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16. Abstract

Hydrated fly ash is produced by allowing a Class C powder fly ash (ASTM C 618) 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.

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 a final report documenting the performance evaluations conducted in the spring of 1997 through the spring of 2001.

Based on visual evaluations, FWD data, and compressive strengths of cores, the hydrated fly-ash test pavements have performed well.

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FIELD PERFORMANCE EVALUATION OF HYDRATED, FLY-ASH BASES IN THE ATLANTA DISTRICT - YEAR 5

by

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Report 2966-5
Project Number 7-2966
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

Sponsored by the
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The contents of this report reflect the views of the author, who is 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). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The engineer in charge of this project was Cindy Estakhri (#77583).

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This research project was sponsored by the Texas Department of Transportation.

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BACKGROUND

Hydrated fly ash is produced by allowing a Class C powder fly ash (ASTM C 618) 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. At the onset of this study district personnel were pleased with the performance of this industrial by-product as a base material; however, its long-term performance was in question. While performance of the material as a base has been acceptable, the district has encountered problems 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 was based on the following types of data:

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

Research report 2966-2 presents results of a laboratory investigation into the cause of 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 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 percent: 85.5 lb/ft^3,

- Los Angeles abrasion: 47, and
- five cycles of freeze-thaw (15 hours freeze-thaw at room temperature for nine 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. These six test pavements are the subject of this study.

Table 1 includes a description of each of the six test sites, their locations, and typical cross sections. 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, several problems 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.

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 One-Course Surface Trt. MC-30 Prime 6.0 - 12.0 in Fly Ash Base 0-6.0 in. Bank-Run RG

VISUAL CONDITION SURVEYS

In this research study, visual condition surveys are performed annually in late spring on all six test pavements. The most recent survey was performed on April 30 and May 1, 2001. Researchers conducted the manual survey in accordance with the procedures set up for a Strategic Highway Research Program (SHRP) Long Term Pavement Performance (LTPP) distress survey (2). 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 0.3 mi south of SH 43. The total length of the project is about 2.5 mi. 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 and there is no longer evidence of alligator cracking at this time. Table 2 shows quantities of distress at each survey location for every year evaluated.

The chip-seal surface exhibited flushing at some locations. Between 1999 and 2000, the flushing of the chip seal seemed to have stabilized. There had been a gradual but progressive increase in rutting from 1997 to 2000. This rutting may have occurred within the hot-mix asphalt concrete overlay and was not necessarily attributed to problems associated with the hydrated fly-ash base.

Between the year 2000 and 2001 the pavement was surfaced with a new Type C hot mix asphalt concrete; therefore, no surface distress was present in 2001 at the time of the pavement evaluation.

Table 2. Loop 390 Distress.

Location (each location represents a 200 ft length)	Alligator* Cracking (sq ft)					Flushing (sq ft)					Rutting (in)									
	1997	1998	1999	2000	2001	1997	1998	1999	2000	2001 **	Left Wheelpath					Right Wheelpath				
											1997	1998	1999	2000	2001 **	1997	1998	1999	2000	2001 **
1	0	0	0	0	0	0	590 (s)	1080 (m)	1200 (s)	0	0	0.1	0.4	0.3	0	0	0.3	0.6	0.5	0
2	0	0	0	0	0	0	97 (s)	960 (m)	1000 (s)	0	0	0.2	0.6	0.3	0	0	0.3	0.4	0.5	0
3	0	0	0	0	0	0	260 (s)	720 (s)	720 (s)	0	0.1	0.1	0.2	0.3	0	0.1	0.1	0.1	0.3	0
4	0	0	0	0	0	0	330 (s)	600 (s)	800 (s)	0	0.1	0.1	0.3	0.2	0	0.1	0.1	0.2	0.3	0
5	0	0	0	0	0	0	260 (s)	720 (s)	720 (s)	0	0.2	0.2	0.8	0.8	0	0.2	0.3	0.8	0.9	0
6	600 (s)	0	0	0	0	600 (s)	800 (s)	860 (s)	860 (s)	0	0.4	0.6	0.5	0.6	0	0.5	0.6	0.4	0.5	0
7	1000 (s)	0	0	0	0	1200 (s)	400 (s)	480 (s)	480 (s)	0	0.5	0.5	0.7	0.6	0	0.5	0.5	0.4	0.4	0
8	1000 (s)	0	0	0	0	1200 (s)	600 (s)	600 (s)	1200 (s)	0	0.4	0.4	0.6	0.8	0	0.4	0.4	0.6	0.6	0
9	600 (s)	0	0	0	0	1000 (s)	300 (s)	300 (s)	300 (s)	0	0.4	0.3	0.4	0.4	0	0.4	0.4	0.2	0.3	0
10	0	0	0	0	0	400 (s) 200 (m)	250 (s)	200 (s)	200 (s)	0	0.1	0.1	0.2	0.2	0	0.1	0.1	0.3	0.4	0
11	0	0	0	0	0	600 (s)	0	0	0	0	0.1	0.1	0.1	0.2	0	0.1	0.1	0.2	0.1	0
12	0	0	0	0	0	0	0	0	0	0	0.1	0.1	0.1	0.1	0	0.1	0.1	0.1	0.2	0
13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0.1	0.1	0

Severity Levels : (s) slight, (m) moderate.
* A Grade 4 seal coat was constructed on the pavement between the 1997 and 1998 evaluations.
** A Type C overlay was constructed between the 2000 and 2001 evaluations.

IH 20 FRONTAGE ROAD

The IH 20 frontage road project begins 0.9 miles east of the Gregg County line and continues eastward for 3000 feet. This pavement is beginning to show signs of some significant distress. There is evidence of raveling in the hot-mix asphalt surface which, of course, is unrelated to the hydrated fly-ash base that is of interest in this study. However, there is some distress which can be attributed to the base and is evident in the form of cracking. Some moderate alligator cracking was observed as shown in Table 3. This represents an increase over that which was observed in 2000. This cracking is still somewhat isolated and not widespread.

Table 3. IH 20 Frontage Road Distress.

Location (each location represents a 200 ft length)	Raveling (sq ft)					Longitudinal Cracking (ft)					Alligator Cracking (sq ft)				
	1997	1998	1999	2000	2001	1997	1998	1999	2000	2001	1997	1998	1999	2000	2001
Core Location 1	43 (s)	43 (s)	43 (s)	200 (s)	1200 (s)	0	0	0	6 (s)	16 (s)	0	5 (s)	5 (s)	8 (s)	30 (m)
Core Location 2	54 (s)	54 (s)	54 (s)	80 (s) 10 (m)	1200 (s)	0	0	0	8 (s)	14 (s)	0	3 (s)	3 (s)	10 (s)	80 (m)
Core Location 3	43 (s)	43 (s)	43 (s)	60 (s)	1200 (s)	0	0	0	0	16 (s)	0	0	0	0	100 (m)

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. This pavement received a Grade 4 lightweight chip seal prior to the evaluation conducted in March of 2000. This seal masked the cracking which had been evident previously as shown in Table 4. There were still no cracks evident on the surface in 2001. Prior to the chip seal, the primary distress of interest on this pavement was some slight transverse cracking. These cracks began in the shoulder and most had not progressed all the way across the main lanes of travel; however, the cracks were very evenly spaced (every 12 to 13 ft) and might be attributable to shrinkage of the fly-ash base. Note in Table

4 that there was no appreciable increase in the amount of cracking observed from 1997 through 1999.

Table 4. 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	1999	2000*	2001	1997	1998	1999	2000*	2001
0 - 200 ft (1st core location)	6 (s)	8 (s)	10 (s)	0	0	0	0	24 (s)	0	0
200 - 400 ft	24 (s)	24 (s)	31 (s)	0	0	0	0	0	0	0
400 - 600 ft	12 (s)	12 (s)	16(s)	0	0	0	0	12 (s)	0	0
600 - 800 ft	17 (s)	7 (s)	7 (s)	0	0	0	0	0	0	0
800 - 1000 ft (2nd core location)	8 (s)	8 (s)	8 (s)	0	0	8 (s)	7 (s)	50 (s)	0	0
1000 -1200 ft	38 (s)	38 (s)	42 (s)	0	0	56 (s)	36 (s)	36 (s)	0	0
1200 -1400 ft	6 (s)	0	2 (s)	0	0	0	0	0	0	0
1400 - 1600 ft	0	0	0	0	0	0	0	0	0	0
1600 - 1800 ft (3rd core location)	0	0	0	0	0	0	0	0	0	0
1800 - 2000 ft	26 (m)	44 (m)	48 (m)	0	0	22 (m)	22 (m)	28 (s)	0	0

Severity Level: (s) slight, (m) moderate.
*A Grade 4 Lightweight Seal Coat was placed prior to the evaluation in March of 2000.

