Continued Monitoring of Instrumented Pavement in Ohio

Final Report
December 2002

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16. Abstract

Performance and environmental data continued to be monitored throughout this study on the Ohio SHRP Test Road. Response testing included three new series of controlled vehicle tests and two sets of nondestructive tests. Cracking in two SPS-2 sections with lean concrete base confirmed observations elsewhere that PCC pavement may not perform well when placed on rigid base. Of the five types of base material used on LOG 33 and evaluated for their effect on AC pavement performance, deflection measurements on the asphalt treated base fluctuated most with changes in temperature. None of the other bases were sensitive to temperature. Cement treated base had the lowest deflection. On unbound material, bases containing large size stone gave the lowest deflection. The preponderance of data collected in the laboratory and at the ERI/LOR 2 site suggests that PCC pavement performs poorly on 307 NJ and CTFD bases. All sections with 25-foot slabs, except those with ATFD base, and the section with 13-foot slabs on 307 NJ base had significant transverse cracking. The 13-foot long slabs with 307 NJ base also had some longitudinal cracking. Considering the relatively short time these pavement sections had been in service, this level of performance was considered unacceptable. The ATFD base appeared to be performing best. On JAC/GAL 35, subgrade stiffness had a significant effect on dowel bar response. Looseness around dowel bars affected their ability to transfer load. Larger diameter and stiffer dowel bars provided better load transfer across PCC joints. The most effective dowel bar in these tests was the 1.5” diameter steel bar. The performance of 1” steel dowel bars were similar to 1.5” fiberglass bars. One-inch diameter fiberglass dowel bars were not recommended for PCC pavement. While undercutting PCC joint repairs initially reduced the forces in dowel bars, the effectiveness of the undercut diminished over time. Dowel bar forces were about the same in the Y and YU types of joint repairs after some time.

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Prepared in cooperation with the

OHIO DEPARTMENT OF TRANSPORTATION

and the

U. S. Department Transportation, FEDERAL HIGHWAY ADMINISTRATION

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1.0 INTRODUCTION

Beginning in 1992, Ohio University (OU), under contracts with the Ohio Department of Transportation (ODOT) and the Federal Highway Administration (FHWA), undertook several research projects to measure the response of various highway pavement structures over a range of environmental and loading conditions. Much of these response data were collected from transducers placed in the pavement during construction. Information gathered from these projects was to be used to refine and improve pavement design and construction procedures in Ohio.

Many of the embedded sensors exceeded their expected useful life and survived past the end of the projects, presenting an opportunity for additional follow-up monitoring. Also, final conclusions on performance were sometimes rather tentative due to a lack of early definitive distress patterns. In order to provide funds for continued performance monitoring activities on test pavements around Ohio, a follow-up project entitled "Continued Monitoring of Instrumented Pavement in Ohio" was initiated with OU on September 3, 1996. The purpose of this project was to build upon the earlier work through extended monitoring and testing of these test pavements, integration of the old and new data for validation and further implementation of the earlier findings.

The following summaries provide project titles, pavement locations and contract dates, along with a brief description of the research activities undertaken in the original studies. Chapter 6 was added to provide updated Falling Weight Deflectometer (FWD) measurements on a section of ATH 33 where various types of dowel bars were installed in a PCC pavement.

1.1 COORDINATION OF LOAD RESPONSE INSTRUMENTATION OF SHRP PAVEMENTS [DEL-23]

Project dates 6/13/94 to 10/13/98 (Final Report May, 1999, No. FHWA/OH-99/009)

The Ohio Department of Transportation constructed an experimental pavement for the Strategic Highway Research Program (SHRP) on U.S. 23 north of Columbus, OH, which included 40 asphalt and Portland cement concrete test sections in the SPS-1, 2, 8 and 9 experiments. These sections contained various combinations of structural parameters known to affect performance.

To enhance the value of this pavement, sensors were installed in 18 test sections to continuously monitor temperature, moisture and frost within the pavement structure, and 33 test sections were instrumented to monitor strain, deflection and pressure generated by environmental cycling and dynamic loading. Two weigh-in-motion systems and a weather station were installed to continuously gather the necessary traffic and climatic information required to properly interpret the performance data. Six universities, including Ohio University which coordinated this effort, were responsible for installing and monitoring the instrumentation. Nondestructive testing conducted with an FWD and Dynaflect, and five series of controlled vehicle tests were performed between 1995 and 1998 to assess the response of these test sections to dynamic loading.

During this project, two reports were published to describe the forensic investigation of one failed SPS-1 section on the DEL 23 test pavement and an analysis of subgrade variability noted during the construction of this pavement. Both studies were outside the scope of existing research contracts. These reports included:

1.1.1 Forensic Study of Section 390101 on Ohio SHRP U.S. 23 Test Pavement

A detailed examination was performed to determine the cause of rutting failures which occurred earlier than expected at a number of locations in this SHRP SPS-1 test section. Specific tests included surface distress surveys, FWD testing and dissection of the pavement structure. TDR probes indicated high moisture levels in the subgrade throughout the life of the pavement and FWD tests showed substantial variability in base and subgrade stiffness at the time of construction. Based upon these factors and internal deformations observed in the pavement structure during a forensic examination, the early failure of this section was attributed to high stresses in the base and subgrade layers. The life of this section predicted from actual measured data and AASHTO equations agreed well with observed performance.

1.1.2 Evaluation of Initial Subgrade Variability on the Ohio SHRP Test Road

During construction of the Ohio SHRP Test Road, subgrade stiffness was measured on forty test sections with an FWD. Areas of low subgrade stiffness appeared to correlate with premature pavement failures. Results suggest that improved pavement performance will result when subgrade uniformity is improved during construction.

1.1.3 Other Ohio SHRP Test Road Publications

Prior to this study, the following publications were printed for dissemination:

“Development of an Instrumentation Plan for the Ohio SPS Test Pavement (DEL 23),”

S. Sargand, et. al., 1994, Research Final Report

"Instrumentation Plan for SPS-2," A. Sharkins, Masters Thesis, November, 1996

"Pavement Response to Environmental Factors," J. VonHandorf, Masters Thesis, 1997

"Final Report on Forensic Study for Section 390101 of Ohio SHRP US 23 Test Pavement," S. Sargand, B. Young, Research Final Report, February, 1998

"Coordination of Load Response Instrumentation of SHRP Pavements – Ohio University," S. Sargand, et. al., May, 1999, Research Final Report

"Early SPS-1 Performance on the Ohio SHRP Test Road," ORITE Tech Note-2, 10/99

"Subgrade Variability on the Ohio SHRP Test Road," ORITE Tech Note-3, 10/99

1.2 A Demonstration Project on Instrumentation of a Flexible Pavement [LOG-33]

Project dates 3/9/92 to 10/9/97 (Final Report April, 1997)

An instrumentation plan and sensor installation techniques were developed for a full-scale asphalt concrete test pavement on U.S. 33 in Logan County. Six test sections were constructed over asphalt-treated base, cement-treated base, New Jersey base, Iowa base, and 304 base. Upon completion of the test sections, moisture, temperature, vertical deflections, pressures, and strains were monitored as dynamic loads were applied with a Falling Weight Deflectometer. The OU-PAVE finite element program predicted maximum deflection and the shape of the deflection basins with reasonable accuracy. Field data indicated that the deflection of asphalt pavement with asphalt-treated base varies significantly with changes in temperature. Asphalt pavement over cement-treated base had the lowest deflections. Among the non-treated bases, those with larger aggregate experienced the least deflection. Based upon the observations to date, New Jersey and cement treated bases are considered to be the best types of base material for flexible pavement.

1.3 Instrumentation of a Rigid Pavement System [ERI/LOR-2]

Project dates 3/9/92 to 11/9/97 (Final Report April, 1997)

This research focused on the development of a comprehensive field instrumentation program to measure the in-situ responses of a Portland cement concrete pavement system subjected to dynamic loading and changing environmental conditions. Measured responses included: slab strain and vertical slab deflection. Environmental conditions monitored included:

temperature gradients through the pavement slab, moisture in the base and subgrade, and pressure at the slab-base interface.

Moisture in the subgrade was found to increase up to 50% once cracks developed in the pavement slabs. Temperature gradients through the pavement slabs were not linear. During FWD testing, deflections were greatest at the joints, and significant stresses and deflections developed in all slab lengths tested. Lowest stresses were recorded in the 21 foot slabs. Strain sensors were able to detect stress relief due to cracking. Pressure at the slab-base interface and moisture level in the base and subgrade did not appear to be significant.

Three-dimensional finite element modeling was shown to be effective for calculating deflections and stresses that develop due to changes in environmental factors and loads applied during non-destructive testing.

1.4 EVALUATION OF PAVEMENT JOINT PERFORMANCE [JAC/GAL-35]

Project dates 1/1/90 to 1/30/94 (Final Report January, 1994)

The field performance of steel and fiberglass dowels for transferring load across rigid pavement repair sections was evaluated using strain gauges cemented to the dowel rods for the determination of shear forces, moments, torques and axial loads in the rods. Concrete in the repair sections was instrumented to measure internal strain. Dynamic loads were applied with an FWD, and single and tandem axle dump trucks traveling between 5 and 65 mph. Variables in the analysis included truck speed, dowel bar diameter and material, and Y or YU types of joint repair. Dominate forces in the dowel rods were bending moments and vertical shear. Field performance data were compared to analytical solutions using modified versions of ILLI-SLAB.

One-inch diameter fiberglass dowel bars were not recommended for rigid pavement, and there were not sufficient benefits to warrant the undercut (YU) type of joint repair. ILLI-SLAB did not accurately calculate the measured joint responses. Recommendations were made for dowels and joint repair in rigid pavement sections.

This report consists of a chapter for each of the four projects cited, including a summary of work performed previously and, where applicable, follow-up data and results from this current contract.

2.0 COORDINATION OF LOAD RESPONSE INSTRUMENTATION OF SHRP PAVEMENTS – DEL 23

2.1 INTRODUCTION

ODOT constructed forty test sections along a 3.5-mile length of US 23 in Delaware County for SPS-1, 2, 8 and 9 experiments in the Strategic Highway Research Program (SHRP). This test pavement was comprised of four new lanes of pavement constructed in the median of an existing four lane pavement. The SPS-1 and SPS-9 experiments were located in the southbound lanes of the new pavement. The SPS-2 experiment was placed in the northbound lanes of the new pavement, and the AC and PCC sections in the SPS-8 experiment were constructed on a ramp coming south from the village of Norton onto the original southbound lanes of U.S. 23. The new pavement carries mainline traffic, while the original lanes serve as a service road for local residents and as alternate mainline lanes when traffic needs to be diverted from the test pavement.

Figure 2.1 shows the project layout, and Tables 2.1 and 2.2 summarize the build-up of the test sections. Figure 2.1 contains information for the sections originally constructed, but includes the additional instrumentation added to Sections 803 and 804 during their reconstruction. Severe early rutting in these sections due to a poorly compacted subgrade layer approximately four feet below the pavement surface, excess moisture in the subgrade, and heavy loads applied during the initial series of controlled vehicle tests required they be totally replaced in the Fall of 1997. Environmental and dynamic sensors were added to Sections 803 and 804 at the time of the replacement, resulting in a total of 20 sections with environmental instrumentation, 17 AC sections with response instrumentation and 17 PCC sections with response instrumentation. Table 2.3 summarizes the number of test variables contained in each SPS experiment on the Ohio SHRP Test Pavement.

Research contracts were initiated with six universities in Ohio to install seasonal instrumentation in 18 sections and load response instrumentation in 33 sections. The contract with Ohio University also included responsibility for coordinating the efforts of the five other universities. The OU project, entitled “Coordination of Load Response Instrumentation of SHRP Pavements – Ohio University,” extended from June 13, 1994 to October 13, 1998. A final report documenting work during that period was published in May 1999.

The continued monitoring project documented in this report was initiated on September 3, 1996, in part, to provide funding for performance monitoring of the test sections on US 23. These efforts included: recording data from the weather station; collecting data from seasonal sensors in the pavement; summarizing skid, roughness and FWD data obtained by ODOT; repairing sensors which fail; and recording response measurements during controlled vehicle tests.

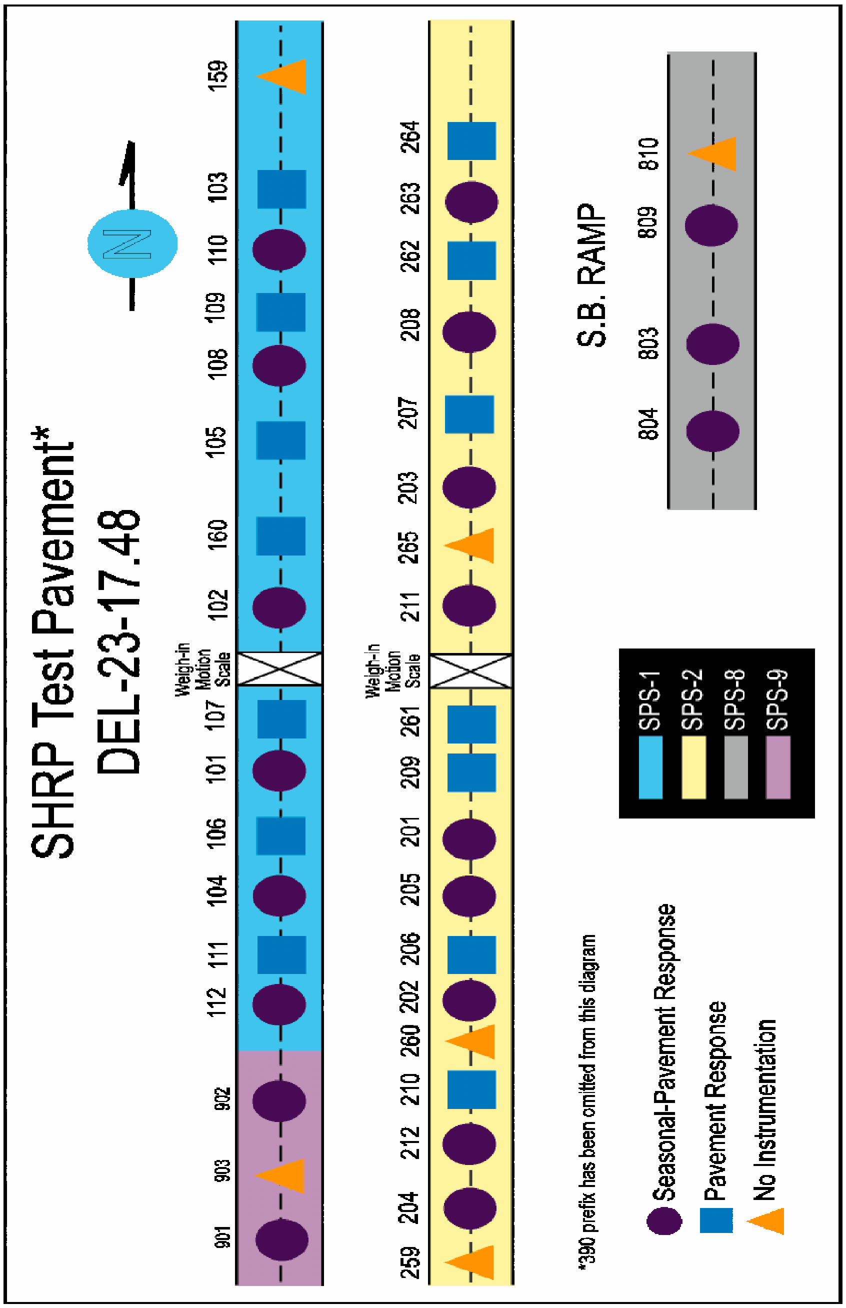


Figure 2.1 Layout of the Ohio SHRP Test Pavement

Table 2.1 Location and Design of the Asphalt Concrete Sections

SPS-1 (Southbound)				
Section	Station	AC Thickness (in.)	Base Type and Thickness	Drains
390101	355+00-350+00	7	8" DGAB	No
390102	375+00-370+00	4	12" DGAB	No
390103	420_75-415+75	4	8" ATB	No
390104	341+00-336+00	7	12" ATB	No
390105	392+50-387+50	4	4" ATB/4" DGAB	No
390106	348+00-343+00	7	8" ATB/4" DGAB	No
390107	363+00-358+00	4	4" PATB/4"DGAB	Yes
390108	399+75-394+75	7	4" ATB/8" DGAB	Yes
390109	406+50-401+50	7	4" PATB/12" DGAB	Yes
390110	413+50-408+50	7	4" ATB/4" PATB	Yes
390111	333+00-328+00	4	8" ATB/4" PATB	Yes
390112	325+00-320+00	4	12" ATB/4" PATB	Yes
390159	433+00-428+00	4	15" ATB/4" PCTB/6" DGAB	Yes
390160	382+00-377+00	4	11" ATB/4"DGAB	Yes
SPS-8 (Ramp)				
Section	Station	AC Thickness (in.)	Base Type and Thickness	Drain
390803	19+19-14+90	4	8" DGAB	No
390804	13+50-8+50	7	12" DGAB	No
SPS-9 (Southbound)				
Section	Station	AC Thickness (in.)	Base Type and Thickness	Drain
390901	282+5-277+75	4 (AC-20)	12" ATB/4" PATB/6" DGAB	Yes
390902	302+50-297+50	4 (PG58-28)	12" ATB/4" PATB/6" DGAB	Yes
390903	291+00-286+00	4 (PG64-28)	12" ATB/4" PATB/6" DGAB	Yes

Table 2.2 Location and Design of the Portland Cement Concrete Sections

SPS-2 (Northbound)						
Section	Station	PCC Layer		Lane Width (ft.)	Base Type and Thickness	Drain
		Strength (psi)	Thickness (in.)			
390201	343+00-348+00	ODOT	8	12	6" DGAB	No
390202	319+00-324+00	900	8	14	6" DGAB	No
390203	384+00-389+00	ODOT	11	14	6" DGAB	No
390204	275+50-280+50	900	11	12	6" DGAB	No
390205	335+75-340+75	ODOT	8	12	6" LCB	No
390206	327+50-332+50	900	8	14	6" LCB	No
390207	391+25-396+25	ODOT	11	14	6" LCB	No
390208	397+75-402+75	900	11	12	6" LCB	No
390209	350+25-355+25	ODOT	8	12	4" PATB/4" DGAB	Yes
390210	303+50-308+50	900	8	14	4" PATB/4" DGAB	Yes
390211	369+00-374+00	ODOT	11	14	4" PATB/4" DGAB	Yes
390212	294+00-299+00	900	11	12	4" PATB/4" DGAB	Yes
390259	265+50-270+50	900	11	12	6" DGAB	Yes
390260	311+50-316+50	ODOT	11	12	4" PATB/4" DGAB	Yes
390261	357+75-362+75	ODOT	11	14	4" PCTB/4" DGAB	Yes
390262	405+25-410+25	ODOT	11	12	4" PCTB/4" DGAB	Yes
390263	414+50-419+50	ODOT	11	14	6" DGAB	Yes
390264	422+50-427+50	ODOT	11	12	6" DGAB	Yes
390265	376+10-381+10	ODOT	11	12	4" PATB/4" DGAB	Yes

SPS-8 (Ramp)						
Section	Station	PCC Layer		Lane Width (ft.)	Base Type and Thickness	Drain
		Strength (psi)	Thickness (in.)			
390809	25+90-20+90	550	8	11	6" DGAB	No
390810	32+50-27+50	550	11	11	6" DGAB	No

Table 2.3 Distribution of Structural Parameters

SHRP Experiment	No. Sections	Number of Test Variables					
		Pavement Thickness	Pavement Mixes	Base Thickness	Base Materials	Lane Widths	Drains
SPS-1	14	2	1	5	4	1	2
SPS-2	19	2	2	2	4	2	2
SPS-8 (AC)	2	2	1	2	1	1	1
SPS-8 (PCC)	2	2	1	1	1	1	1
SPS-9	3	1	3	1	1	1	1

2.2 Seasonal Instrumentation

The seasonal program conducted between 1994 and 1997 on the DEL-23 project involved monitoring of the on-site weather station as well as installation and monitoring of 18 seasonal sections for the Seasonal Monitoring Program (SMP). Temperature, moisture, and frost depth were monitored to a depth of six feet in these sections.

2.2.1 Temperature

It is important that temperature be monitored in subgrade and base layers to determine if they are frozen. Temperature plays a major role in the deflection and fatigue life of asphalt concrete pavements, as it directly affects resilient modulus and ultimate tensile strength. For Portland cement concrete pavements, the variation of temperature throughout the slab depth creates distortion or curling of the slabs which impacts support from the underlying layers and magnifies load related stresses during certain times of the day. In addition, temperature changes also result in the expansion and contraction of PCC slabs which affect cracking in long slabs and joint performance.

Temperature variations on the test road were monitored with thermistors, or temperature-sensitive resistors. Slight changes in temperature create major variations in the resistance of the thermistors. To find this resistance, a known voltage is applied to the thermistor and the output voltage is read between the thermistors leads. By knowing the change in resistance, temperature can be calculated with a correlation equation. A thermistor probe was chosen to obtain pavement and soil temperatures. This device consists of individual, but interconnected probes for both pavement and soil temperature measurements. A metal rod containing up to four thermistors was used for the pavement layer followed by a six foot long, clear PVC pipe housing 15 thermistors for temperature measurements below the pavement.

2.2.2 Volumetric Moisture Content

The moisture content of a soil is required for many important design considerations such as settlement, resilient modulus, and freeze-thaw capacity. Based upon results obtained from other test pavements, time-domain reflectometry probes (TDR) were chosen as the best instruments available to monitor volumetric water content. Installed every six to twelve inches down to a depth of six feet, TDR probes consist of a coaxial cable leading to a three-pronged probe installed in the subgrade. When an electromagnetic wave is carried to the probe, the time for the pulse to travel from one end of the probe to the other is recorded. The pulse is displayed graphically by a cable tester where an initial inflection point represents the wave entering the probe, and a second inflection point is produced when the signal reflects at the end of the probe. The time of travel between these two points is a function of the dielectric constant of the soil. The dielectric constant of the soil is, in turn, calibrated with the volumetric moisture content.

2.2.3 Frost Depth

Since the DEL-23 project is located in a geographic area that experiences multiple freeze/thaw cycles during the winter season, it was necessary to measure the depth of frost penetration in the subgrade soil as well as the number of freeze/thaw cycles during the winter. This depth is important in determining the thickness of base layers that will limit or prevent frost heave in the soil and pavement. Also, since soil stiffness tends to decrease after each freeze/thaw cycle, mechanistic design procedures will require this information to provide a more durable pavement cross section.

After studying the methods available for monitoring frost depth, the FHWA considered electrical resistance and resistivity methods to be the most reliable for SHRP. A probe developed by the U.S. Army Corps of Engineers' CRREL was chosen for the program. This probe consists of a 73-inch long solid PVC pipe upon which 36 metal wire electrodes are mounted and spaced every two inches (Rada et al, SMP 1994, II-8). When a function generator creates an AC current in two outer electrodes, voltage drop and resistance are measured and compared across the two inside electrodes. Bulk, or apparent, resistivity can then be computed by the product of the resistance times the geometric factor for the electrode array. Since ice has a much greater electrical resistivity than water, areas of high resistivity will correspond to frozen layers in the subgrade soil.

Temperatures were recorded hourly using a datalogger and the necessary electrical components required for automatic data storage on site. Because moisture content and frost levels were not expected to vary much throughout the day, these readings were recorded monthly with mobile monitoring equipment. When using mobile equipment, the user connected all necessary cables to monitor and download the data to a personal computer. This equipment consisted of a Cable Tester - datalogger/controller; and two Multiplexers plus an interface board for resistivity measurements. Lead cables from the soil and base moisture sensors were connected to the multiplexers, and the corresponding traces were displayed on the cable tester screen. The datalogger communicated with the cable tester and multiplexers to monitor and record data. Data were then downloaded to the microcomputer from the mobile unit using specialized software.

2.3 GROUND WATER TABLE

Fourteen and one-half foot long, slotted observation piezometers were used to measure the depth to the water table along the outside pavement shoulder. Made of two individual 1-inch diameter PVC pipes coupled together, the piezometers were threaded to a metal floor flange and anchored at the bottom of a bore-hole. When necessary, this pipe also served as a swell-free benchmark for surface level measurements. A total of nine piezometers were installed at the locations and elevations shown in Table 2.4, and water table measurements from these piezometers are shown in Figure 2.2.

Table 2.4 Piezometer Locations

Southbound Lane			Northbound Lane		
Section	Station	Pavement Elevation*	Section	Station	Pavement Elevation*
390103	417+02	955.4	390204	279+85	955.6
390108	397+00	953.4	390212	298+01	957.2
390102	372+00	953.7	390201	346+00	954.9
390104	337+00	956.0	390208	401+00	954.4
390901	279+50	955.2			

* Pavement elevation nearest piezometer well head, ft. above sea level

2.4 WEATHER STATION

To assist in monitoring climatic changes along the test road, a weather station was installed near the north end and on the east side of the test road. This station had the capacity to monitor solar radiation, air temperature, wind speed, wind direction, relative humidity, and rainfall amount. Air temperature and relative humidity were monitored with one probe containing a thermistor and a capacitive relative humidity sensor. The cable was connected to a datalogger, which monitored and stored all weather-related measurements.

A pulse-type tipping bucket rain gauge installed a few feet away from the weather station to monitor rainfall. The bucket was equipped with a heating device to melt accumulated snowfall. A propeller type gauge was used to measure wind speed and direction. As the propeller rotated, sine wave signals were produced with a frequency proportional to wind speed. Wind direction was determined by the azimuth angle of the vane. As the vane rotated, a potentiometer produced an output voltage proportional to the angle. A pyranometer was used to monitor incoming solar radiation in terms of energy per surface area. This conversion was performed with a silicon photovoltaic detector that produced an output current based on levels of radiation. A resistor in the cable then converted this current to a voltage recorded by the datalogger.

