a means of applying torque to the specimen. A uniform torsional deformation rate of 2.9E-04 radians per second was applied to the top of the sample while holding the bottom portion stationary until failure occurred. Specimens were tested at 77°F.

Experiment Design

There were two types of variables which were investigated in this laboratory experiment: (1) priming materials, and (2) curing conditions.

The priming materials which were used in this experiment were selected in cooperation with district personnel and included the following:

- No Prime (control);
- MC-30 (Lion Oil Company, El Dorado, Arkansas);
- SS-1 (Ergon Asphalt and Emulsions, Mt. Pleasant, Texas);
- CRS-2 (Ergon Asphalt and Emulsions, Mt. Pleasant, Texas);
- HFRS-2p (Ergon Asphalt and Emulsions, Mt. Pleasant, Texas); and
- EPR-1 (Blacklidge International, Houston, Texas).

There were three types of curing conditions which were simulated in the laboratory:

- *Curing Condition 1* was an attempt to simulate field practice. The base samples were cured for 24 hours after the base was compacted. The primed base was cured an additional 24 hours prior to application of the surface treatment and then tested the following day.

- *Curing Condition 2* was the same as the first condition except that the base was cured for 72 hours prior to applying the prime (to allow a chance for some of the moisture to escape).
- *Curing Condition 3* was the same as the first condition except that the primed base was allowed to cure for 72 hours prior to application of the surface treatment. Note: All curing took place at 104°F.

The above variables provided for a 3 x 6 full factorial experiment and a total of three samples for each condition were produced, except that the control specimens which had no prime were tested under curing conditions 1 and 2 but not 3 (since there was no prime added). For the Control specimens cured under condition 2, the base samples were simply cured 72 hours prior to application of the surface treatment. A total of 51 samples was produced. Two of the samples at each factor were tested using the torsional shear test and the third sample was visually evaluated (by using hand tools such as a knife/spatula to determine if the surface treatment could be easily *peeled* from the base (which was often the case where some of the field problems existed).

Torsional Shear Test Results

Results of the torsional shear strength tests are shown below in Table 2. A statistical analysis was performed to analyze the data in this table. Results of an analysis of variance revealed that there is no statistical difference between the different curing conditions and no significant difference in the priming materials. A visual plot of the data in Table 2 is shown in Figure 4. In this figure, each bar represents the mean of the two values shown in Table 2 . As in the statistical analysis, this plot also does not reveal a clear distinction between any of the prime materials or curing conditions.

Table 2. Laboratory Test Results of Torsional Shear Test

Priming Materials	Torsional Shear Strength, lbf-in		
	Curing Condition 1 *Current Practice (prime w/in 24 hrs and test w/in 24 hrs of priming).*	**Curing Condition 2** *Cure Base for 72 hrs prior to applying prime then apply surface treatment and test w/in 24 hrs.*	**Curing Condition 3** *Apply prime within 24 hours of base construction but allow prime to cure for a few days before testing.*
MC-30	1103.7 992.2	1098.4 734.6	997.5 965.6
EPR-1	1111.7 971.0	988.7 901.9	1094.9 955.0
HFRS-2p	1181.6 913.4	1294.9 1293.1	1093.1 974.5
SS-1	989.5 731.1	977.2 1134.7	929.4 1094.9
None	932.9 1157.7	930.2 1106.4	None. Same as Condition 2 (since no prime was applied).
CRS-2	1118.8 1357.7	1065.7 1089.6	1007.2 894.0

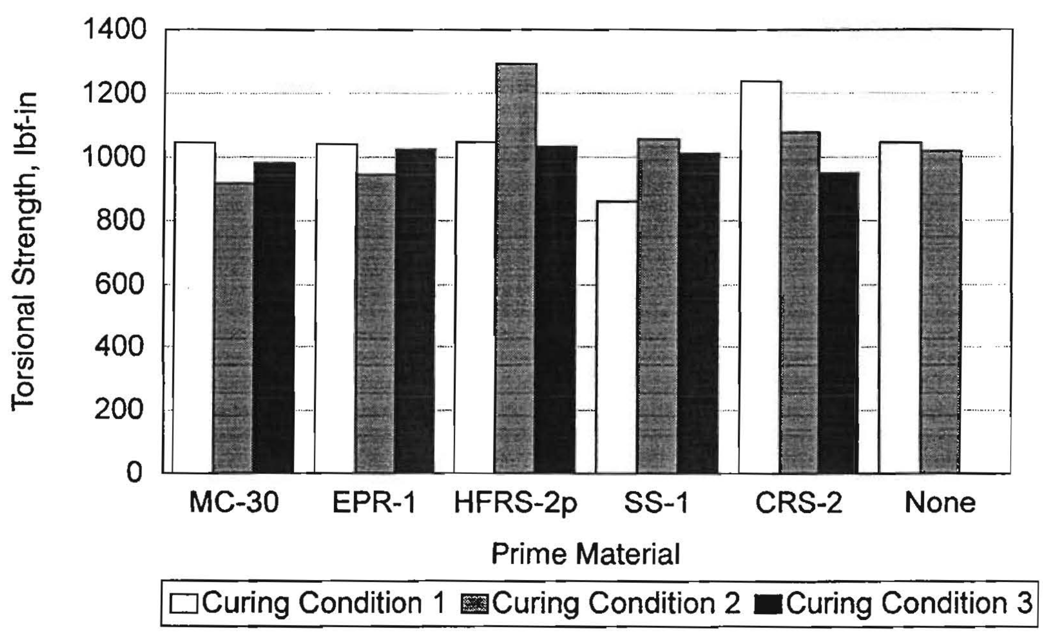

Figure 4. Torsional Strength at Interface of Different Types of Prime and Fly-Ash Base Cured Under Different Conditions

A typical photograph of two of the failed specimens is shown in Figure 5 below. This photo shows the specimens for the CRS-2 prime material and curing condition 1. As shown in the photograph, failure occurred just below or at the interface of the prime and the base material. Note that the shorter specimens in front were sheared from specimens in back, i.e., failure plane is shown in photograph.

Figure 5. Failed Torsional Shear Test Specimens - CRS-2, Curing Condition 1.

As mentioned previously, one sample for each of the priming materials/curing conditions was not tested but was visually and subjectively evaluated. Using tools such as a knife and spatula, attempts were made to remove the seal from the base material by hand. In the field, when some of these pavements were constructed, the bond of the surface treatment to the underlying base was so poor, the surface treatment could literally be *peeled* from the pavement. In the laboratory study, however, the surface treatment seemed very well bonded to the base material in all cases.

South African Durability Test

Since the torsional shear test experiment did not show any differences in the variables examined in the experiment, researchers tried to incorporate the effects of traffic into an experiment. It was postulated that perhaps traffic on the seal might cause damage at the interface of the seal if the base had not yet developed its full strength. TTI's South African Durability Test was used to simulate traffic.

For this test the hydrated fly ash was compacted into a beam mold (17.7 in x 3.0 in x 3.0 in) in three equal layers. Each layer was compacted with 56 blows using the modified compaction method (ASTM D 1557). A static load of 10,000 lb was applied and cycled five times to provide maximum density and a smooth, finished surface. The beam was removed from the mold and then subjected to accelerated curing by placing it in a sealed chamber (with about 2 oz of water) and storing it in a 160°F room for seven days. After curing, the beam was cut, using a diamond blade saw, to a length of approximately 10.6 in and molded (sides and bottom) with gypsum. The beam was then cut to a height of 2.0 in and the surfaces of the specimens were treated with the different prime materials, cured for 24 hours at 104°F, and topped with the Grade 4 surface treatment. The prime materials used were the same as those shown in Table 2. One specimen was produced for each prime for a total of six specimens.

The molded beam was then placed in the water bath of the erosion testing device. It was allowed to soak for 1.5 hours prior to testing for durability. It was then subjected to 5000 wheel load repetitions.

At the end of the test, none of the test specimens showed any degradation. Keep in mind, however, that the test was performed after the fly-ash base was cured (a condition which may not always exist in the field). This was necessary because the conditions of the test require that the specimen be trimmed or cut using a diamond blade saw and this could not have been done on an uncured specimen. This test does, however, indicate that there were no apparent problems with the surface-treatments after the base was fully cured.

Efflorescence

Efflorescence is a crystalline deposit of water-soluble salts that sometimes appears on the surface of brick masonry. The result of this phenomenon has been seen on the hydrated fly-ash base materials: both on unsurfaced as well as asphalt-surfaced bases. Although efflorescence on brick masonry is unsightly, it is usually not harmful (3).

Efflorescence occurs when water-soluble salts in solution are brought to the surface and deposited there by evaporation. Certain simultaneous conditions must exist in order for efflorescence to occur. Soluble salts must be present in the system. There also must be a source of water in contact with the salts for sufficient time to permit them to dissolve. There must be migration of salt solutions to the surface in an environment which allows evaporation.

Some have postulated that the efflorescence which is appearing on the surface of the pavement is actually the active stabilizing agent in the fly ash which is leaching to the surface. If this is the case, one would expect that under wet conditions, the base might be losing strength. Field information collected thus far in this study, however, does not indicate that the base materials are losing strength.

VISUAL CONDITION SURVEYS

In this research study, visual condition surveys are performed annually on all six test pavements in late spring. The most recent survey was performed during the last week of April in 1998. The manual survey was conducted in accordance with the procedures set up for a SHRP LTPP distress survey (4). In addition to measuring the quantity of each distress at each severity level, a map showing the location of crack-distress was also produced.

Loop 390

This project begins at US 59 in Marshall and extends to 0.5 km south of SH 43. The total length of the project is about 4.0 km. For visual condition surveys, the project was evaluated at 13 locations (200 ft survey length per location) in the eastbound travel lane. In 1997 there were three types of distress beginning to be evident on Loop 390: alligator cracking, a slight flushing of the seal coat surface, and rutting. However, between the 1997 and 1998 evaluations, a Grade 4 chip seal was placed on the surface so there is no longer evidence of alligator cracking at this time. Quantities of distress at each survey location are shown below in Table 3.

The surface is exhibiting a slight amount of flushing at some locations. Some locations also showed a slight increase in rutting depths from the previous year; however, overall the pavement is in good condition.