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. This pavement is exhibiting a significant amount of transverse cracking as shown in Table 5.

Table 5. FM 1326 Distress.

Location, ft	Transverse Cracking, linear ft.					Longitudinal Cracking, linear ft.				
	1997	1998	1999	2000	2001	1997	1998	1999	2000	2001
0 - 100	0	0	0	36 (s)	140 (s)	0	0	0	0	8 (s)
100 - 200	0	0	0	96 (s)	136 (s)	0	0	0	0	100 (s)
200 - 300	0	0	0	48 (s)	144 (s)	0	0	0	0	100 (s)
300 - 400	0	0	0	0	144 (s)	0	0	0	0	50 (s)

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

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 is almost no change in the pavement since last year and is considered to be performing very well.

Table 6. FM 1520 Distress.

Location (each location represents a 200 ft length)	Flushing (sq ft)					Rutting (in)									
						1997		1998		1999		2000		2001	
	1997	1998	1999	2000	2001	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP
1	1000 (s)	1000 (s)	1000 (s)	1000 (s)	1000 (s)	0	0	0	0	0	0.1	0	0.1	0	0.1
2	1200 (s)	1200 (s)	1200 (s)	1200 (s)	1200 (s)	0	0	0	0	0	0.1	0	0.1	0	0.1
3	1500 (s)	1500 (s)	1500 (s)	1500 (s)	1500 (s)	0	0	0	0	0.1	0.1	0.1	0.1	0.1	0.1
4	320 (s)	320 (s)	320 (s)	320 (s)	320 (s)	0	0	0	0	0.1	0.1	0.1	0.1	0.1	0.1
5	0	0	0	0	0	0	0	0	0	0.1	0.1	0.1	0.1	0.1	0.1
6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Severity Level: (s) slight, (m) moderate.
Left Wheel Path (LWP), Right Wheel Path (RWP)

FM 560

The FM 560 project is located near Hooks and begins at Barkman Creek and Relief and extends north for 2300 feet. This pavement received an overlay prior to the 1999 evaluation; therefore, there was no evidence of any distress during the April 1999 evaluation, none in the March 2000 evaluation, and still none in May of 2001. Previous distress data is shown in Table 7. This pavement is performing well.

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	1999 *	2000	2001	1997	1998	1999 *	2000	2001	1997	1998	1999 *	2000	2001
1 Core Location 1	1000 (m)	1000 (m)	0	0	0	0	12 (s)	0	0	0	0	23 (s)	0	0	0
2 Core Location 2	150 (m) 120 (s)	150 (m) 120 (s)	0	0	0	5 (s)	5 (s)	0	0	0	10 (s)	10 (s)	0	0	0
3 Core Location 3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

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

* An overlay was constructed on the pavement between the 1998 and 1999 evaluations.

FIELD CORE AND FIELD TESTING DATA

TxDOT staff attempted to obtain three 6-inch diameter 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. Adjustment factors were used to facilitate comparing cores of different thickness as described in Tex 418-A. Table 8 shows results of the field core strength tests. Figure 1 compares results with previous years' results.

At the time the pavements were visually evaluated, Atlanta District personnel also performed Falling Weight Deflectometer (FWD) testing. The FWD is a test that nondestructively measures stiffness and relative deflection of the various layers of a pavement system. A load that 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 9 through 14. The moduli values for the base (E2) are of particular interest for this project.

TxDOT personnel provided researchers with roughness (IRI) data from the Pavement Management Information System (PMIS) database. The available data are shown in Figures 8 through 12.

Table 8. Field Cores - Unconfined Compressive Strengths.

Sample ID	Sample Height (in)	Failure Load (lbs)	Adjustment Factor	Corrected Failure Stress (psi)
FM 1520 Core 1	5.6	19,922	0.85	599.0
FM 1520 Core 2	5.3	33,106	0.83	972.0
FM 1520 Core 3	5.4	73,242	0.83	2150.4
IH 20 Core 1	5.9	64,258	0.87	1977.5
IH 20 Core 2	6.4	46,143	0.90	1469.0
IH 20 Core 3	6.6	36,865	0.90	1173.6
SH 154 Core 1	12.3	12,402	1.00	438.7
SH 154 Core 2	12.1	54,395	1.00	1924.1
SH 154 Core 3	11.6	30,811	1.00	1089.9
FM 1326 Core 1	4.1	62,304	0.72	1586.8
FM 1326 Core 2	4.6	49,951	0.76	1342.9
FM 1326 Core 3	5.0	10,693	0.76	287.5
FM 560 Core 1	6.4	36,425	0.90	1159.6
FM 560 Core 2	6.6	22,559	0.90	718.2
FM 560 Core 3	7.3	9326	0.92	303.5

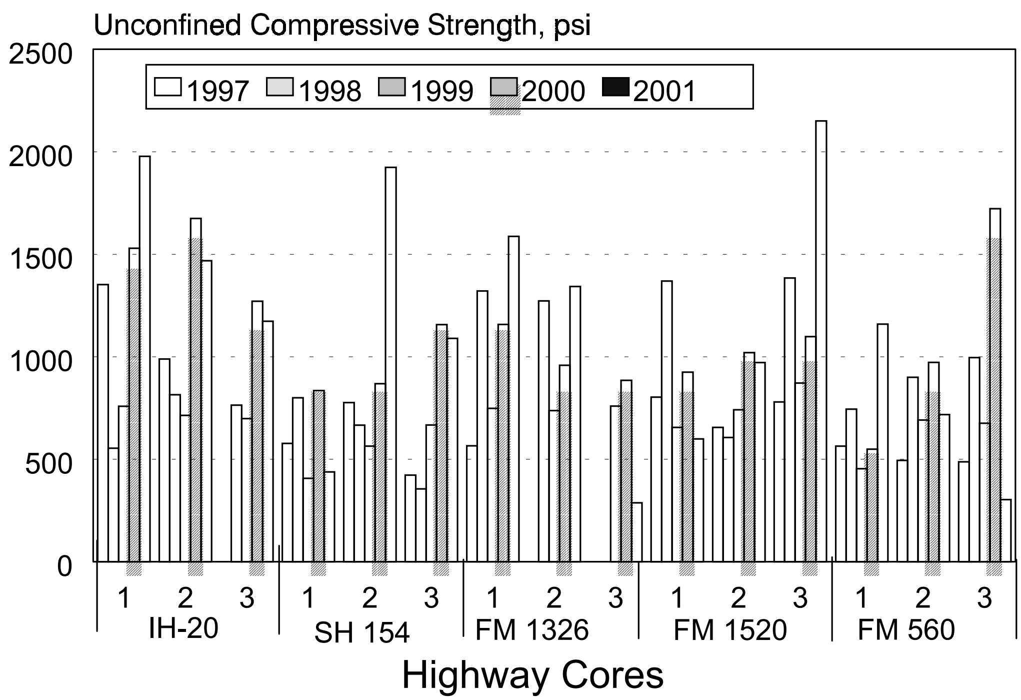

Figure 1. Unconfined Compressive Strength of Highway Cores.

Table 9. FWD Data Analysis - Loop 390.