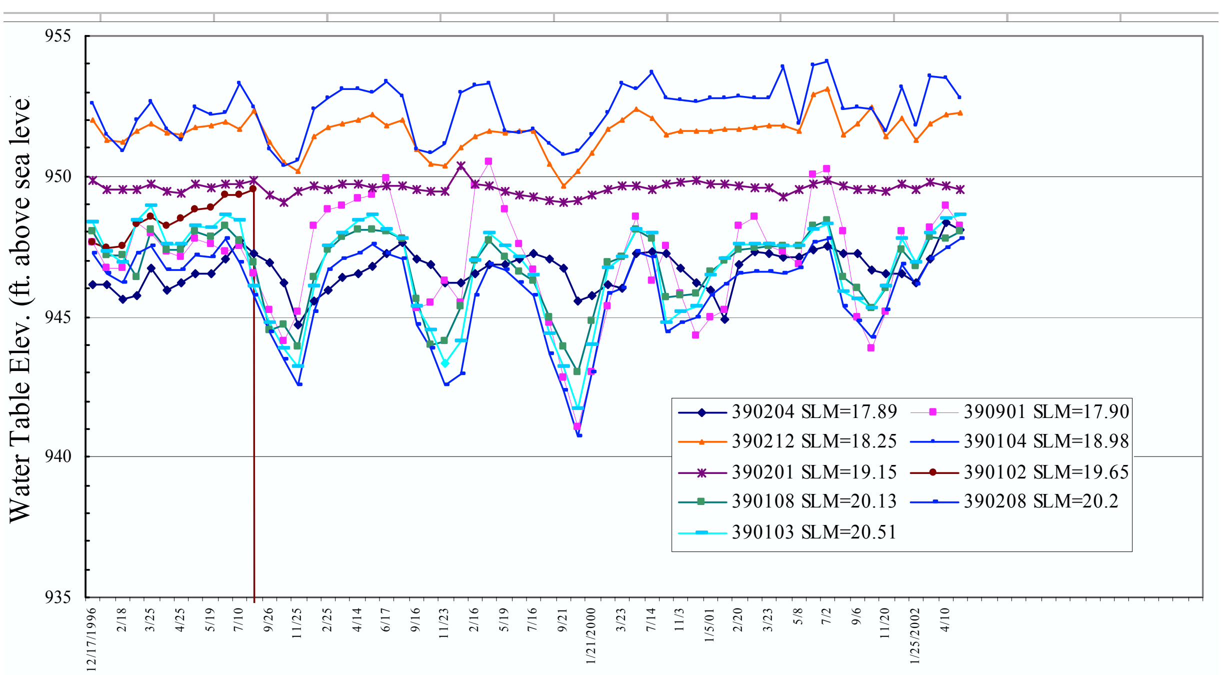


Figure 2.2 Water Table – Ohio Test Road

2.5 SEASONAL AND WEATHER STATION ANALYSIS

Once data was downloaded from the onsite dataloggers at both the seasonal sites and the weather station, it was checked and edited for quality in accordance with SHRP protocol. If the data met quality assurance checks, it was sent to the FHWA coordinator. The programs SMPCheck (Seasonal Monitoring Program Check), and AWSCheck (Automated Weather Station Check) were developed by SHRP to analyze and edit data in a consistent format.

2.5.1 SMPCheck Program

This program checks the data files for accuracy within prescribed ranges, adjusts for overlap and time corrections, and then prepares six graphs that include the following:

1) daily average, min, max air temperature and rainfall data,
2) daily average air, rainfall, and first 5 MRC sensors temperature data,
3) daily all 18 MRC sensors average temperatures,
4) daily all 18 MRC sensors maximum temperatures,
5) daily all 18 MRC sensors minimum temperatures, and
6) hourly air temperature, rainfall, and first 5 MRC sensors temperature data.

Figure 2.3 shows an example of Option 4 above which illustrates daily average temperatures for all eighteen MRC thermistors. Although the sensor numbers are not visible in the graph, it can be seen that temperatures near the pavement surface fluctuated intensely while temperatures in the subgrade undergo little change. The mobile data analysis was conducted in the same manner. Once selected, plots of the TDR traces and resistivity values were displayed as shown in Figure 2.4. For mobile data, however, it was only necessary to choose those plots which were valid by typing the corresponding number under the graph.

2.5.2 AWSCheck Program

The AWSCheck Program used to monitor weather station data followed the same format and procedure as previously described for the SMPCheck Program. Again, data were displayed graphically whereupon the user removed corrupt data points, and an upload file was created providing the data passes the Level D check. The program is capable of displaying the following options for viewing:

1) daily average, min, max air temperature and precipitation data,
2) daily relative humidity, solar radiation, and precipitation data,
3) daily wind information,

4)	hourly temperature and precipitation data,

5)	hourly relative humidity and precipitation data,

6)	hourly solar radiation and precipitation data, and

7)	hourly wind information.

Figures 2.5 to 2.7 provide samples of Options 2, 3, and 6 respectively. As with the Onsite data, the weather station data must be checked for quality and consistency. Options 1 through 3 above only permit viewing of the data, while Options 4 through 7 permit the user to edit the data manually.

ODOT is currently making arrangements for the environmental data to be available through its web site.

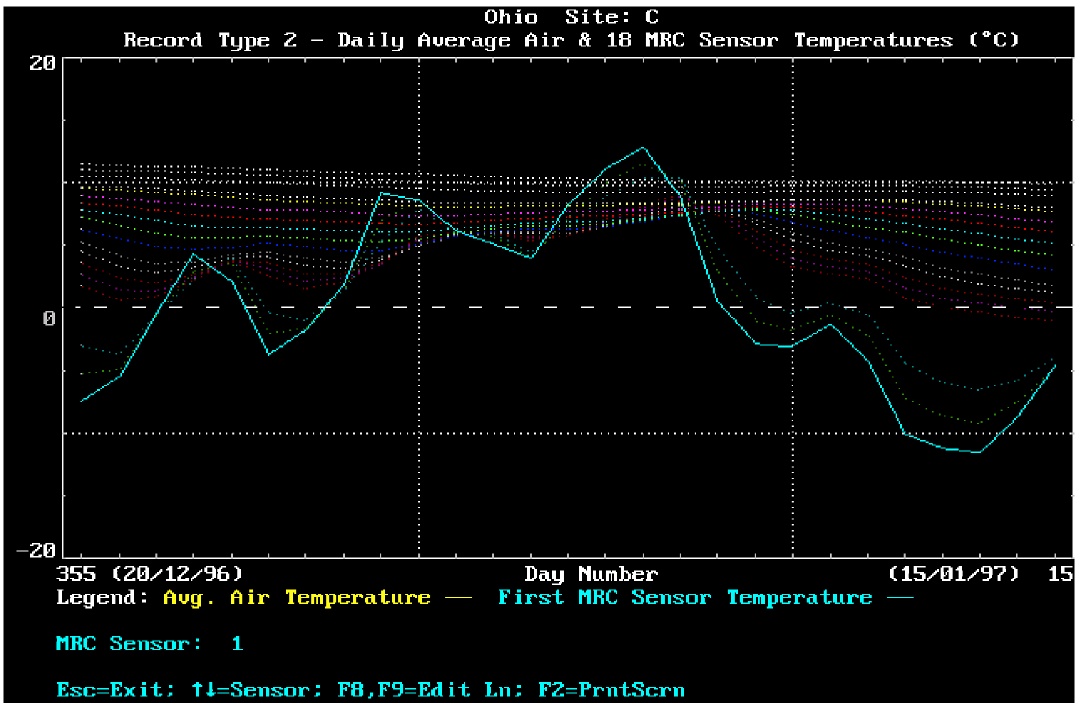


Figure 2.3 Typical SMPCheck Display of Daily Average Thermistor Temperatures

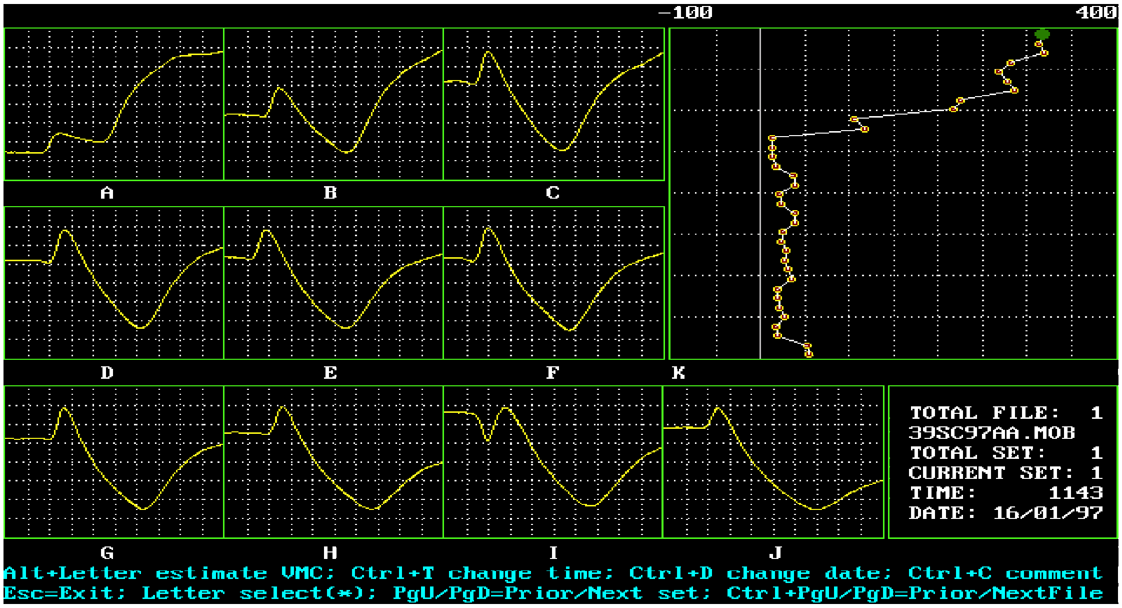


Figure 2.4 Typical SMPCheck Display of TDR and Resistivity Data

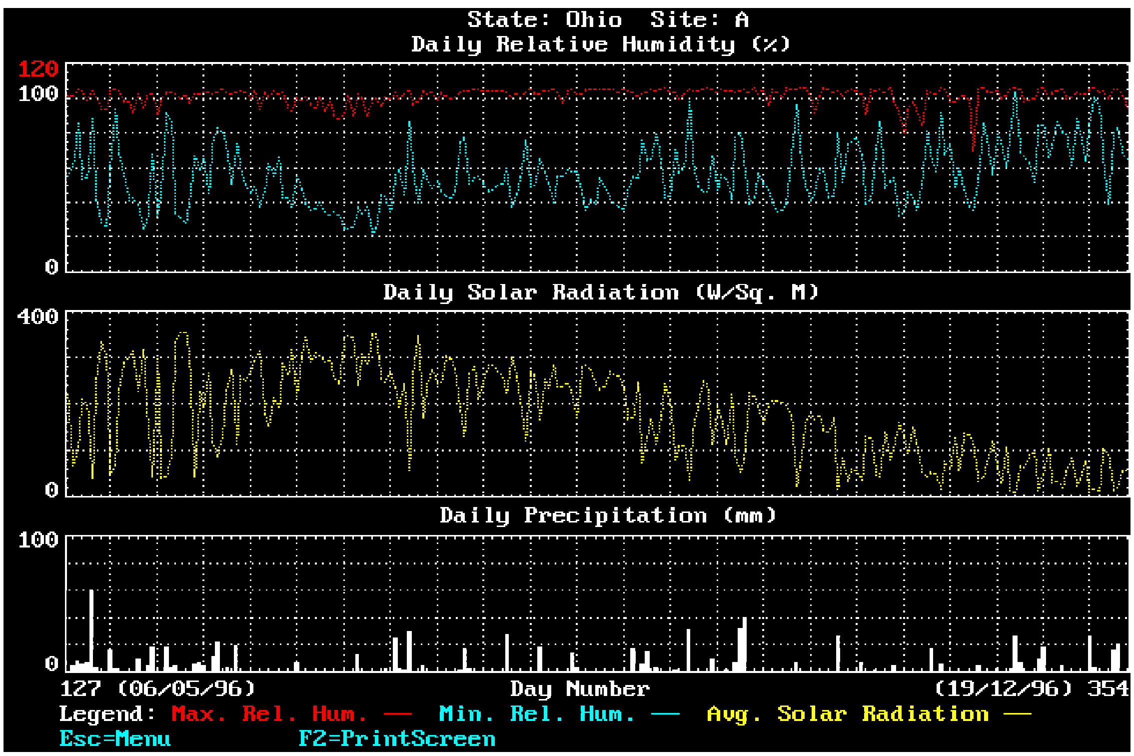


Figure 2.5 Typical AWSCheck Display of Daily Humidity, Radiation, and Precipitation Data

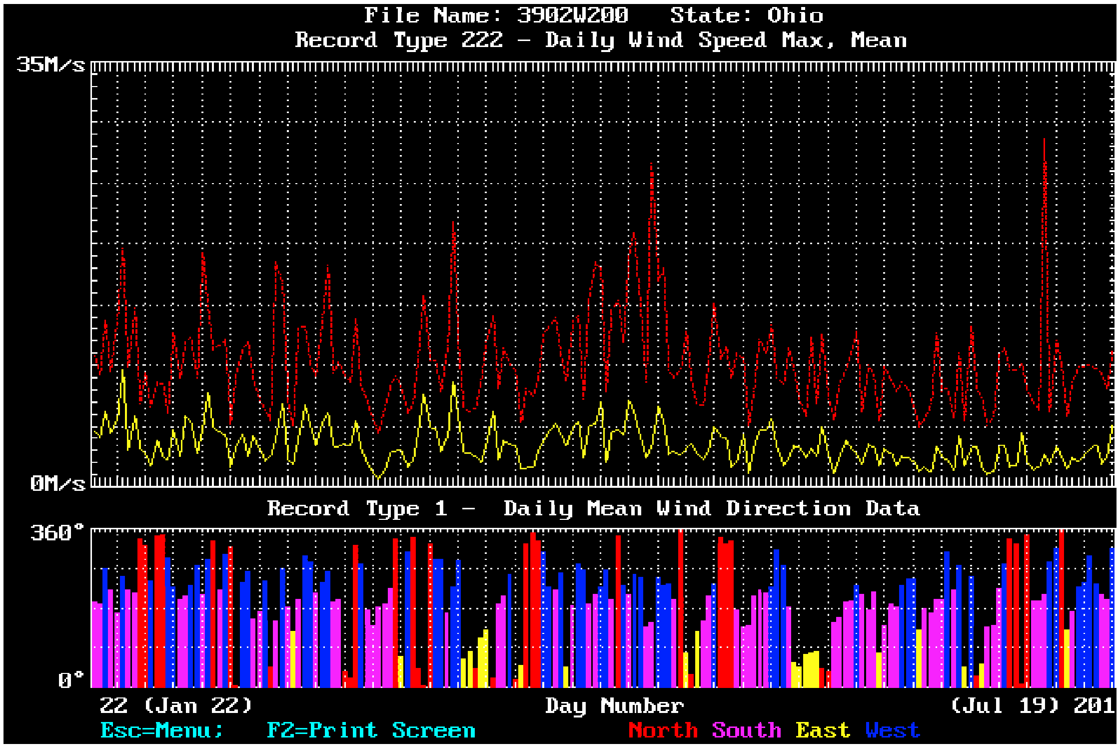


Figure 2.6 Typical AWSCheck Display of Daily Wind Information

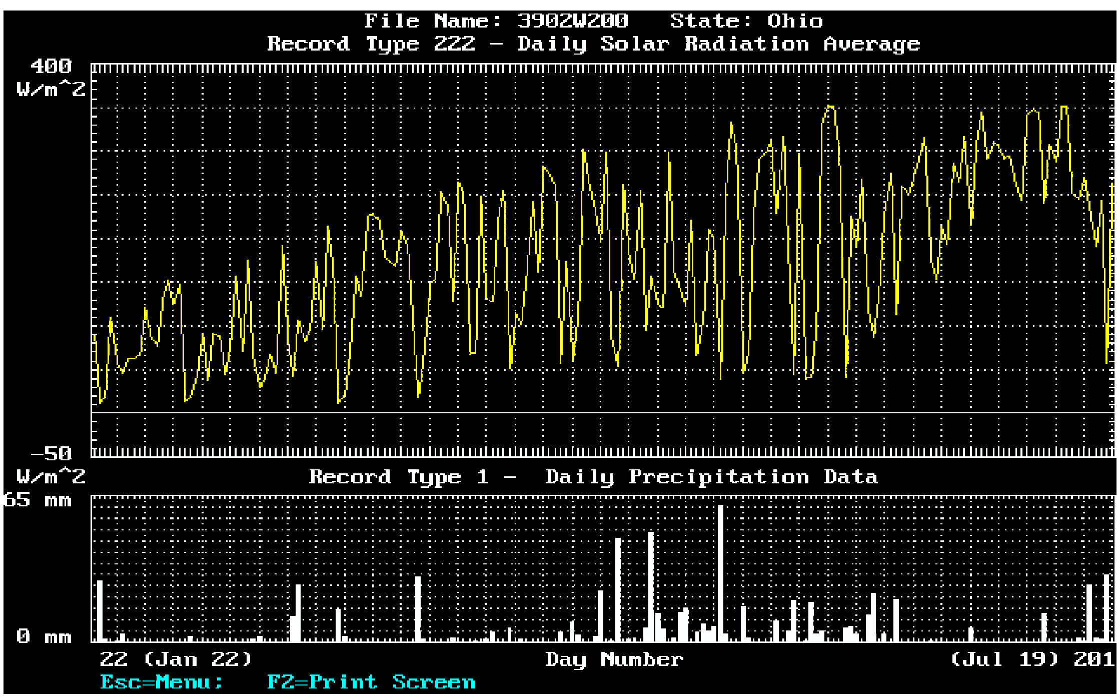


Figure 2.7 Typical AWSCheck Display of Hourly Radiation and Precipitation Data

2.6 CONTROLLED VEHICLE TESTING

In recent years, much attention has been given to developing a mechanistic empirical design procedure for highway pavements. In procedures of this type, the build-up of a pavement structure is based upon the mechanical properties of materials used in the structure, environmental conditions typical of the location, and anticipated traffic loading. To develop, calibrate and verify mathematical models for mechanistic design, multiple pavement response parameters such as strain, deflection and pressure are essential. Traditionally, one parameter such as deflection has been used in the development of design procedures for pavement systems. Unfortunately, one-parameter systems cannot adequately describe response over a wide range of conditions. Climatic influence is essential in modeling the performance of pavement systems and, therefore, parameters such as temperature, moisture and frost depth must be integrated into the design procedure.

Since completion of the Ohio SHRP Test Road in 1996, eight series of controlled vehicle tests have been run to monitor dynamic response under known loading and environmental conditions. Because of the numerous parameters known to affect response, the size of a matrix required to examine all load associated response parameters in a series of controlled vehicle tests becomes unwieldy. For this reason, SHRP reduced the testing to a few of the more significant vehicle related parameters on a limited number of sections.

SHRP targeted four core sections in each of the SPS-1 and SPS-2 experiments for the installation of sensors to monitor dynamic pavement response during controlled vehicle testing. These sections included J2 (390102), J4 (390104), J8 (390108), and J10 (390110) in SPS-1; and J1 (390201), J5 (390205), J8 (390208), and J12 (390212) in SPS-2. Tests were to be performed with single and tandem-axle dump trucks. The rear axle on the single-axle truck was to be loaded to approximately 18 and 22 kips, while total loads on the rear axles of the tandem-axle dump truck were to be approximately 32 and 42 kips. Both trucks were to run over the instrumented sections at 50 (30), 65 (40), and 80(50) km/hr (mph) in the morning and afternoon to obtain information on the effects of temperature. With a minimum of three repetitions being required for each cell in the matrix, a total of 72 runs were necessary to complete a single series of SHRP tests with the two trucks.

SHRP requested states to perform these tests in the spring and summer when moisture conditions in the base and subgrade, and temperature in the pavement layer are typically quite different. The ODOT goal was to follow the SHRP testing protocol on all core sections and to include as many of the other 25 instrumented sections as possible at the time these tests were being run. ODOT also conducted additional tests with a research tank truck operated by the Canadian National Research Council to gather supplementary information on the effects of tridem axles, axle spacing, and dual versus super single tires. Runs at 8 (5) km/hr (mph) were also added to obtain low speed data.

Table 2.5 summarizes the basic parameters included in each of the eight series of controlled vehicle tests and the following text briefly describes each series. Pavement temperature, soil moisture and lateral position are inherent variables within each series of tests. Wheel geometry and weights for the ODOT test trucks are shown in Tables A-1 to A-3 for all test series. In the first five series of tests, tires were checked visually and air was added to under-inflated tires, but individual pressures were not measured and recorded during the tests. In the 1999 and 2001 tests, tire pressures were recorded, as shown in Table A-4.

Table 2.5 Summary of Controlled Vehicle Tests

Test Date	Test Series	Load Device	No. Passes	No. of Sections Monitored		Dynamic Parameters					
				AC	PCC	Load	Speed	No. Axles	Axle Spacing	Tire	Vehicle Dynamics
12/95 - 3/96	I*	CNRC	144	1	1	X	X	X	X	X	
8/96	II	Single	85	6	5	X	X				
		Tandem	87								
6/97	III	CNRC	127	7	8	X	X	X	X	X	X*
		Tandem	122	7	8	X	X				
7/97 - 8/97	IV	Single	77	12	14	X	X				
		Tandem	77								
10/98	V	Single	72	8	9	X	X				
		Tandem	60								
9/99 10/99	VI	Single	86	8	8	X	X				
		Tandem	86								
10/99	VII	Single	30-60/sect	7	7	X	X				
		Tandem	30-60/sect.								
		FWD	50 drops/sect	7	7						
		Dynaflect	20read/sect	7	7						
4/01 - 5/01	VIII	Single	80	10	12	X	X				
		Tandem	80								

* Funded by FHWA

2.6.1 Series I Testing - CNRC (12/95 and 3/96)

Toward the end of 1995, FHWA requested permission to conduct a series of controlled vehicle tests on Sections 390803 (AC) and 390809 (PCC) constructed and instrumented the previous year for SPS-8. They were in the process of preparing a document on size and weight regulations for commercial trucks in which axle configuration and types of tires were to be included. Dynamic response data obtained from these sections would provide valuable input as to how these parameters affect pavement performance. ODOT agreed and a special research truck was brought down from the Canadian National Research Council (CNRC) to perform the tests. This truck can be configured with tandem or tridem axles on the trailer, axle spacing can be adjusted, and either dual or super single tires can be mounted on the trailer axles. Specified axle weights are achieved by filling selected tanks in the trailer with water and by adjusting lead weights on the rear of the trailer.

For this series of tests, tandem axles were typically spaced 48 inches on centers, with a few tests being run at 96 and 114-inch spacings. A tridem axle configuration was achieved by lowering the lift axle and spacing it 54 inches in front of the 48-inch spaced tandem axles. Standard dual tires were used with the tandem configuration, and both standard dual and super single tires were used with the tridem configuration. Tire pressure was set at 100 psi for all tests.

2.6.2 Series II Testing - ODOT Single and Tandem Axle-Dump Trucks (8/96)

ODOT conducted a series of basic SHRP controlled vehicle tests on the SPS-1 and SPS-2 core sections using ODOT single and tandem-axle dump trucks just prior to the test pavement being opened to traffic. Because of the anticipated early distress in Sections J5 (390105) and J7 (390107), however, they were added to this test series so data could be obtained before those gauges became inoperative and the sections failed. Test sections in the SPS-1 and SPS-9 experiments were opened to main-line traffic on August 14, 1996. The SPS-2 sections were opened one day later. Approximately three weeks after being opened to traffic, Sections J1 (390101), J2 (390102), and J7 (390107) in the SPS-1 (asphalt concrete) experiment began to exhibit measurable wheel path rutting.

2.6.3 Series III Testing - CNRC and ODOT Tandem-Axle Dump Truck (6/97)

Because of the high quality of pavement response data obtained on the two SPS-8 sections during Series I testing with the CNRC truck in 1996, and because 31 additional instrumented test sections were available on the mainline pavement, ODOT contracted with the

Canadian National Research Council to bring their research tank truck back to Ohio for an expanded series of tests in June 1997. One month of testing was believed to be adequate to complete a comprehensive matrix of truck parameters, including number of axles, axle spacing, load, speed, tire configuration, and lateral position on the pavement. FHWA also funded the monitoring of vehicle dynamics on the CNRC truck for a few runs during this series of tests. Unfortunately, this was an extremely wet time in Ohio and testing could not be performed while it was raining because of potential damage to the data acquisition systems. Even most of the weekends were wet.

It soon became apparent that the planned testing sequence would have to be modified to accommodate the weather and still obtain the maximum benefit from the CNRC truck within the allotted time. The first step taken was to select the optimum number of sections in SPS-1 and SPS-2 that could be monitored simultaneously with the nine data acquisitions available. There was not going to be sufficient time to conduct one complete series of tests on SPS-1 and another on SPS-2 as originally planned. By monitoring sections as the truck traveled northbound and southbound, seven and eight of the highest priority sections in SPS-1 and SPS-2 could be monitored within a few minutes of each other.

Because the ODOT tandem-axle dump truck would be involved in routine controlled vehicle testing as long as the pavement sensors remained functional, it was also run in the Series III tests to serve as a control vehicle for comparison with the CNRC truck. Axle load and speed on the ODOT truck were adjusted to simulate conditions for the CNRC truck as closely as possible. With this arrangement, the CNRC truck would make a pass on the SPS-1 sections and pass over the SPS-2 sections as it returned. The ODOT truck would follow behind in such a way as to be traveling in the opposite direction of the CNRC truck. Pavement response was monitored on both sides of the highway. The time differential between comparable runs for the two vehicles was typically less than 10 minutes.