IH-20 Frontage Road

The IH-20 Frontage Road project begins 0.9 miles east of the Gregg Co. Line and continues eastward for 3000 feet. This pavement is in very good condition. Raveling which was observed in 1997 had not progressed any further in 1998. There were some isolated spots of alligator cracking as shown in Table 4. The project was evaluated at three locations (200 ft length at each location) in the eastbound lane. The quantity of distress present at each location is shown in Table 4.

Table 3. Loop 390 Distress

Location (each location represents a 200 ft length)	Alligator * Cracking, sq ft		Flushing, sq ft		Rutting, in			
					Left Wheelpath		Right Wheelpath	
	1997	1998	1997	1998	1997	1998	1997	1998
1	0	0	0	590 (s)	0	0.1	0	0.3
2	0	0	0	97 (s)	0	0.2	0	0.3
3	0	0	0	260 (s)	0.1	0.1	0.1	0.1
4	0	0	0	330 (s)	0.1	0.1	0.1	0.1
5	0	0	0	260 (s)	0.2	0.2	0.2	0.3
6	600 (s)	0	600 (s)	800 (s)	0.4	0.6	0.5	0.6
7	1000 (s)	0	1200 (s)	400 (s)	0.5	0.5	0.5	0.5
8	1000 (s)	0	1200 (s)	600 (s)	0.4	0.4	0.4	0.4
9	600 (s)	0	1000 (s)	300 (s)	0.4	0.3	0.4	0.4
10	0	0	400 (s) 200 (m)	250 (s)	0.1	0.1	0.1	0.1
11	0	0	600 (s)	0	0.1	0.1	0.1	0.1
12	0	0	0	0	0.1	0.1	0.1	0.1
13	0	0	0	0	0	0	2	0

Severity Levels : (s) slight, (m) moderate.

* A Grade 4 Seal Coat was constructed on the pavement between the 1997 and 1998 evaluations.

Table 4. IH 20 Frontage Road Distress

Location (each location represents a 200 ft length)	Raveling, sq ft		Alligator Cracking, sq ft	
	1997	1998	1997	1998
1 Core Location 1	43 (s)	43 (s)	0	5 (s)
2 Core Location 2	54 (s)	54 (s)	0	3 (s)
3 Core Location 3	43 (s)	43 (s)	0	0

Severity Level: (s) slight, (m) moderate.

SH 154

This project is located in Diana beginning 0.1 mi east of US 259 and extending to 0.5 mi east of US 259. The entire length of this pavement was visually evaluated in the westbound lane. The primary distress of interest on this pavement is some slight transverse cracking. These cracks are beginning in the shoulder and most have not progressed all the way across the main lanes of travel; however, the cracks are very evenly spaced (every 12 to 13 ft) and might be attributable to shrinkage of the fly-ash base. A summary of the distress is shown in Table 5 below. Note that there is no appreciable increase in the amount of cracking observed from 1997 to 1998. In fact, it appears that some of the cracks observed in 1997 may have healed by 1998.

Table 5. SH 154 Distress

Location (beginning at east end of project)	Transverse Cracking in westbound lane, linear ft		Longitudinal Cracking in westbound lane, linear ft	
	1997	1998	1997	1998
0 - 200 ft (1st core location)	6 (s)	8 (s)	0	0
200 - 400 ft	24 (s)	24 (s)	0	0
400 - 600 ft	12 (s)	12 (s)	0	0
600 - 800 ft	17 (s)	7 (s)	0	0
800 - 1000 ft (2nd core location)	8 (s)	8 (s)	8 (s)	7 (s)
1000 -1200 ft	38 (s)	38 (s)	56 (s)	36 (s)
1200 -1400 ft	6 (s)	0	0	0
1400 - 1600 ft	0	0	0	0
1600 - 1800 ft (3rd core location)	0	0	0	0
1800 - 2000 ft	26 (m)	44 (m)	22 (m)	22 (m)

Severity Level: (s) slight, (m) moderate.

FM 1326

The FM 1326 project begins about 3.0 mi north of US 82. It was constructed by district maintenance forces and is about 400 feet in length. The entire length of pavement (both lanes) was evaluated visually. No distress of any kind was evident in the seal coat surface.

FM 1520

The FM 1520 project is located in Camp County and begins 0.1 miles east of Pickett Spring Branch extending to FM 1521. Its total length is about 7800 feet. This project was visually evaluated at eight locations as shown below in Table 6. There was virtually no change in the condition of the pavement from 1997 to 1998.

Table 6. FM 1520 Distress

Location (each location represents a 200 ft length)	Flushing, sq ft	
	1997	1998
1	1000 (slight)	1000 (slight)
2	1200 (slight)	1200 (slight)
3	1500 (slight)	1500 (slight)
4	320 (slight)	320 (slight)
5	0	0
6	0	0
7	0	0
8	0	0

FM 560

The FM 560 project is located near Hooks and begins at Barkman Creek and Relief and extends north for 2300 feet. The primary distress evident on this pavement is a moderate amount of flushing in the wheelpaths. The surface treatment under the hot-mix overlay was constructed using a multi-grade asphalt (10W30) and appears to be flushing through the hot mix to the surface. There was also a very slight amount of cracking in the northbound lane. At about 1500 feet north of where the project begins (Barkman Creek), four transverse cracks appeared in the center of the northbound lane in 1997. Each crack was less than three feet in length. There was also one longitudinal crack five feet long. In 1998 there was a bit more cracking as shown in Table 7 below; however, the pavement is still in very good condition. The project was evaluated at three locations (200 ft length at each location) in the northbound lane. The quantity of distress present at each location is shown below in Table 7.

Table 7. FM 560 Distress

Location (each location represents 200 ft in length)	Flushing, sq ft		Longitudinal Cracking, linear ft		Transverse Cracking, linear ft	
	1997	1998	1997	1998	1997	1998
1 Core Location 1	1000(m)	1000(m)	0	12 (s)	0	23 (s)
2 Core Location 2	150 (m) 120 (s)	150 (m) 120 (s)	5 (s)	5 (s)	10 (s)	10 (s)
3 Core Location 3	0	0	0	0	0	0

Severity Level: (s) slight, (m) moderate.

FIELD CORE AND FIELD TESTING DATA

Attempts were made to obtain three cores from each of the six test pavements. Laboratory staff from the Atlanta District performed the coring operations using district coring equipment. Water was used to cool the bit during the coring operations. It was not possible to obtain as many cores as desired because, in some cases, the cores were not retrievable. They broke into pieces when attempting to remove them from the pavement or core bit.

TTI performed unconfined compressive strength testing on the field cores. Plaster was used to cap the ends of the specimens prior to testing. For unconfined compressive strength, it is desirable to have a sample length (L) to diameter (D) ratio of at least 2. However, some of the cores were very short and L/D ratios varied from 0.76 to 2.2. Adjustment factors were used to facilitate comparing cores of different thickness as described in Tex 418-A . These results are compared with last year's results in Figure 6.

At the time the pavements were visually evaluated, falling weight deflectometer (FWD) testing was also performed by the Atlanta District personnel. The FWD is a test which nondestructively measures stiffness and relative deflection of the various layers of a pavement system. A load which simulates a truck load is applied to the pavement through a 12 inch diameter load plate. Pavement deflection is measured by geophones placed at various distances from the plate, yielding a "deflection bowl." Deflection magnitudes and bowl shape are used to calculate stiffness and relative deflection of each layer. In general, the lower the deflection and higher the stiffness, the better the pavement's ability to distribute and carry load without rutting and cracking. FWD deflections were measured at regular intervals along the length of each test pavement.

Moduli values of the pavement layers were calculated using the TTI Modulus Analysis System (Version 5.1). Results of the analysis are presented in Tables 8 through 13. Of particular interest for this project is the moduli values for the base (E2). TTI experience has shown that for stabilized bases, moduli values between 145,000 and 500,000 psi are optimum in terms of field performance. Bases with moduli values between

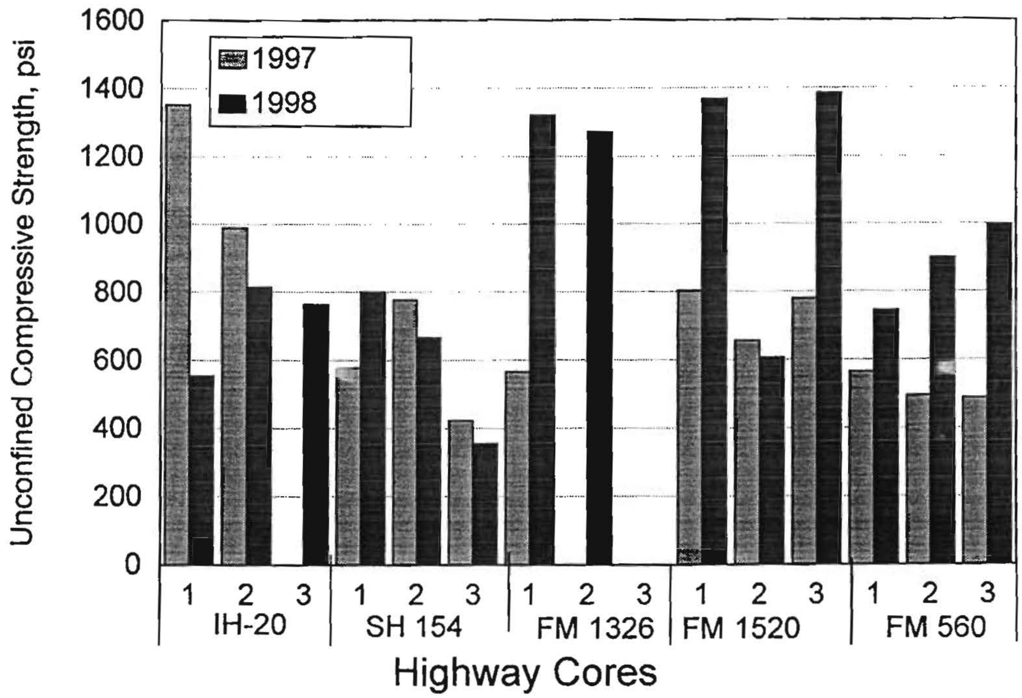

Figure 6. Unconfined Compressive Strength of Highway Cores

Table 8. FWD Data Analysis - Loop 390

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19
County: 103
Highway/Road: SL0390