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

District: 19				MODULI RANGE(psi)		
County: 103		Thickness(in)	Minimum	Maximum	Poisson Ratio Values	
Highway/Road: SL0390	Pavement:	2.00	199,980	200,020	H1: u = 0.35	
	Base:	10.00	30,000	600,000	H2: u = 0.20	
	Subbase:	8.00	4,000	500,000	H3: u = 0.15	
	Subgrade:	135.90	16,100		H4: u = 0.40	

Station	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	Dpth to Bedrock
321.000	9,346	8.02	5.94	4.59	3.70	2.21	1.38	1.05	200.	600.0	7.3	24.5	11.94	110.82
842.000	9,307	7.80	5.22	2.94	2.72	1.11	0.80	0.72	200.	397.2	17.5	30.5	9.69	300.00
1370.000	9,366	7.13	4.53	3.05	2.68	1.57	1.25	1.10	200.	332.2	156.1	20.7	4.88	300.00
1900.000	9,144	10.72	8.27	5.74	4.30	2.76	1.93	1.31	200.	476.0	10.0	13.3	2.47	153.62
2430.000	9,287	8.75	5.36	3.37	2.71	1.55	1.20	1.03	200.	239.5	66.1	20.8	5.03	300.00
2962.000	9,017	12.14	9.04	5.18	3.59	1.91	1.31	1.10	200.	259.1	6.1	20.2	6.23	86.40
3485.000	9,366	5.42	4.38	3.58	3.11	1.85	1.43	1.13	200.	600.0	403.7	15.6	9.58	107.88
4011.000	9,481	16.39	11.15	5.87	3.98	2.36	1.77	1.41	200.	135.3	8.8	15.6	6.32	111.89
4539.000	9,366	13.99	9.82	5.83	4.31	2.68	2.11	1.68	200.	189.2	17.4	12.8	5.65	131.84
5019.000	9,374	10.38	6.43	3.67	2.74	1.80	1.40	1.19	200.	183.9	46.0	19.3	5.19	171.47
5090.000	9,620	12.90	8.69	4.22	2.80	1.78	1.56	1.29	200.	156.3	16.3	20.3	9.57	300.00
5596.000	9,263	14.24	10.19	5.63	3.70	2.25	1.61	1.29	200.	182.8	7.7	16.9	5.86	122.48
6023.000	9,402	9.93	4.89	2.44	2.29	1.64	1.35	1.08	200.	110.4	238.0	25.2	13.23	300.00
6651.000	9,195	13.66	9.52	5.59	3.90	2.45	1.76	1.34	200.	191.5	12.3	14.5	4.34	139.23
7181.000	9,136	7.19	5.14	3.35	2.72	1.83	1.45	1.17	200.	483.7	77.0	18.3	4.20	198.78
7721.000	9,267	11.89	8.25	4.33	3.22	2.17	1.55	1.24	200.	193.7	20.0	17.0	6.82	300.00
7907.000	9,084	13.28	8.42	4.56	3.30	2.28	1.74	1.35	200.	133.4	27.3	15.4	6.67	300.00
8236.000	9,128	9.36	6.33	4.42	3.42	2.33	1.85	1.43	200.	278.4	92.3	14.0	2.85	224.63
8766.000	8,897	12.42	8.31	4.82	3.26	1.95	1.45	1.09	200.	186.0	12.9	17.1	4.72	114.24
9292.000	8,977	9.54	5.91	3.12	2.13	1.33	1.02	0.83	200.	200.8	25.3	25.3	5.18	131.68
9819.000	9,080	6.62	4.13	2.56	2.09	1.39	1.08	0.89	200.	325.0	133.0	24.5	4.49	300.00
10349.000	8,826	12.19	7.76	4.69	3.23	2.07	1.58	0.96	200.	165.4	23.4	15.5	4.07	152.85
10880.000	8,973	10.41	6.47	3.80	2.56	1.67	1.22	0.78	200.	192.4	27.0	20.1	3.50	167.25
11403.000	8,750	7.23	5.04	3.21	2.43	1.61	1.20	0.87	200.	425.9	49.2	20.3	3.25	180.35
11937.000	8,790	12.49	9.11	5.57	3.68	2.33	1.69	1.24	200.	239.4	9.2	15.2	5.01	146.51
12462.000	9,485	18.54	13.14	6.16	3.25	1.84	1.27	0.93	200.	107.2	4.7	22.6	10.52	95.93
12990.000	8,604	16.09	10.53	5.38	3.10	1.84	1.35	1.05	200.	109.9	7.0	18.6	7.42	114.64
13524.000	8,353	20.88	13.37	6.17	3.26	1.89	1.40	0.89	200.	66.3	5.1	17.0	10.02	97.56
Mean:		11.41	7.69	4.42	3.15	1.94	1.45	1.12	200.	255.7	54.5	19.0	6.38	155.92
Std. Dev:		3.72	2.58	1.16	0.61	0.40	0.29	0.22	0.	146.7	87.5	4.3	2.83	62.98
Var Coeff(%):		32.55	33.59	26.26	19.31	20.59	19.96	19.81	0.	57.3	100.0	22.5	44.41	40.39

Table 10. FWD Data Analysis - IH 20 Frontage Road.

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: u = 0.35
	11.00	100,000	2,000,000	H2: u = 0.20
	8.00	20,000	700,000	H3: u = 0.15
	58.90	12,000		H4: u = 0.40

Station	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	Dpth to Bedrock
TTI Load 200.000	8,317	34.18	16.15	3.17	1.13	0.91	0.92	0.75	200.	100.0	20.0	18.9	42.11	36.00
401.000	9,180	2.98	2.31	1.81	1.50	1.11	0.86	0.64	200.	2000.0	578.3	13.5	2.65	220.88
675.000	9,303	2.59	2.06	1.57	1.15	0.81	0.64	0.37	200.	2000.0	284.2	20.9	4.50	36.00
800.000	9,394	3.44	2.35	1.80	1.35	0.96	0.74	0.49	200.	1572.4	290.0	18.2	1.09	36.00
999.000	9,215	2.74	2.31	1.83	1.45	1.10	0.86	0.69	200.	2000.0	148.6	19.7	13.30	300.00
Base: 1200.000	9,259	8.53	6.42	4.17	3.18	2.24	1.61	1.17	200.	354.6	116.4	7.9	4.37	204.97
Subbase: 1234.000	9,231	7.04	5.57	3.79	2.79	2.03	1.63	1.17	200.	592.2	123.4	8.3	5.68	300.00
Subgrade: 1401.000	9,183	7.39	4.63	2.74	2.30	1.78	1.44	1.06	200.	207.5	700.0	9.3	8.37	300.00
1604.000	9,044	9.28	5.79	3.58	2.55	1.87	1.39	1.11	200.	156.4	286.3	9.0	6.59	287.53
2010.000	9,851	12.33	5.44	2.90	2.33	1.82	1.51	1.13	200.	100.0	700.0	10.2	12.90	300.00
2201.000	9,406	8.69	5.63	3.14	2.23	1.71	1.37	1.09	200.	157.5	451.9	10.1	9.81	300.00
2248.000	9,426	9.61	5.75	2.83	2.18	1.69	1.35	1.01	200.	112.1	700.0	10.3	11.43	300.00
2347.000	9,315	13.04	8.11	3.73	2.18	1.59	1.26	1.09	200.	100.0	63.3	11.7	14.29	157.78
2400.000	8,993	9.50	5.95	3.27	2.41	1.81	1.50	1.03	200.	117.4	661.6	8.6	9.50	300.00
2601.000	8,882	9.39	6.42	3.71	2.57	1.79	1.35	1.04	200.	188.4	119.1	9.6	8.37	276.31
2799.000	8,846	8.39	5.49	3.19	2.15	1.50	1.22	0.98	200.	193.1	166.1	11.1	9.67	278.27
3000.000	9,791	14.86	8.19	2.72	1.45	1.02	0.76	0.75	200.	100.0	35.6	19.2	24.09	36.00
3118.000	9,493	12.28	7.20	2.82	1.56	1.21	1.04	0.59	200.	100.0	73.6	16.4	20.88	72.27
3172.000	9,569	12.24	6.67	2.93	1.80	1.25	0.96	0.85	200.	100.0	100.2	14.9	14.60	137.21
3400.000	9,235	2.39	1.73	1.15	0.83	0.53	0.46	0.26	200.	2000.0	398.7	31.2	6.00	24.00
3602.000	8,830	2.52	1.65	1.07	0.70	0.45	0.36	0.26	200.	1680.2	249.5	36.8	7.41	24.00
3807.000	9,033	1.69	1.21	0.93	0.67	0.43	0.39	0.24	200.	2000.0	441.7	42.2	9.36	16.00
Mean:		8.87	5.32	2.67	1.84	1.35	1.07	0.81	200.	724.2	304.9	16.3	11.23	79.90
Std. Dev:		6.94	3.26	0.95	0.70	0.53	0.40	0.32	0.	830.5	236.8	9.5	8.81	89.86
Var Coeff(%):		78.25	61.31	35.67	38.26	39.33	37.37	39.77	0.	100.0	77.7	58.1	78.50	112.47

Table 11. FWD Data Analysis - SH 154.