For the CNRC truck, it was most efficient to perform all tests with the same arrangement of lead weights on the back of the trailer. Consequentially, three of the four boxes of weights were evenly distributed across the back of the truck throughout the Series III tests. Tests were grouped to minimize the movement of axles and the changing of tires. Tanks of water were filled at the District 6 garage so the heaviest load would be run first. One or two tanks were then emptied into a catch basin at the site in preparation for the next lightest axle load. This procedure minimized the necessity of having to return to the district garage to fill tanks. Similarly, the ODOT tandem-axle dump truck was loaded heavy in the morning at a nearby maintenance garage and unloaded as necessary by returning to this garage. While not as efficient as dumping material at the site, this process reduced the potential problem of having to find an equipment operator at the garage to load the truck during the day when most everyone was out. Unloading typically takes less time than loading. Also, the trucks were gassed up either in the morning or at the end of the day to reduce down time. Wheel loads on the trucks were weighed with portable PAT scales in the test lane where any effects of pavement slope would be taken into account.

2.6.4 Series IV - ODOT Single and Tandem-Axle Dump Trucks (7/8/97)

The fourth series of truck tests was performed mainly to fulfill SHRP requirements. However, it was also an excellent opportunity to monitor a number of other pavement sections along with the core sections. To complete these tests, 12 sections in the SPS-1 experiment were monitored first. Single and tandem-axle dump trucks were loaded with the light load, and all speeds and repetitions were run in the morning and afternoon of July 2, 1997. The load was increased on July 3 and the same test sequence was performed. A similar procedure was followed for 14 sections in SPS-2 later in July and early August.

2.6.5 Series V - ODOT Single and Tandem-Axle Dump Trucks (10/98)

The Series V controlled vehicle tests were also performed for SHRP. All core sections, with the exception of Section 390102 which was removed and replaced earlier as Section 390161, were included along with a few additional sections to obtain supplementary data. By the time these tests were run, there had been a significant drop in the number of sensors that were still operable. In the thinner SPS-1 sections, very few strain gauges were functional, except for Section 390162 (replacement for 390107), which was constructed in the fall of 1997. Overall, the pressure cells appeared to be performing satisfactorily and 90% of the LVDTs, which had been removed after the last series of truck tests and remounted for these tests, provided acceptable data. As noted in the earlier tests, a higher percentage of sensors were operational in the thicker pavement sections. In the PCC sections (SPS-2), the number of operable pressure cells and LVDTs was comparable to that in the thicker AC sections. None of the rosettes, about half of the Dynatest gauges, and approximately 90% of the KMB-100 gauges were operational.

The full SHRP matrix of load parameters was completed on nine SPS-2 sections. Because of time constraints and mechanical problems, only a few runs were completed with the tandem-axle truck on the eight AC sections being monitored.

2.6.6 Series VI - ODOT Single and Tandem-Axle Dump Trucks (9/99 and 10/99)

The Series VI controlled vehicle tests were also performed for SHRP. All core sections, with the exception of Section 390102, which was removed and replaced earlier, were included along with a few additional sections to obtain supplementary data. By the time these tests were conducted, four years had passed since the installation of strain gauges in the pavement. The life expectancy of most gauges was one to two years, so there had been a significant drop in the number of sensors that were still operable. In the thinner SPS-1 sections, very few strain gauges were functional. Overall, the pressure cells appeared to be performing satisfactorily and 80% of the LVDTs, which had been removed after the last series of truck tests and remounted for these tests, provided valid data. In the PCC sections (SPS-2), none of the rosettes, about 40% of Dynatest gauges, and approximately 70% of the KMB-100 gauges were operational.

In addition to the sensors still functioning, surface mounted strain gauges were installed on all core SPS-1 and SPS-2 sections being monitored in accordance with SHRP/LTPP guidelines. Sensors installed in the non-core sections were mounted at critical locations in the wheel path in order to measure maximum stress as trucks passed over the sections. Additional gauges were mounted transversely in Sections 390206, 390205, and 390208. The full SHRP matrix of load parameters was completed on eight SPS-2 sections.

2.6.7 Series VII - ODOT Single and Tandem-Axle Dump Trucks, FWD and Dynaflect (10/99)

The Series VII tests were special investigative efforts performed for ODOT. Six SPS-1, one SPS-9, and seven SPS-2 sections were chosen for these tests. FWD and Dynaflect loads were applied over embedded and surface gauges (see series VI), and this was followed by single and tandem-axle dump truck runs at 8 (5), 50 (30), 65 (40), and 80(50) km/hr (mph). Lateral tire offsets and strain gauge responses were recorded for all dynamic loading conditions.

2.6.8 Series VIII – ODOT Single and Tandem-Axle Dump Trucks

This series of tests were run mainly for SHRP. Surface gauges were mounted in 10 AC sections and 12 PCC sections prior to testing. Runs include those specified by SHRP with the addition of creep speed.

2.6.9 Summary

Techniques used to install response sensors in the SPS-1, SPS-2, SPS-8, and SPS-9 pavement sections were quite successful with over 95% surviving construction, over 90% of those still functional after one year, and a significant number of surviving in the thicker pavement sections after two years. Strain gauges failed quickest in the thinner SPS-1 (AC) sections with no drainage. Repeated heavy loads applied by mainline traffic on these sections over-stressed the transducers and caused visible distress in the pavement after a rather short period of time. Table 2.6 summarizes the instrumented AC sections and Table 2.7 summarizes the instrumented PCC sections monitored in each of the nine series of controlled vehicle tests conducted on the Ohio SHRP Test Road.

Table 2.6 Instrumented Pavement Sections Monitored During the Controlled Vehicle Tests – AC Sections

Asphalt Concrete (AC)								
Section No.	Test Series/Date							
	I 12/95-3/96	II 8/96	III 6/97	IV 7/97	V 10/98	VI 9/99-10/99	VII 10/99	VIII 4/01-5/01
SPS-1								
390101			X	**	**	**	**	**
390102*		X		**	**	**	**	**
390103				X				
390104*		X	X	X	X	X	X	X
390105		X	X	X	**	**	**	**
390106			X	X	X	X	X	X
390107		X		**	**	**	**	**
390108*		X	X	X	X	X	X	X
390109				X	X	X	X	X
390110*		X		X	X	X	X	X
390111			X	X		X	X	X
390112			X	X		X		X
390160				X				X
390162***					X			
390165***					X			
SPS-8								
390803	X							
SPS-9								
390901				X				X
390902				X	X	X	X	X

* SHRP Core Section ** Section failed/removed from service *** Replacement section

Table 2.7 Instrumented Pavement Sections Monitored During the Controlled Vehicle Tests – PCC Sections

Portland Cement Concrete (PCC)								
Section No.	Test Series/Date							
	I 12/95-3/96	II 8/96	III 6/97	IV 7/97	V 10/98	VI 9/99-10/99	VII 10/99	VIII 4/01-5/01
SPS-2								
390201*		X	X	X	X	X		X
390202			X	X		X		X
390203				X			X	
390204				X	X		X	X
390205*		X	X	X	X	X		X
390206			X	X		X		X
390207				X			X	
390208*		X		X	X	X		X
390209		X	X		X			
390210			X	X	X	X		X
390211				X			X	
390212*		X	X	X	X	X		X
390261			X		X		X	
390262				X	X	X		X
390263				X			X	X
390264				X			X	X
SPS-8								
390809	X							

* SHRP Core Section

2.7 SPS PERFORMANCE

In general, SPS-1 and SPS-2 test sections on the Ohio SHRP Test Road were placed so those expected to fail early were located toward the middle of the project, except where construction sequencing required sections containing some unique design feature or material be put in the same area. Sections with the longest life expectancy were located at the ends of the project where traffic control at the intersection of the old and new lanes would be more difficult during rehabilitation or replacement.

2.7.1 Projected Performance of SPS Sections

Projected services lives of SPS-1 and SPS-2 sections included in the original SHRP matrix are shown in Table 2.8 in terms of ESALs. These preliminary estimates were derived from AASHTO equations prior to construction using assumed structural properties for materials being incorporated into the pavement sections. By coincidence, the ESAL count on U.S. 23 was estimated to be about one million annually, thereby making the ESALs to failure count (in millions) shown in Table 2.8 to also be the approximate number of years of expected service, assuming no unusual environmental conditions or material degradation. Sections are listed in order of service life. Service lives projected in the table were subject to considerable error due to the design assumptions involved and the actual values are not as significant as the relative order of predicted failure. Obviously, the extremely long lives predicted for some of the stiffer sections are unrealistic. Actual material properties, in-situ stiffness and environmental data obtained after construction brought the calculated service lives of the failed sections much closer to observed performance. State sections added by ODOT to the SPS-1 and SPS-2 experiments were designed to provide performance information for standard ODOT designs. It should be noted that the first four sections to fail in the SPS-1 experiment were the four sections listed below with the shortest projected service lives

Table 2.8 Projected Design Lives of SHRP Test Sections

SPS-1		**SPS-2**	
Section No.	ESAL (million)	Section No.	ESAL (million)
390107	0.2	390201	0.9
390102	0.9	390205	1.1
390105	1.6	390209	3.2
390101	2.4	390202	6.7
390108	6.4	390206	7.8
390103	7.2	390203	10.7
390110	10.0	390207	12.2
390109	15.5	390210	23.2
390111	17.2	390204	32.7
390106	75.2	390208	36.5
390112	118.1	390211	36.9
390104	215.4	390212	112.2

2.7.2 Visual Distress - SPS-1

Construction of the SPS-1 and SPS-9 sections was functionally complete and mainline traffic was moved onto the test pavement on August 14, 1996. Within a few days, noticeable rutting was detected in Sections 390102 and 390107 in SPS-1, and there was concern these sections might deteriorate rapidly over the upcoming Labor Day weekend. Fortunately, there were no serious problems, but there was considerable doubt as to whether the sections would remain intact during the spring thaw. The prospect of having to perform emergency repairs on a major highway during the winter or early spring while the weather was cold and wet, and access to materials was limited prompted the consideration of some type of immediate remedial repair. After some deliberation, it was decided to remove the 4-inch thick AC pavement layer and some base material from both sections and replace these materials with a thicker layer of temporary AC pavement to get them through the winter. The southbound lanes were closed on September 3, 1996 to complete this work. A total removal of the temporary pavement and replacement with more robust supplemental sections of interest to the state was planned for 1997. While the distress in Sections 390102 and 390107 occurred somewhat earlier than expected using ODOT design parameters, the AASHTO equations did forecast these sections to be the first to fail.

During the rehabilitation of Section 390107, a portion of the underdrains originally installed to drain the pavement were observed to be not connected to outlet pipes, thus making the section partially drained and partially undrained. SHRP was notified of this oversight so it would be properly documented and perhaps the section would be removed from the database. Shortly after placement of the temporary pavement in Sections 390102 and 390107, and reopening of the southbound lanes on September 11, 1996, rutting also began to develop in Section 390101. To avoid a midwinter or early spring failure in this section and to preserve the integrity of dynamic response sensors in the thinner AC sections for the 1997 controlled vehicle tests, these lanes were closed again on December 3, 1996, and not re-opened until November 11, 1997.

During the winter of 1996-97, plans were prepared for removal of the three distressed SPS-1 sections and installation of heavier sections similar to those in SPS-9. Replacement of two distressed SPS-8 AC sections was included in the same contract. Prior to preparation of the construction drawings, ODOT contacted SHRP to see if there was any interest in having the sections rehabilitated in some particular way to further achieve their goals. ODOT was informed that SHRP had no follow-up plans for distressed sections in SPS-1 or SPS-2.

Visual observations of the three distressed SPS-1 sections indicated severe rutting throughout, with localized areas also exhibiting wheel path cracking. Because it was not possible visually to determine the specific causes of the distress, ODOT personnel and ORITE staff and students conducted a forensic investigation to more clearly define the failure mechanism in Section 390101. Appendix B contains a synopsis of the report documenting this effort entitled, "Final Report on Forensic Study for Section 390101 of Ohio SHRP U.S. 23 Test Pavement." Results of the forensic study showed the following:

- Essentially all of the rutting could be attributed to the base and subgrade, with none being observed in the AC layers.
- AC debonding was observed in the most severely distressed areas. The AC lifts were not tacked during construction.
- Subgrade moisture was consistently higher than anticipated throughout the short life of the section.

Judging by the nature and timing of distress in the other two sections, their modes of failure were likely to be very similar.

A sudden and rather dramatic failure occurred at Station 2+30 in Section 390105. Within a few hours after the distress was first reported to ODOT by passing motorists on May 29, 1998, considerable AC material from an area approximately 20 feet long and covering the right half of the right lane had been removed by traffic and scattered along the roadside. The two lifts of AC had debonded from the ATB and from each other over a 3-foot wide by 6-foot long oval at the center of the failed area. The ATB was also broken and in danger of being removed at that point. Away from the most distressed area, debonding was still evident, but less severe. Heavy rain the previous day likely precipitated the failure.

Over the next few days, an ODOT maintenance crew shut the driving lane down in Section 390105 only, removed the severely debonded AC over a 6-foot wide by 40-foot long area in the right side of the lane, and patched it with hot mix AC. Severe rutting was noted in other areas of the section and in the instrumented area immediately preceding the section. Consequently, other portions of the section were expected to fail in a short period of time. FWD and Dynaflect measurements obtained three weeks prior to this failure confirmed the area between Stations 2+00 and 2+50 to be particularly weak in the right wheelpath, with mid-lane measurements showing good uniformity throughout the section length.

Section 390105 was removed and replaced with a pavement identical to Section 390108, but with the addition of underdrains and a geosynthetic fabric placed on the surface of the finished subgrade. That is, a 7-inch thick asphalt concrete pavement (1-3/4" ODOT 446, Type I AC over 5-1/4" ODOT 446, Type II AC) on a 12-inch thick base (4" PATB/8" DGAB) on Geogrid laid on the subgrade. The Geogrid was not stapled or otherwise affixed to the subgrade prior to installation of the base. To facilitate construction and permit completion of a fifth series of controlled truck tests, the entire test pavement was shut down and traffic diverted back to the original lanes between September 8 and October 20, 1998. This replacement section was identified as Section 390164. Overall, surface raveling and longitudinal cracking, which appeared to be related more to construction techniques used in placement of the AC than structural distress, were common throughout the SPS-1 sections. These cracks were very straight, located at the same lateral position over the length of several sections, and not positioned where maximum stresses would be expected to occur. They probably reflect the segregation of aggregate along the edge of the paver as it placed AC on the pavement.

Localized distress in Section 390103 became severe enough by March 8, 2002 that it was closed to traffic. Upon further investigation, distress in Sections 390108, 390109 and 390110 had also progressed to the point where it was necessary to close the entire length of the southbound lanes on April 24, 2002. Plans are currently being prepared for replacing all four of these sections with more robust AC pavement designs being considered for implementation elsewhere on the ODOT pavement network. The sale date for this project is scheduled for November 20, 2002, and construction is expected in 2003.

2.7.3 Visual Distress - SPS-2

The SPS-2 test sections were opened to traffic on August 15, 1996. Traffic was moved back to the original lanes on December 2, 1996 for testing and rehabilitation of distressed SPS-1 sections. To facilitate completion of the fifth series of controlled vehicle tests, traffic was removed from the SPS-2 sections between September 8 and October 20, 1998. During the 1998 truck tests, early signs of distress were observed in Sections 390205 and 390206, the two 8-inch thick PCC sections with a lean concrete base. Among the types of distress noted were transverse and longitudinal cracking, faulting, and pumping.

Various aspects of the distresses observed in Sections 390205 and 390206 are of interest. As noted above, both sections have a 6-inch thick lean concrete base. Section 390205 has a 12-foot lane width and ODOT Class C concrete, while Section 390206 has a 14-foot lane width and high strength concrete. Both sections show evidence of pumping at contraction joints and along the pavement/shoulder interface. Both sections contain a longitudinal crack that starts near the pavement edge and passes continuously through several slabs as it moves to the right wheel path and back near the pavement edge. The location of a transverse crack at SHRP Station 4+38 in

Section 390205 appears to correspond to the location of a transverse crack noted in the lean concrete base prior to placement of the PCC pavement. As of the summer of 2002, the distress in these sections has progressed some, but not to the point of being dangerous or objectionable to motorists. The other SPS-2 sections did not show signs of distress by the fall of 2002.

In February 2003, researchers visiting the site observed numerous transverse cracks in Sections 390201, 390202, 390209 and 390210, all of which had eight-inch thick pavement. Section 390212, which had an eleven-inch thick pavement, also had a few transverse cracks. In Central Ohio, the winter of 2002-03 was consistently cold with considerable snowfall and snow covering the ground most of the time.

2.7.4 Visual Distress - SPS-8

The four test sections in SPS-8 were opened to traffic on November 18, 1994. Sections 390803 and 390804 (AC) displayed premature rutting very quickly. While these sections were exposed to a very low volume of truck traffic during 1995, the Series I controlled vehicle tests performed for FHWA in December 1995 and March 1996 accelerated the rutting process through the repeated application of some very heavy loads. ORITE staff completed a set of Cone Penetrometer Tests (CPT) tests along both sections and discovered a layer of poorly consolidated clay subgrade approximately four feet below the pavement surface. This was the depth of undercutting required in the area during construction and the level for placement of the first lift of material. CPT tests suggested the compaction effort on this first lift was inadequate. Also, the subgrade under the SPS-8 sections was undrained and appeared to be quite wet most of the time. The presence of excessive moisture, the poorly compacted subgrade layer, and the truck tests performed for FHWA all contributed to the premature rutting on these sections.

In August of 1997, Sections 390803 and 390804 were removed and replaced with sections similar to the original SPS-8 AC construction. The only differences were that the subgrade was undercut to a greater depth and treated with lime as it was replaced, and the surface and leveling courses were both constructed of ODOT Type I asphalt concrete. An array of response sensors similar to those incorporated in the other AC sections was installed just outside both replaced sections, and one additional environmental array was placed near the interface of the two sections. Because of pavement geometry on the ramp where this SPS-8 experiment was located, only local traffic could use Section 390809 and 390810 while Sections 390803 and

390804 were being replaced. This included some construction traffic. The ramp was re-opened on October 15, 1998.

By 2002, surfaces of the PCC sections in SPS-8 were scaling quite noticeably. These sections were designed for 550 psi concrete, which was included in the SHRP matrix as a material variable. To achieve this low strength, fly ash was added as a replacement for cement during placement of the SPS-8 PCC sections until the texture of the mix became rather coarse and porous. Because of concerns regarding the ability of this low strength mix to resist freeze-thaw cycling on the mainline pavement, ODOT, with SHRP concurrence, used standard Class C concrete as the low strength mix on the mainline pavement. A 900 psi concrete was developed for the high strength mix. The difference in strength between these mixes satisfied the intent of SHRP to construct pavements with two distinct concrete strengths.

2.7.5 Visual Distress - SPS-9

The three SPS-9 sections were constructed with a 22-inch thick base to provide extended service. The only difference between these sections was the grade of asphalt cement used in the 4-inch thick pavement layer. Section 390901 contained standard AC-20, Section 390902 contained PG 58-28, and Section 390903 contained PG 64-28. The AC surface course mix designed for Section 390903 with Superpave Level I specifications resulted in an extremely fine mix resembling sand asphalt. Skid resistance, as measured with the ODOT K.J. Law Skid Trailer, has remained satisfactory on these sections and about the same as the standard ODOT mix used on the SPS-1 sections. Aside from the fine texture on the pavement surface, there have been no indications of distress in the SPS-9 sections by 2002. Dates when these sections have been opened and closed to traffic are the same as sections in the SPS-1 experiment.

2.8 NON-DESTRUCTIVE TESTING (NDT)

Test sections were typically constructed in groups by location on the project, by types of material being incorporated into the sections, and by particular design features common to the sections. The FWD was brought in for testing as each lift of material was completed throughout the project. Because FWD test dates for the various layers and sections were quite different, any comparison of deflections between sections must be made cautiously. The most obvious variable throughout the season is subgrade moisture, which has a significant impact on NDT measurements. Temperatures within the AC or PCC pavement layer can also affect these data. While both the driving and passing lanes were constructed identically, all SHRP sections were located in the right hand or driving lane. Deflection data were obtained in the right wheel path and in the centerline of this right lane.

All FWD deflections shown in this report have been normalized to 1,000 lbs. for easier comparison with other FWD data and to facilitate any comparisons with Dynaflect data where deflections were obtained with a uniform 1,000 lb. sinusoidal loading. While the magnitude of FWD loading is approximated during testing by mounting specified weights on the trailer and by adjusting the drop height of the weights, the actual applied load is affected somewhat by the stiffness of the pavement structure. For a given combination of weights and drop height, the applied loads will tend to be higher with stiffer pavements. Dynaflect testing was limited to the completed pavement sections and subsequent in-service measurements.

2.8.1 FWD Subgrade Measurements

Falling Weight Deflectometer (FWD) measurements obtained on the finished subgrade are an important indicator of how pavements are likely to perform in the future. Because preliminary borings taken prior to construction suggested a relatively uniform soil structure throughout the U.S. 23 site, subgrade stiffness was expected to be reasonably similar in all of the test sections. However, the remains of old basements, wells, cisterns and other abandoned structures left when U.S. 23 was upgraded from a two-lane to a four-lane facility in the 1960's, and the occurrence of a few other localized areas where the naturally occurring moisture would, if left in place, have resulted in highly variable subgrade stiffness throughout these SPS experiments. In an attempt to improve subgrade uniformity, substantial undercutting was performed to remove these features and borrow was imported from a pit adjacent to the project. Even then, actual subgrade stiffness measurements obtained with the FWD still varied dramatically between sections and even within certain sections.

FWD testing on the finished subgrade consisted of two drops at each of four load ranges at 50-foot intervals along the centerline and right wheelpath of the sections. The highest loads on the subgrade ranged between 4,000-6,000 lbs. Profiles of this deflection along the section length are indicative of subgrade uniformity. With the exception of Sections 390159 and 390264, which were not finished until the following year, the subgrade for all SPS-1 and SPS-2 sections was completed and tested in the summer of 1995. Subgrade stiffness was highly variable between sections and, in some instances, within individual sections. FWD deflections obtained on the subgrade under Sections 390159 and 390264 are much higher than the remaining sections constructed one year earlier. These data illustrate how, even on projects with such a high visibility as this SHRP project, current ODOT specifications are limited in their ability to ensure the construction of stiff, uniform pavement subgrades. Stiffness-based specifications based on non-destructive testing (FWD/Dynaflect) might provide a better method for controlling subgrade uniformity than density-based specifications using nuclear density techniques.

2.8.2 Subgrade Variability

As part of this project "Continued Monitoring of Instrumented Pavement in Ohio," an interim report entitled "Evaluation of Subgrade Variability on the Ohio SHRP Test Road" was published in October 1999 to document an analysis of subgrade stiffness measured with the FWD. A synopsis of that report is included in Appendix C. A related technical note on early performance of the Ohio SHRP Test Road was also published and it is included in Appendix D.

2.8.3 NDT Testing of the Finished Pavement

FWD testing of the newly finished mainline pavement sections in June 1996 consisted of one drop at each of three load levels in the centerline and right wheelpath of the right lane. The sensor under the load plate (Df1) was used as the indicator of stiffness for the middle drop, being 9,000-11,000 lbs. on AC sections and 11,000-14,000 lbs. on PCC sections. This difference in load range on the two types of pavement was due to the effect of pavement stiffness on the response of the FWD. On AC sections, readings were taken every 50 feet in both test paths. On PCC sections, mid-slab readings were taken at about 50-foot intervals along the centerline and, in the right wheelpath, joint measurements were obtained about every 100 feet.

While considerable variability existed within the subgrade, data obtained when the sections were completed but not yet opened to traffic indicated that, as expected, much of this variability was masked once stiffer materials were placed over the subgrade. Also, the degree of masking is dependent upon the stiffness of the overlying pavement structure. Specifically, 1) the addition of any pavement structure will bridge over localized areas of subgrade weakness to some extent and reduce stiffness variability within the total structure, and 2) stiffer pavement structures provide more bridging action, thus reducing variability to a greater extent. As any pavement structure carries traffic, however, internal stresses will be higher in areas of diminished subgrade support. These areas will fatigue faster than the remaining pavement, and variability will begin to emerge again as a problem in the structure, both in terms of stiffness and visible distress.

By the very nature of the SPS experiments, significant differences in non-destructive deflection measurements would be expected between the completed test sections. This variation was due to the wide range in designs incorporated into the SPS experiments and any inherent differences in the subgrade. As on the subgrade, FWD data on finished pavement are an important indicator of how well sections can be expected to perform under traffic. Sections designed for limited service are less stiff and give much higher deflections than the more robust sections.

It is interesting to note that the first four sections to fail (390101, 390102, 390105, and 390107) were new pavement sections with the highest normalized Df1 deflection (1.62, 3.36, 1.38, and 1.90 mils, respectively). Also, the area in Section 390101 with the most severe distress at the time of the forensic investigation was Station 2+65 which was close to Station 3+00 with the highest initial normalized deflection (2.17 mils). As would be expected, mid-lane and right wheelpath measurements were quite similar on these new AC pavement sections.

It is noteworthy that the Dynaflect also shows Sections 390101, 390102, 390105, and 390107 to have the highest average deflections when the sections were new and identified Station 2+50 as being the weakest location in Section 390101. Overall, the correlation between FWD and Dynaflect measurements appears to be excellent for ranking pavement stiffness or identifying areas of weakness. One-to-one correlations would not be expected because of differences in the geometry of the loaded areas and in distances from the load to the sensors.