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	2.00	199,980	200,020	H1: PR = 0.35
Base:	10.00	30,000	500,000	H2: PR = 0.30
Subbase:	8.00	5,000	500,000	H3: PR = 0.25
Subgrade:	207.50	15,000		H4: PR = 0.40

Station ft	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Depth to Bedrock
114.000	11,341	22.24	14.74	8.50	5.17	3.30	2.26	1.69	200.	126.4	5.4	17.8	3.13	158.23
642.000	11,341	8.36	5.48	3.37	2.39	1.65	1.11	0.78	200.	450.8	29.5	34.0	2.26	155.71
1171.000	12,139	14.70	8.60	4.08	2.49	1.75	1.35	1.11	200.	147.4	18.1	33.8	6.99	226.18
1698.000	11,086	12.76	9.33	6.91	4.96	3.48	2.57	1.86	200.	419.1	18.1	14.5	0.60	209.38
2226.000	10,991	11.91	7.71	4.19	2.59	1.78	1.35	1.09	200.	222.3	16.3	30.2	5.19	247.91
2754.000	11,023	10.07	6.84	4.20	2.81	1.97	1.46	1.05	200.	336.4	24.0	26.7	3.09	197.66
3281.000	11,317	11.34	7.46	4.80	3.38	2.57	2.04	1.59	200.	255.0	57.5	20.4	3.75	300.00
3811.000	10,630	16.27	9.82	5.90	3.71	2.63	2.02	1.59	200.	133.8	17.7	19.0	3.14	300.00
4338.000	10,701	15.95	8.71	5.19	3.72	2.84	2.17	1.74	200.	101.2	58.6	18.1	4.50	300.00
4634.000	12,222	17.80	8.58	4.74	3.33	2.57	2.08	1.67	200.	91.8	48.1	23.4	6.86	300.00
4871.000	11,110	20.32	11.09	5.52	3.65	2.64	2.00	1.53	200.	81.6	17.1	20.5	5.20	287.88
5394.000	11,130	14.45	9.41	5.65	3.57	2.40	1.76	1.38	200.	200.0	12.9	22.5	2.81	211.66
5923.000	11,793	11.67	7.48	4.65	3.51	2.65	2.15	1.65	200.	224.2	80.2	20.7	4.43	300.00
6449.000	11,023	10.13	7.54	5.23	3.43	2.25	1.65	1.22	200.	500.0	10.4	24.9	3.44	175.33
6980.000	11,202	8.91	5.04	3.35	2.41	1.81	1.44	1.24	200.	240.3	134.7	29.4	2.96	300.00
7506.000	11,265	13.17	6.32	4.15	3.07	2.24	1.71	1.33	200.	109.6	159.4	24.0	2.81	300.00
8035.000	11,269	15.69	8.11	5.07	3.50	2.56	1.96	1.59	200.	106.4	59.7	20.7	2.95	300.00
8562.000	10,034	18.90	12.95	7.89	4.96	3.21	2.19	1.65	200.	150.8	5.1	16.4	2.33	171.48
9093.000	11,162	10.43	5.28	2.85	1.92	1.38	1.05	0.85	200.	173.4	47.5	39.1	4.50	300.00
9677.000	11,317	9.73	5.43	3.09	2.14	1.49	1.18	0.96	200.	229.9	49.0	35.9	3.92	266.19
10147.000	10,490	12.83	8.69	4.73	3.04	2.16	1.59	1.33	200.	204.1	14.6	24.1	5.63	276.13
10673.000	10,943	11.57	7.29	4.41	2.84	1.82	1.28	0.96	200.	255.9	15.4	29.2	1.22	151.38
11203.000	10,562	9.31	5.88	3.79	2.54	1.70	1.25	0.96	200.	326.3	27.9	28.9	1.10	196.67
11731.000	10,653	12.06	6.86	3.98	2.42	1.54	1.04	0.72	200.	194.0	15.6	34.0	0.90	146.10
12259.000	11,213	12.14	8.09	4.26	2.47	1.66	1.24	0.97	200.	230.3	12.2	33.9	6.16	148.45
12984.000	10,681	20.83	12.54	6.25	3.39	2.14	1.57	1.20	200.	91.1	7.3	23.3	6.33	106.03
Mean:		13.60	8.28	4.88	3.21	2.24	1.67	1.30	200.	215.5	37.0	25.6	3.70	227.57
Std. Dev:		3.88	2.43	1.37	0.85	0.58	0.43	0.33	0.	113.1	38.3	6.7	1.81	84.37
Var Coeff(%):		28.52	29.31	28.19	26.51	25.99	25.95	25.73	0.	52.5	100.0	26.2	48.94	37.07

Table 9. FWD Data Analysis - IH 20 Frontage Road

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19
County: 103
Highway/Road: IH0020

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	2.00	199,980	200,020	H1: PR = 0.35
Base:	11.00	100,000	6,000,001	H2: PR = 0.35
Subbase:	8.00	20,000	700,000	H3: PR = 0.25
Subgrade:	45.00	15,000		H4: PR = 0.40

Station ft	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Depth to Bedrock
423.000	10,351	3.06	2.30	1.84	1.44	1.11	0.87	0.69	200.	5284.7	83.2	11.1	2.71	36.00
665.000	10,661	2.79	1.89	1.48	1.15	0.88	0.69	0.55	200.	3293.4	563.6	15.3	3.63	24.00
895.000	10,216	5.00	4.01	3.10	2.34	1.66	1.18	0.87	200.	2088.6	20.6	9.1	2.00	203.89
1037.000	10,240	2.92	2.20	1.75	1.41	1.12	0.90	0.75	200.	4860.5	305.2	10.5	2.81	36.00
1103.000	10,053	5.85	3.87	2.60	1.87	1.44	1.13	0.91	200.	438.5	441.0	9.5	6.72	300.00
1193.000	10,427	9.85	6.54	4.17	3.03	2.26	1.69	1.21	200.	218.2	236.3	6.3	6.34	192.12
1401.000	10,832	8.72	5.18	3.14	2.35	1.79	1.41	1.15	200.	186.0	646.0	7.8	7.93	300.00
1598.000	10,633	9.44	5.85	3.67	2.66	2.00	1.59	1.22	200.	183.2	425.9	6.8	7.74	277.05
2035.000	11,043	11.15	6.40	3.62	2.62	1.94	1.46	1.17	200.	135.6	351.8	8.1	9.17	300.00
2200.000	10,570	11.41	6.30	3.46	2.46	1.86	1.43	1.16	200.	117.2	331.9	8.3	10.55	300.00
2364.000	10,761	12.06	6.26	3.17	2.17	1.72	1.35	1.11	200.	100.0	294.9	9.7	13.04	300.00
2603.000	10,264	11.18	6.89	3.64	2.50	1.87	1.45	1.13	200.	131.6	218.7	8.1	11.50	300.00
2801.000	10,121	10.98	5.88	3.36	2.30	1.68	1.31	1.02	200.	114.6	312.8	8.7	9.19	300.00
2999.000	10,876	11.48	5.46	1.97	1.18	0.86	0.55	0.43	200.	100.0	152.4	19.6	20.85	24.00
3140.000	10,689	11.14	6.17	2.61	1.36	1.03	0.92	0.76	200.	139.4	63.0	16.4	22.80	36.00
3357.000	9,819	2.84	1.70	1.21	0.87	0.64	0.50	0.42	200.	1411.4	645.6	21.8	6.63	24.00
Mean:		8.12	4.81	2.80	1.98	1.49	1.15	0.91	200.	1175.2	318.3	11.1	8.97	66.02
Std. Dev:		3.67	1.86	0.90	0.66	0.49	0.37	0.28	0.	1771.4	192.6	4.7	5.96	68.71
Var Coeff(%):		45.25	38.60	32.17	33.10	32.54	32.27	30.74	0.	100.0	60.5	42.1	66.39	104.07

Table 10. FWD Data Analysis- SH 154

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19
County: 230
Highway/Road: SH0154

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.50	199,980	200,020	H1: PR = 0.35
Base:	13.00	15,000	1,500,000	H2: PR = 0.30
Subbase:	0.00	0	0	H3: PR = 0.25
Subgrade:	146.90	15,000		H4: PR = 0.40