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: u = 0.35
Base:	13.00	15,000	2,500,000	H2: u = 0.20
Subbase:	0.00	0	0	H3: u = 0.15
Subgrade:	242.90	16,400		H4: u = 0.40

Station	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	Dpth to Bedrock
150.000	8,115	40.54	21.33	7.50	4.21	2.96	2.30	1.78	200.	21.5	0.0	11.1	9.64	59.25
201.000	8,381	30.30	14.52	3.74	1.44	1.20	1.00	0.85	200.	24.7	0.0	22.7	22.10	62.38
299.000	8,234	41.50	23.57	9.15	3.81	2.00	1.54	1.40	200.	19.7	0.0	12.8	22.78	65.38
399.000	9,176	6.64	5.50	4.15	3.17	2.26	1.71	1.35	200.	703.9	0.0	18.6	2.56	300.00
499.000	9,942	6.46	5.06	4.12	3.11	2.35	1.69	1.29	200.	859.6	0.0	20.3	1.18	223.15
512.000	9,418	5.82	5.37	4.11	3.16	2.38	1.69	1.43	200.	1022.0	0.0	18.4	4.95	205.71
610.000	9,688	4.91	4.46	3.59	3.06	2.27	1.82	1.39	200.	1724.7	0.0	18.9	2.35	300.00
702.000	9,819	3.63	3.44	2.87	2.53	2.22	1.49	1.39	200.	2500.0	0.0	23.0	4.93	150.83
803.000	9,839	5.98	4.13	3.87	2.99	2.72	1.92	1.73	200.	1356.0	0.0	19.2	7.13	300.00
900.000	8,941	4.53	4.23	3.50	2.82	2.30	1.98	1.52	200.	2055.4	0.0	16.8	3.93	273.88
1037.000	9,950	5.23	4.37	3.95	3.54	2.98	2.38	2.07	200.	2446.7	0.0	15.0	1.76	300.00
1102.000	8,953	4.68	4.54	3.91	3.24	2.68	2.23	1.73	200.	2378.8	0.0	14.1	3.64	293.66
1230.000	8,607	6.03	5.17	4.20	3.35	2.62	2.19	1.52	200.	1106.1	0.0	15.1	3.08	300.00
1251.000	9,493	5.46	4.75	3.88	3.20	2.60	2.15	1.70	200.	1584.5	0.0	16.7	2.62	300.00
1301.000	8,886	5.11	4.85	4.04	3.36	2.63	2.17	1.81	200.	1789.2	0.0	14.6	3.42	300.00
1400.000	9,164	4.57	4.38	3.72	3.01	2.43	1.85	1.59	200.	2050.7	0.0	16.7	4.40	300.00
1500.000	8,750	2.17	2.80	2.17	1.57	1.07	0.73	0.37	200.	2500.0	0.0	38.2	13.55	
1600.000	8,635	5.34	5.26	4.28	3.56	2.78	2.24	1.74	200.	1622.4	0.0	13.5	4.32	298.41
1700.000	9,446	19.54	10.72	6.57	4.14	2.65	2.05	1.56	200.	89.4	0.0	14.5	1.53	158.13
1800.000	9,005	7.74	6.14	4.59	3.39	2.56	1.93	1.44	200.	538.6	0.0	16.7	2.08	239.11
1903.000	9,517	7.07	6.23	4.77	3.58	2.59	1.90	1.48	200.	736.8	0.0	16.8	4.63	261.51
2070.000	9,581	8.96	6.42	4.17	2.79	1.87	1.35	1.06	200.	297.7	0.0	21.6	3.26	201.68
Mean:		10.56	7.15	4.40	3.14	2.37	1.83	1.46	200.	1246.7	0.0	18.0	5.90	256.40
Std. Dev:		11.58	5.56	1.51	0.67	0.49	0.42	0.36	0.	872.0	0.0	5.5	6.04	199.54
Var Coeff(%):		99.99	77.76	34.41	21.20	20.50	22.83	24.45	0.	69.9	0.0	30.6	102.43	77.82

Table 12. FWD Data Analysis - FM 1326.

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	199,980	200,020	H1: u = 0.35
Base:	5.50	20,000	1,500,000	H2: u = 0.20
Subbase:	8.00	4,000	180,000	H3: u = 0.15
Subgrade:	69.60	3,000		H4: u = 0.40

Station	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	Dpth to Bedrock
0.000	8,310	47.55	20.44	6.49	4.27	2.39	1.93	1.68	200.	26.6	11.9	7.5	18.28	53.95
51.000	8,226	41.80	21.54	8.54	5.01	2.95	2.49	1.98	200.	36.8	16.1	6.0	12.75	80.67
99.000	8,810	14.63	10.06	5.43	3.75	1.86	1.39	1.23	200.	223.0	88.7	9.5	7.95	78.47
149.000	8,945	13.08	10.37	7.10	5.27	2.89	1.99	1.63	200.	1324.2	67.1	6.7	3.61	91.99
200.000	8,588	12.91	8.67	5.87	4.59	2.58	1.91	1.57	200.	1081.4	78.5	7.5	8.50	95.51
249.000	8,468	12.88	9.72	6.04	4.47	2.47	1.81	1.53	200.	913.3	73.6	7.4	6.22	92.95
280.000	8,564	15.85	9.85	5.35	3.69	1.87	1.48	1.26	200.	119.3	104.7	10.0	7.29	80.68
301.000	8,619	12.77	9.79	6.35	4.73	2.53	2.00	1.72	200.	1113.4	71.4	7.1	6.72	87.45
349.000	8,389	14.71	9.76	5.67	3.85	2.07	1.49	1.30	200.	172.2	109.3	8.9	5.34	88.89
389.000	8,310	18.01	12.50	7.36	4.89	2.55	1.69	1.34	200.	207.9	63.0	6.8	4.78	86.16
439.000	8,135	40.44	20.22	7.69	3.93	2.30	1.74	1.57	200.	42.8	11.7	7.2	12.15	70.38
450.000	8,079	40.31	20.19	7.82	4.25	2.27	1.74	1.44	200.	42.2	12.2	6.9	10.01	74.20
Mean:		23.75	13.59	6.64	4.39	2.39	1.80	1.52	200.	441.9	59.0	7.6	8.63	83.58
Std. Dev:		14.06	5.26	1.04	0.52	0.35	0.30	0.22	0.	504.1	36.7	1.2	4.12	14.13
Var Coeff(%):		59.22	38.67	15.70	11.91	14.51	16.43	14.48	0.	100.0	62.2	16.0	47.76	16.91

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Table 13. FWD Data Analysis - FM 1520.

MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19
County: 32
Highway/Road: FM1520

	Layer	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
	Pavement:	0.50	199,980	200,020	H1: u = 0.35
	Base:	10.00	20,000	400,000	H2: u = 0.20
	Subbase:	8.00	4,000	150,000	H3: u = 0.15
	Subgrade:	157.90	15,000		H4: u = 0.40

Station	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	Dpth to Bedrock
0.000	9,430	13.92	9.10	4.74	2.79	1.81	1.21	0.90	200.	199.2	10.3	24.3	3.85	160.16
201.000	10,125	10.61	6.90	3.83	2.52	2.00	1.49	1.31	200.	282.3	29.0	23.7	7.56	300.00
600.000	9,708	22.30	13.39	6.24	3.84	2.67	1.96	1.41	200.	92.2	11.2	16.3	7.23	267.92
1201.000	9,859	15.82	8.74	4.46	3.17	2.33	1.75	1.12	200.	123.1	32.3	19.1	7.92	300.00
2078.000	9,918	27.59	14.02	6.85	5.14	3.58	2.67	1.86	200.	55.4	23.3	12.7	7.53	300.00
2429.000	8,886	34.50	20.79	10.04	5.85	4.57	3.79	2.62	200.	50.0	7.3	9.3	11.15	178.05
2999.000	9,819	27.29	15.82	8.63	4.85	2.97	2.01	1.40	200.	87.1	6.2	14.8	1.69	132.44
3600.000	9,434	25.61	16.56	7.93	4.28	2.66	1.93	1.44	200.	89.1	5.5	15.9	6.18	107.56
4077.000	9,251	10.37	4.87	2.44	1.83	1.14	0.90	0.70	200.	166.4	49.7	32.0	10.53	300.00
4210.000	9,799	25.26	11.25	5.10	2.97	2.37	2.20	1.65	200.	51.5	21.5	18.0	13.49	157.13
4800.000	9,760	26.42	13.94	5.63	4.39	3.32	2.30	1.56	200.	53.7	20.4	14.6	12.29	81.39
5401.000	9,835	19.13	13.53	6.66	3.63	2.35	1.63	1.23	200.	148.2	6.5	19.7	6.52	109.73
6001.000	9,724	16.36	9.93	5.27	3.38	2.25	1.64	1.28	200.	144.7	16.5	18.4	3.34	197.22
6447.000	9,760	9.89	6.84	4.41	3.32	2.61	2.08	1.78	200.	321.1	75.7	16.4	6.27	300.00
6612.000	9,855	10.28	6.33	4.09	3.06	2.38	1.89	1.32	200.	251.3	94.6	17.6	7.08	174.17
7200.000	8,886	17.58	13.24	9.16	4.73	2.58	1.53	0.95	200.	195.5	4.0	17.0	7.77	93.72
7801.000	9,922	10.84	5.25	3.06	2.99	2.02	1.65	1.36	200.	185.3	115.4	22.6	18.38	300.00
8400.000	9,692	22.19	13.35	8.08	5.95	4.40	3.13	2.02	200.	93.2	32.2	10.2	3.37	147.12
8760.000	8,246	23.35	18.76	12.39	7.35	4.25	2.46	1.61	200.	130.5	4.0	8.7	10.26	118.30
Mean:		19.44	11.72	6.26	4.00	2.75	2.01	1.45	200.	143.1	29.8	17.4	8.02	176.43
Std. Dev:		7.35	4.58	2.58	1.38	0.91	0.67	0.43	0.	80.1	32.2	5.6	4.00	84.12
Var Coeff(%):		37.81	39.08	41.16	34.47	33.02	33.27	30.00	0.	55.9	100.0	32.3	49.92	47.68

Table 14. FWD Data Analysis - FM 560.

MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19
County: 19
Highway/Road: FM0560

Layer	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	4.00	200,000	2,000,000	H1: u = 0.35
Base:	6.50	15,000	1,000,000	H2: u = 0.20
Subbase:	6.00	10,000	700,000	H3: u = 0.15
Subgrade:	220.30	12,900		H4: u = 0.40

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Station	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	Dpth to Bedrock
0.000	8,365	16.76	12.05	7.51	5.65	3.85	3.05	2.40	992.	22.8	101.4	9.0	2.49	272.28
150.000	8,361	21.31	13.24	7.42	5.29	3.61	2.88	2.29	453.	15.0	182.4	9.5	2.43	273.00
299.000	8,814	4.29	3.91	3.44	3.19	2.39	1.79	1.14	2000.	1000.0	419.0	16.3	5.86	161.82
450.000	8,409	17.81	12.56	7.46	5.13	3.46	2.58	1.96	977.	18.7	47.2	10.3	1.45	246.36
600.000	8,385	22.23	15.89	9.41	6.24	3.97	2.89	2.30	904.	15.0	17.4	8.9	1.39	164.66
759.000	8,417	15.52	11.35	7.69	5.29	3.50	2.54	1.88	704.	100.0	12.6	10.5	0.81	213.42
899.000	8,401	16.42	11.08	6.74	4.76	3.34	2.53	1.92	801.	22.4	111.0	10.6	1.08	300.00
1050.000	8,532	10.21	7.15	4.87	3.76	2.61	2.05	1.71	1200.	54.0	242.8	13.4	1.55	300.00
1200.000	8,568	8.71	5.42	3.89	3.03	2.13	1.72	1.41	405.	106.5	452.1	17.3	1.73	300.00

MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 5.1)

District: 19
County: 19
Highway/Road: FM0560

Layer	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	4.00	200,000	2,000,000	H1: u = 0.35
Base:	9.50	15,000	1,000,000	H2: u = 0.20
Subbase:	3.50	10,000	1,000,000	H3: u = 0.15
Subgrade:	219.80	14,600		H4: u = 0.40

TTI

Station	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	Dpth to Bedrock
1350.000	8,552	10.31	6.18	4.34	3.31	2.28	1.81	1.40	254.	110.4	311.5	16.3	1.76	300.00
1500.000	8,445	12.33	8.31	4.98	3.63	2.56	2.00	1.53	1012.	34.4	332.7	13.6	1.73	300.00
1614.000	8,786	8.85	6.72	4.85	3.69	2.60	1.99	1.51	1499.	145.8	43.5	14.5	1.08	300.00
1801.000	8,540	10.37	6.91	4.15	3.22	2.18	1.95	1.57	1082.	43.1	1000.0	14.9	4.18	260.20
1950.000	8,449	11.28	7.48	4.81	3.23	2.20	1.76	1.35	761.	68.6	35.7	16.1	2.01	275.93
2037.000	8,540	8.91	6.70	4.81	3.50	2.35	1.80	1.27	1956.	105.3	31.1	15.3	1.64	243.31
2081.000	8,536	10.12	7.59	5.21	3.69	2.41	1.82	1.33	2000.	74.3	16.0	15.1	1.53	196.77
2250.000	8,727	3.18	1.62	1.02	1.05	0.71	0.67	0.61	669.	350.4	1000.0	66.9	15.64	36.00
2401.000	8,397	14.18	9.64	5.10	3.44	2.31	1.77	1.35	1084.	18.2	742.1	14.8	2.40	238.30
2549.000	8,051	42.37	27.29	12.94	6.63	3.78	2.66	2.15	200.	15.0	10.0	6.4	24.17	91.59

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 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 2 through 7, the base moduli values are plotted for each test pavement and compared with previous years' data.

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

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

LOOP 390

No cores were obtained from this pavement. Unsuccessful attempts were made in 1997, 1998, 1999, 2000, and again in 2001. As shown in Figure 2, there is some variation in the moduli values since 1997; however, it does not appear that the base is exhibiting a deteriorating strength overall. Some locations indicate an increase in stiffness while others show a decrease.

IH 20 FRONTAGE ROAD

Three cores were obtained from this pavement as shown in Figure 1. The pavement core strengths are slightly less than but comparable to the core strengths measured last year. There is very little change in the FWD data exhibited in Figure 3 since 1997. Note in Figure 3 that the last data point may coincide with the beginning of a different type of pavement section.

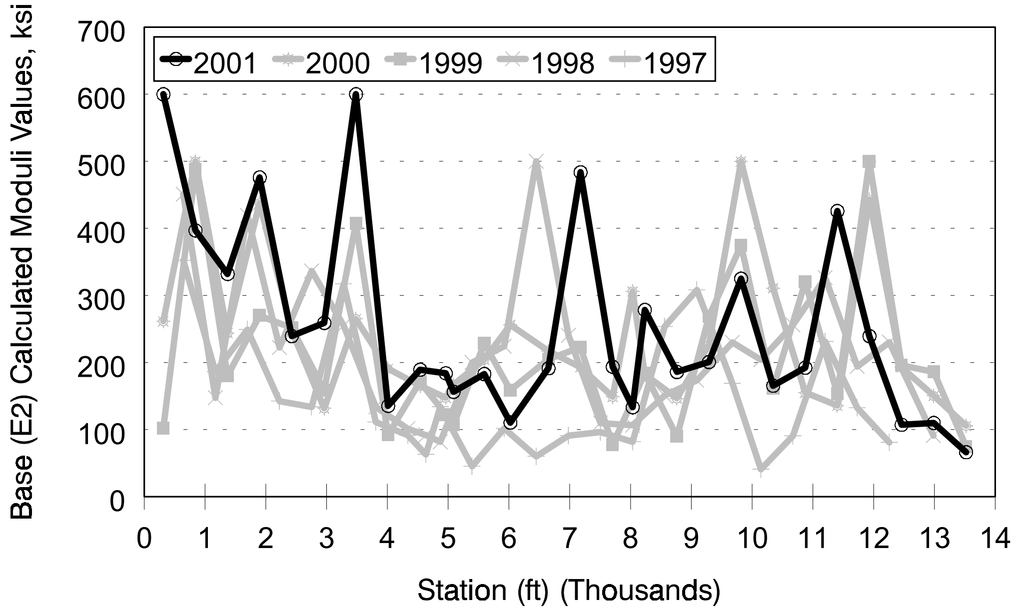

Figure 2. Base Moduli Values for Loop 390.