MODULUS4.2 was used to back calculate moduli on the SPS test sections from the FWD measurements. The selection of MODULUS4.2 was based on comparative results with other programs, including MODCOMP3 and EVERCALC5.0, and lower user dependency. In the back calculation analysis for deflection basin data from entire test sections, MODULUS4.2 matched the deflection basins well with low matching errors, especially on the asphalt concrete sections. More information on this back calculation analysis is provided the report "Determination of Pavement Layer Stiffness on the Ohio SHRP Test Road Using Non-Destructive Testing Techniques," Report No. FHWA/OH 2002/31.

2.8.4 NDT Testing in May 1998

The May 1998 FWD readings were taken in a manner similar to those taken in June 1996. One drop at each of three heights was made at 50-foot intervals throughout the test sections. Time did not permit the completion of both mid-lane and right wheelpath measurements in all sections. Because readings at the center of the load plate (Df1) were quite erratic and often less than those at the edge of the plate (Df2) on the PCC sections, Df1 was used for the SPS-1 sections and Df2 was used for the SPS-2 sections. This pattern of Df1 readings was unusual and believed to have been due to a malfunction in the FWD. The Df2-Df7 readings appeared to be correct. It is interesting that the average normalized deflection on Section 390105 was 1.52 mils and the normalized deflection at Station 2+50 was 2.76 mils less than four weeks before failure occurred near that location on May 29. Tables showing FWD Df1 profiles are included in Appendix E.

2.8.5 NDT Testing in April 2001

ODOT performed a comprehensive series of FWD and Dynaflect measurements in April 2001. Tables F-1 and F-2 in Appendix F show normalized FWD measurements on the AC and PCC sections, respectively, at loads approximating 9000 lbs. Tables F-3 and F-4 show comparable Dynaflect measurements on the AC and PCC sections.

- Despite variations in load application and geophone spacings, a good correlation appears to exist between the FWD and Dynaflect for the 1996 and 2001 data, as shown in Figure 2.8.
- AC Sections 390103 and 390108 have the highest deflections. Deflections in Sections 39A803, 390109 and 390164 have also increased during their period of service.
- PCC Sections 390202 and 390809 have high midslab deflections. FWD and Dynaflect measurements at Station 2+50 in Section 390809 are exceptionally high, possibly indicating a void under the slab. Load transfer is good in all sections.
- While Sections 390205 and 390206 have some cracks and display moderate to severe pumping, midslab and joint deflections appear to be normal. Load transfer across three cracks in Section 390205, however, ranged from 15 to 30%.

2.9 PAVEMENT ROUGHNESS

Another indicator of pavement performance is the manner in which surface roughness increases over time. As pavements degrade, they tend to become rougher and more uncomfortable for vehicle drivers and passengers. A K.J. Law Non-Contact Profilometer was used by ODOT at the completion of the SPS sections and periodically thereafter to monitor section roughness. Data shown in Tables 2.9 and 2.10 represent a summary of section roughness in Mays and PSI numbers when the pavement was new and at various times during service.

2.10 RUT DEPTH

Table 2.11 presents rut depths measured in the right wheel path of the northern SPS-1 sections with a straightedge and a rolling-wheel profilometer developed by Ohio University. Straightedge measurements are maximum depth to the bottom of the rut measured from the bottom of a straight edge laid across the right wheel path. The 4/29/99 and 12/20/00 data were measured by ODOT with a six-foot long aluminum straightedge.

The 9/14/01 data were obtained with the Ohio University contact profilometer using the edge of the right edge paint line as the starting point. This instrument produces a set of elevations to +/- 0.01 inch approximately every ½ inch over a nine-foot long track. Rut depths shown in Table 2.11 are the maximum of the set of calculated distances between elevations measured in the right wheel path and the nearest point on a "virtual" straightedge resting on the right edge of the travel lane and tangent to the hump between the left and right wheel paths. The point where the virtual straight edge is tangent to the hump was determined by maximizing the slope of the line (the virtual straightedge) between the right lane edge (represented by the first member of the set of elevations produced by the profilometer) and measured elevations in a range of elevations determined by examination to represent the hump of the profile plot. The range of elevations defining the rut was likewise determined by examination, because not all plots were typical W-shaped rut profiles. The data reduction program first smoothed the profilometer data by substituting for the raw data a running average of two adjacent elevations. The seven unsmoothed profile plots taken with the profilometer on Section 390103 are shown in Figure 2.9. An additional profile was recorded at Station 0+50 because of distress at that location. The reason for the varied elevations of the right edge of pavement is that the outboard support of the profilometer rests on the shoulder about 20 inches outside the edge of the traveled lane when the profiler's sensor begins its travel at the edge of the traveled lane. Therefore any profile features between the outer support and the first recorded elevation can cause apparent discrepancies, which are nullified when a virtual straight edge is used to calculate rut depths.

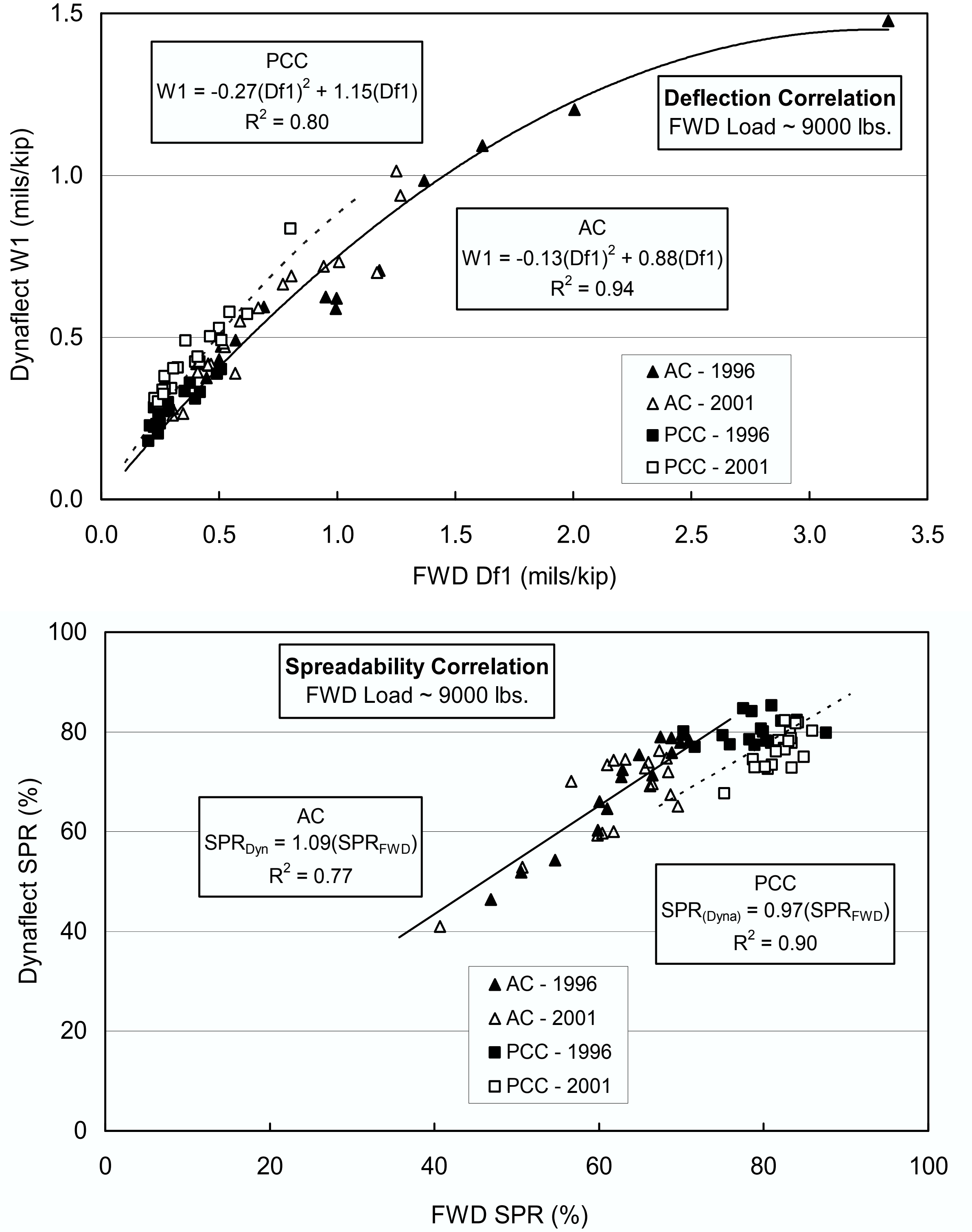


Figure 2.8 Correlation of FWD and Dynaflect Responses on SHRP Test Sections

Table 2.9 Pavement Roughness - Mays

Section No.	Mays Ride Number							
	8/16/96	8/27/96	9/18/96	10/28/96	11/28/97	6/4/98	5/17/99	5/31/00
SPS-1								
390159	Data Not Available (DNA)						58.4	68.6
390103	Data Not Available						126.3	137.3
390110	68.1	68.8	71.6	72.9	64.8	79.3	88.4	86.8
390109	43.0	43.3	45.0	46.3	49.7	61.6	76.3	95.3
390108	53.3	53.4	55.9	67.6	72.4	87.1	110.3	107.1
390105	57.3	60.6	75.1	75.9	97.7	126.3		
390164							98.0	107.9
390160	63.1	65.8	65.0	69.4	110.1	108.2	125.4	121.7
390102	83.1	146.0						
390161					58.4	48.6	43.8	57.1
390107	70.4	81.5						
390162					49.5	45.8	47.5	48.1
390101	86.8	111.8	134.7	189.2				
390163					75.3	64.8	65.5	68.3
390106	71.2	71.3	73.9	76.7	140.9	123.0	115	134.3
390104	45.2	47.2	48.0	46.8	74.0	91.2	74.8	104.8
390111	44.3	45.5	46.8	45.3	58.9	64.1	68.3	74.2
390112	53.7	53.0	53.8	52.3	71.2	83.3	88.2	87.7
SPS-2								
390259	Data Not Available						54.0	52.7
390204	51.4	61.2	53.4	50.9	55.2	49.3	53.4	61.3
390212	67.9	71.3	62.2	60.7	68.3	74.9	72.2	69.2
390210	65.3	73.1	61.5	58.7	66.9	71.6	78.7	79.3
390260	66.6	73.4	64.0	61.1	66.3	68.1	70.4	64.9
390202	71.6	79.1	70.7	71.4	80.7	86.9	88.1	90.5
390206	76.3	70.1	69.6	68.0	79.3	86.0	89.4	84.5
390205	69.8	68.3	67.1	65.9	69.5	66.6	77.1	77.3
390201	71.8	70.8	71.9	71.4	79.1	78.0	87.0	91.5
390209	59.9	58.3	58.9	57.0	64.9	65.7	71.0	65.5
390261	76.3	75.8	76.1	74.4	87.8	93.4	75.2	85.1
390211	85.6	83.1	80.1	80.3	86.4	84.1	91.7	85.3
390265	84.0	81.3	82.5	80.1	86.6	86.4	96.0	95.9
390203	63.1	61.1	60.1	56.2	65.5	65.6	64.0	61.0
390207	80.0	77.1	74.8	74.5	76.8	84.7	86.4	82.8
390208	79.9	79.1	79.0	75.1	81.8	89.3	88.4	83.0
390262	75.0	73.1	73.6	66.1	74.6	78.7	77.3	62.7
390263	Data Not Available						76.8	75.4
390264	Data Not Available						78.1	113.0
SPS-8								
390810 (PCC)	Data Not Available							
390809 (PCC)	Data Not Available							
390803 (AC)	Data Not Available							
39A803 (AC)					DNA			
390804 (AC)	Data Not Available							
39A804 (AC)					DNA			
SPS-9								
390902	47.4	47.2	48.0	47.5	45.7	49.6	49.2	50.7
390903	41.7	40.8	41.6	41.0	45.9	49.1	54.6	51.9
390901	46.7	46.5	47.9	47.0	46.1	48.5	50.4	53.7

Table 2.10 Pavement Roughness - PSI

Section No.	Present Serviceability Index (PSI)							
	8/16/96	8/27/96	9/18/96	10/28/96	11/28/97	6/4/98	5/17/99	5/31/00
SPS-1								
390159	Data Not Available						4.1	4.1
390103	Data Not Available						3.3	3.2
390110	4.0	4.0	3.9	3.9	4.1	3.8	3.7	3.7
390109	4.2	4.2	4.2	4.2	4.3	4.0	4.0	3.7
390108	4.1	4.1	4.1	4.0	4.2	3.8	3.6	3.7
390105	4.0	4.0	3.8	3.8	3.7	3.3		
390164							3.8	3.8
390160	4.0	4.0	4.0	4.0	4.1	3.8	3.7	3.8
390102	3.9	3.2						
390161					4.6	4.3	4.3	4.2
390107	4.1	3.8						
390162					4.4	4.3	4.3	4.2
390101	3.9	3.6	3.4	2.8				
390163					4.4	4.1	4.1	4.1
390106	4.0	4.0	3.9	4.0	3.9	3.7	3.9	3.7
390104	4.1	4.0	4.0	4.1	4.1	3.9	4.0	3.8
390111	4.1	4.1	4.1	4.1	4.2	4.0	4.0	3.9
390112	3.9	4.0	3.9	4.0	4.0	3.9	3.8	3.8
SPS-2								
390259	Data Not Available						4.2	4.2
390204	4.1	3.9	4.1	4.1	4.1	4.1	4.1	3.9
390212	3.9	3.9	4.1	4.1	4.0	3.9	3.9	4.0
390210	3.9	3.9	4.0	4.1	4.0	3.9	3.8	3.8
390260	3.9	3.8	3.9	4.0	3.9	3.9	3.8	3.9
390202	4.0	4.0	4.1	4.1	4.1	4.0	4.0	4.0
390206	3.9	4.0	4.0	4.0	3.9	3.8	3.8	3.9
390205	4.0	4.0	4.0	4.1	4.0	4.0	3.9	3.9
390201	4.0	4.1	4.0	4.1	4.0	4.1	4.0	4.0
390209	4.1	4.1	4.1	4.1	4.1	4.1	4.0	4.1
390261	3.9	4.0	4.0	4.0	3.9	3.9	3.9	4.0
390211	3.8	3.9	4.0	3.9	3.9	3.9	3.7	3.8
390265	3.8	3.8	3.8	3.9	3.8	3.8	3.7	3.8
390203	4.0	4.0	4.0	4.1	4.0	4.0	4.0	4.0
390207	3.8	3.9	3.9	3.9	3.9	3.8	3.8	3.8
390208	3.8	3.8	3.8	3.9	3.8	3.8	3.7	3.7
390262	3.8	3.9	3.8	4.0	3.9	3.9	3.8	4.1
390263	Data Not Available						3.9	3.9
390264	Data Not Available						3.7	3.5
SPS-8								
390810 (PCC)	Data Not Available							
390809 (PCC)	Data Not Available							
390803 (AC)	Data Not Available							
39A803 (AC)					DNA			
390804 (AC)	Data Not Available							
39A804 (AC)					DNA			
SPS-9								
390902	3.9	4.0	3.9	4.0	4.1	4.0	4.0	4.0
390903	4.2	4.2	4.2	4.2	4.3	4.1	4.1	4.2
390901	4.0	4.0	4.0	4.0	4.3	4.0	4.0	4.0

Table 2.11 SPS-1 Rut Depth Measurements

SHRP Section	Station	RWP Rut Depth (in.)				SHRP Section	Station	RWP Rut Depth (in.)			
		4/29/99	12/20/00	9/14/01	7/24/02			4/29/99	12/20/00	9/14/01	7/24/02
390103	0+00			0.40	0.41	390160	0+00			0.01	
	1+00	<2/16	0.2	0.29	0.27		1+00	<1/16		N.A.	
	2+00		0.1	0.32	0.27		2+00			0.17	
	3+00		0.3	0.40	0.41		3+00			0.08	
	4+00		0.5	0.64	0.53		4+00			0.06	
	5+00			0.48	0.43		5+00			0.06	
	Average		0.3	0.42	0.39		Average			0.13	
390108	0+00			0.57	0.58	390161	0+00			0.18	
	1+00	>4/16	0.4	0.57	0.44		1+00	<1/16		0.08	
	2+00		0.3	0.47	0.42		2+00			0.05	
	3+00		0.2	0.31	0.30		3+00			0.05	
	4+00		0.3	0.36	0.35		4+00			0.10	
	5+00			0.50	0.47		5+00			0.07	
	Average		0.3	0.46	0.43		Average			0.09	
390109	0+00			0.41	0.43	3901162	0+00			0.20	
	1+00	>1/16	0.2	0.17	0.20		1+00	<1/16		0.17	
	2+00		0.3	0.33	0.31		2+00			0.22	
	3+00		0.2	0.32	0.26		3+00			0.24	
	4+00		0.1	0.12	0.16		4+00			0.25	
	5+00			0.22	0.22		5+00			0.24	
	Average		0.2	0.26	0.26		Average			0.22	
390110	0+00			0.12	0.06	390164	0+00			0.20	
	1+00	>1/16	0.1	0.25	0.20		1+00	<1/16	0.1	0.20	
	2+00		0.1	0.20	0.13		2+00		0.1	0.22	
	3+00		0.0	0.09	0.07		3+00		0.3	0.26	
	4+00		0.0	0.11	0.07		4+00		0.2	0.27	
	5+00			0.21	0.11		5+00			0.29	
	Average		0.0	0.16	0.11		Average		0.18	0.24	

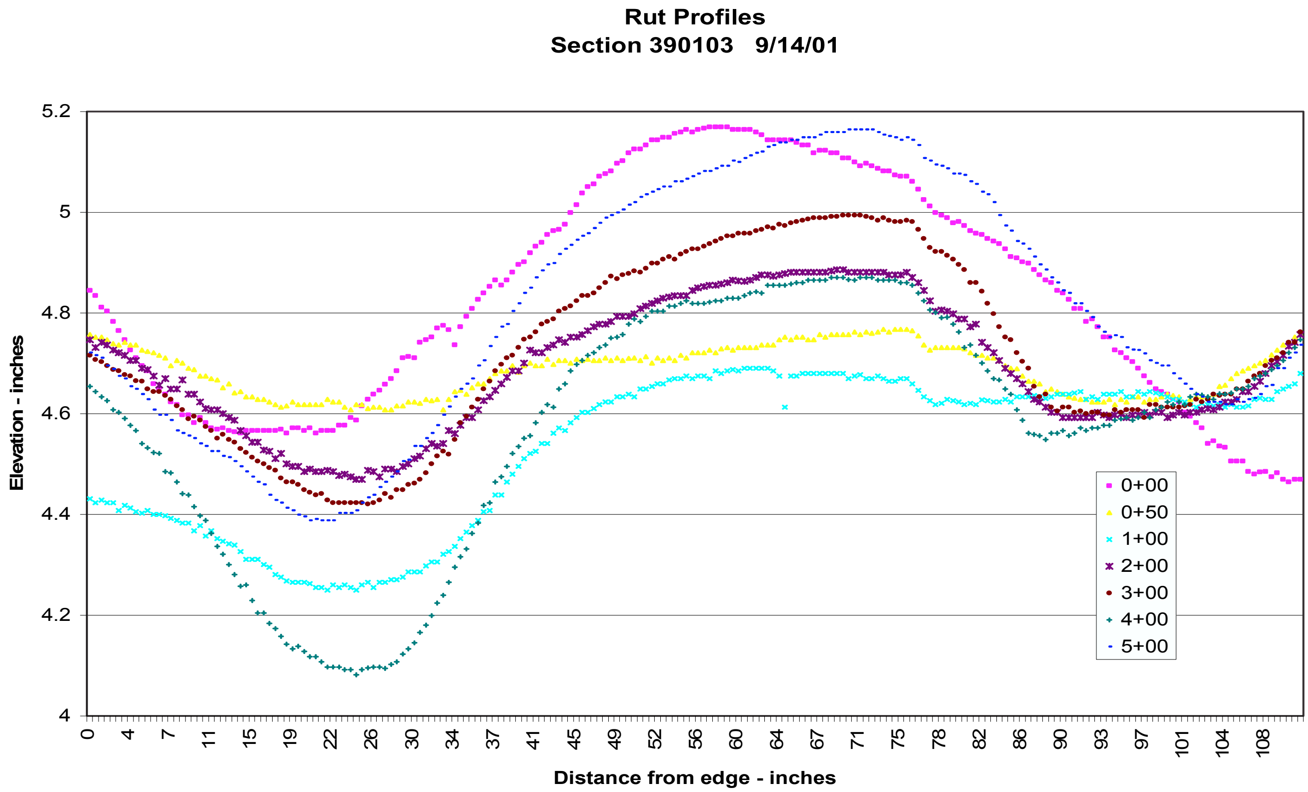


Figure 2.9 Rut Profiles Section 390103 9/14/01

2.11 SKID RESISTANCE

Skid resistance is a measure of the friction generated by a locked test tire skidding across a pre-wetted pavement surface under standard ASTM test conditions. It is expressed as skid number or SN40 when testing at the standard speed of 40 mph. Skid resistance is affected by many variables, including the texture of the pavement surface. AC and PCC surfaces generally exhibit high skid resistance soon after being opened to traffic. On AC pavements, friction can increase as the bituminous coating wears off the aggregate and then decrease as sharp edges are worn off the aggregate particles. On PCC pavements, friction decreases as the initial texture and aggregate particles wear over time. Seasonal variations of 3-5 skid numbers are common as grits added for snow and ice control in the winter months tend to rough up the surface and increase skid resistance. In the summer, friction is reduced as this roughness is worn down and, on AC pavements, a film of asphalt cement may be tracked on to the aggregate particles.

Table 2.12 summarizes skid data obtained to date on individual sections at 40 mph and Figures 2.10-2.13 show plots of how skid resistance changed in the four SPS experiments over time. The following trends have emerged:

- The replacement sections on SPS-1 generally have lower skid resistance, possibly because they have been exposed to less traffic and the surface mix in these sections may contain more asphalt cement than the original mixes.
- Section 390901 contained standard AC-20 asphalt cement, Section 390902 contained PG58-28 asphalt cement and Section 390903 contained PG64-28 asphalt cement. The three SPS-9 sections had about the same skid resistance as the SPS-1 sections, indicating equal performance between traditional ODOT AC mixes and the SuperPave mixes used on this project.
- The 900 psi concrete sections in SPS-2 are consistently 5-10 skid numbers lower than the sections constructed with Standard ODOT Class C concrete. The 550 psi concrete in SPS-8 is some 20 skid numbers higher than the ODOT concrete, possibly because of surface scaling which increases friction but is indicative of poor concrete durability.
- The sharp increase in SN40 on the SPS-1 and SPS-9 sections during the 6/19/02 tests was highly unusual, first because of the magnitude of the increase, second because the same jump was not present on the SPS-2 and SPS-8 sections (including AC) and, third because the higher skid numbers were duplicated on 7/3/02.