Station ft	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Depth to Bedrock
100.000	10,546	36.41	19.19	7.12	3.77	2.89	2.25	1.85	200.	34.4	0.0	13.0	12.70	64.25
199.000	9,577	40.63	21.04	8.17	4.14	2.83	2.15	1.71	200.	27.2	0.0	11.1	12.39	74.51
300.000	10,014	35.29	17.17	4.63	1.74	1.43	1.28	1.14	200.	27.4	0.0	19.9	23.26	58.96
400.000	11,178	37.21	20.60	7.06	3.34	2.43	1.88	1.43	200.	34.2	0.0	14.6	14.62	56.82
498.000	12,342	7.37	6.05	4.76	3.65	2.71	1.70	1.39	200.	813.7	0.0	20.6	4.68	124.20
573.000	12,226	5.76	4.81	3.71	2.84	2.04	1.42	1.11	200.	1069.2	0.0	25.7	3.08	180.92
699.000	12,485	5.52	4.85	3.87	3.03	2.32	1.78	1.39	200.	1500.0	0.0	21.8	2.87	300.00
800.000	12,898	4.49	4.05	3.51	3.02	2.11	1.61	1.31	200.	1500.0	0.0	27.3	8.51	265.08
905.000	12,226	6.01	5.54	4.69	3.84	2.98	2.29	1.74	200.	1500.0	0.0	16.7	3.48	255.13
1001.000	11,408	7.31	5.67	4.43	3.40	2.59	2.04	1.69	200.	837.1	0.0	19.0	1.96	300.00
1100.000	12,314	6.54	3.85	2.68	1.63	1.11	0.81	0.69	200.	459.0	0.0	43.7	4.38	36.00
1200.000	12,016	6.04	5.15	4.24	3.43	2.71	2.15	1.75	200.	1500.0	0.0	17.8	1.98	300.00
1310.000	12,258	6.34	5.71	4.54	3.57	2.81	2.22	1.78	200.	1380.0	0.0	17.4	3.25	300.00
1449.000	11,718	5.91	4.76	4.04	3.29	2.65	2.11	1.74	200.	1500.0	0.0	18.2	1.93	300.00
1500.000	11,730	5.35	4.79	4.12	3.25	2.41	1.94	1.41	200.	1500.0	0.0	19.2	4.16	300.00
1600.000	11,170	14.76	10.61	5.91	2.07	1.16	0.59	0.42	200.	107.4	0.0	25.0	28.74	36.00
1711.000	12,318	7.09	5.87	4.79	3.82	2.98	2.33	1.91	200.	1198.7	0.0	16.9	1.41	300.00
1800.000	12,326	10.03	7.15	4.83	3.33	2.38	1.79	1.43	200.	382.7	0.0	22.5	3.37	300.00
1901.000	11,277	9.52	6.34	4.51	3.43	2.54	1.82	1.40	200.	420.7	0.0	20.8	3.78	217.08
1999.000	11,944	10.11	7.19	5.22	3.82	2.72	2.03	1.52	200.	428.9	0.0	19.5	1.47	241.45
2100.000	11,885	12.48	8.66	5.63	4.00	2.90	2.17	1.70	200.	280.6	0.0	18.2	3.61	300.00
2141.000	11,702	13.63	9.30	6.38	4.07	2.68	1.90	1.43	200.	218.9	0.0	17.6	3.77	182.94
2199.000	12,131	17.04	10.19	5.75	3.43	2.20	1.58	1.21	200.	132.3	0.0	19.9	4.64	155.30
Mean:		13.51	8.63	4.98	3.30	2.42	1.82	1.44	200.	732.7	0.0	20.3	6.70	160.49
Std. Dev:		11.70	5.45	1.29	0.68	0.54	0.45	0.36	0.	598.4	0.0	6.3	7.15	152.23
Var Coeff(%):		86.54	63.15	25.85	20.63	22.33	24.78	25.06	0.	81.7	0.0	31.2	106.81	94.85

Table 11. FWD Data Analysis - FM 1326

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19
County: 19
Highway/Road: FM1326

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	0.50	199,960	200,020	H1: PR = 0.35
Base:	5.50	20,000	800,000	H2: PR = 0.30
Subbase:	8.00	4,000	180,000	H3: PR = 0.35
Subgrade:	114.20	15,000		H4: PR = 0.40

Station ft	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Depth to Bedrock
0.000	11,305	45.28	17.65	8.11	4.72	3.20	2.51	2.17	200.	39.4	18.4	12.2	6.59	162.36
50.000	10,681	46.12	21.64	8.02	4.30	3.26	2.63	2.29	200.	56.0	9.9	11.5	9.58	64.45
100.000	11,301	24.11	14.60	7.50	4.16	2.77	2.11	1.70	200.	213.4	26.5	13.8	3.43	119.15
150.000	11,801	16.23	12.52	7.57	4.96	3.30	2.35	1.88	200.	688.4	64.5	12.2	3.61	199.43
200.000	12,072	15.24	11.43	7.57	4.96	3.17	2.06	1.50	200.	800.0	73.7	12.8	0.99	144.65
225.000	11,241	15.60	9.91	6.27	4.06	2.72	1.93	1.47	200.	255.9	104.6	15.6	1.29	207.22
300.000	11,396	16.70	10.77	5.54	3.06	2.00	1.54	1.31	200.	393.0	35.9	18.7	3.56	113.66
321.000	11,619	17.68	12.68	6.80	4.28	2.83	2.32	1.70	200.	478.1	43.5	14.2	5.92	190.22
350.000	11,940	19.46	12.51	7.59	4.77	3.17	2.28	1.80	200.	273.1	65.5	13.7	1.47	199.45
400.000	11,130	26.56	16.12	7.43	3.83	2.28	1.61	1.28	200.	246.3	12.3	14.8	2.45	89.06
417.000	11,702	18.98	13.81	8.11	4.65	2.76	1.83	1.38	200.	722.5	21.9	13.2	1.74	115.80
450.000	10,196	51.40	24.39	8.11	3.61	2.42	1.91	1.63	200.	58.7	5.1	12.6	6.44	56.42
Mean:		26.11	14.84	7.39	4.28	2.82	2.09	1.68	200.	352.1	40.2	13.8	3.92	128.27
Std. Dev:		13.46	4.43	0.80	0.58	0.42	0.34	0.32	0.	267.3	30.8	1.9	2.67	60.43
Var Coeff(%):		51.54	29.87	10.88	13.55	14.93	16.35	19.15	0.	75.9	76.8	14.1	68.05	47.11

Table 12. FWD Data Analysis - FM 1520

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19			MODULI RANGE(psi)		
County: 32		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road: FM1520	Pavement:	0.50	199,980	200,020	H1: PR = 0.35
	Base:	10.00	20,000	400,000	H2: PR = 0.30
	Subbase:	8.00	4,000	150,000	H3: PR = 0.25
	Subgrade:	158.10	15,000		H4: PR = 0.40

Station ft	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Depth to Bedrock
212.000	12,449	10.52	6.48	4.10	2.86	2.01	1.43	1.17	200.	329.5	65.2	24.6	2.96	207.98
800.000	11,809	15.72	8.77	5.15	3.22	2.28	1.76	1.27	200.	174.4	30.0	21.2	4.54	299.39
1399.000	12,286	11.66	7.32	4.31	3.08	2.35	1.79	1.54	200.	306.7	50.5	22.3	7.14	300.00
2000.000	9,176	53.70	25.19	7.24	3.94	3.24	2.61	2.44	200.	22.0	4.7	13.3	19.57	56.30
2599.000	11,138	33.46	20.28	9.77	4.58	4.07	3.76	2.87	200.	69.7	7.5	12.9	15.20	73.46
3200.000	11,654	14.88	11.58	7.15	4.50	2.93	1.87	1.34	200.	397.9	4.7	22.8	2.57	137.14
3800.000	11,809	32.25	18.59	8.83	6.24	3.82	2.87	2.26	200.	73.9	11.2	12.9	5.19	300.00
4183.000	12,671	16.15	6.00	2.07	1.95	1.60	1.32	1.02	200.	125.8	45.9	35.6	28.78	54.35
4400.000	11,849	19.46	10.00	5.56	3.46	2.10	1.40	1.27	200.	109.4	28.4	20.9	2.15	120.30
5002.000	11,567	14.83	8.13	5.06	3.27	2.39	1.84	1.39	200.	178.7	44.5	19.5	6.15	300.00
5601.000	11,809	12.59	6.36	3.17	2.22	1.82	1.79	1.43	200.	199.9	48.0	28.9	14.80	300.00
6200.000	11,436	22.03	14.36	8.44	5.09	2.98	2.16	1.70	200.	171.7	6.2	17.2	1.35	110.80
6583.000	12,183	15.55	9.44	6.57	4.57	3.10	2.36	1.62	200.	190.4	71.9	14.9	2.32	224.41
6820.000	12,342	8.55	5.86	4.06	2.88	2.09	1.54	1.22	200.	400.0	27.4	27.4	13.14	264.63
7400.000	11,754	18.22	10.17	6.94	3.94	2.35	1.55	1.18	200.	192.2	11.3	21.4	5.16	116.14
8002.000	12,493	14.04	5.59	4.02	2.99	2.17	1.66	1.23	200.	176.3	85.0	24.8	16.23	222.56
8600.000	11,158	13.07	8.70	5.26	3.77	2.44	1.76	1.40	200.	268.5	27.7	18.6	2.51	159.25
Mean:		19.22	10.75	5.75	3.68	2.57	1.97	1.55	200.	199.2	33.5	21.1	8.81	176.64
Std. Dev:		11.18	5.67	2.11	1.08	0.68	0.63	0.51	0.	109.8	25.1	6.1	7.80	122.20
Var Coeff(%):		58.17	52.73	36.68	29.35	26.61	31.84	32.67	0.	55.1	74.9	28.9	88.58	69.18

Table 13. FWD Data Analysis - FM 560

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19
County: 19
Highway/Road: FM0560

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	2.00	199,980	200,020	H1: PR = 0.35
Base:	6.50	20,000	1,000,000	H2: PR = 0.30
Subbase:	6.00	10,000	700,000	H3: PR = 0.35
Subgrade:	274.20	15,000		H4: PR = 0.25

Station ft	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	ERR/Sens	Absolute Depth to Bedrock
0.000	10,272	28.76	16.78	9.18	6.07	4.48	3.44	2.81	200.	77.7	15.3	11.6	3.10	300.00
103.000	9,617	29.14	16.09	9.16	6.17	4.41	3.32	2.71	200.	59.5	19.5	10.9	1.32	300.00
300.000	10,236	5.27	4.45	3.93	3.32	2.72	2.15	1.79	200.	1000.0	700.0	23.1	12.81	300.00
450.000	10,193	29.07	16.42	9.19	5.78	4.00	2.94	2.28	200.	81.1	10.7	12.6	0.59	300.00
606.000	9,692	27.70	17.92	10.45	6.68	4.52	3.32	2.65	200.	107.0	10.0	10.3	1.59	255.78
758.000	9,748	19.46	13.79	8.93	5.85	3.89	2.74	2.21	200.	300.6	10.0	12.5	1.15	209.84
904.000	9,716	19.57	12.31	7.72	5.04	3.46	2.51	2.07	200.	168.0	19.8	13.8	0.84	268.35
1045.000	9,787	15.70	9.72	5.65	3.76	2.77	2.15	1.77	200.	175.0	34.3	17.8	2.89	300.00
1200.000	9,728	14.66	7.86	4.31	3.00	2.29	1.81	1.56	200.	113.9	60.8	22.4	4.22	300.00

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	2.00	199,980	200,020	H1: PR = 0.35
Base:	9.50	20,000	400,000	H2: PR = 0.30
Subbase:	3.50	5,000	400,000	H3: PR = 0.35
Subgrade:	273.70	19,600		H4: PR = 0.25

Station ft	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	ERR/Sens	Absolute Depth to Bedrock
1350.000	9,783	12.96	7.93	4.94	3.29	2.43	1.89	1.61	200.	179.8	22.5	20.8	2.63	300.00
1444.000	9,609	21.45	12.22	6.35	4.02	2.89	2.23	1.85	200.	78.6	5.9	17.5	4.23	300.00
1500.000	9,628	19.49	11.62	6.93	4.58	3.17	2.35	1.86	200.	103.7	8.4	15.6	1.23	285.34
1666.000	9,756	21.79	11.80	6.69	4.38	3.11	2.38	1.87	200.	73.6	11.3	16.0	1.82	300.00
1807.000	9,744	15.85	5.84	1.87	0.65	0.49	0.53	0.54	200.	45.6	45.6	45.6	44.51	24.00
1963.000	9,597	18.21	9.07	4.63	2.84	2.09	1.65	1.41	200.	77.0	9.8	23.6	3.22	247.34
2099.000	9,787	19.07	9.91	5.74	3.54	2.36	1.75	1.41	200.	87.6	8.1	20.8	0.96	202.89
2249.000	9,446	30.33	14.59	6.81	3.86	2.53	1.89	1.52	200.	36.8	5.0	17.2	4.66	135.76

500,000 and 1,000,000 psi give variable field performance and values above 1,000,000 psi seem to be too stiff and exhibit transverse/shrinkage cracking. In Figures 7 through 12, the base moduli values are plotted for each test pavement.