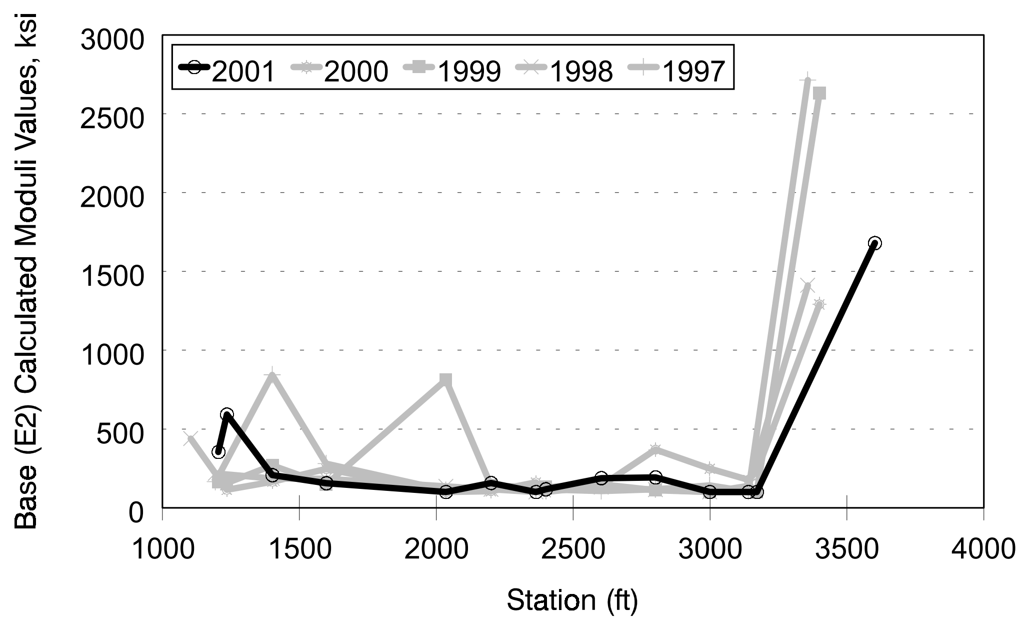

Figure 3. Base Moduli Values for IH 20 Frontage Road.

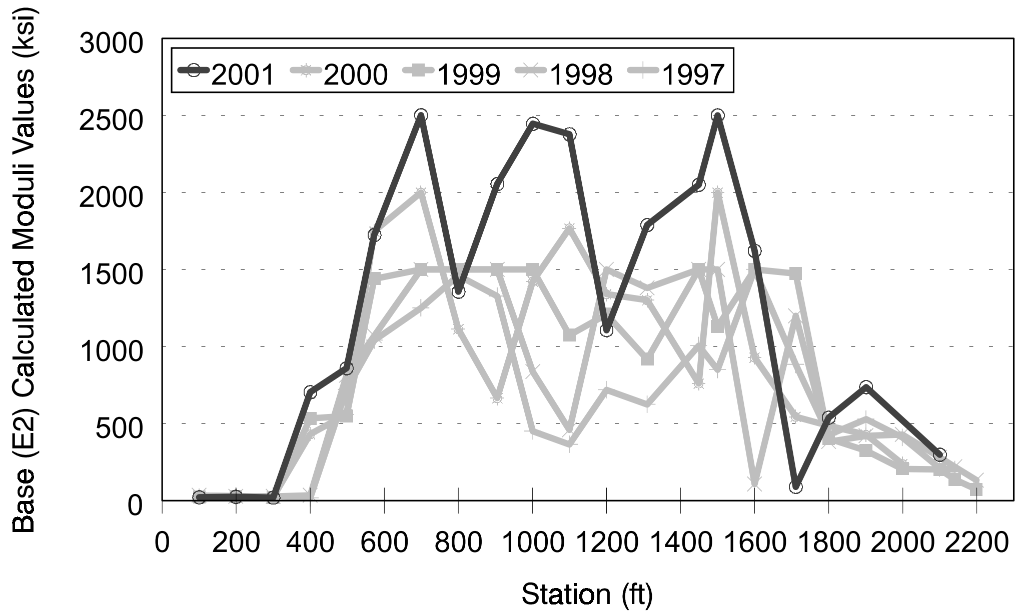

Figure 4. Base Moduli Values for SH 154.

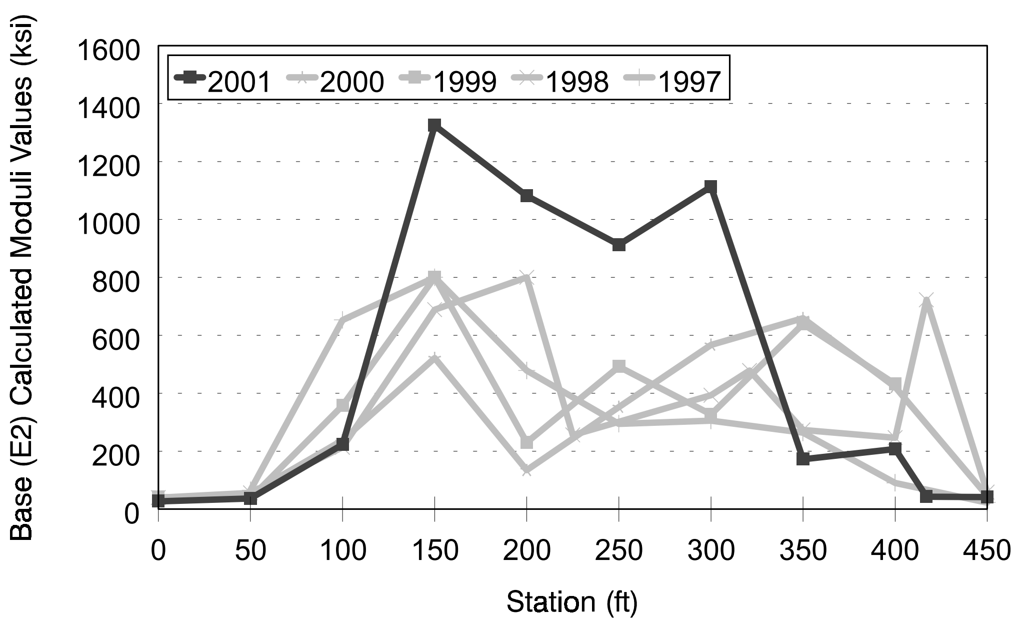

Figure 5. Base Moduli Values for FM 1326.

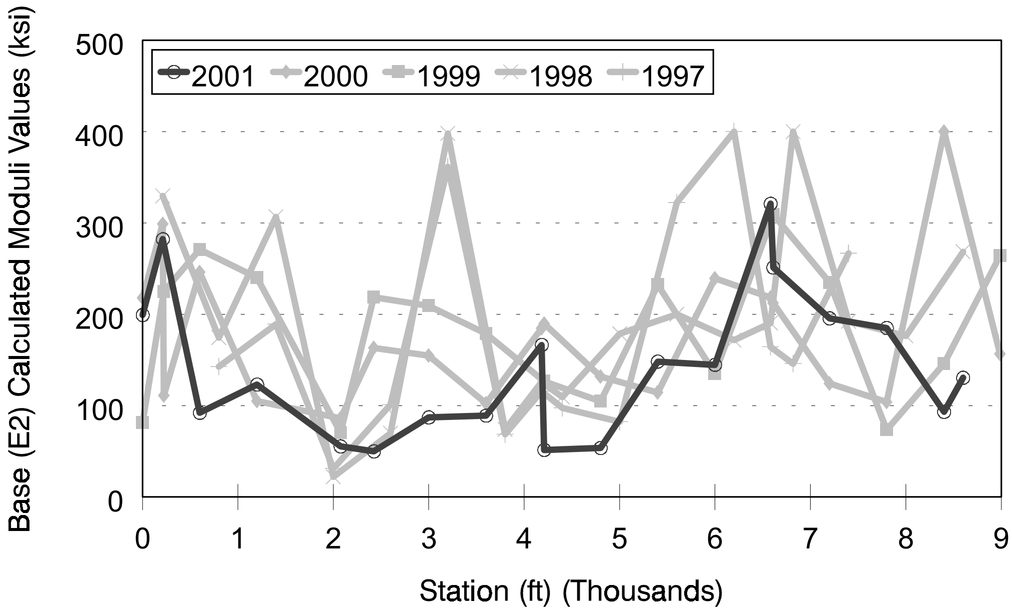

Figure 6. Base Moduli Values for FM 1520.