Table 2.12 Skid Resistance on Ohio SHRP Test Road

Section No.	Skid Number (SN40) on Date											
	8/16 1996	5/6 1997	5/5 1998	10/27 1998	5/13 1999	11/5 1999	7/28 2000	10/23 2000	6/19 2001	12/4 2001	6/19 2002	7/3 2002
SPS-1												
390159	NA	50.3	53.2	48.9	47.6	45.0	46.7	48.5	42.6	43.8	46.9	47.0
390103	NA	67.3	54.0	55.3	47.7	50.6	44.9	44.6	47.2	43.8	56.8	58.9
390110	NA	69.7	56.3	57.5	50.7	51.6	51.5	50.3	47.7	47.3	56.9	59.5
390109	NA	70.0	54.4	54.9	49.1	52.1	49.2	49.5	49.1	43.3	54.6	58.5
390108	NA	69.0	55.8	55.3	49.6	52.6	50.4	51.3	48.2	45.5	56.2	57.8
390105	NA	72.0	53.1	Section Replaced								
390164				52.9	501	49.3	44.7	45.1	45.5	41.5	52.8	56.3
390160	NA	71.0	57.2	49.0	49.6	52.5	49.7	50.0	49.3	47.5	54.8	56.6
390102	NA	68.0	Section Replaced									
390161			54.9	34.0	42.8	46.0	42.7	43.8	43.5	39.8	50.7	51.0
390107	NA	63.0	Section Replaced									
390162			52.0	38.9	44.0	44.0	42.9	43.1	44.1	40.3	50.2	51.7
390101	NA	72.0	Section Replaced									
390163			52.6	41.9	41.8	44.4	41.3	41.5	43.4	37.6	53.2	55.3
390106	NA	70.3	56.1	54.8	50.9	53.5	50.7	53.1	50.1	45.6	57.4	59.1
390104	NA	65.7	55.4	53.6	52.0	52.6	50.0	51.9	50.7	46.9	56.9	58.6
390111	NA	69.7	55.7	55.3	50.4	52.8	51.3	52.5	49.8	45.7	57.0	59.2
390112	NA	71.7	56.4	53.8	51.3	52.7	50.9	51.0	19.2	46.0	57.4	58.1
SPS-2												
390259*	39.7	49.3	39.9	32.2	35.4	33.7	33.2	33.5	31.1	33.8	35.8	
390204*	45.0	52.3	41.9	30.0	33.8	30.7	31.0	32.0	29.9	32.2	32.4	
390212*	46.0	58.7	47.4	33.3	32.6	32.0	31.9	31.8	30.0	32.4	31.8	
390210*	49.3	60.7	49.0	32.3	34.8	33.6	31.5	31.9	30.6	31.9	32.1	
390260	52.0	61.0	53.5	41.0	42.9	40.3	40.0	40.7	39.9	39.6	40.3	
390202*	32.0	54.0	40.7	26.0	30.8	28.9	31.5	30.6	31.0	31.4	31.1	
390206*	39.3	54.0	42.5	28.6	33.3	29.2	30.3	29.9	30.8	28.6	29.1	
390205	49.3	61.0	53.9	41.6	44.2	41.7	40.7	42.8	43.4	40.6	39.9	
390201	40.3	59.7	47.6	36.2	39.4	37.7	38.9	40.0	42.6	40.0	43.3	
390209	44.3	59.7	49.9	39.6	41.6	40.1	41.2	42.7	42.6	42.3	43.1	
390261	43.3	58.3	48.9	38.2	39.7	38.9	41.1	41.3	42.2	40.7	43.1	
390211	43.0	57.0	50.0	37.4	40.3	38.5	40.4	40.2	42.4	40.6	43.2	
390265	48.3	59.3	50.1	40.3	42.7	40.4	41.6	40.2	43.1	39.6	42.6	
390203	45.3	59.0	48.1	38.7	41.9	39.6	42.2	42.7	44.9	42.6	44.3	
390207	47.7	57.3	54.3	43.4	43.9	42.4	41.7	42.0	43.0	40.6	43.6	
390208*	35.0	54.0	37.6	30.3	31.8	30.1	32.1	31.4	34.2	33.5	34.1	
390262	47.7	58.3	56.3	45.2	48.1	44.6	44.6	44.4	44.9	41.0	44.1	
390263	45.0	58.0	50.4	40.2	43.0	40.8	42.8	43.5	44.0	42.1	45.0	
390264	38.3	55.3	53.4	43.9	45.8	41.1	476.0	45.0	41.7	41.1	43.0	
SPS-8												
390810**	NA	NA	61.8	57.6	59.4	57.4	59.8	62.6	55.3	62.9	60.7	
390809**	NA	NA	63.1	56.9	61.1	59.5	63.5	63.8	58.2	64.7	62.7	
390803	NA	NA	Section Replaced									
39A803			67.8	64.6	67.3	62.5	68.1	70.6	63.3	68.0	68.1	67.2
390804	NA	NA	Section Replaced									
39A804			64.6	62.5	64.6	63.7	66.6	67.1	63.4	67.8	68.6	66.8
SPS-9												
390902	NA	67.0	52.9	55.7	50.3	52.3	47.4	48.9	49.6	43.3	60.7	62.3
390903	NA	74.0	58.0	56.5	50.8	50.7	50.6	51.8	49.4	48.6	57.7	58.4
390901	NA	69.3	57.8	57.4	51.4	52.7	50.0	48.7	49.5	46.5	58.8	60.2

NA - Data not available * 900 psi concrete ** 550 psi concrete

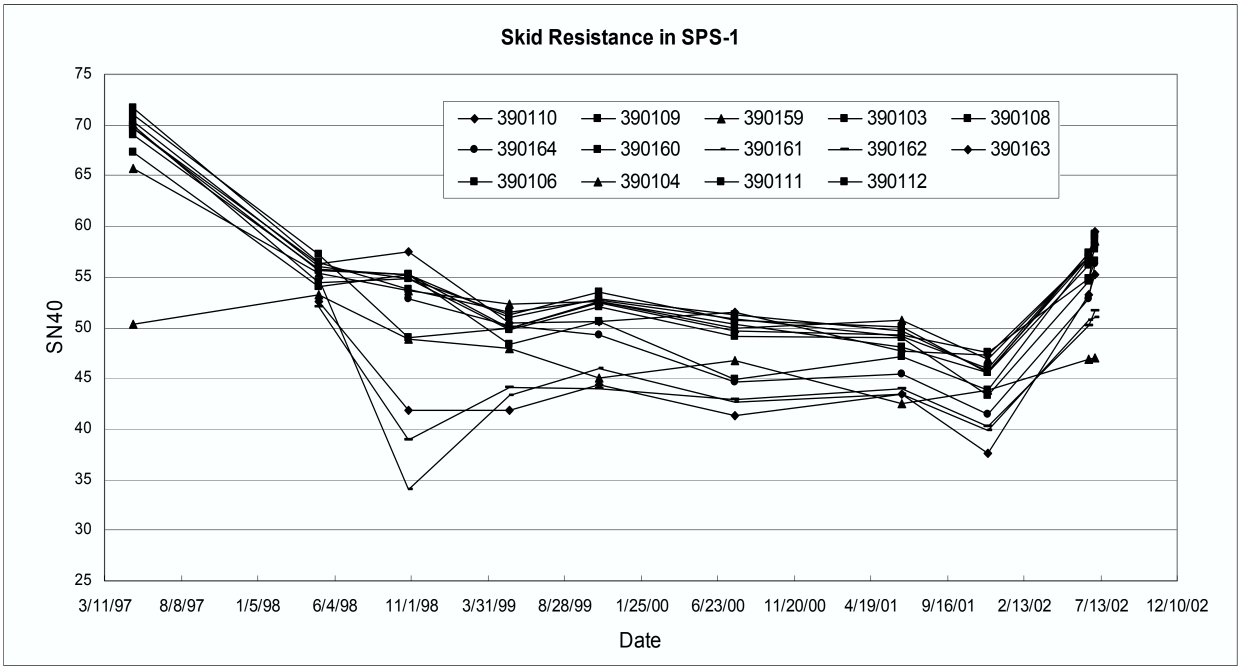


Figure 2.10 Skid Resistance in SPS-1

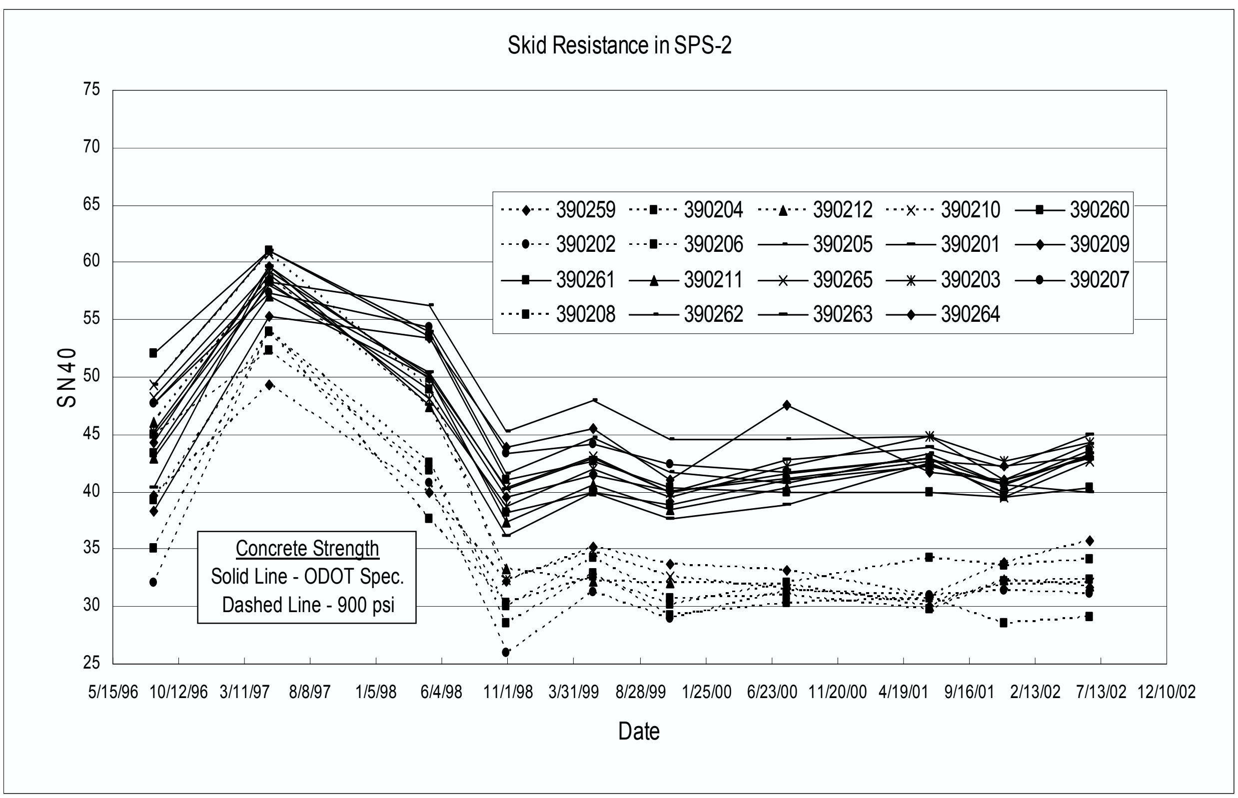


Figure 2.11 Skid Resistance in SPS-2

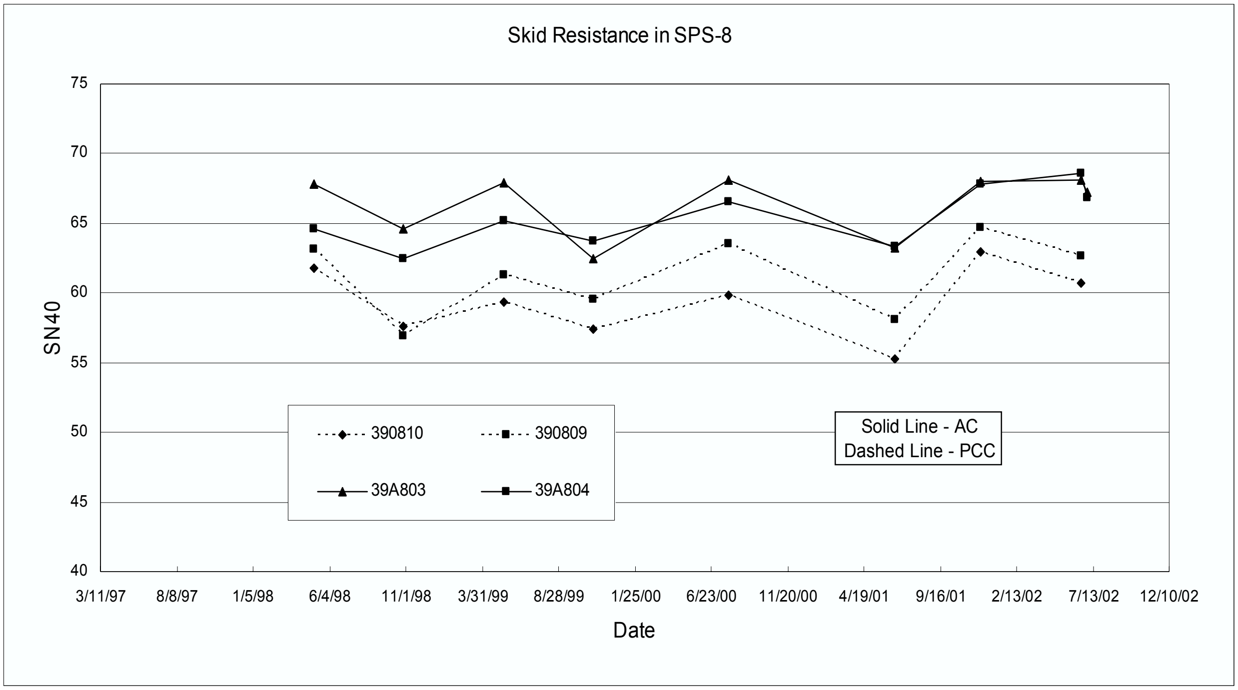


Figure 2.12 Skid Resistance in SPS-8

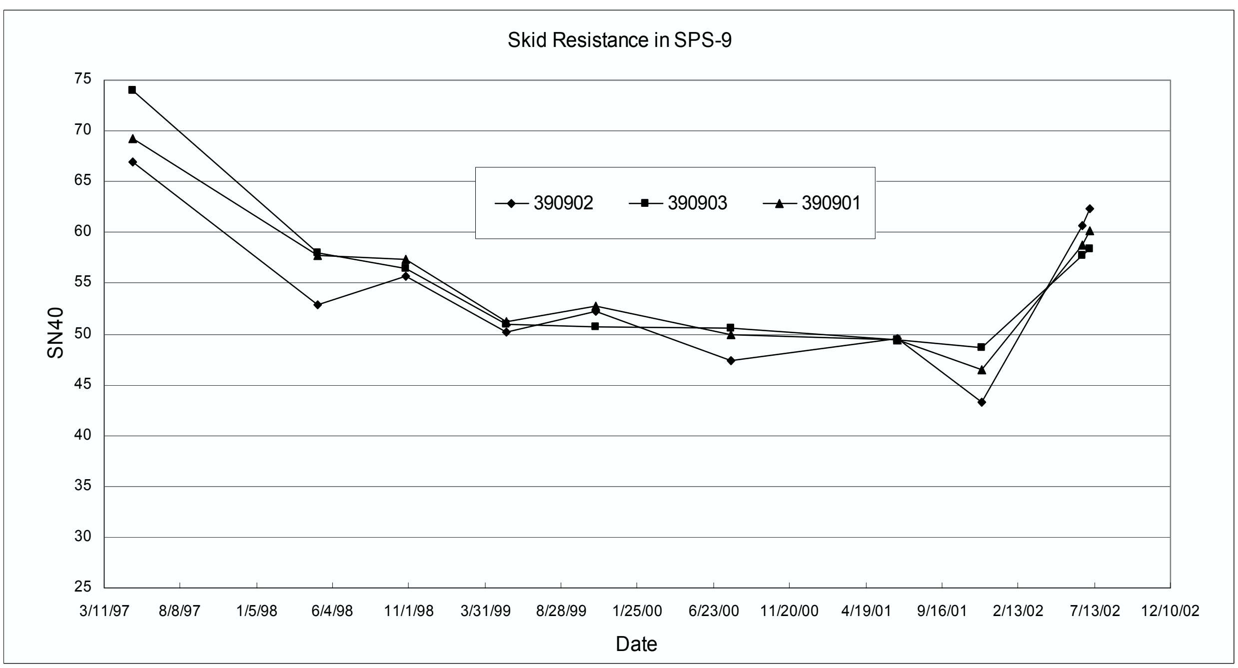


Figure 2.13 Skid Resistance in SPS-9

2.12 REPLACEMENT SECTIONS

A total of six test sections failed and were replaced soon after they were opened to traffic. Four of these sections in the SPS-1 experiment were replaced with more robust sections of interest to ODOT with underdrains. Two AC sections in the SPS-8 experiment were replaced with identical designs, except that lime was added to the subgrade to improve the stiffness of that layer. No underdrains were added to the SPS-8 replacement sections. Table 2.13 summarizes the pavement buildup, design features and station limits of the replacement sections.

In Section 390164, PG 64-28 binder was used in both the TI and TII mixes, a bituminous prime coat was used between the PATB and the TII layers, and a tack coat was used between the TII and TI layers. No reclaimed material was used in the TI or TII mixes. Underdrains were installed and unstapled Tensar BX1100 fabric was placed between the subgrade and DGAB.

In April 2002, the southbound lanes were closed because of an immanent failure in Section 390103 and significant rutting in Sections 390108, 390109 and 390110. These four contiguous sections will be replaced in 2003 with one AC pavement designed for extended service.

Table 2.13 Design and Station Limits of Replacement Sections

New SHRP Section	Replaced SHRP Section	Station Limits		Pavement Buildup	Design Features
		Start	End		
39A803	390803	19+90	14+90	1.75" TI/ 2.25" TII/ 8" DGAB over lime stabilized subgrade	No Recycled Material
39A804	390804	13+50	8+50	1.75" TI/ 5.25" TII/ 12" DGAB over lime stabilized subgrade	No Recycled Material
390161	390102	375+00	370+00	1.25" TI/ 1.75" TII/ 12"ATB/4" PATB/ 6" DGAB	SUPERPAVE Level I Design & 20% RAP in both TI and TII. Underdrains installed
390162	390107	363+00	358+00	1.25" TI/ 1.75" TII/ 12"ATB/ 4" PATB/ 6" DGAB	No Recycled Material & Gravel Coarse Agg. In Both T1 and TII. Underdrains installed
390163	390101	355+00	350+00	1.25" TIH/ 1.75" TII/ 12"ATB/ 4" PATB/ 6" DGAB	No Recycled Material in TIH, Polymer added to TIH. Underdrains installed
390164	390105	392+50	387+50	1.75" TI/ 5.25" TII/ 4"PATB/ 8"DGAB/ Geogrid	Geogrid - Tensar BX1100 Underdrains installed
390165	390103	420+75	415+75	1.5"TI/ 1.75"-TII/ 9.5" ATB/4"304NJ/6" DGAB/16" cement stabilized soil	
	390108	399+75	394+75		
	390109	406+50	401+50		
	390110	413+50	408+50		

TI - Asphalt Concrete Surface Course TIH – TI w/coarse aggregate TII - Asphalt Concrete Intermediate Course

2.13 ACCUMULATED TRAFFIC

Figures 2.14 and 2.15 are plots of accumulated Equivalent Single Axle 18 Kip Loads (ESALs) measured by the weigh-in-motion scales mounted in the northbound and southbound pavement lanes, respectively. The number of ESALs in the southbound lanes containing the SPS-1 and SPS-9 experiments has been revised substantially upward from that shown earlier in the report entitled "Coordination of Load Response Instrumentation of SHRP Pavements – Ohio University," published in May 1999. The northbound and southbound ESAL counts are amazingly close with both directions, averaging about 500,000 ESALs annually. Traffic data obtained on the ramp with a different WIM system was difficult to process and is not presented here, although informal on-site observations indicate traffic volumes were extremely low with only an occasional sighting of a commercial vehicle using the ramp.

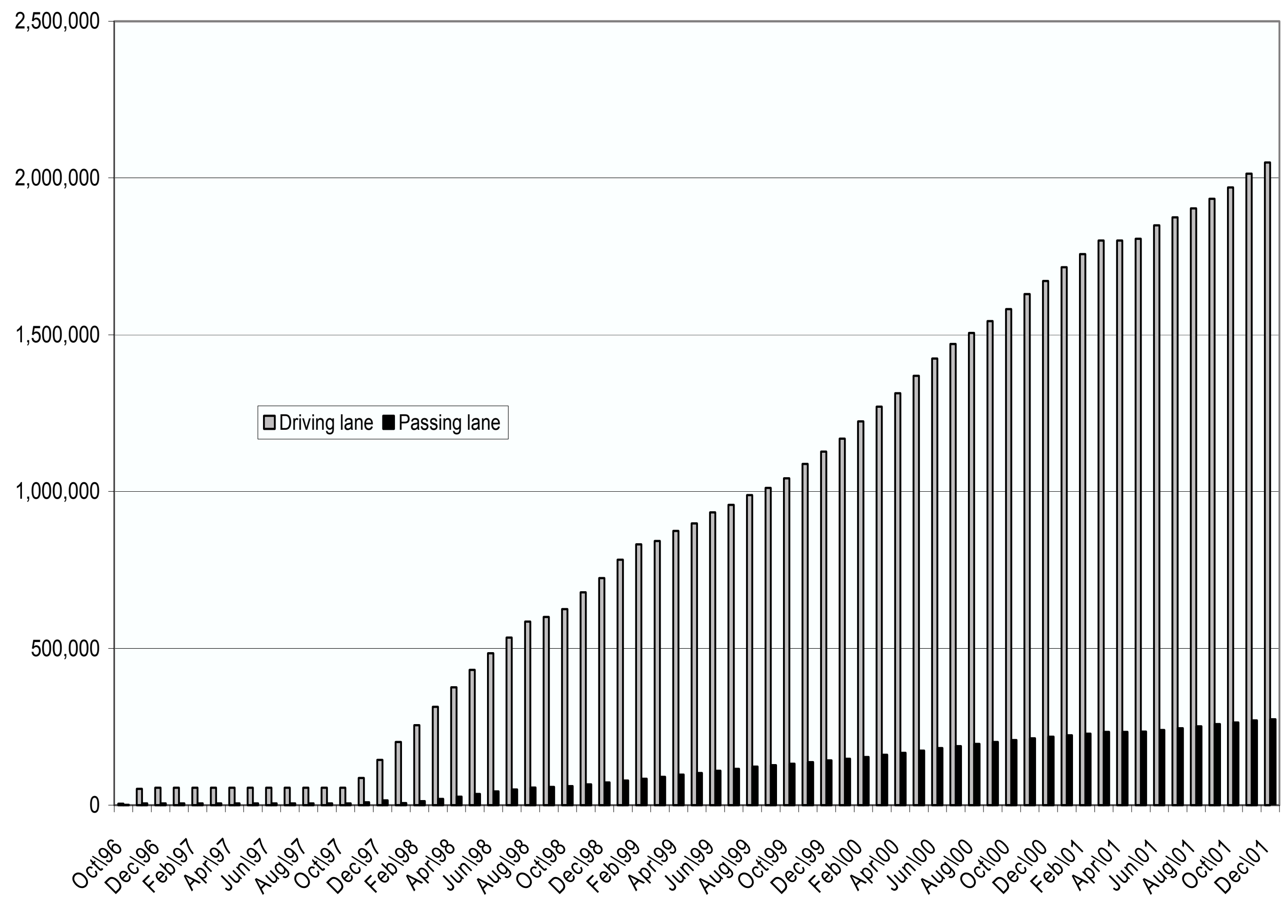


Figure 2.14 Accumulated ESALs Northbound (PCC)

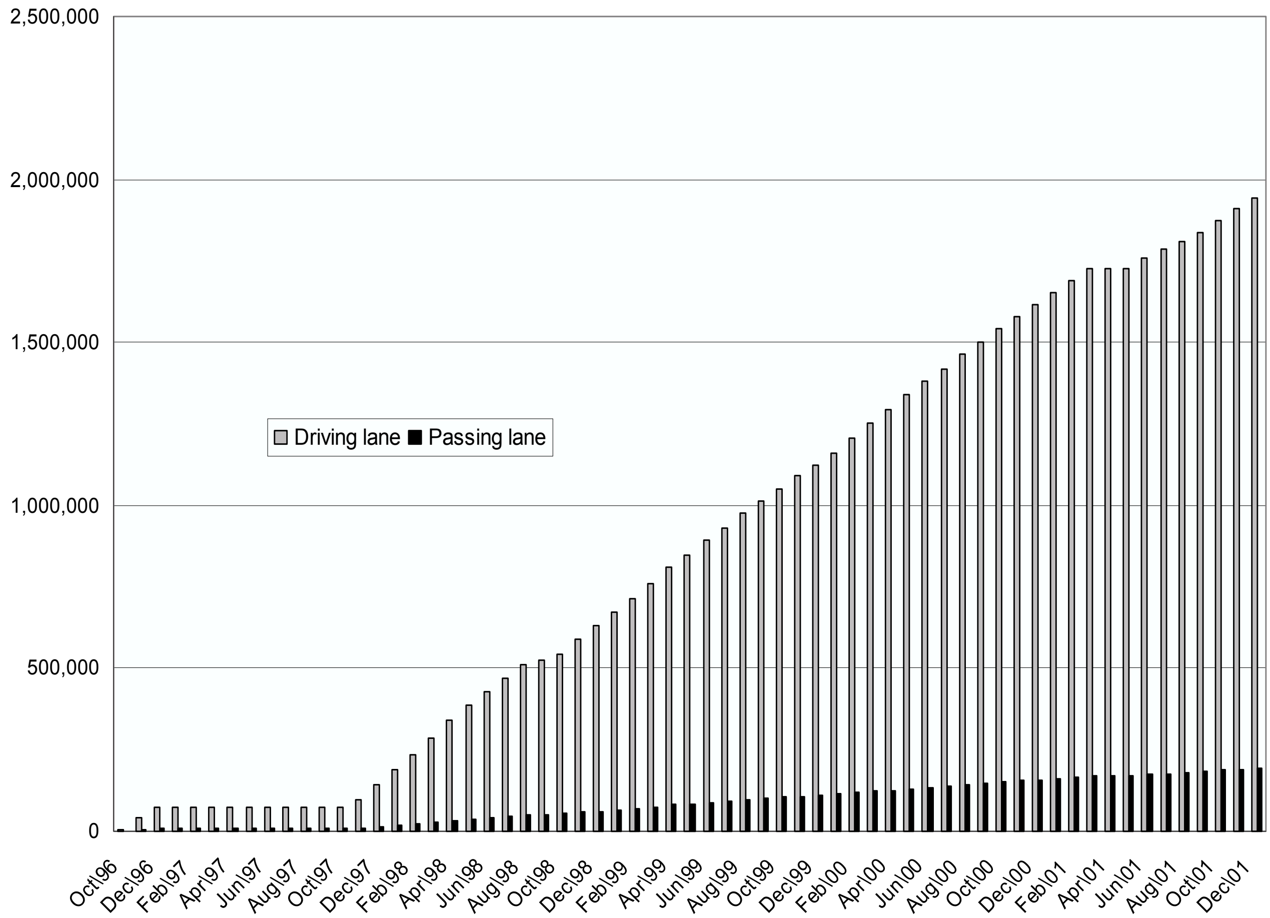


Figure 2.15 Accumulated ESALs Southbound (AC)

3.0 A DEMONSTRATION PROJECT ON INSTRUMENTATION OF A FLEXIBLE PAVEMENT (LOG U.S. 33)

3.1 INTRODUCTION

An instrumentation plan was developed for a full-scale asphalt concrete test pavement on U.S. 33 in Logan County. Six asphalt pavement test sections were constructed over asphalt-treated, cement-treated, New Jersey, Iowa, and six and eight-inch thick 304 dense-graded aggregate bases. Upon completion of test sections, moisture, temperature, vertical deflections, pressures, and strains were measured as the pavement was subjected to FWD loading. Sensor measurements were compared to predictions from a finite element model. Field data indicated that the deflection of asphalt with asphalt-treated base varies significantly with changes in temperature. Deflection of the pavement over cement-treated base was the lowest. Among the non-treated bases, those with larger aggregate showed less deflection. Predictions made with the OU-PAVE finite element program agreed well with measurements made in the field.