Another parameter which should be noted is the ratio of the base to the subgrade (E2/E4). It is desirable (in stabilized bases) for this ratio to be greater than 3. Between 2-3 is marginal and below 2 is considered poor.

For subgrades, moduli values less than 4000 psi are considered poor while good values are those greater than 16,000 psi.

Ground penetrating radar (GPR) data were obtained for all six test pavements in February of 1998 by Department of Transportation (DOT) Design Division personnel.

Below is a discussion of the FWD and GPR test results and the field core data.

Loop 390

No cores were obtained from this pavement. Unsuccessful attempts were made in 1997 and again in 1998.

FWD data shown in Table 8 and Figure 7 indicate that the base layer is weak in some areas which also coincided with areas where alligator cracking was observed in 1997. As shown in Figure 7, there is some variation in the moduli values between 1997 and 1998; however, the difference does not seem to warrant concern that the base is exhibiting a deteriorating strength.

IH 20 Frontage Road

Three cores were obtained from this pavement as shown in Figure 6. Last year, this pavement exhibited the highest compressive strength but there was a loss in strength as noted with the cores taken in 1998. However, there doesn't seem to be an appreciable difference in the base moduli values from 1997 to 1998 (Figure 8). Note in Figure 8, that the last data point may coincide with the beginning of a different type of pavement section.

SH 154

With indications of what appears to be shrinkage cracking, one would expect this pavement to be the stiffest of the six. This is true in terms of FWD data (Figure 9). Base moduli values along the pavement exceed 1,000,000 psi in some locations. Base moduli values in 1998 appear to be similar to that in 1997 with some places showing significantly higher moduli than the previous year. Compressive strength of the cores is also close to the values obtained the previous year (Figure 6).

FM 1326

Two cores were obtained from FM 1326 which could be tested and the compressive strength was significantly higher than the single core which was tested in 1997. FWD data (Table 12 and Figure 10) indicate that the base is not deteriorating but exhibits an overall similar or better modulus than the previous year.

FM 1520

Three cores were obtained from FM 1520 and two of the three cores showed a significantly greater compressive strength than the previous year. FWD data (Figure 11) on this pavement indicates that there is no significant change between 1997 and 1998.

FM 560

All three cores obtained from FM 560 had a higher compressive strength than the cores obtained the previous year. The base on this pavement has two different thicknesses along its length: 9 inches and 16 inches. Because of the difference in thicknesses, two separate FWD analyses were performed as shown in Table 14. Results from both analyses, however, were combined for Figure 12. Moduli values for this pavement do not appear to be as variable as on some of the others; however, the values are lower than the desired minimum of 145,000 psi. Also, however, there seems to be little change in moduli values between 1997 and 1998.