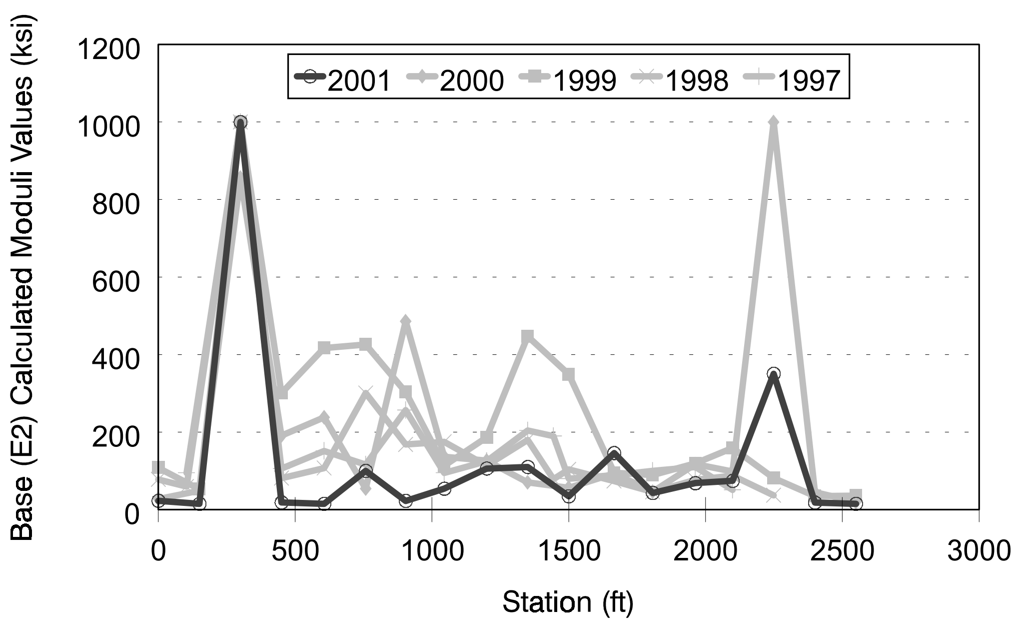

Figure 7. Base Moduli Values for FM 560.

SH 154

From what has appeared 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 4). Base moduli values along the pavement exceed 1,000,000 psi in some locations. Base moduli values in 2001 are somewhat greater than values observed in previous years. Compressive strength for one of the cores taken in 2001 is much greater than previous years; whereas, the other cores are comparable to previous measurements.

FM 1326

Two of the three cores obtained from FM 1326 show an increase in strength over that measured in 2000. One of the cores has a significantly lower strength than last year's. The base moduli values as calculated from FWD data (shown in Figure 5) show an increase in stiffness at some locations and a decrease in other locations.

FM 1520

Three cores were obtained from FM 1520, and these cores had an average strength higher than last year's core data. FWD data (Figure 6) on this pavement indicate that there may be a general decrease in moduli values since last year; however, most of the values still fall between 50,000 and 300,000 psi.

FM 560

Two of the three cores obtained from FM 560 had lower compressive strengths and one had a higher strength than the cores obtained in 2000. 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 7. Moduli values for this pavement are lower in 2001 than measured in 2000.

IRI / Roughness Data

TxDOT personnel provided researchers with roughness (IRI) data from the PMIS database. The available data are shown in Figures 8 through 12. Most of the pavements generally exhibit a roughness of more than 80 to 100 inches per mile. To draw any significant conclusions from this type of data, one would need roughness information soon after construction.

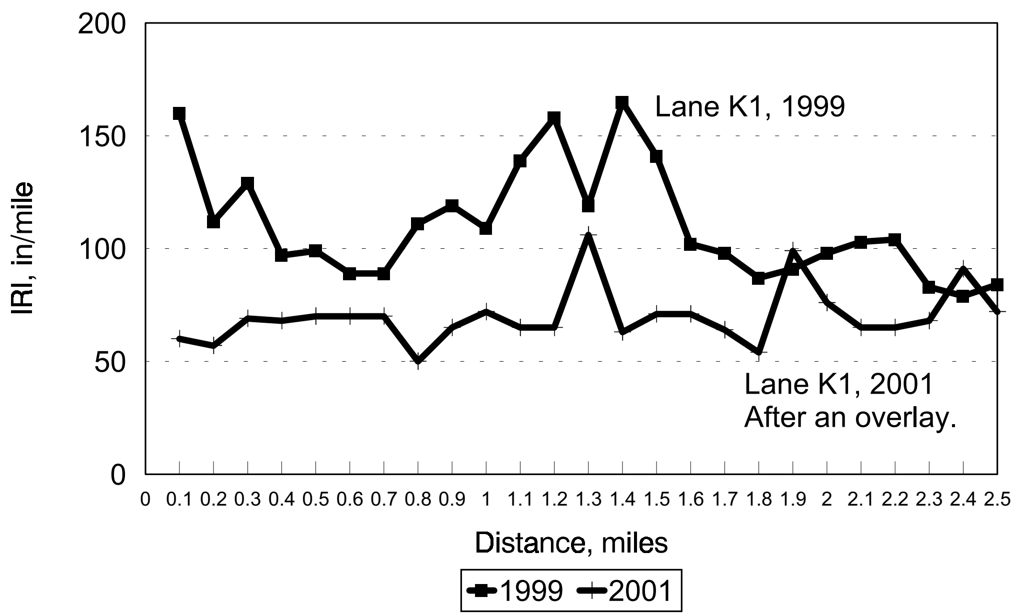

Figure 8. Roughness Data for Loop 390.

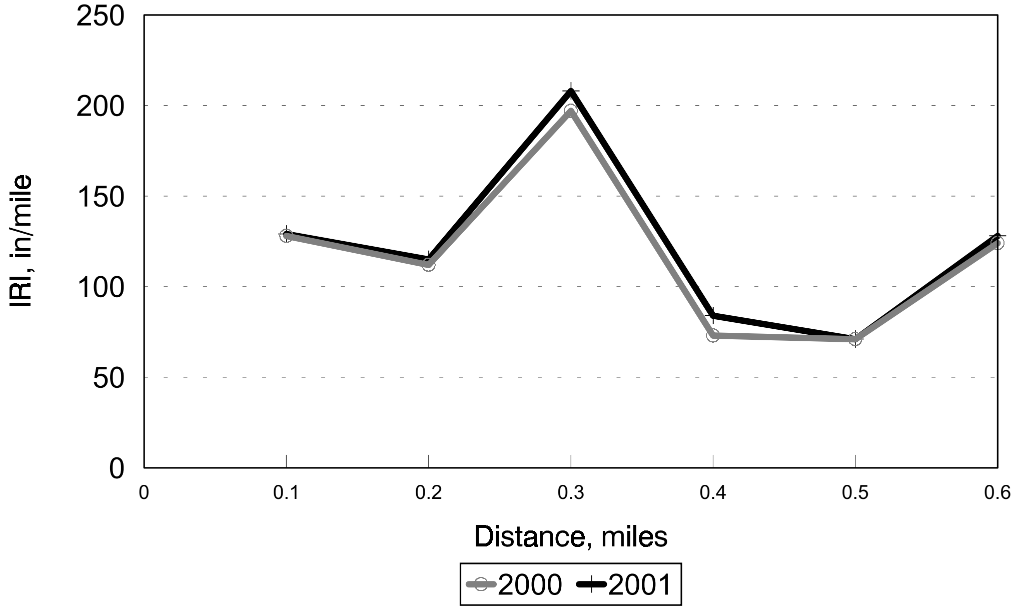

Figure 9. Roughness Data for IH 20 Frontage Road.

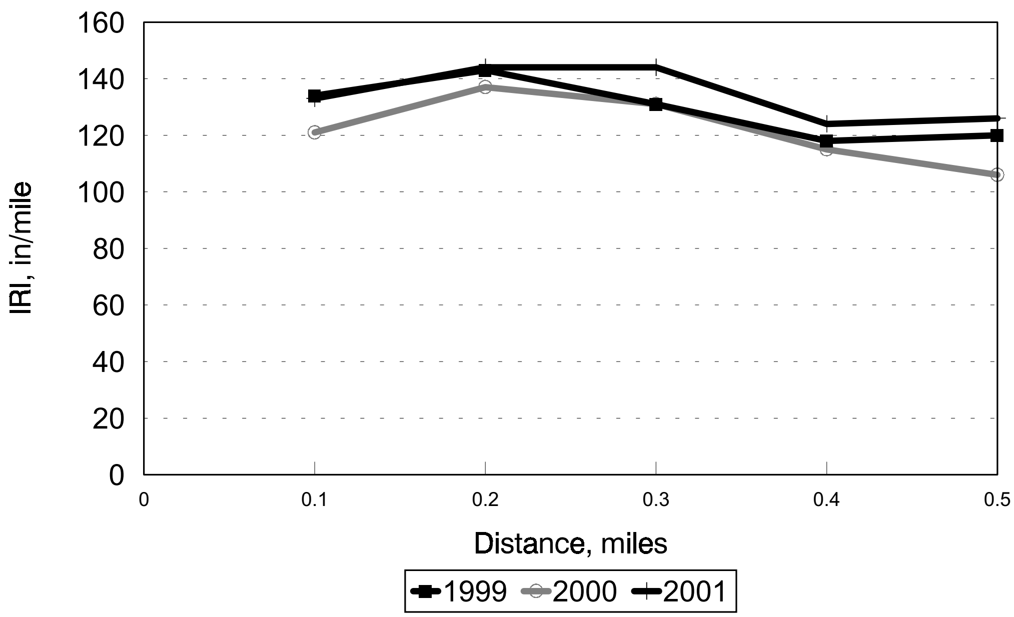

Figure 10. Roughness Data for SH 154.