This project was located on SR 33, east of the city of Bellefontaine Ohio in Logan County. The road was expanded from two lanes to four lanes. Six sections along a four-mile long stretch of asphalt concrete pavement were instrumented in November of 1993. Environmental conditions and dynamic loading response of these flexible pavement sections were initially recorded in December 1993, with subsequent data collection efforts being completed in April 1994, September 1994, and January 1995.

3.2 DESCRIPTION OF THE SECTIONS

On one portion of the project, an eleven-inch thick asphalt concrete pavement was placed over five different types of base, each eight inches thick. Another portion of the project had a thirteen-inch thick asphalt concrete pavement over a six-inch thick base. The individual test sections are described below and summarized in Table 3.1. Table 3.2 shows target gradations provided by the contractor to the material suppliers.

Section 1: Four-inch thick, free-draining Asphalt Treated Base (ATFDB) placed over a four-inch thick standard 304 limestone aggregate base.

Section 2: Four-inch thick, Portland cement-treated free-draining base (PCTB) placed over four inches of standard 304 limestone aggregate base.

Section 3: Four inches of New Jersey (NJ) base placed over four inches of standard 304 limestone aggregate base. NJ was a non-stabilized permeable aggregate base. Of the bases tested, NJ had the least amount of fines, as shown in Table 3.2.

Section 4: Four inches of Iowa type (IA), non-stabilized drainage base placed over four inches of 304 limestone aggregate base. The Iowa base was similar in composition to the 304 base, except that it had fewer fine particles.

Sections 5 and 6: Standard 304 limestone aggregate base eight and six inches thick.

Table 3.1 Description of Pavements and Bases

Section	Beginning Station	Base Type	Base Thickness (inches)	Asphalt Pavement Thickness (inches)
1	954 + 50	Asphalt Treated Free Draining	4	11
		304 Aggregate Base	4	
2	986 + 00	Cement Treated Free Draining	4	11
		304 Aggregate Base	4	
3	1049 + 00	Non-Stabilized Drainage Base-Type NJ	4	11
		304 Aggregate Base	4	
4	1056 + 00	Non-Stabilized Drainage Base-Type IA	4	11
		304 Aggregate Base	4	
5	1115 + 00	304 Aggregate Base	8	11
6	1132 + 00	304 Aggregate Base	6	13

Table 3.2 Description of Non-Stabilized Base Materials

Sieve Size	Base Type					
	Percent Passing (ODOT)			Percent Passing (Specification)		
	IA*	NJ**	304	IA*	NJ**	304
2 in.			100%			100%
1-1/2 in.			100%		100%	
1 in.	100%	100%	88%	100%	95-100%	70-100%
3/4 in.			75%			50-90%
½ in.	73%	74%	59%		60-80%	
3/8 in.						
No. 4		46%	49%		40-55%	30-60%
No. 8	24%	19%	32%	10-35%	5-25%	
No. 16		5%	24%		0-8%	
No. 30			18%			7-30%
No. 40						
No. 50	5%	3%	10%	0-15%	0-5%	
No. 70						
No. 200	3.5%	2.4%	6.4%	0-6%		0-13%

*IA (Iowa) Non-Stabilized Free Draining Base - Proposed Aggregate Blend of 50% No. 6 Limestone, 33.5% No. 9 Limestone and 16.5% Limestone Screenings

**NJ (New Jersey) Non-Stabilized Free Draining Base - Proposed Aggregate Blend of 50% No. 6 Limestone, 37.5% No. 9 Limestone, and 12.5% Limestone Screenings

3.3 INSTRUMENTATION

The locations of the instrumented sections were selected by personnel from the Ohio Department of Transportation (ODOT) and the Ohio Research Institute for Transportation and the Environment (ORITE). All instrumented sections were in the eastbound driving lane. Driving and passing lanes were 12 feet wide. Berms along the driving and passing lanes were 10 feet and 4 feet wide, respectively.

The foremost concern of this project was to ensure that the instrumentation would withstand installation and environmental factors, and still perform within the sensitivity range. The instrumentation was expected to endure elevated temperatures, compaction, moisture, and repeated heavy loading. High temperature (200-300°C) at installation and saturated moisture conditions over an extended time period were particular concerns. Instrumentation was selected to monitor the following: 1) pressure between the base and the subgrade layers; 2) pressure between the pavement and base layers; 3) deflection in the pavement wheelpath; 4) strain in the pavement wheelpath; 5) temperature profile within the pavement; and, 6) volumetric moisture content of the base and subgrade material.

3.3.1 Strain

Pavement strain was measured along the wheelpath with Dynatest PAST-IIAC strain gauges and the Hottinger Baldwin Messtechnik (HBM) DA 3 encapsulated strain gauges. Careful installation of each gauge was made to ensure the desired bonding of the asphalt concrete and gauge to obtain accurate readings. Service life was expected to exceed three years.

3.3.2 Deflection

Deflection of the asphalt concrete was measured using accelerometers and Linear Variable Displacement Transformers (LVDTs). The accelerometers were positioned in the asphalt concrete and the LVDTs were placed in a Single Layer Displacement (SLD) unit. The reference rod was placed deep enough (10 ft.) so displacement at that depth due to surface loading was negligible in the deflection measurement. Based upon field experience and elastic theory, reference rods should not be anchored less than 6 ft. below the top of the subgrade. The reference rod is then assumed to be stationary while the LVDT monitors pavement deflection. The SLD has the advantage over the accelerometer in that it can measure both static and dynamic deflections, as well as permanent deformations.

3.3.3 Moisture

The volumetric moisture content of the soil was measured with Time Domain Reflectometry (TDR) units developed by Campbell Scientific, Inc., of Logan, Utah. This process involves the sending of pulses along a coaxial cable to two parallel, 12-inch long stainless steel rods embedded in the soil, and observing the velocity of the reflected waveform. The velocity of waves traveling along a coaxial cable or waveguide is affected by the type of material surrounding the conductor. If the dielectric constant of the material is high, the signal propagates slower. Because the dielectric constant of water is much higher than (about 80 in water versus 0 in air), a signal within a wet or moist soil propagates slower than in the same soil when dry. Thus, moisture content can be determined by measuring the propagation time over a fixed length probe embedded in the soil beneath the pavement.

3.3.4 Temperature

Single point thermocouples, distributed by Measurement Instruments East, Inc., of Blairsville, Pennsylvania, were used on this project to monitor pavement temperature. Thermocouples were placed in the top and bottom layers of the asphalt concrete (the same layers as the strain gauges) and at the subgrade/base interface. Each thermocouple was sealed in a stainless steel tube for protection from moisture in the pavement system and high temperature of the asphalt concrete during installation.

3.3.5 Interface Pressure

Two types of pressure cells, distributed by GEOKON, Inc., of Lebanon, New Hampshire, were used to measure vertical pressure under the pavement. These included the Model 3650 strain gauge pressure cell and the Model 4800E vibrating wire pressure cell. Both cells were used to measure changes in pressure at the base/subgrade interface and at the pavement/base interface. The strain gauge pressure cells were used to measure pressure changes caused by both static and dynamic loading. The vibrating wire pressure cells were only used for static loading conditions. Both cells consist of two 9-inch diameter circular stainless steel plates, welded together and spaced apart by a narrow cavity filled with an antifreeze solution. External pressure acting on the cell was recorded with a data acquisition system. Both cells have an operating range of 0-30 psi.

3.4 INSTRUMENTATION LAYOUT AND PERFORMANCE

Sensors were installed in a straight line along the wheelpath of the driving lane where the largest values of strain, deflection, and pressure were expected to occur. Depths for the four LVDT reference rods were 10, 6, 4, and 2 feet. Different depths were used to compare the deflection of the pavement with the mid-point of various pavement layers. Strain gauges were installed one inch above the bottom of the pavement and one inch below the top of the pavement. Gauges were spaced 2½ feet apart along the wheelpath. After placement of the final layer of AC in the summer of 1994, the top gauges were approximately 2¼ inches from the top of the asphalt concrete. Both types of pressure cells were aligned vertically at the subgrade/ base interface and at the pavement/base interface. Pavement moisture was monitored six inches from the top of the subgrade and at the mid-point of the total base thickness with probes aligned horizontally.

Since most of these sensors were highly sensitive, extreme care was taken during installation to minimize erroneous data. However, since the installation procedures utilized in this project were new, some problems and losses were encountered. The condition of sensors at the completion of testing is shown in Table 3.3.

Table 3.3 Sensor Status at the Completion of Project

Section	Functional	Non-Functional
1	ACC SM01/02 HBM01 SGPC01/02 LVDTs TC01/02/03 VWPC02/01	DYN01/02 HBM02
2	ACC DYN01/02 HBM01/02 SGPC1/02 SM01/02 TC01/02/03 VWPC01/02	
3	ACC HBM01 LVDTs SGPC01/02 TC01/02/03 VWPC01/02	DYN01/02 HBM02 SM01/02
4	ACC DYN01 HBM01/02 LVDTs SGPC01/02 SM01/02 TC01/02/03 VWPC01/02	DYN02
5	ACC DYN01 LVDTs SGPC01/02 SM01/02 TC02/03 VWPC01/02	DYN02 HBM01/02 TC01
6	ACC DYN01/02 HBM01 LVDT 02/03/04 SGPC01/02 SM01/02 TC01/02/03 VWPC01/02	HBM02 LVDT01

Where, ACC = accelerometers, HBM = HBM strain gauges, DYN = Dynatest strain gauges, LVDT = Schlumberger Linear Variable Differential Transformers, VWPC = Geokon vibrating wire pressure cells, SGPC = strain gauge pressure cells, SM = soil moisture probes, and TC = Campbell Scientific thermocouples

It was discovered through the course of the project that the LVDTs were inoperative at temperatures below freezing. This was due to moisture freezing the core to the coil housing, resulting in no movement of the core and no change in voltage. When the cores were heated, the LVDTs functioned normally. LVDT data from the 10-foot deep rods were not available for Section 6 because the rod fell from the LVDT contact during installation. Attempts to adjust the contact point failed to give reliable readings.

The Dynatest gauge located at the top of the pavement in Section 1 was lost due to the burning of the outside gauge coating by hot asphalt concrete when the paver stopped over the gauge. The bridge balance was found to be out of the amplifier range. The HBM gauges in

Section 2 did not respond during the January test and the top HBM gauge did not respond in April. Prior to the April tests, the wiring to all instrumentation in Section 3 was cut while power lines to the section were being laid. The HBM gauges were spliced to their original wiring. The Dynatest gauges, however, could not be spliced together and produced no further data. Although strain data were collected for the remaining gauges, the splices cast doubt into the validity of the readings. There were not sufficient data available to plot the top Dynatest gauge. In Section 4, the top Dynatest gauge was non-responsive during all four FWD tests and the bottom gauge did not function in December. During the grading of the berm along Section 5, the wires to all instrumentation were severed. They were spliced together and testing was performed with satisfactory results. The top Dynatest gauge failed during the January tests. In Section 6, the top HBM gauge was lost during installation. The encapsulated strain gauge was rendered inoperative from hot asphalt concrete burning through the wire. It was discovered that the protective coating on the connecting wire to the HBM gauges stopped just short of the gauge. This left a small end of wire unshielded. Subsequent installation of HBM gauges was performed successfully using heat resistant shrink tubing placed over the exposed wire.

3.5 NONDESTRUCTIVE TESTING

Non-Destructive Testing (NDT) was performed with a Dynatest Model 8000 Falling Weight Deflectometer (FWD) applying an impulse load of 140 to 175 psi to simulate traffic loading. Measured deflections from the FWD were compared with response data collected from sensors installed in the pavement. Before each drop of the weight, seven geophones were lowered and placed on the pavement surface to measure surface deflection at 0, 8, 12, 18, 24, 26, and 60 inches from the center of the loaded area. After testing was completed, a hardcopy of all data was printed out, including the air temperature, all seven deflection measurements, the applied load pressure and exact load equivalent, station number, time of day, and sensor number where the load was applied.

Three FWD drops were applied over the strain gauges, pressure cells, accelerometers, LVDTs, and the Dynatest and HBM strain gauges. Their responses were monitored with the EGAA data acquisition system by manually triggering it to capture the dynamic response. The trigger was activated one second prior to the impact of the load on the pavement to ensure that the entire sensor response was obtained. A sampling frequency of 2000 data points per second was used throughout the tests.

Results of the FWD analysis show the cement treated base to be the stiffest of the five bases, with the asphalt concrete treated base being the most flexible at elevated temperatures. Analysis of the “304 type bases” (i.e., NJ, IA, 304) displayed varying results. Each base appeared best under specific conditions. Overall, the NJ base appeared to have the highest stiffness. Agreement between the FWD geophone and LVDT output showed that both measurement techniques are consistent and may be used interchangeably for particular applications. It appears that strain gauge damage does occur after repeated loading. At warm temperatures, aggregate particles may move and damage the strain gauges. The Dynatest gauges performed better than HBM and had fewer casualties. Gauges installed in the top and bottom of pavement were not the same because of temperature differences and because friction between base and asphalt concrete affected the strain field.

3.5.1 April 1994 FWD Data

In the April 1994, deflections basins calculated by finite element methods (FEM) matched FWD deflection basins recorded in Section 3 containing the NJ base. This confirms other recorded data where contact pressures of 4.1 and 6.5 psi were measured at the pavement/base and base/subgrade interfaces. These pressures were 5 and 6 times larger than the values measured in January 1995.

3.5.2 September 1994 Data

The September 1994 calculations obtained for Section 1 (Asphalt Treated Base) with OU-PAVE closely resembled FWD measurements in the field. However, the FEM calculations showed deflections approximately 1 mil less than the FWD measurements. The September field deflections were considerably less than the April 1994 measurements, while FEM results, based on input data for April, exhibited the same deflection profile. The September results for Section 5 (crushed 304) showed a localized basin, whereas, the FEM model predicted a wide shallow basin. Maximum deflections calculated by the FEM were 14.0, 8.2, and 9.2 mils for Sections 1, 3 and 5, respectively. Corresponding maximum measured geophone deflections were 12.2, 6.5, and 11.7 mils respectively.

3.5.3 January 1995 Data

While FWD deflection profiles for Section 1 (ATB), Section 3 (NJ) and Section 5 (304) in January 1995 matched the FEM calculations reasonably well, discrepancies existed between the measured and calculated maximum deflections. Maximum deflections measured for Sections

1, 3, and 5 were 6.3, 4.1, and 5.4 mils, whereas, the corresponding FEM maximum deflections were 6.4, 6.1, and 6.4 mils.

3.5.4 June 1998 Data

FWD measurements were conducted at 50-foot intervals along a 500-foot length of all six test sections to determine the relative stiffness of AC pavements with the various bases constructed on this pavement. Figure 3.1 shows the profiles of normalized DF1 measurements along the 500-foot lengths monitored and Table 3.4 summarizes the average section Df1 measurements obtained at loads approximating 9,000 lbs. In Section 5, a bridge deck between Stations 2+50 and 4+00 caused an interruption in the profile and the last point at Station 5+00 was not recorded in Section 2.

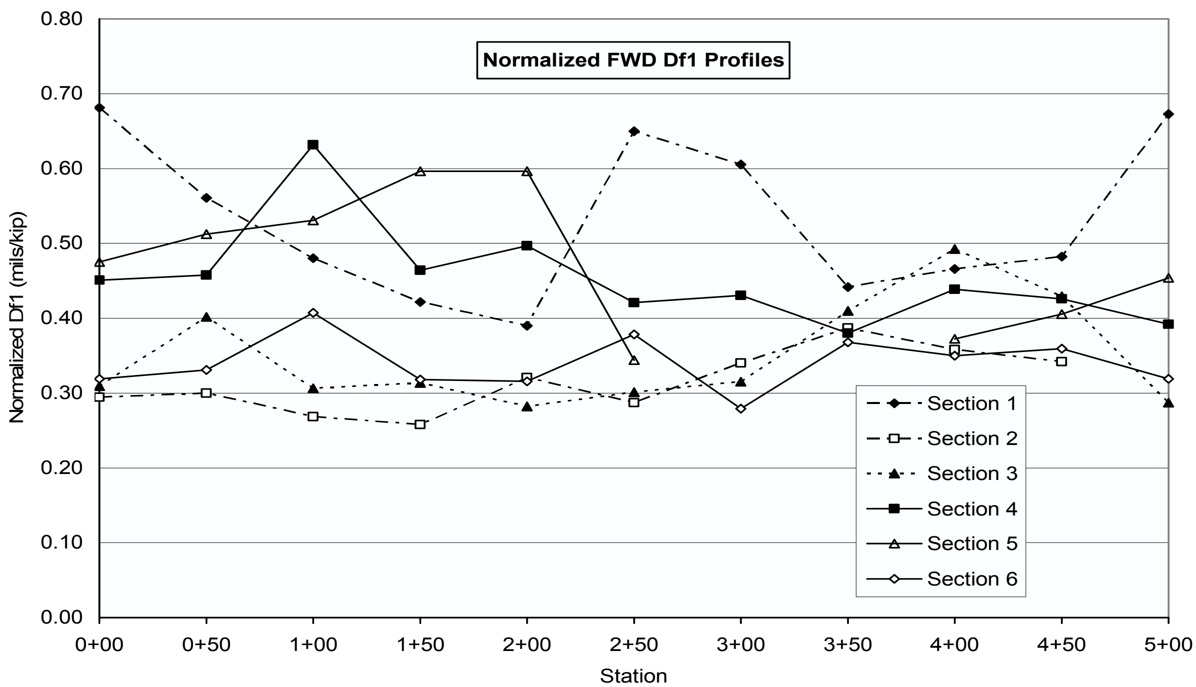


Figure 3.1 Normalized Df1 Profiles on LOG 33, June 1998

Table 3.4 Average FWD Responses on LOG 33, 6/17/98

Section	Pvt. Thick. (inches)	Base	Normalized Deflection (mils/kip)							SPR (%)
			Df1	Df2	Df3	DF4	Df5	Df6	Df7	
1	11	4" ATFDB/4" 304	0.53	0.42	0.37	0.31	0.26	0.18	0.09	58.51
2	11	4" PCTB/4'" 304	0.32	0.27	0.25	0.23	0.21	0.17	0.10	69.58
3	11	4" 304NJ/4" 304	0.35	0.28	0.25	0.22	0.18	0.13	0.06	59.72
4	11	4"304IA/4"304	0.45	0.38	0.34	0.28	0.22	0.14	0.05	58.99
5	11	8" 304	0.48	0.40	0.36	0.30	0.25	0.17	0.08	61.63
6	13	6" 304	0.34	0.28	0.26	0.22	0.19	0.14	0.08	63.43

In general, stiffer pavement structures are indicated by lower maximum normalized deflection (Df1) and higher Spreadability. Using this criteria, Section 2 with the 4” PCTB/4” 304 base is stiffest of the six sections, followed by Sections 3 and 6, Sections 4 and 5, and Section 1 being the least stiff of the six sections. Section 6 with 6” of 304 and a 13” AC pavement was stiffer than Section 5 with 8” of 304 and an 11” AC pavement.

3.6 ENVIRONMENTAL DATA

The environmental condition of the pavement, base and subgrade materials must be known at the time of testing for valid comparisons of data throughout the different seasons. The top and bottom pavement temperature, the temperature of the base material, and the volumetric moisture content of the subgrade and base were recorded during the FWD measurements.

First, the soil moisture probes were connected so the Campbell data acquisition system could initialize itself and obtain the moisture readings. After the system was initialized, the displayed moisture results were recorded on field notes for the section being tested. While the soil moisture was being read, each thermocouple was connected to an Omega HH21 and the corresponding temperatures were recorded in the field notes, along with soil moisture. This process took approximately 10 minutes per section. All instrumentation cables were individually coded in each section.

Table 3.5 displays the environmental conditions for each section during the FWD tests. SM-01 was installed mid-depth in the 304 base and SM-02 was installed six inches deep in the subgrade. This includes moisture in the base and subgrade, and temperature of the base and top and bottom layers of the asphalt concrete. Moisture in the subgrade and base stayed fairly constant in Sections 1 and 2 during the FWD tests. However, moisture in the remaining sections varied with the seasons. Moisture variations were not anticipated since the asphalt concrete was new and no cracking was observed. Water was expected to drain off the pavement with little to no infiltration, thus maintaining uniform moisture throughout the seasons. This was true in Sections 1 and 2, but moisture levels changed in Sections 3, 4, and 5. Moisture levels appeared to correlate with section elevations.

The average elevations of the test sections above sea level were as follows: Section 1 – 1461 feet; Section 2 – 1411 feet; Section 3 – 1320 feet; Section 4 – 1410 feet; Section 5 – 1166 feet; and, Section 6 – 1165 feet. The two-foot long LVDT rods measured deformation in the pavement and base layers under the application of FWD loading. As expected, Section 2 had the lowest deflection and moisture among the sections tested. Moisture remained between 7.4 and 15% by weight except for Section 5 where the moisture was 17.6%. The two-foot deflections appeared to be related to seasonal conditions and base type rather than moisture content. Subgrade moisture in Sections 1 and 2 was constant. Total pavement deflection appeared to be related more to temperature and pavement type than moisture

Table 3.5 Environmental Data for Asphalt Concrete

Test Month	Section	Soil Moisture Probe				Temperature (°F)		
		Volumetric Moisture (%)		Weight Moisture (%)		Thermocouple		
		SM-01	SM-02	SM-01	SM-02	TC-01	TC-02	TC-03
December 1993	1	14.7	22.3	8.1	9.6	34	32	39
	2	32.2	17.9	17.8	7.7	33	33	34
	3	23.9	21.7	13.2	9.3	35	32	34
	4	23.3	34.9	12.9	15.0	33	31	32
April 1994	2	32.5	16.1	18.0	6.9	63	66	71
	3	*	*	*	*	63	67	75
	4	31.7	28.4	17.5	12.2	65	71	84
September 1994	1	15.2	16.8	8.4	7.2	66	62	63
	2	31.9	16.8	17.6	7.2	65	63	64
	3	*	*	*	*	68	66	68
	4	40.4	22.9	22.3	9.8	68	69	71
	5	42.8	41.9	23.6	18.0	69	70	73
	6	30.7	26.6	17.0	11.4	68	70	73
January 1995	1	15.0	31.9	8.3	13.7	33	37	37
	2	31.4	16.5	17.4	7.1	35	35	37
	3	*	*	*	*	38	37	38
	4	29.8	28.2	16.5	12.1	35	37	38
	5	32.9	41.5	18.2	17.8	36	37	38

* Nonfunctional

Larger pavement deflections, pressures, and strains were measured during the warmer months of April and September than in December and January. During these tests, temperature profiles were uniform in the pavement. In April, the temperature at the surface of the asphalt concrete was warmer than the bottom of the asphalt concrete. In Section 4, this difference was 13°F. Temperature was the most significant factor affecting deflection, pressure, and strain measurements on this flexible pavement. The modulus of asphalt concrete changes significantly with temperature. In the range of temperatures observed during the tests, the modulus of asphalt concrete varied from 1800 ksi in December 1993 to 200 ksi in April 1994. In addition, temperatures of 32°F measured at the pavement/base interface indicated there may have been some frozen material present during the December tests.

3.7 CORRELATION OF FWD DATA AND FEM MODELS

To validate the accuracy of finite element models (FEM) for simulating pavement response to dynamic loading, surface deflections calculated with OU-PAVE, a FEM program developed at Ohio University, were compared to deflection measurements obtained with the FWD. Temperature and moisture conditions measured at the time of the tests were combined with material properties determined in the laboratory to provide stiffness data required in OU-PAVE for the various pavement layers. Table 3.6 summarizes these data.

Table 3.6 Comparison of Laboratory and Backcalculated Layer Moduli

Material	Modulus (psi)	
	Laboratory	FWD
Asphalt Concrete	678,000	527,000
Asphalt Treated Base	10,000	52,000
Cement Treated Base	2,800,000	1,096,000
New Jersey Base	16,000	61,000
Iowa Base	15,000	6,000
Standard 304 Base	15,000	18,000

3.8 ACCUMULATED TRAFFIC

Figure 3.2 shows the accumulated number of ESALs calculated from a weigh-in-motion scale located in eastbound US 33 at SLM 19.87 in Logan County.