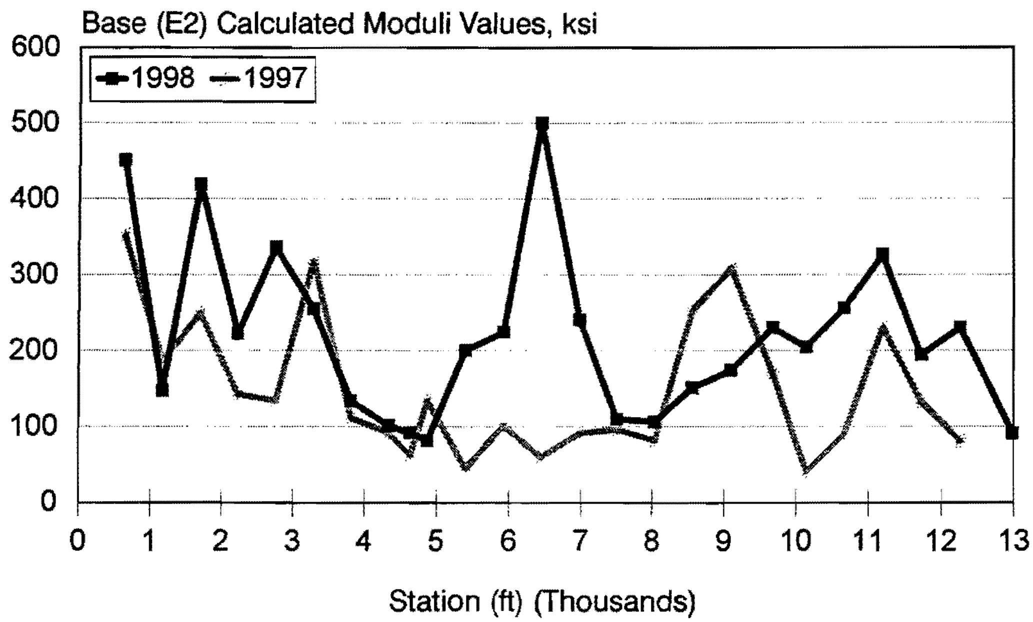

Figure 7. Base Moduli Values for Loop 390

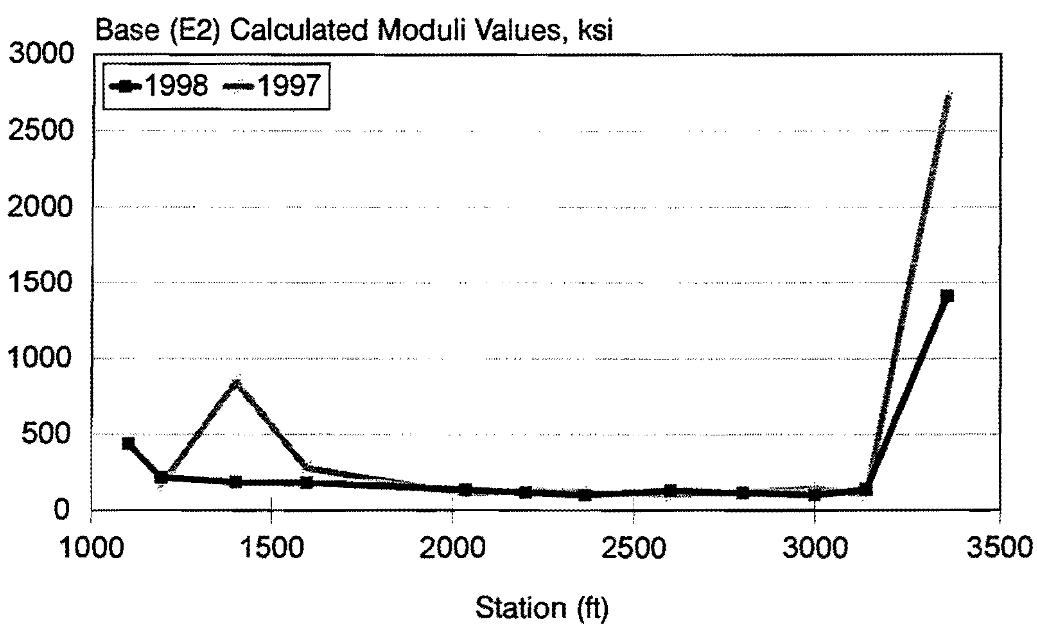

Figure 8. Base Moduli Values for IH 20 Frontage Road

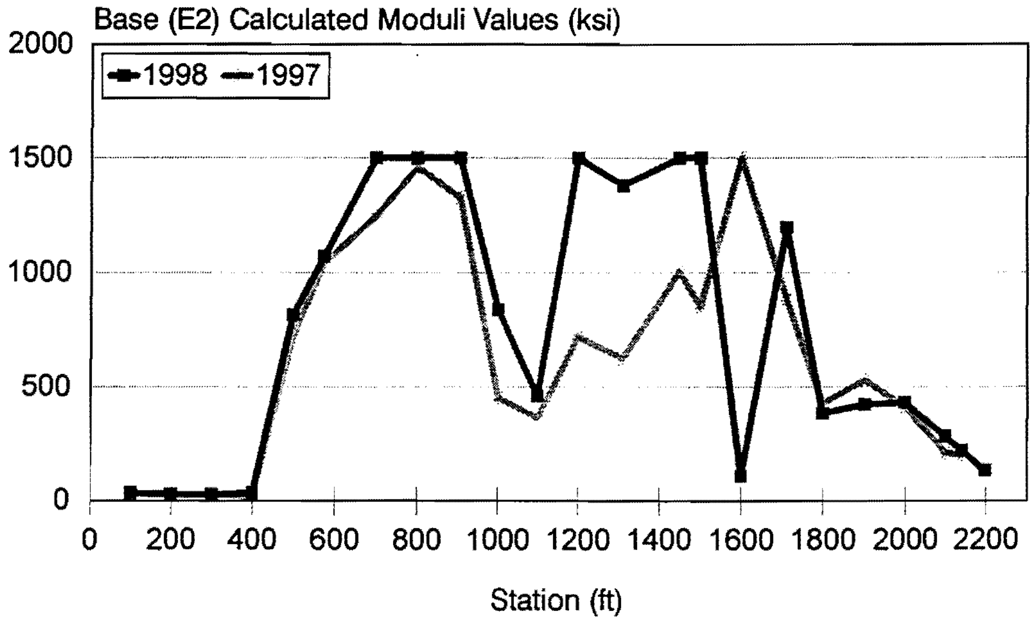

Figure 9. Base Moduli Values for SH 154

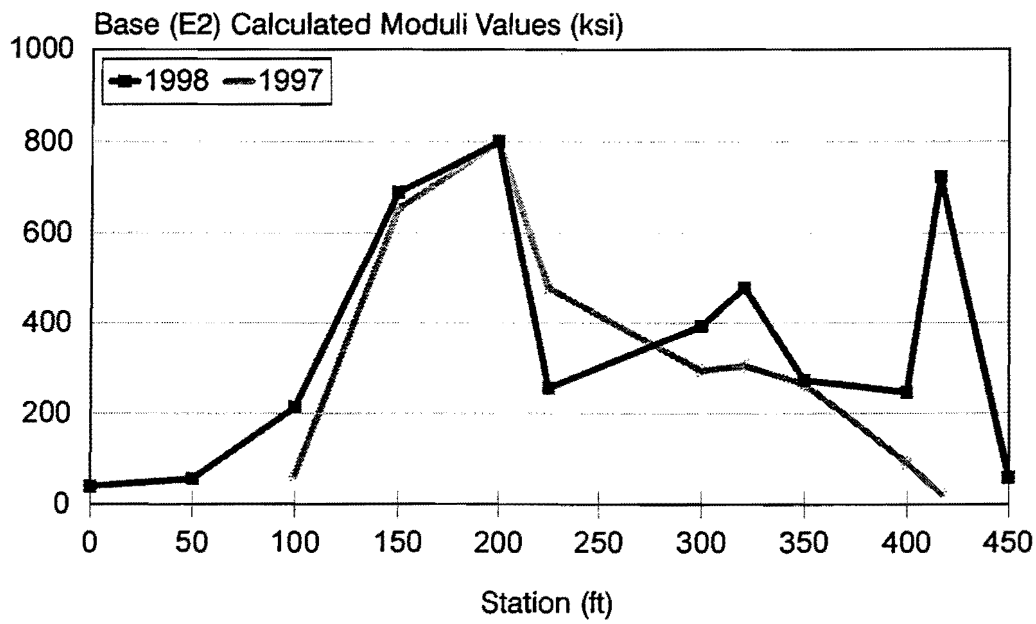

Figure 10. Base Moduli Values for FM 1326

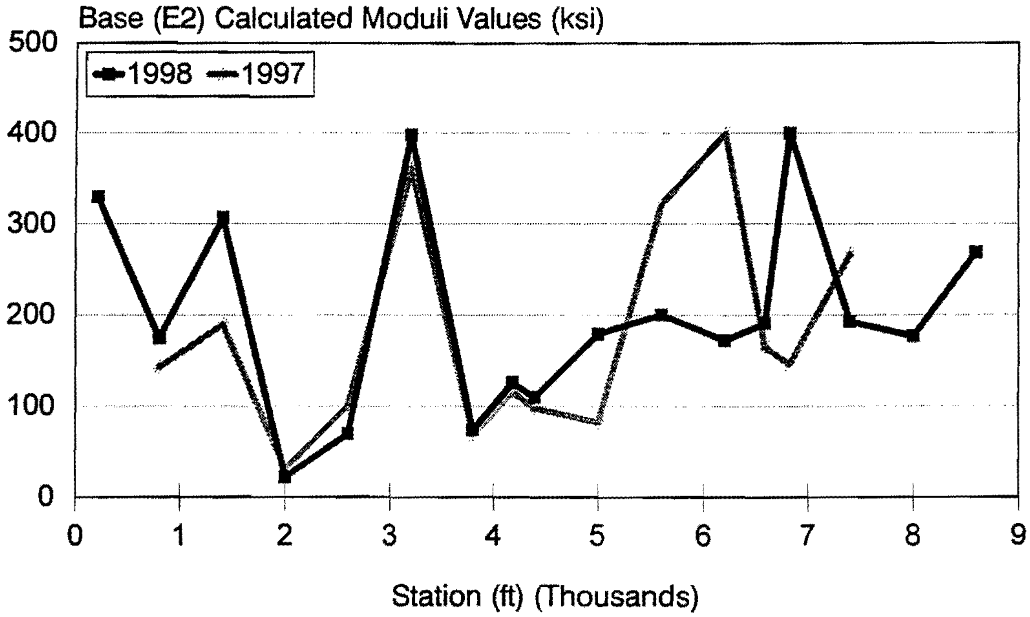

Figure 11. Base Moduli Values for FM 1520

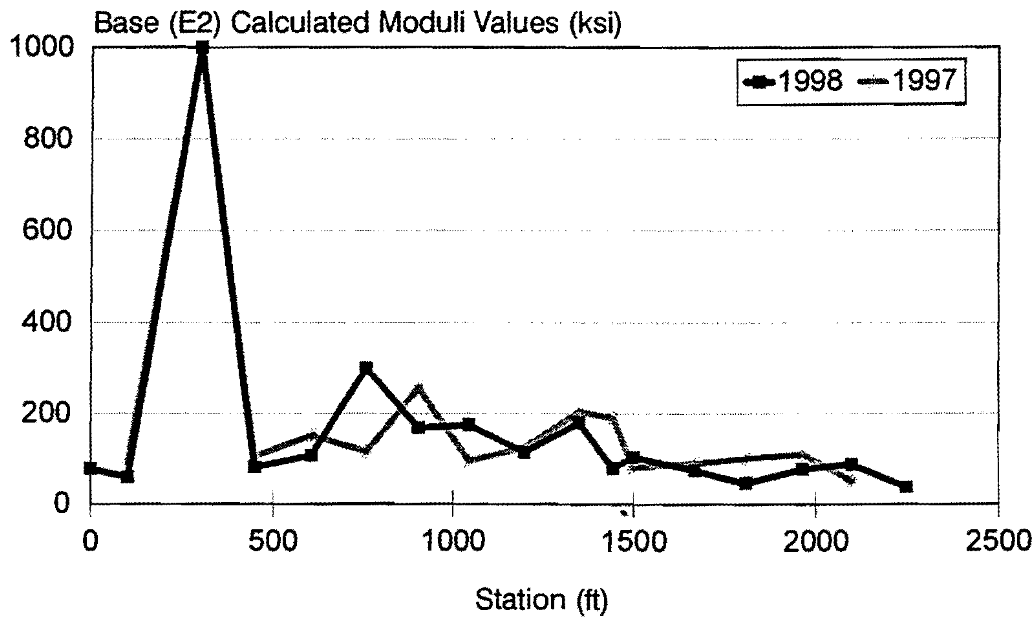

Figure 12. Base Moduli Values for FM 560.

Ground Penetrating Radar Data

Ground penetrating radar data surveys were collected by TxDOT's Design Division personnel on February 9, 1998. Some typical dielectric constants for the fly-ash base are shown below in Table 14.

Table 14. Typical Dielectric Constants for Hydrated Fly-Ash Bases

Pavement Section	Station Location	Dielectric Constant for Hydrated Fly-Ash Base
Loop 390	1909 ft 2266 ft	11.3 16.0
IH-20 Frontage Road	2086 ft 2423 ft	16.5 12.8
SH 154	92 ft 991 ft	17.6 18.3
FM 1326	239 ft 253 ft	20.6 20.2
FM 1520	607 ft	23.3
FM 560	1158 ft 2034 ft	19.5 15.0

CONCLUSIONS AND RECOMMENDATIONS

Laboratory Study

A laboratory study was conducted to evaluate the bond strength of surface treatments to hydrated fly-ash base materials. Variables in the experiment included (1) type of prime material used and (2) curing conditions for the base material. Tests used to evaluate the bond strength included a torsional shear test, a South African durability test, and visual/subjective evaluations. The torsional shear test did not show any differences between the different prime materials used or the different curing conditions. A visual evaluation was done also on samples for each prime material and curing condition and there appeared to be a very good bond of the surface treatment to the base in all cases.

Based on the above laboratory data, researchers attempted to include the effects of traffic on evaluating the bond strength. For this evaluation, the South African durability test was used. This is a test that is typically used to evaluate the durability of stabilized base materials. For the purposes of this study, the base materials were compacted at optimum moisture into beam-shaped molds, cured and topped with different types of prime materials and finally a surface treatment. The samples were then placed in a water bath and trafficked under a loaded wheel for 5000 repetitions. All of the samples (produced with different prime materials) performed very well and the bond strength of the surface treatment to the base material seemed to be very good. Curing condition was not a variable in this experiment. Curing of the samples for seven days prior to testing is a necessity for this test because the samples must be trimmed with a saw prior to testing.

Based on the laboratory study, no confident solution can be provided to the problem experienced in the field regarding the surface treatment not bonding to the base material. Originally, one problem was thought to be the use of MC-30 as a prime material; however, the laboratory study showed that the MC-30 is an effective prime material in addition to the other prime materials that were used in the lab study. Even though *curing time* of the base was a variable in the experiment, it may be that even the lowest level of curing in the laboratory was more than what was experienced in the field prior to construction of the surface treatment and application of traffic. Researchers believe that the curing time of the base prior to application of the surface treatment may be the key to achieving a good bond.

The hydrated fly-ash base material has an optimum moisture content which can be as high as 35%. Compared to other types of base materials, this is an extremely high moisture content. If the surface of the base material is sealed soon after construction, moisture may accumulate in the upper portion of the base, weakening the base material near the interface. As in concrete, where excess water creates a high water cement ratio (and lower strength), excess moisture in this type of stabilized base might also cause a strength reduction.

Hydrated fly-ash base develops strength with time. If enough strength has not developed in the surface at the time traffic has been placed, excess fines may be generated in the base surface (by the action of traffic) causing a debonding of the surface treatment.

The laboratory study showed that it is possible to develop a good bond of the surface treatment to the hydrated fly-ash base using various types of prime materials, including MC-30. Inadequate bond of surface treatments to hydrated fly-ash base materials is probably not attributable to the type of prime material used.