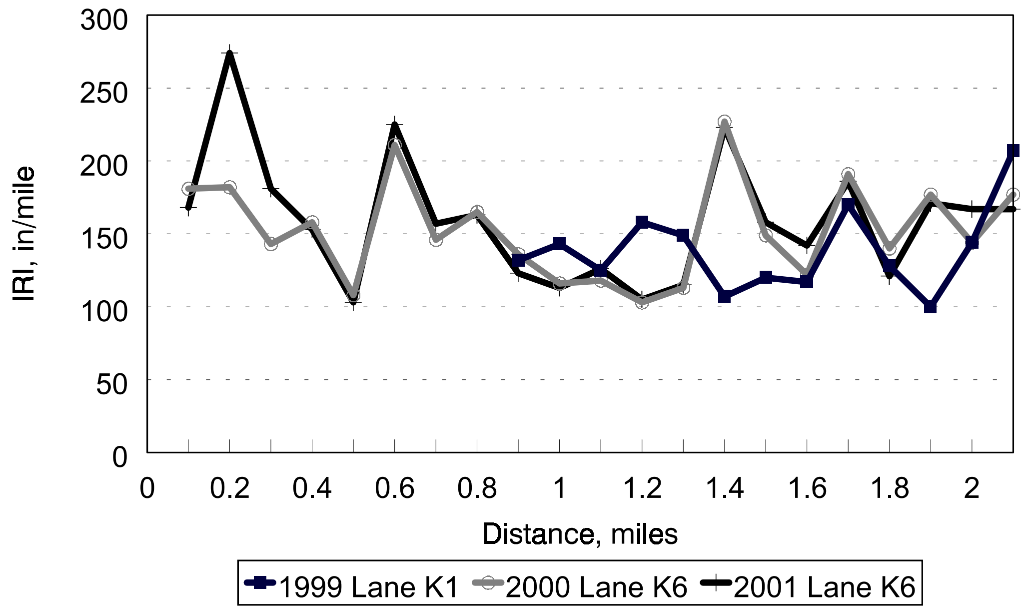

Figure 11. Roughness Data for FM 1520.

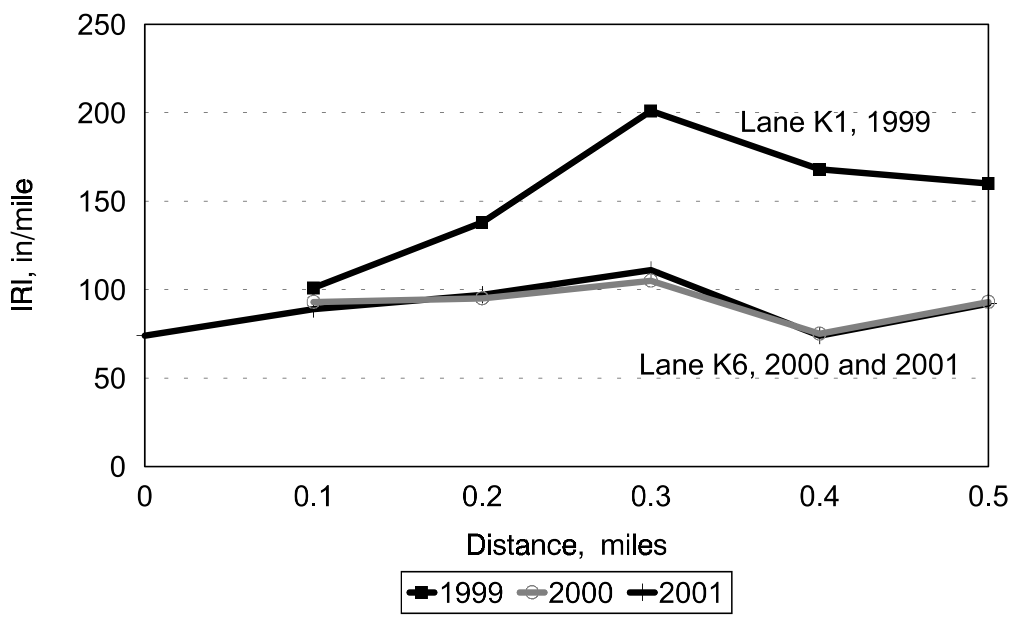

Figure 12. Roughness Data for FM 560.

CONCLUSIONS

- All of the hydrated fly-ash test pavements have performed well throughout this research project. Cracking distress has been exhibited in four of the six test pavements; however, to a significant degree in only two pavements: FM 1326 and IH 20. This significant amount of cracking has occurred in the final year of the study. For all of the pavements except FM 1326, the distress is generally in isolated areas, and the distress is not affecting the serviceability of the roadway.
- There has been little change observed in the performance of the six pavements since 1997. Four of the six hydrated fly-ash test pavements have exhibited distress that 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 no further cracking distress has been evident. Loop 390 also previously exhibited some rutting, but it appeared it may have been within the hot-mix asphalt concrete layer. SH 154 has exhibited transverse cracking (which appears to be from shrinkage of the base), and the FWD data indicate this pavement is very stiff. This pavement was chip-sealed in 1999 and no distress is currently exhibited on the surface. IH 20 is beginning to exhibit some alligator cracking which could be attributed to the base. In 2001, FM 1326 began to exhibit a significant degree of transverse cracking which would be attributable to the base.
- Year 2001 FWD data were compared to that taken in 2000, 1999, 1998, and 1997. Moduli of the fly-ash base materials were back-calculated from the FWD data. There is no indication of any significant weakening of these base materials with time. In the past year, there appears to be some weakening exhibited in FM 560 and FM 1520.
- Cores were taken on all of the test pavements except Loop 390. No intact core could be obtained from Loop 390 throughout the five years of this research study. Compressive strengths for the cores from the other five test pavements were comparable to the strengths observed in the year 2000 and previous years.

- The six test pavements evaluated in this research study range in age from 6 to 8 years. Based on visual evaluations, FWD data, and compressive strengths of cores, the hydrated fly-ash test pavements have performed well with only one pavement exhibiting a significant amount of distress and that was in it's eighth year of service.

RECOMMENDATIONS

Based on five years of monitoring for these fly-ash test pavements, performance results are very promising. Pavement base materials have not exhibited any significant deterioration over the study period. Researchers, therefore, can recommend use of such material in the applications and highway types as used in the Atlanta District. Concern is warranted regarding the fly ash material variability as exhibited in moduli values from FWD data; however, this variability has not adversely affected performance thus far. Methods used to *hydrate* the fly ash do not necessarily produce a consistent material.

Another concern regarding the use of this type of fly ash is that fly ash produced from one plant is not the same as one produced at another. The type of fly ash used for this study is known as a Class C fly ash. A fluidized bed ash should not be used in paving applications.

Ground penetrating radar (GPR) data taken in 1999 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 percent. It appears that typical *rule of thumb* criteria which we typically apply to conventional pavements may not be applicable to fly-ash bases.

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 ensure that adequate curing occurs prior to application of the surface treatment (especially on higher-traffic roadways). Once the base has been compacted at optimum moisture content, any additional water sprayed on the surface for finishing 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.

For a better surface treatment bond to the base, researchers recommend the following modifications to *Special Specification No. 2011 - Fly Ash Base*. Article 6 *Finishing* on page 3-4 should be amended by deleting the following items as stated below:

6. <u>Finishing</u>. 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 the 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 the fy 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.~~

Article 7 *Curing* on page 304 shall be deleted as stated below:

7. <u>Curing.</u> ~~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 gallons per square yard unless the plans require otherwise.~~

~~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.~~

Article 7 should be replaced with the following:

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

Performance results for the hydrated fly ash test pavements evaluated in this study are very promising. However, as mentioned previously, there is cause for concern regarding the variability of the material and concern regarding the appropriate extent of curing which is needed prior to surfacing the base material. These concerns should be addressed and/or confirmed in any future trials with hydrated fly ash.

REFERENCES

1. Nash, P. T., P. Jayawickrama, S. Senadheera, J. Borrelli, and A. Ashek Rana, 1995. *Guidelines for Using Hydrated Fly Ash as a Flexible Base,* Research Report 0-1365-1F, College of Engineering, Texas Tech University, Lubbock.

2. *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.