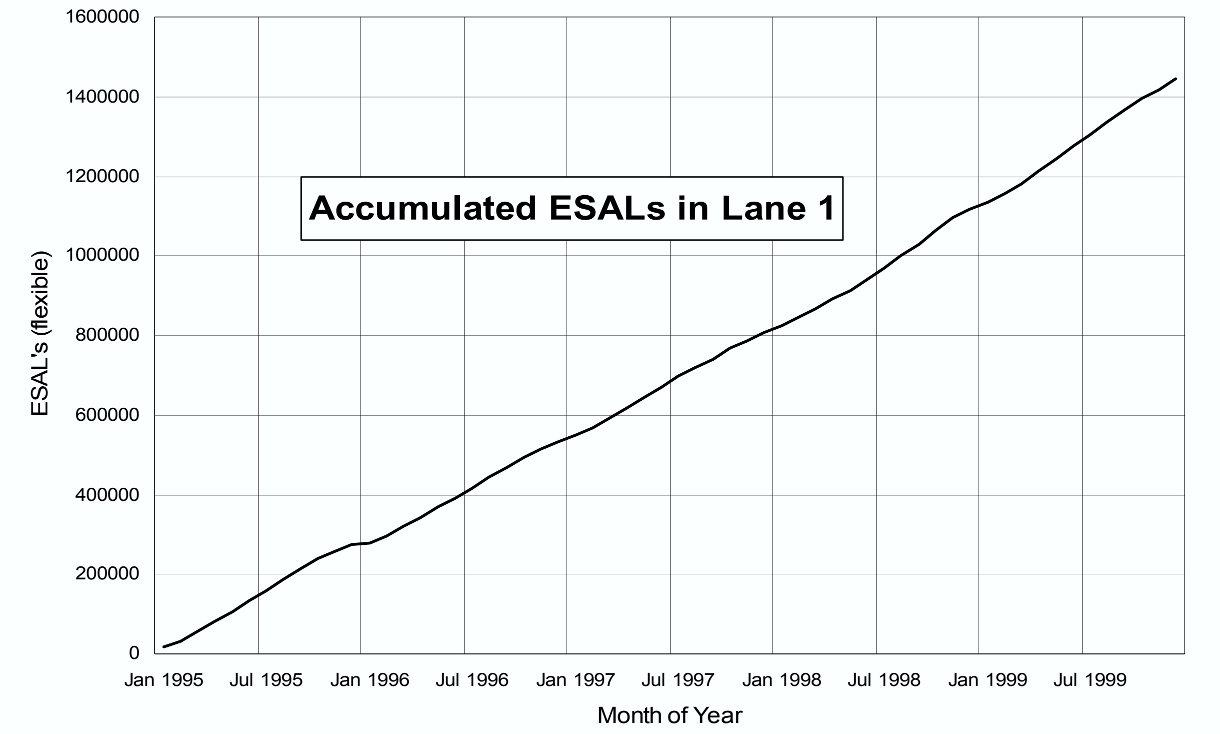


Figure 3.2 Accumulated ESALs on LOG 33 - Lane 1

3.9 CONDITION SURVEY

A condition survey was performed at the Logan 33 site in June of 2000. The most visible distresses in Sections 1 and 2 were longitudinal cracks 0.25 to 0.50 inches wide running along the C/L of the pavement, and along the middle of the driving lane. These sections also showed some minor transverse cracking from the outside edge of the driving lane towards the center. These cracks did not extend more than six inches. Section 3 only contained some minor longitudinal cracking. Section 4 had some minor longitudinal and transverse cracking. Section 5 had a 0.25 to 0.50 inch wide longitudinal crack running the length of the section at the C/L of the pavement, and some small transverse cracks from the edge of the driving lane towards the center of the pavement. Section 6 did not show any distress.

3.10 CONCLUSIONS AND RECOMMENDATIONS

Of the five types of base material used on Route 33 and evaluated for their effect on pavement performance, deflection measurements on the asphalt treated base fluctuated most with changes in temperature. None of the other bases were sensitive to temperature. Cement treated base had the lowest deflection. On unbound material, bases containing large size stone gave the lowest deflection. Before making any final conclusions on the relative performance of each base, however, they must be observed for several years to determine their ability to resist long-term traffic loading. Of particular interest is the stability of the asphalt and cement treated bases over long periods of time.

Subgrade moisture remained fairly constant throughout this study because the asphalt concrete pavement was new and resisted the infiltration of surface water; however, this parameter should be monitored as the pavement ages. The effectiveness of the base materials in removing water also should be investigated as the pavement ages. Even though the cement treated base reduced displacement, long-term performance should be evaluated by conducting periodic distress surveys. This is a very important point because the performance of asphalt and concrete pavements is affected by the type of base material placed under them.

Pressure readings were consistent with deflection measurements in that larger values were recorded during April and September than in December and January. FWD tests on ATFDB in September yielded a smaller pressure at the pavement/base interface than on the other bases. The highest pressures were recorded at the pavement/base interface on the PCTB during September and April. Data obtained from the December tests showed compressive and tensile strains measured in the ATFDB to be larger than strains in the other sections. This supports conclusions drawn from the deflection and pressure data that, even during the colder months, ATFDB was the most flexible of the bases tested.

An attempt was made in this investigation to verify the usefulness of OU-PAVE in calculating pavement deflection and strain. This program can be utilized for back calculation when material constants are available from nondestructive testing. Using an axisymmetric mesh configuration, FEM deflection profiles agree reasonably well with FWD measurements. Based upon this agreement, the FEM pavement model can be used with high confidence. The results of this investigation concerning future instrumentation, data collection, and data analysis of flexible pavements should be implemented according to the following recommendations:

- The installation procedure used here to install pavement sensors in flexible pavement was successful and should be followed in future investigations.
- The single anchor LVDT was reliable for measuring FWD induced deflections on flexible pavement.
- Pavement design procedures should continue to account for high contact pressures that occur on the subgrade during the warmer months.

- A nonlinear model for calculating asphalt concrete response should be formulated for dynamic loading.
- The long term performance of base materials under this asphalt concrete pavement should be monitored for several years to determine how the permeability of base material changes with time, and how the distress of asphalt concrete pavement changes with respect to time and type of base material.

3.11 ADDITIONAL RESEARCH:

A dissertation was completed on this same project by Sedtha Mahasantipiya of Ohio University as partial fulfillment of his doctorial degree. A synopsis of this dissertation is included in Appendix G.

4.0 EFFECTIVENESS OF BASE TYPE ON THE PERFORMANCE OF PCC PAVEMENT ON ERI/LOR-2

4.1 INTRODUCTION

In 1974, an experimental section of pavement was constructed on SR 2 in Erie and Lorain Counties near Vermilion, Ohio to determine the effect of various materials and design features on the occurrence of D-cracking in Portland cement concrete (PCC) pavements. The major findings of this research were that: (1) certain coarse aggregates fracture more readily than others when exposed to moisture and freeze-thaw cycling and (2) when the size of susceptible aggregates is reduced from #57 to #8, D-cracking is significantly reduced. However, the frequency of transverse slab cracking appeared to increase with this smaller aggregate.

In September 1993, nine slabs in the westbound lane between Stations 114+00 and 124+00 in Lorain County were replaced with embedded instrumentation to monitor vertical pavement deflection, pavement strain, pavement temperature, base and subgrade moisture, and pressure at the slab/base interface. A research project entitled, "Instrumentation of a Rigid Pavement System," was initiated on March 9, 1992 to document the findings of this effort.

Another set of test sections was constructed under Project 6000(92) in 1993 in the westbound lanes of Erie and Lorain Counties for the project entitled "Effectiveness of Base Type on the Performance of PCC Pavement on ERI/LOR 2." The objectives of this project were to investigate the effects of base type on D-cracking, slab length on transverse slab cracking, and natural versus manufactured sand on skid resistance. Although D-cracking has not been observed to date in these test sections, significant transverse cracking developed in sections with particular combinations of base type and joint spacing. An interim report was published in April 2000 to document these observations. Highlights of the interim report are presented here with additional information from a crack survey in April 2002.

4.2 PROJECT LAYOUT

A matrix consisting of six base types and two coarse aggregate sources for a 10-inch thick PCC pavement was established to address the effect of these parameters on D-cracking. The pavement contained reinforcing mesh to control the growth of any transverse cracks that might occur. One of the coarse aggregates was resistant to D-cracking and the other was susceptible to D-cracking. #57 coarse aggregate from Martin-Marietta in Woodville, Ohio was selected as the source of D-cracking resistant aggregate and #57 coarse aggregate from Sandusky

Crushed Stone in Parkertown, Ohio was selected as the source of D-cracking susceptible aggregate. A joint spacing of 13 feet was used with the Parkertown coarse aggregate and a joint spacing of 25 feet was used with the Woodville coarse aggregate.

Base materials included ODOT 304 and 310 (both dense graded aggregate), ODOT 307IA and 307NJ (both unstabilized drainable aggregate), and asphalt and cement treated free-draining bases. Test sections were located in the westbound lanes of SR 2 between Station 1835+10 in Erie County and Station 90+23 in Lorain County. Limits of the individual sections are shown in Table 4.1.

Table 4.1 Summary of Test Section Parameters

Station Limits		Base/Subbase		Joint Spacing (ft.)	PCC Coarse Aggregate
Begin	End	Thickness (in.)	Type		
1835+10	0+01	4/6	310/304	13	Parkertown (D)
0+01	5+00	4/6	310/304	25	Woodville (ND)
5+00	9+80	4/6	307IA/304	25	Woodville (ND)
9+80	56+06	4/6	307IA/304	13	Parkertown (D)
56+06	60+33	4/6	304/304	13	Parkertown (D)
60+33	64+60	4/6	304/304	25	Woodville (ND)
64+60	68+87	4/6	307NJ/304	25	Woodville (ND)
68+87	73+14	4/6	307NJ/304	13	Parkertown (D)
73+14	77+41	4/6	ATFDB*/304	13	Parkertown (D)
77+41	81+68	4/6	ATFDB*/304	25	Woodville (ND)
81+68	85+96	4/6	CTFDB**/304	25	Woodville (ND)
85+96	90+23	4/6	CTFDB**/304	13	Parkertown (D)

(D) D-cracking susceptible (ND) D-cracking resistant
*Asphalt treated free draining base **Cement treated free draining base

4.3 PCC Mix Design

Tables 4.2 and 4.3 provide aggregate gradations and mix designs used in the 451 mesh reinforced PCC pavement as obtained from test reports issued at the time of construction, and ODOT specifications applicable at the time. The mesh was W8.5 x W4 – 6 x 12 smooth steel wire and the concrete joints were square.

Table 4.2 Gradation of Aggregates in PCC Pavement

Sieve No.	% Passing							
	#57 Ls Coarse Aggregate			Fine Aggregate				
				Natural Sand*		Man. Ls Sand**		Spec.
	D	ND	Spec.	41P	42P	2D	7D	
1 ½”	100	100	100					
1”	100	100	95-100					
¾”	77	76						
½”	34	27	25-60					
3/8”	14	8		100	100	100	100	100
4	1	1	0-10	100	100	100	100	95-100
8	1	1	0-5	95	96	79	89	70-100
16	1	1		69	73	38	47	38-80
30				34	39	17	22	18-60
40				20	25	12	14	
50				11	15	8	9	5-30
70				6	8	5	5	
100				4	5	4	4	1-10
200				2.0	2.4	3.0	2.6	0-5
Spec. Gravity	2.62	2.69		2.57	2.57		2.64	
Absorption (%)	1.77	1.55		1.56	1.56		1.45	
Fineness Modulus						3.54	3.29	

* Two samples of natural sand supplied by Norwalk Sand and Gravel in Norwalk, Ohio

** Two samples of manufactured limestone sand supplied by Sandusky Crushed Stone in Parkertown, Ohio

Table 4.3 Mix Designs for PCC Pavement

Material	451 PCC Pavement (corrected lbs./cu.yd.)	
	D*	ND**
#57 Coarse aggregate	1635	1637
Natural sand fine aggregate	1242	1242
Type I cement (M. B. Guran)	510	510
Class “F” flyash (Avon Lake)	90	90
PCC Admixtures		
Air (Axim Caterol AE 260)	15 oz.	15 oz.
Range of water/cement ratio	43-50	41-49

* PCC with D-cracking susceptible coarse aggregate

** PCC with D-cracking resistant coarse aggregate

4.4 BASE MATERIALS

Table 4.4 is a summary of the gradation of aggregates used in the various bases on this project. ODOT permits the use of #57 or #67 aggregate in asphalt and cement treated free draining bases. #57 was used on this project.

Table 4.4 Gradation of Base Courses

Sieve No.	% Passing											
	304*	Spec.	307IA	Spec.	307NJ*	Spec.	310*	Spec.	ATFDB*	CTFDB**	#57	#67
2”	100	100					100	100				
1 ½”	100					100	100		100	100	100	
1”	92	70-100	100	100	100	95-100	100	100	100	100	95-100	
¾”	86	50-90	91		93		100		87	82		90-100
½”	73		56	50-80	65	60-80	100		37	30	25-60	
3/8”	65		36		49		100	80-100	8	5		20-55
4	44	30-60	31		42	40-55	100	60-100	1	1	0-10	1-10
8	22		25	10-35	14	5-25	83	45-85	1	1	0-5	0-5
16	22		14		4	0-8	83		1	1		
30	10	7-30	7		2		83					
40	10		5				16	15-50				
50	10		3	0-15	1	0-5	16					
70			2									
100	10		2				16					
200	6.6	0-13	1.3	0-6			2.3	0-10				

* Supplied by Wagner Quarries in Sandusky, Ohio
** Supplied by Sandusky Crushed Stone in Parkertown, Ohio

The 304 and 310 base materials were placed as delivered from Wagner Quarries. The 307 Type IA base was manufactured by blending 70%, #57 limestone with 30% limestone sand. Moisture was between 5.5 and 6.3% and no water was added at the plant. The 307 Type NJ base was manufactured by blending 55%, #57 limestone with 45%, #9 limestone. Moisture in this material was between 4.0 and 5.0% and, again, no water was added at the plant. The stabilized bases were 100%, #57 aggregate with either asphalt cement or Portland cement added to bind the stone together. The following table shows mix designs for the stabilized bases.

Table 4.5 Mix Designs for Stabilized Base

Material	CTFDB (lbs./cu.yd.	ATFDB (% by wt.)
#57 Coarse aggregate	2580	97.7
Type I cement (M.B. Guran)	220	
AC-20 Asphalt cement		2.3
Water reducer (Axim Type A)	4.40 oz.	

4.5 TRAFFIC LOADING

Table 4.6 is a summary of total monthly traffic loading beginning in 1994 on the two westbound lanes of SR 2 at the location of this experimental pavement in terms of ESALs.

Table 4.6 Monthly ESAL Counts

Year	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
1994	25632	36692	44115	44585	45754	48110	37024	46048	41542	40746	39950	36247	486445
1995	27682	38159	45879	46368	55416	53976	42882	49581	50525	51904	43497	38945	544814
1996	42127	45584	56398	76082	59770	72389	50995	76699	69046	77487	63654	59028	749259
1997	61617	69495	84055	80790	86619	84973	85037	81642	81175	88385	82758	84006	970552
1998	61432	70851	73012	71800	89769	76917	86708	87717	89523	81084	67044	67998	923855
1999	61248	72207	61969	59544	63960	89950	83726	82921	79092	81668	89684	90548	916517
2000	77150	66355	66111	67150	94900	95035	77093	92729	71902	83486	77675	48013	917599
2001	71669	60502	70252	74756	81966	88908	81739	82749	58647	98008	79404	66357	914957

A significant increase in ESAL loading in March 1997 may have been about the time tolls were raised and construction was in progress on the Ohio Turnpike. In general, traffic loading was higher during the spring and summer months than during the fall and winter. Figure 4.1 shows the number of ESALs accumulated on this pavement since the test sections were constructed.

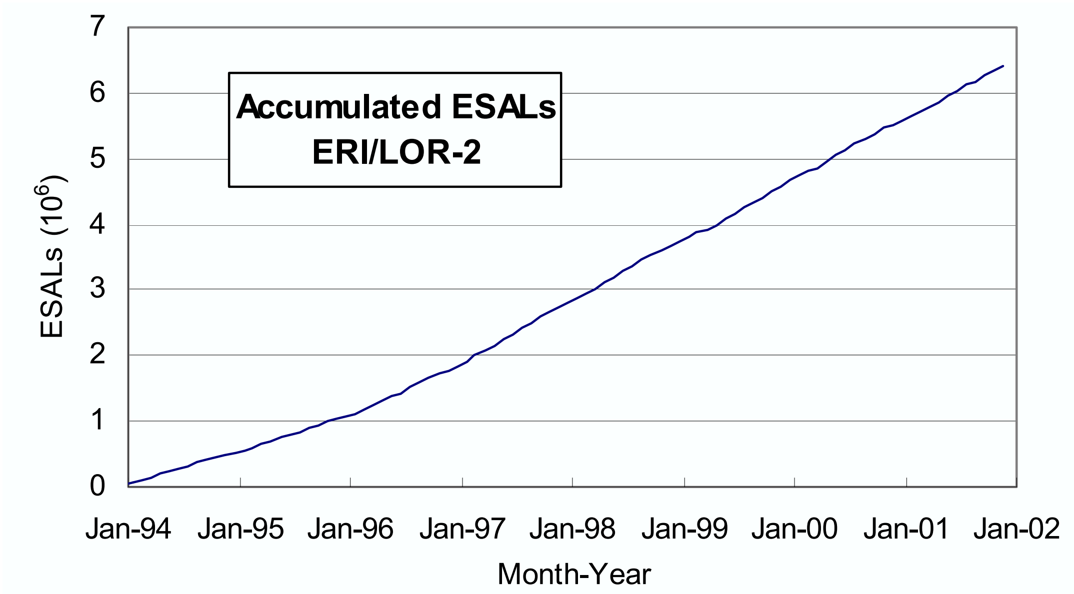


Figure 4.1 Accumulated ESALs on ERI/LOR 2

4.6 Laboratory Testing

Laboratory tests were performed on three of the unstabilized base materials used in this field evaluation; ODOT 307NJ, 307IA and 304. Because the in-situ base has been in service for several years, it was likely contaminated with fines from the underlying subgrade. Virgin aggregate was purchased for this project from the same sources and blended to match sample gradations obtained at the time of construction. Samples were compacted equally using the modified Proctor method. Triaxial and resilient modulus tests were performed to confirm, and possibly explain, differences in measured and observed performance in the field. No tests were performed on the ATFDB or CTFDB materials. Permeability tests on the unstabilized materials were conducted by the University of Toledo and documented in a report entitled "Permeability and Stability of Base and Subbase Materials." Because this was a test of the effect of base type on D-cracking, no laboratory tests were performed on the concrete mix or the aggregate in the concrete.

4.6.1 Triaxial Testing

Moisture was determined as samples were being prepared for testing. All specimens were nominally six inches in diameter and twelve inches long. Table 4.7 summarizes the results of triaxial tests performed on these materials.

Table 4.7 Summary of Triaxial Test Results

Sample No.	Moisture (%)	Dry Dens. (pcf)	Confining Stress (psi)	Deviator Stress @ Failure (psi)	Axial Strain @ Failure (%)	Angle of Internal Friction (degrees)
NJ-1	1.94	105.5	5.0	34.0	4.8	50.6
NJ-2	2.12	106.6	6.0	36.5	5.2	48.8
Avg.	2.03	106.1	5.5	35.3	5.0	49.7
IA-2	2.95	114.1	5.0	31.5	7.0	49.4
IA-3	3.90	111.9	6.0	39.0	6.8	49.9
Avg.	3.43	113.0	5.5	35.3	6.9	49.7
304-1	5.35	116.9	5.0	25.0	6.5	45.6
304-2	5.16	117.1	7.0	33.5	5.0	44.6
Avg.	5.26	117.0	6.0	29.3	5.8	45.1

Several trends can be observed from these data, including the following:

1. Moisture was lowest in the 307 NJ base, probably because of the lack of fine-grained material.
2. Density was significantly lower in the 307 NJ base, because of the lack of aggregate passing the #8 sieve to fill voids between the larger aggregate particles.
3. Axial strain at failure was highest for the 307 IA base and lowest for the 307 NJ base, with the 304 base falling in between. This suggests lower shear strength in the 307 NJ base.

4.6.2 Resilient Modulus Testing

Resilient modulus testing at the Ohio Research Institute for Transportation and the Environment (ORITE) was performed in accordance with SHRP LTPP protocol using a large triaxial chamber, an electro-servo controlled actuator, and computerized command generation and data acquisition. Test specimens were nominally six inches in diameter, twelve inches long and weighed approximately 22 - 26 lbs. Resilient moduli were determined at three deviatoric stresses applied at confining stresses of 3, 5, 10, 15 and 20 psi. Table 4.8 summarizes moisture, dry density, permanent strain and modulus constants measured on each sample of base material. K and n are constants obtained from a linear best-fit line drawn for all confining and deviatoric stresses on that sample on a log-log plot. The values of r^2 indicate how well the line represents these data. M_R shown for the five deviatoric stresses were calculated from the equation of that line.

Table 4.8 Summary of Resilient Modulus Tests

Sample No.	Moisture (%)	Dry Dens. (PCF)	Perm. Strain (%)	M_R (psi) = K $(\sigma_d)^n$			M_R (psi) @ Deviatoric Stress of:				
				K	n	r^2	2 psi	5 psi	10 psi	15 psi	20 psi
NJ-3	2.21	107.2	.71	2690	.487	.790	3770	5890	8256	10058	11571
NJ-4	2.64	107.6	.39	2074	.502	.840	2937	4653	6589	8076	9331
NJ-5	1.54	109.8	.42	2563	.498	.850	3619	5712	8068	9873	11395
NJ-6	na	na	.61	3340	.349	.621	4254	5858	7461	8595	9502
Avg.	2.13	108.2	.53				3645	5528	7594	9151	10450
IA-5	3.90	116.8	.97	2645	.448	.780	3608	5440	7420	8898	10122
IA-6	2.85	118.8	.44	3172	.305	.774	3919	5182	6402	7245	7910
IA-7	4.68	117.0	.74	2334	.511	.797	3326	5313	7571	9314	10789
IA-8	3.17	119.8	.71	2000	.566	.868	2961	4974	7364	9264	10901
Avg.	3.65	118.1	.72				3454	5227	7189	8680	9931
304-5	3.65	112.0	.76	3348	.440	.806	4542	6798	9222	11024	12512
304-6	na	na	.57	2175	.557	.839	3199	5330	7841	9828	11536
Avg.	3.65	112.0	.67				3871	6064	8532	10426	12024

Apparent trends from these data are as follows:

1. As in the triaxial tests, the 307 NJ base had the lowest moisture content and lowest dry density.
2. Permanent strain measured for these three unstabilized materials were similar, especially when considering the range of strain measured for each base type.
3. Considering the variation of M_R within each base type, the averages shown are about the same with both the 307 IA and 307 NJ being slightly less than the 304.

4.6.3 Laboratory Summary

From tests conducted in the laboratory, the stiffness characteristics of 304, 307IA and 307NJ unstabilized base materials appear to be quite similar. However, the 307 NJ base was particularly difficult to compact in the laboratory, apparently due to the presence of large angular particles, and contractors have remarked about how difficult it is to compact in the field. For these reasons, laboratory test results for 307NJ base may be less representative of field placed 307NJ base than laboratory tests for other unstabilized materials.

As repeated traffic loads are applied in the field, the lack of fine-grained material in the 307 NJ base could permit some reorientation of aggregate particles and, hence, densification of the base layer. Densification will result in voids being created under the PCC slab and a loss of support, especially at joints as slabs curl and warp during curing and environmental cycling. This loss of support will result in higher stresses being induced in the slab, thereby increasing the probability of transverse cracking.

4.7 CRACK EVALUATION

While the principal objective of the 6000(92) project was to determine the impact of base type on D-cracking, none of the test sections have exhibited any of these symptoms to date. However, a number of unexpected transverse cracks were observed in certain sections. As shown in Table 4.9, sections with a 25-foot joint spacing and a CTFD, 307NJ, 307IA, 310 or 304 base, and the section with a 13-foot joint spacing and the 307NJ base have extensive transverse cracking and some longitudinal cracking after a few years of service. None of these cracks appeared soon after construction and, therefore, were not attributed to conditions existing at the time of placement. Slabs with a 13-foot joint spacing on any base except 307NJ, and the 25-foot joint spacing on ATFD base have performed reasonably well to date. The test sections in this table are listed in order of increasing number of cracks per slab in 1999 and are grouped into three levels of performance. These cracks were large enough to be easily seen when walking along the pavement.

Table 4.9 Transverse Cracking Survey on ERI/LOR 2

Base Type	Joint Spacing (ft.)	June 1999			April 2002		
		No. Slabs Observed	Total No. Trans. Cracks	Avg. No. Cracks/ Slab	No. Slabs Observed	Total No. Trans. Cracks	Avg. No. Cracks/ Slab
ATFDB	13	33	0	0.00	32	2	0.06
304	13	33	1	0.03	31	8	0.26
310	13	23	2	0.09	24	2	0.08
ATFDB	25	16	3	0.19	18	14	0.78
CTFDB	13	16	3	0.19	34	10	0.29
307IA	13	36	7	0.19	36	15	0.42
307NJ	13	20	19	0.95	33	29	0.88
304	25	17	17	1.00	17	28	1.65
310	25	20	22	1.11	20	24	1.20
307IA	25	19	23	1.21	19	24	1.26
307NJ	25	17	34	2.00	17	39	2.29
CTFDB	25	17	41	2.41	16	44	2.75

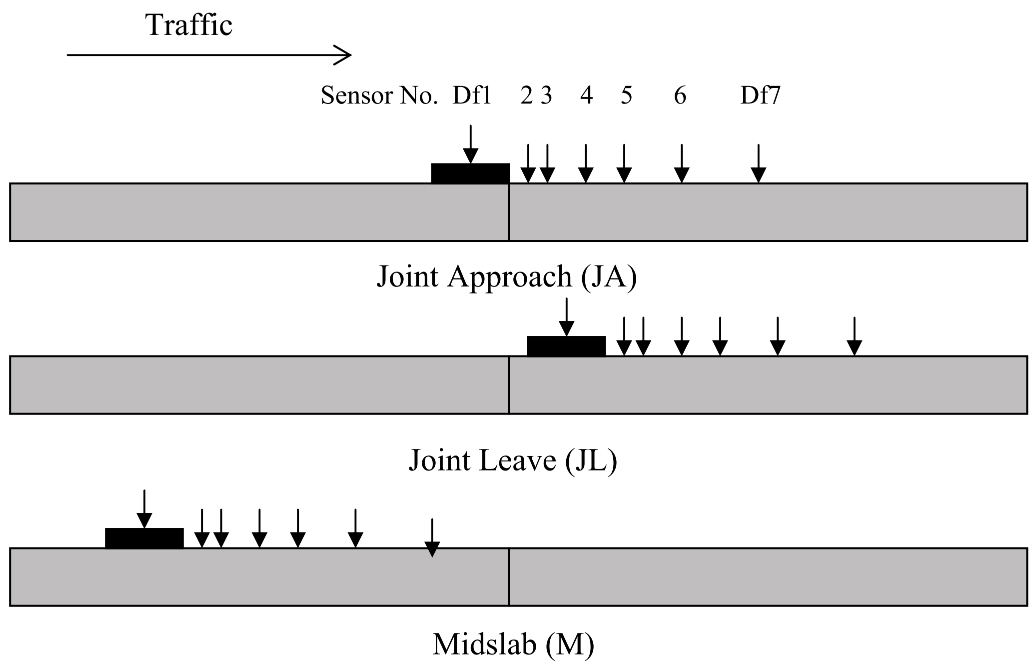
The number of slabs shown in some sections was different between June 1999 and April 2002 because of the interpretation of section limits by observers. Only transverse cracks were included in the table. Generally, slabs with several transverse cracks also contained some longitudinal cracks. The correlation between base type and number of cracks per slab seems to have remained the same in 2002 as in 1999 except that, perhaps, the ATFDB with 25 foot joint spacing might better be placed with the second performance group in 2002. Overall, the 13 foot slabs are performing much better than the 25 foot slabs on all bases, the CTFDB should not be used with 25 foot slabs, and the 307NJ base should not be used at all under PCC pavement.