Field Evaluation

- Most of the hydrated fly-ash test pavement are performing very well at this time. Those pavements which have distress are in isolated areas and the distress is not affecting the serviceability of the roadway.
- Very little change was seen in the performance of the six pavements between the 1997 and 1998 evaluations. Two of the six hydrated fly-ash test pavements have exhibited distress which might be attributable to deficiencies in the fly-ash base material. In 1997 Loop 390 exhibited a small amount of alligator cracking in an area where the FWD data indicated the base is weak. However, by 1998, the surface had a new seal coat and there was apparent surface distress at the time of evaluation in 1998. SH 154 is exhibiting transverse cracking (which appears to be from shrinkage of the base) and the FWD data indicates this pavement is excessively stiff. Researchers observed that the cracking had not progressed further in 1998 and, in fact, there was slightly less

cracking in 1998 than in 1997. This indicates there may be a tendency of the cracks toward autogenous healing in this type of base material.

- 1998 FWD data were compared to that taken in 1997. Modulus of the fly-ash base materials were back-calculated from the FWD data. There is no indication of any weakening of these base materials with time. Modulus values, however, are dependent on moisture conditions of the base and the 1998 FWD data were taken on the heels of a dry spring (compared with the 1997 data).
- Cores were taken on all of the test pavements except Loop 390. No intact core could be obtained from Loop 390. For the other five pavements, unconfined compressive strengths were about the same or higher than the compressive strengths of the previous year.
- Ground penetrating radar (GPR) surveys of all six test pavements indicate a very high dielectric constant for the fly-ash base materials. Values of this magnitude typically indicate the presence of excessive amounts of moisture and would generally warrant a great deal of concern by pavement engineers. However, one must remember that the optimum moisture content for these pavements was 35% compared with moisture contents of, say, 7% for more typical base materials. Therefore, these high dielectric constants may not necessarily be cause for alarm.
- Hydrated fly ash is a new material and is different from other stabilized base materials. Given this fact, it may not be appropriate to apply field testing criteria associated with conventional materials. For this material and its respective traffic conditions, values shown in this report may be acceptable (since the pavements are performing very well). This will become more evident as performance is monitored over the next three years.

Recommendations

Based on a second year of monitoring for these fly-ash test pavements, performance results are very promising. Concern, however, is warranted regarding the fly ash material variability as exhibited in moduli values from FWD data. GPR data showed alarmingly high dielectric constants for the bases indicating excessive moisture in the base. This may not be cause for concern, though, since original optimum moisture content was as high as 35%.

It appears that typical *rule of thumb* criteria which we typically apply to conventional pavements may not be applicable to fly ash bases. Since appropriate criteria is not established for this type of material, it is recommended that the Atlanta District continue the current course of action: monitoring the performance of these pavements as scheduled through this research project. If any new construction with fly-ash base is initiated soon, it is recommended that the construction be limited to pavements that do not have heavy truck traffic (until more is understood about these base materials).

Inadequate bond of surface treatments to fly ash base materials does not appear to be related to the type of prime material used. Researchers believe that the bonding problem is related to the curing extent of the base material. The fly-ash base develops strength with time and care should be taken to insure that adequate curing occurs prior to application of the surface treatment (especially on higher-trafficked roadways). Also, once the base has been compacted at optimum moisture content, any additional water sprayed on the surface could weaken the base near the surface. If it is necessary to spray additional water on the surface for finishing, care should be taken not to trap any water (by an asphalt membrane) in excess of that needed for hydration.

At the onset of the study, researchers consulted with other hydrated fly-ash suppliers. In a letter from Don King (President of DePauw Fly Ash suppliers in Amarillo) to TTI dated April 1, 1996, Mr. King states that *Special Specification No. 2011 - Fly Ash Base is in need of further development, especially in the area of curing conditions and bonding mechanism with surface courses.* DePauw recommends that *Article (6) Finishing* on page 3-4 be amended by deleting items *(1), (2)* and *(3)* as shown in Figure 13. DePauw also suggests that *Article (7) Curing* on page 3-4, be deleted and replaced with the following:

> *Prior to placing the surfacing on the completed base, the base shall be cured to the extent as directed by the Engineer.*

Researchers concur with this recommendation.

1993 SPECIFICATIONS

CSJ 0801-02-012

SPECIAL SPECIFICATION

ITEM 2011

FLY ASH BASE

1. DESCRIPTION. THIS ITEM SHALL CONSIST OF A BASE COURSE COMPOSED OF THE ITEMS DESCRIBED UNDER ARTICLE 2. MATERIALS. THIS ITEM SHALL ALSO INCLUDE THE PLACEMENT, COMPACTION, FINISHING AND SHAPING OF THE BASE COURSE IN ACCORDANCE WITH THE REQUIREMENTS OF THIS SPECIFICATION AND THE PLANS AND TO THE LINES AND GRADES AS ESTABLISHED BY THE ENGINEER.

2. MATERIALS.

(1) CRUSHED, CURED FLY ASH, A FLY ASH WHICH HAS SET, CURED, BEEN MINED, CRUSHED AND SIZED. THE CRUSHED, CURED FLY ASH SHALL BE FREE OF INJURIOUS OR HAZARDOUS PRODUCTS AND FREE OF ORGANIC MATERIAL OR OTHER FOREIGN MATTER. THE CONTRACTOR IS RESPONSIBLE FOR FURNISHING THE ENGINEER WITH THE FOLLOWING:

1. CERTIFICATION THAT THE CRUSHED, CURED FLY ASH COMPLIES WITH EITHER CLASS 2 OR 3 INDUSTRIAL WASTE REQUIREMENTS SET FORTH IN 30 TAC 335.506 & 30 TAC 335.507. THE CERTIFICATION REQUIRED BY THIS SUBPARAGRAPH SHALL BE BASED ON LABORATORY TESTING OF THE CRUSHED, CURED FLY ASH. THE SAMPLING FREQUENCY OF THE CRUSHED, CURED FLY ASH SHALL COMPLY WITH THE QC REQUIREMENTS SET FORTH IN EPA SW846, CHAPTER 9.

2. DOCUMENTATION THAT THE GENERATOR OF THE FLY ASH BY-PRODUCT HAS COMPLIED WITH THE NOTIFICATION REQUIREMENTS FOR RECYCLING ACTIVITIES AS REQUIRED BY 30 TAC 335.24(H) AND 30 TAC 335.6.

THE SOURCE OF THE CRUSHED, CURED FLY ASH SHALL BE APPROVED BY THE ENGINEER PRIOR TO ITS USE.

(2) WATER MEETING THE MATERIAL REQUIREMENTS OF ITEM 204, "SPRINKLING".

(3) ASPHALT MEETING THE REQUIREMENTS OF ITEM 300, "ASPHALTS, OILS AND EMULSIONS.

3. STRENGTH REQUIREMENT. WHEN TESTED IN ACCORDANCE WITH TEST METHOD TEX-117-E, THE TRIAXIAL CLASS SHALL NOT BE LESS THAN CLASS 1.0.

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5. CONSTRUCTION METHODS.

(1) GENERAL. IT IS THE PRIMARY REQUIREMENT OF THIS SPECIFICATION TO SECURE A COMPLETED BASE COURSE OF FLY ASH BASE UNIFORMLY COMPACTED TO THE SPECIFIED DENSITY WITH NO LOOSE OR POORLY COMPACTED AREAS, WITH UNIFORM MOISTURE CONTENT, WELL BOUND THROUGHOUT ITS FULL DEPTH AND WITH A SURFACE FINISH SUITABLE FOR PLACING A SURFACE COURSE. IT SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR TO REGULATE THE SEQUENCE OF WORK, MAINTAIN THE WORK, AND REWORK THE COURSES AS NECESSARY TO MEET THE REQUIREMENTS OF THIS SPECIFICATION.

(2) PREPARATION OF SUBGRADE. THE ROADBED SHALL BE EXCAVATED AND SHAPED IN CONFORMITY WITH THE TYPICAL SECTIONS SHOWN ON THE PLANS TO THE LINES AND GRADES ESTABLISHED BY THE ENGINEER. ALL SUITABLE OR OTHERWISE OBJECTIONABLE MATERIAL OR ROOTS SHALL BE REMOVED FROM THE SUBGRADE AND REPLACED WITH APPROVED MATERIAL. ALL HOLES, RUTS AND DEPRESSIONS SHALL BE FILLED WITH APPROVED MATERIAL AND, IF REQUIRED, THE SUBGRADE SHALL BE THOROUGHLY WETTED WITH WATER AND RESHAPED AND ROLLED TO THE EXTENT DIRECTED IN ORDER TO PLACE THE SUBGRADE IN AN ACCEPTABLE CONDITION TO RECEIVE THE BASE MATERIAL. THE SURFACE OF THE SUBGRADE SHALL BE FINISHED TO LINES AND GRADES AS ESTABLISHED AND SHALL BE IN CONFORMITY WITH THE TYPICAL SECTIONS SHOWN ON THE PLANS. A SUBGRADE PLANER MAY BE USED. ANY DEVIATION IN EXCESS OF ONE-HALF INCH IN CROSS SECTION OR ONE-HALF INCH IN A LENGTH OF 16 FEET MEASURED LONGITUDINALLY SHALL BE CORRECTED BY LOOSENING, ADDING OR REMOVING MATERIAL, RESHAPING AND RECOMPACTING BY SPRINKLING AND ROLLING. SUFFICIENT SUBGRADE SHALL BE PREPARED IN ADVANCE TO INSURE SATISFACTORY PROSECUTION OF THE WORK. MATERIAL EXCAVATED IN PREPARATION OF THE SUBGRADE SHALL BE UTILIZED IN THE CONSTRUCTION OF ADJACENT SHOULDERS AND SLOPES OR OTHERWISE DISPOSED OF AS DIRECTED BY THE ENGINEER. WORK REQUIRED FOR PREPARATION OF SUBGRADE WILL BE MEASURED AND PAID FOR UNDER ITEM 110, "EXCAVATION" AND ITEM 132, "EMBANKMENT" OR IN ACCORDANCE WITH THE PROVISIONS OF OTHER APPLICABLE BID ITEMS.

(3) PLACING. THE FLY ASH BASE SHALL BE PLACED IN UNIFORM LAYERS ON THE PREPARED SUBGRADE TO PRODUCE THE DEPTH SPECIFIED ON THE PLANS. THE MATERIAL SHALL BE CONSOLIDATED WITH ROLLERS CAPABLE OF COMPACTING FROM THE BOTTOM UP. THE DEPTH OF LAYERS SHALL BE AS APPROVED BY THE ENGINEER. TO INSURE HOMOGENEOUS DISTRIBUTION OF THE FLY ASH BASE MATERIAL IN EACH LAYER, THE MATERIAL SHALL BE PLACED USING AN APPROVED SPREADER. THE SPREADING OPERATIONS SHALL BE DONE IN SUCH A MANNER AS TO ELIMINATE NESTS OR POCKETS OF MATERIAL OF NONUNIFORM GRADATION RESULTING FROM SEGREGATION IN THE HAULING OR DUMPING OPERATIONS AND IN SUCH A MANNER AS TO ELIMINATE PLANES OF WEAKNESS.

THE FLY ASH BASE SHALL NOT BE PLACED WHEN THE AIR TEMPERATURE IS BELOW 40 F AND IS FALLING, BUT MAY BE PLACED WHEN THE AIR TEMPERATURE IS ABOVE 35 F, AND IS RISING, THE TEMPERATURE BEING

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TAKEN IN THE SHADE AND AWAY FROM ARTIFICIAL HEAT AND WITH FURTHER PROVISION THAT FLY ASH BASE SHALL BE MIXED OR PLACED ONLY WHEN WEATHER CONDITIONS IN THE OPINION OF THE ENGINEER ARE SUITABLE FOR SUCH WORK.

(4) CONSTRUCTION JOINTS. IF A ROAD SECTION IS NOT COMPLETED AT THE END OF A CONSTRUCTION DAY, A STRAIGHT TRANSVERSE CONSTRUCTION JOINT SHALL BE FORMED BY CUTTING BACK INTO THE COMPLETED WORK TO FORM A VERTICAL FACE.

(5) COMPACTION. UNLESS OTHERWISE SHOWN ON THE PLANS, THE FLY ASH BASE SHALL BE SPRINKLED AS REQUIRED AND COMPACTED TO A DENSITY OF NOT LESS THAN 95 PERCENT OF COMPACTION RATIO DENSITY, TEST METHOD TEX-113-E AND SHALL BE CHECKED IN THE FIELD BY TEST METHOD TEX-115-E. THE MOISTURE CONTENT OF THE MIXTURE DURING COMPACTION OPERATIONS SHALL BE MAINTAINED WITHIN A RANGE FROM OPTIMUM PERCENTAGE TO TWO (2) PERCENTAGE POINTS ABOVE OR 3.5 PERCENTAGE POINTS BELOW THE OPTIMUM PERCENTAGE OR WITHIN THE RANGE DIRECTED BY THE ENGINEER. IF THE OBTAINED DENSITY DOES NOT SATISFY REQUIREMENTS, THE CONTRACTOR SHALL MAKE ADJUSTMENTS IN ROLLER WEIGHT, LIFT THICKNESS OR MATERIAL MOISTURE LEVEL OR REPLACE THE MATERIAL IN QUESTION. THE MATERIAL SHALL NOT BE COMPACTED UNTIL THE NECESSARY SHAPE AND THICKNESS HAS BEEN ACHIEVED BY GRADING. WHEN ADDITIONAL LIFTS ARE NECESSARY, THE EXISTING LAYER SHALL BE LIGHTLY SPRINKLED PRIOR TO PLACING THE ADDITIONAL COURSES.

(6) FINISHING. AFTER THE FINAL COURSE OF THE FLY ASH BASE, EXCEPT THE TOP MULCH, IS COMPACTED, THE SURFACE SHALL BE FINISHED TO GRADE AND SECTION BY BLADING AND SHALL BE SEALED WITH APPROVED PNEUMATIC TIRE ROLLERS. WHEN DIRECTED BY THE ENGINEER, SURFACE FINISHING METHODS MAY BE VARIED FROM THIS PROCEDURE PROVIDED A DENSE UNIFORM SURFACE IS PRODUCED AND FURTHER PROVIDED THAT THE CONSTRUCTION OF COMPACTION PLANES IS AVOIDED. UNLESS OTHERWISE SHOWN ON PLANS, (1) NOT MORE THAN 90 MINUTES SHALL ELAPSE BETWEEN THE START OF MIXING AND THE TIME OF STARTING THE COMPACTION OF THE FLY ASH BASE ON THE PREPARED SUBGRADE, (2) THE MIXTURE OF FLY ASH BASE AND WATER THAT HAS NOT BEEN COMPACTED SHALL NOT BE LEFT UNDISTURBED FOR MORE THAN 60 MINUTES, AND (3) ALL FINISHING OPERATIONS SHALL BE COMPLETED WITHIN A PERIOD OF FIVE (5) HOURS AFTER WATER IS ADDED TO THE FLY ASH BASE. Delete

(7) CURING. IMMEDIATELY AFTER THE FLY ASH BASE HAS BEEN BROUGHT TO LINE AND GRADE, AN ASPHALTIC MEMBRANE SHALL BE PLACED ON THE FLY ASH BASE TO PREVENT EVAPORATION OF WATER AND PROVIDE CURING. THE ASPHALT USED FOR CURING SHALL BE OF THE TYPE AND GRADE SHOWN ON THE PLANS OR AS APPROVED BY THE ENGINEER AND SHALL BE APPLIED AT THE RATE OF APPROXIMATELY 0.1 GALLON PER SQUARE YARD UNLESS THE PLANS REQUIRE OTHERWISE. Delete

IF THERE IS A TIME DELAY PRIOR TO APPLICATION OF THE ASPHALT MEMBRANE WHICH IS SUFFICIENT TO CAUSE SURFACE DRYING, THE ENGINEER MAY REQUIRE THE SURFACE TO BE MOISTENED.

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(8) TRAFFIC. THE FLY ASH BASE SHALL BE OPENED TO TRAFFIC AS SPECIFIED ON THE PLANS OR AS DIRECTED BY THE ENGINEER.

5. MAINTENANCE.

THE CONTRACTOR WILL BE REQUIRED WITHIN THE LIMITS OF HIS CONTRACT TO MAINTAIN THE FLY ASH BASE IN GOOD CONDITION UNTIL ALL WORK HAS BEEN COMPLETED AND ACCEPTED. MAINTENANCE SHALL INCLUDE IMMEDIATE REPAIR OF ANY DEFECTS THAT MAY OCCUR. THIS WORK SHALL BE DONE BY THE CONTRACTOR AT HIS ENTIRE EXPENSE AND SHALL BE REPEATED AS OFTEN AS MAY BE NECESSARY TO KEEP THE AREA CONTINUOUSLY INTACT. REPAIRS TO FLY ASH BASE SHALL BE EFFECTED BY REPLACING THE FLY ASH BASE FOR ITS FULL DEPTH RATHER THAN BY ADDING A THIN LAYER OF FLY ASH BASE TO THE LAYER OF BASE IN NEED OF REPAIR.

6. MEASUREMENT. THIS ITEM WILL BE MEASURED BY THE CUBIC YARD IN THE COMPLETED AND ACCEPTED FINAL POSITION. THE VOLUME OF BASE COURSE WILL BE COMPUTED IN PLACE BETWEEN THE ORIGINAL SUBGRADE OR SUBBASE SURFACES, AND THE LINES, GRADES AND SLOPES OF THE ACCEPTED BASE COURSE AS SHOWN ON THE PLANS BY THE METHOD OF AVERAGE END AREAS.

THIS IS A PLAN QUANTITY MEASUREMENT ITEM AND THE QUANTITY TO BE PAID FOR WILL BE THAT QUANTITY SHOWN IN THE PROPOSAL AND ON THE "ESTIMATE AND QUANTITY" SHEET OF THE CONTRACT PLANS, EXCEPT AS MAY BE MODIFIED BY ARTICLE 9.8. IF NO ADJUSTMENT IS REQUIRED, ADDITONAL MEASUREMENTS OR CALCULATIONS WILL NOT BE REQUIRED. NO PAYMENT WILL BE MADE FOR THICKNESS OR WIDTH EXCEEDING THAT SHOWN ON THE TYPICAL SECTION OR PROVIDED ON THE PLANS.

7. PAYMENT. THE WORK PERFORMED AND MATERIALS FURNISHED IN ACCORDANCE WITH THIS ITEM AND MEASURED AS PROVIDED UNDER "MEASUREMENT" WILL BE PAID FOR AT THE UNIT PRICE BID FOR "FLY ASH BASE (DENSITY CONTROL)" OF THE DEPTH SPECIFIED.

THIS PRICE SHALL BE FULL COMPENSATION FOR SECURING AND FURNISHING ALL MATERIALS; INCLUDING ALL ROYALTY, FREIGHT AND STORAGE INVOLVED; FOR ALL PROCESSING, CRUSHING AND LOADING; FOR ALL HAULING, DELIVERING, STOCKPILING, PLACING, SPREADING, BLADING, MIXING, STRIPPING, DRAGGING, FINISHING, CURING AND MAINTAINING; FOR ALL FINE GRADING; FOR WETTING AND COMPACTING AND ALL MANIPULATION, LABOR, TOOLS AND INCIDENTALS NECESSARY TO COMPLETE THE WORK.

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Figure 13. Special Specification Item 2022, Fly-Ash Base with Recommended Deletions

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2. Mantilla, C.A. and J.W. Button, *Prime Coat Methods and Materials to Replace Cutback Asphalt,* Report FHWA/TX-94-1334-1F, Texas Transportation Institute, Texas A&M University, College Station, November 1994.

3. *Principles of Brick Masonry,* Brick Institute of America, 11490 Commerce Park Drive, Reston, Virginia 22091.

4. *Distress Identification Manual for the Long-Term Pavement Performance Project,* 1993. Report SHRP-P-338, Strategic Highway Research Program, National Research Council, Washington, D.C.