4.8 NONDESTRUCTIVE TESTING

Nondestructive testing was performed in June and August of 1999 with the ODOT Dynatest Falling Weight Deflectometer (FWD) to determine the stiffness characteristics of PCC slabs constructed on different base materials. Included in this evaluation was an examination of how well the transverse contraction joints in these sections transferred load to adjacent slabs. The results of these evaluations are presented in the following sections. All tests were performed in the right wheelpath of the driving lane.

4.8.1 June 29, 1999 Tests

In this set of FWD measurements, a few slabs were selected for testing in each section containing a particular combination of joint spacing and base type. The load plate was placed on both sides of the joints and at one or more locations along the interior of the slab. In these configurations, the geophones measuring vertical surface deflection were located as shown in Figure 4.2. Readings were initiated at 8:40 am at which time the surface temperature of the pavement was 69º F and the pavement gradient would be expected to be minimal.



Sensor spacing from center of load plate: 0, 8, 12, 18, 24, 36 and 60 inches

Figure 4.2 FWD Load Plate and Sensor Positioning – June 1999

Df1 deflections measured with the FWD load plate located in the middle portion of the slab reflect the composite vertical stiffness of the entire layered pavement structure, including the pavement, base and subgrade. When slabs are cracked, there is likely to be some reduction in stiffness. Every effort was made to have the FWD load plate and all geophones on an uncracked section of pavement; otherwise, there would likely be a discontinuity in the FWD deflection profile. Deflections measured with the FWD load plate located near a joint are indicative of the vertical stiffness of the slab ends at the time of the measurements. The presence of temperature and/or moisture gradients in PCC slabs causes the slab ends to curl and warp at the ends, thereby affecting the degree to which they are supported by the underlying layers. Therefore, the stiffness of slab ends can be low in the morning when they are curled upward and acting as a cantilever, and high in the afternoon as the pavement surface warms and brings the slab back into contact with the base layer. Once in contact with the base, slab end stiffness at joints is affected by moisture conditions in the base and subgrade around the joints.

Load transfer mechanisms, such as aggregate interlock and/or dowel bars, increase the stiffness of PCC slab ends by transferring vertical shear and horizontal bending forces to adjacent slabs. When the pavement is warm (> ~70º F), PCC slabs typically are sufficiently expanded horizontally to be in contact with neighboring slabs. The irregular aggregate surfaces at the slab ends then interlock and transfer load across the joint. At lower temperatures, load transfer will become less as slabs contract and aggregate interlock is lost. Dowel bars also improve stiffness and the magnitude of load transfer at joints under all temperature and moisture conditions.

If free water is present under the slab, fine material may be removed from the subgrade and/or base by the process of pumping as heavy traffic loads force the slab ends downward and expel this water containing suspended fines up through a joint or crack. When pumping occurs, material is generally removed more from under the leave side of joints and cracks, and FWD deflections there are higher than on the approach side. Severe pumping often leads to slab faulting where the leave slab drops below the approach slab.

Load transfer across PCC joints and cracks can be quantified with the FWD by placing the load plate and sensors in the joint approach position shown in Figure 4.1, and comparing deflection measured at the center of the load plate with deflection measured on the unloaded slab. The second sensor is sometimes moved to a position 12 inches behind the center of the load plate to measure load transfer in the joint leave position. For consistency, therefore, the third sensor will be used to calculate load transfer at all times in the joint approach position (LT_A = Df3/Df1) and the second sensor will be used to measure load transfer in the joint leave position (LT_L = Df2/Df1) when it is placed behind the load plate. In these equations, Sensors 2 and 3 are the same distance from the load plate. While load transfer, as defined at joints and cracks, is not a relevant term in the middle of an uncracked slab, the ratio of Df3/Df1 at midslab is indicative of slab bending stiffness and can be used to further refine the assessment of load transfer.

For example, if the average Df3/Df1 ratio is 0.70 at midslab and 0.65 at the joints on Pavement 1 and 0.85 at midslab and 0.70 at the joints on Pavement 2, which pavement has better load transfer at the joints? Pavement 1 does not distribute load as well as Pavement 2 at midslab, as indicated by the lower Df3/Df1 ratio. The joints on Pavement 1 have an average stiffness across the joints equal to 0.65/0.70 = 0.93 or 93% of the midslab bending stiffness. The joints on Pavement 2 have an average stiffness of 0.70/0.85 = 0.82 or 82% of the midslab bending stiffness. Therefore, while the joints on Pavement 2 have a higher magnitude of load transfer, they have lost more of their potential ability to transfer load, assuming that the Df3/Df1 ratio at midslab and at the joints was approximately equal when the pavement was new. Table 4.10 shows a summary of FWD deflection measurements collected during the June 1999 evaluation.

Values shown in the table were obtained at loads approximating 9000 lb. and normalized to a 1000 lb. load.

Table 4.10 Summary of June 1999 FWD Measurements

Base Type	Avg. Norm. Df1 Deflection in FWD Position (mils)			Load Transfer (Df3/Df1) in FWD Position (%)	
	JA	JL	M	JA	M
13-foot Joint Spacing					
304	0.51	0.70	0.25	41.6	77.1
310	0.66	0.60	0.35	49.2	87.3
307NJ	1.42	2.32	0.73	5.5	84.8
307IA	0.51	0.53	0.30	103.2	85.7
ATFDB	0.55	0.32	0.20	43.2	82.5
CTFDB	2.09	1.17	0.49	6.6	92.8
25-foot Joint Spacing					
304	0.57	1.09	0.36	48.8	82.0
310	0.73	0.52	0.33	28.1	88.5
307NJ	1.32	1.28	0.69	7.1	89.4
307IA	0.51	0.56	0.36	62.2	82.2
ATFDB	0.24	0.28	0.11	61.6	76.6
CTFDB	0.37	0.43	0.27	91.9	86.0

Several interesting observations can be made from Table 4.10, as follows:

1. With the exception of slabs on CTFD base, the midslab vertical deflection (Df1) of 13 and 25-foot long slabs with the same type of base material was similar, with the ATFD base having the lowest deflection and the 307NJ base having the highest deflection in both cases. Base type had a greater effect on FWD deflection than slab length in these tests.
2. Df1 deflection in the joint leave (JL) position is typically equal to or greater than the Df1 deflection in the joint approach (JA) position on in-service PCC pavements. Past NDT in Ohio suggests that, when deflection on the leave side becomes two to three times greater than the approach side, faulting is likely to occur as the slab on the leave side of the joint settles. On the ERI/LOR 2 test sections, joint leave deflections were generally larger than the joint approach deflections, except on the stabilized ATFD and CTFD bases with a 13-foot joint spacing, and the 310 base with a 25-foot joint spacing. In these sections, deflections on the approach side were much higher than on the leave side. It is doubtful the slab end on the approach side of these joints will

settle much below the slab end on the leave side of the joints because of impact forces being imposed by traffic on the elevated leave slab that would tend to also force it down. There is no obvious reason why the approach readings were higher on these sections.

3. As joints deteriorate, stiffness and load transfer at the slab ends both tend to decrease on intact, in-service PCC pavements. High deflections and extremely low load transfer were observed at the ends of both the 13 and 25-foot long slabs on the 307NJ base, and the 13-foot long slabs on the CTFD base. These three sections also had the lowest average composite stiffness (highest deflection) at midslab.

   One reason for the high deflections mentioned above at certain slab ends may be due to a loss of support from curling and warping of the PCC slabs. The presence and the condition of transverse cracks in the slabs would undoubtedly affect how the ends respond to dynamic loading. However, the 307NJ sections with 13 and 25-foot long slabs average one and two cracks per slab respectively, the CTFDB section with 13-foot long slabs has minimal cracking, and several other sections with significant cracking ($\geq$ 0.95 cracks/slab) showed reasonably good slab end deflection and load transfer. With Df1 being significantly higher on the leave side than on the approach side of joints in the 307NJ section with 13-foot long slabs, some joint faulting may become evident in this section in the near future.

4. While the average load transfer of 103.2% measured on the section with 13-foot long slabs and 307IA base appears to be too high, load transfer at the three consecutive joints used to obtain this average were 100.2%, 97.4% and 112.1%. This consistency of unusually large values of load transfer at PCC joints may have been caused by some rocking phenomenon in the slabs. Unfortunately, no additional data are available to support this premise.

NOTE: Cores taken in the sections with ATFD base at the time of the FWD testing showed extensive stripping of the asphalt cement in the base to the point where there was essentially no bonding of the aggregate.

4.8.2 August 11, 1999 Tests

A second set of FWD measurements was conducted in August 1999 to verify some results obtained at joints during the first set of measurements in June, and to provide additional information on load transfer and rocking of the 13-foot long slabs. In this evaluation, Sensor 2 was moved from 8 inches in front of the center of the load plate to 12 inches behind the center of the load plate. With the load plate in the joint approach position, load transfer was defined in the forward direction as Df3/Df1 and, with the load plate in the joint leave position, load transfer was defined in the reverse direction as Df2/Df1. While it seems that load transfer should be about the same in both directions, it is occasionally different. Readings were initiated at 9:30 am when the pavement temperature was 68º F.

Another parameter being investigated during the August 1999 FWD measurements was slab rocking. To see if a cracked slab was rocking, the load plate was positioned in either the joint approach or joint leave positions, and a remote geophone with a long cable was connected to the connector for Sensor 7. This geophone was placed manually just inside the nearest joint or crack in the slab with the load plate. In this configuration, the load plate with Sensor 1 was on one end of the slab and Sensor 7 was on the other end of the uncracked slab or cracked partial slab. If the slab was rocking, it was expected that there would be a measurable negative deflection at the slab end opposite the load. Unfortunately, the FWD only records the peak downward deflection for each drop and, therefore, geophones located in the area of the slab moving upward would measure zero as the peak downward deflection. In hindsight, a deflection history should have been run with the FWD during these runs to actually determine this negative deflection. Another possible test for a rocking slab would be to position the remote sensor at various distances along the length of the slab, plot the positive (downward) maximum deflections measured over that portion of the slab moving downward, and extrapolate these values across the upward moving portion of the slab. Because of the stiffness of the PCC slab, these deflections should plot close to a straight line. The point of zero deflection would be the fulcrum over which the slab was rocking. Figure 4.3 shows the positioning of the load plate and geophones in the August 1999 readings.

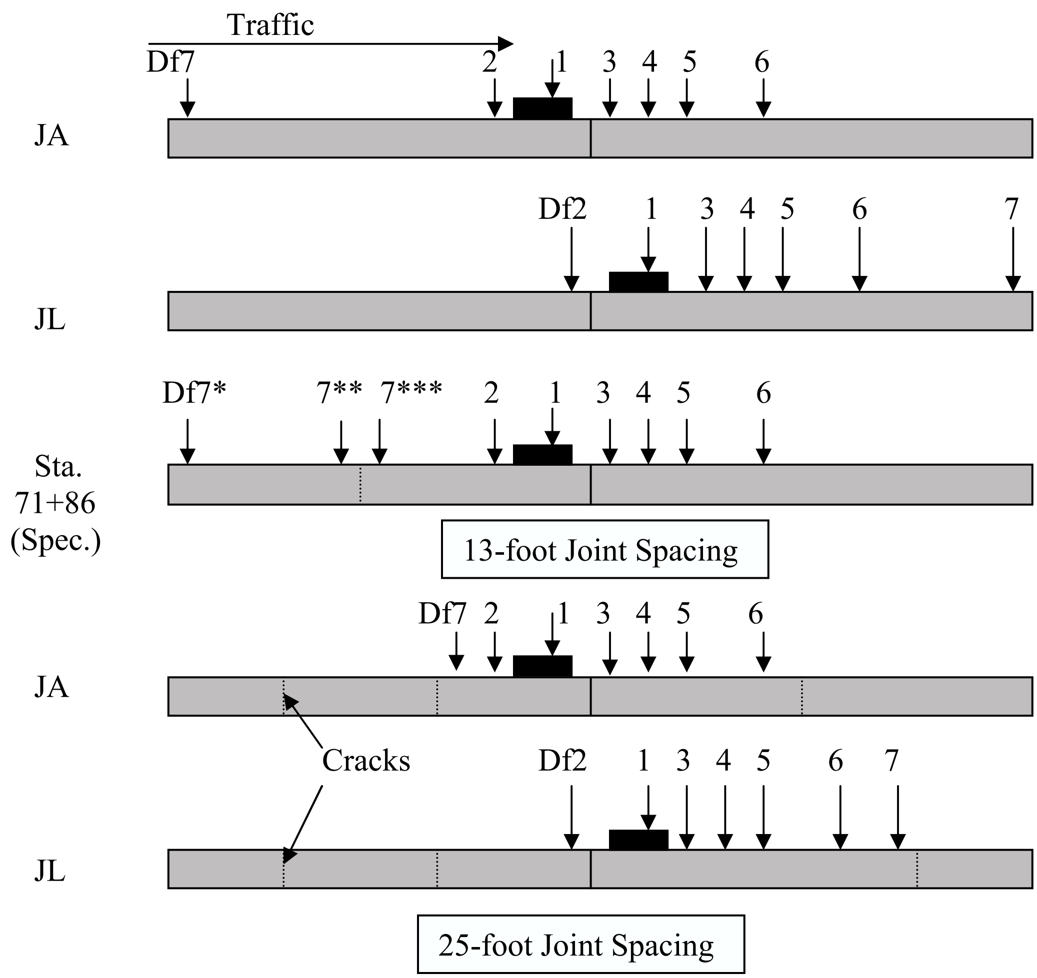
Table 4.11 summarizes the results of the August 1999 measurements. As can be seen in this table, these tests were limited to sections with 13-foot long slabs and the one section of 25-foot long slabs on the 307NJ base. Normalized Df7 measurements on the CTFD and 307NJ bases suggest the entire partial slabs on which the FWD load plate was located moved downward under the load.

Table 4.11 Summary of August 1999 FWD Measurements

Base Type	Avg. Norm. Df1 Defl. in FWD Position (mils)		Avg. Norm. Df7 Defl in FWD Position (mils)		Load Transfer (Df3/Df1) in FWD Position (%)	
	JA	JL	JA	JL	JA	JL
13-foot Joint Spacing						
304	0.47	1.06	0.00	0.00	34.7	17.7
310	0.53	0.49	0.02	0.02	69.1	75.3
307NJ	1.39	2.00	0.09	0.14	7.0	5.6
307IA	0.66	0.79	0.02	0.03	61.1	56.2
ATFDB	0.55	0.51	0.01	0.01	44.0	54.8
CTFDB	1.75	1.32	0.28	0.26	15.2	18.3
25-foot Joint Spacing						
307NJ	1.38	1.04	0.03	0.09	9.8	16.9

Observations from the August 1999 FWD measurements include the following:

1. As was seen during the June 1999 FWD measurements, Df1 in the JL position was generally about the same or greater than Df1 in the JA position with the following exceptions: the 13-foot long slabs on both stabilized bases and 25-foot long slabs on 310 base. In August, joint deflections on the 13-foot long slabs with a CTFD base remained higher on the approach side, but deflections on the ATFD base had equalized. Df1 was slightly higher in the JA position than the JL position on 25-foot long slabs with a 307NJ base. The 310 base with 25-foot long slabs was not tested in August.
2. High slab end deflections continued to be associated with low load transfer across joints. Though not a precise correlation, it is interesting to note in the August measurements that the higher average Df1 measured in either the JA or JL position on each section also had a lower load transfer in that position.
3. In June and August, all slab ends with an average normalized deflection of over 1.00 mils had an average load transfer of less than 20% in that position. The same three sections with extremely low load transfer in June showed the same trend in August. Load transfer in the 307IA/13’ section decreased to 61.1% in August from the unrealistically high levels registered in June.



Sensor spacing from center of load plate: -12, 0, 12, 18, 24 and 36 inches, with Df7 being placed just inside the nearest crack or joint on the loaded slab

Figure 4.3 FWD Load Plate and Sensor Positioning – August 1999

4.9 DYNAMIC CONE PENETROMETER

The Dynamic Cone Penetrometer (DCP) was used to determine resilient modulus of the in-situ unstabilized base and DGAB subbase materials and, to the extent possible, the underlying subgrade in sections with the different base materials and joint spacing. Table 4.12 shows the results of these tests.

Table 4.12 DCP Results on ERI/LOR 2

Base Type	Joint Spacing (ft.)	No. Tests	Average Mr (ksi)		
			Base (t=4")	DGAB (t=6")	Subgrade
307	13	6	86.34	77.70	72.63
	25	2	92.22	143.92	133.90
310	13	6	53.27	54.39	26.58
	25	2	62.75	75.09	63.88
IA	13	4	57.27	51.12	67.26
	27	2	76.05	58.45	73.49
NJ	25	1	71.53	145.27	134.86

4.10 PERFORMANCE SUMMARY

Data presented earlier in this chapter on cracking frequency and FWD deflections have been combined together in Table 4.12. The June and August 1999 FWD readings are both included as shown with a slash separating them. Since somewhat different types of data were obtained each time and since not all of the sections with 25-foot long slabs were tested in August, all data were not duplicated. When data are not available, 'na' was inserted in the table.

Unless considerable background information is available with FWD data, it is difficult to determine from this table alone how the sections are performing. To better visualize overall performance, qualitative ratings were established for each measured parameter, as shown in Table 4.13. It is important to note that these ratings are not standards, nor were they obtained from other sources. They are ranges of measured performance based solely on the experience of the authors from NDT results obtained around Ohio.

Table 4.13 Quantitative Summary of Section Performance

Base Type	Slab Length (feet)	Trans. Cracking (Avg. # Cracks/Slab)		FWD Measurements – June/August 1999				
				Df1 Deflection (mils/kip)			Load Transfer (%)	
		6/99	4/02	Midslab	JA	JL	LT_A	LT_L
304	13	0.03	0.26	.25/-	.51/.47	.70/1.06	41.6/34.7	-/17.7
	25	1.00	1.65	.36/-	.57/-	1.09/-	48.8/-	-/-
310	13	0.09	0.08	.35/-	.66/.53	.60/.49	49.2/69.1	-/75.3
	25	1.11	1.20	.33/-	.73/-	.52/-	28.1/-	-/-
307NJ	13	0.95	0.88	.73/-	1.42/1.39	2.32/2.00	5.5/7.0	-/5.6
	25	2.00	2.29	.69/-	1.32/1.38	1.28/1.04	7.1/9.8	-/16.9
307IA	13	0.19	0.42	.30/-	.51/.66	.53/.79	103.2/61.1	-/56.2
	25	1.21	1.26	.36/-	.51/-	.56/-	62.2/-	-/-
ATFDB	13	0.00	0.06	.20/-	.55/.55	.32/.51	43.2/44.0	-/54.8
	25	0.19	0.78	.11/-	.24/-	.28/-	61.6/-	-/-
CTFDB	13	0.19	0.29	.49/-	2.09/1.75	1.17/1.32	6.6/15.2	-/18.3
	25	2.41	2.75	.27/-	.37/-	.43/-	91.9/-	-/-

Table 4.14 Descriptive Ranges of Performance

Rating	Trans. Cracking (Avg.# Cracks/Slab)	FWD Measurements-June/August 1999		
		Df1 Deflection (mils/kip)		Load Transfer (%)
		Midslab	Joints (JA and JL)	
Excellent (Ex)	0-0.05	0-0.20	0-0.40	91-100
Good (Gd)	0.06-0.25	0.21-0.40	0.41-0.70	71-90
Fair (Fr)	0.26-0.50	0.41-0.60	0.71-1.00	51-70
Poor (Pr)	0.51-1.50	0.61-0.80	1.01-1.50	30-50
Very Poor (VP)	>1.50	>0.80	>1.50	<30

Table 4.14 is a duplicate of Table 4.12, except that qualitative ratings were used instead of the actual data, and the excellent and good rankings have been highlighted for easier visualization. ‘na’ was inserted in the table when data were not available.

Table 4.15 Qualitative Summary of Section Performance

Base Type	Slab Length (ft.)	Trans. Cracking (Avg. # Cracks/Slab)	FWD Measurements - June/August 1999				
			Df1 Deflection (mils)			Load Transfer (%)	
			Mdsb.	JA	JL	LT_A	LT_L
304	13	Excellent	Gd/na	Gd/Gd	Gd/Pr	Pr/Pr	na/VP
	25	Poor	Gd/na	Gd/na	Pr/na	Pr/na	na/na
310	13	Good	Gd/na	Gd/Gd	Gd/Gd	Pr/Fr	na/Gd
	25	Poor	Gd/na	Fr/na	Gd/na	VP/na	na/na
307NJ	13	Poor	Pr/na	Pr/Pr	VP/VP	VP/VP	na/VP
	25	Very Poor	Pr/na	Pr/Pr	Pr/Pr	VP/VP	na/VP
307IA	13	Good	Gd/na	Gd/Gd	Gd/Fr	Ex/Fr	na/Fr
	25	Poor	Gd/na	Gd/na	Gd/na	Fr/na	na/na
ATFDB	13	Excellent	Ex/na	Gd/Gd	Ex/Gd	Pr/Pr	na/Fr
	25	Good	Ex/na	Ex/na	Ex/na	Fr/na	na/na
CTFDB	13	Good	Fr/na	VP/VP	Pr/Pr	VP/VP	na/VP
	25	Very Poor	Gd/na	Ex/na	Gd/na	Ex/na	na/na

* Cores revealed severe stripping of the asphalt cement from the base aggregate

Three major conclusions can be drawn from Table 4.14, as follows:

1. None of the parameters measured on the two sections with 307NJ base are classified as being good or excellent. This included transverse slab cracking, midslab and joint deflection, and load transfer at joints. Similarly, the section with 13-foot long slabs on CTFD base, though largely uncracked, also had high FWD deflections throughout. This would suggest that additional cracking may become evident soon. The 25-foot long slabs on CTFD base, while having good FWD response, are the most severely cracked on the project. Based on these data, both 307NJ and CTFDB sections can be considered to be performing poorly.
2. Both sections on the ATFD base received the highest ratings for the parameters measured, even though load transfer was marginal. This would suggest that these sections are performing the best at this point in time. Severe stripping was observed in cores taken from the AC base.
3. Excellent to good FWD response (low deflection) at slab ends is not always indicative of good load transfer. Though somewhat related, some deviations are expected since numerous mechanisms are involved. Deflection at slab ends is sensitive to internal temperature gradients, which cause the slabs to curl, and load

transfer is sensitive to average slab temperature, which affects aggregate interlock at the slab faces. Curling is most prominent in the spring and fall when the seasons are changing, and during days when there are either significant changes in temperature or rainfall events. Load transfer is generally high in the summer when the slabs are warm and expanded, and low in the Winter when the slabs are cold. Most of the time, slab deflections and load transfer tend to be inversely proportional, but there are times when both parameters can be good or bad At the time of the FWD measurements on ERI/LOR 2, it appears the slab ends were not curled severely, as indicated by the sections providing low deflections, and the slab temperatures were not high enough to consistently provide aggregate interlock on all sections.

4.11 SKID RESISTANCE

Two additional short sections of PCC pavement were placed in the eastbound lanes of LOR 2 between Stations 153+12 and 167+20 to evaluate the effect of natural and manufactured sand on skid resistance. Specifically, manufactured sand was used between Stations 153+12 and 160+16 and natural sand was used between Stations 160+16 and 167+20. The D-cracking resistant coarse aggregate from Woodville and a 21-foot joint spacing were used throughout both sections. Skid tests were performed with the ODOT K.J. Law Skid Trailer since 1994 on the two sections. Typically, a test series consisted of averaging three individual tests in each of the driving and passing lanes in each of the two sections. Table 4.15 summarizes the results obtained to date.

Skid numbers shown in Table 4.15 are typical of that normally observed on PCC pavement. First, the new pavement had excellent skid resistance provided by coarse texture built into the pavement at the time of construction, regardless of aggregate type. Second, as traffic wore off the initial rough mortar texture, skid resistance decreased in accordance with the traffic volume and the frictional characteristics of the aggregate in contact with vehicle tires. When the ERI/LOR 2 pavement was opened to traffic, skid numbers in both test sections were probably around 60 in all four lanes, which were close to that measured in the passing lane in March 1994. With more traffic using the driving lanes, they wore faster and, by March, were 12 skid numbers lower than the passing lane on the section with natural sand and 21 skid numbers lower than the passing lane in the section with manufactured sand. From that time on, skid resistance in all four lanes has continued to drop. Field notes for the 6/23/95 readings indicated that the pavement surface was contaminated with mud from a nearby construction project. Judging by the trends observed in skid numbers over time, some residue from this contamination may have remained on the pavement at the time of the 10/6/95 readings. The numbers recorded on both of these dates would be expected to be slightly higher than the 6/7/96 readings.

Table 4.16 Skid Resistance using Natural and Manufactured Sand

Date	Average Skid Number			
	Manufactured Sand		Natural Sand	
	Driving Lane	Passing Lane	Driving Lane	Passing Lane
3/25/94	37.7	59.0	57.3	45.0
7/15/94	33.0	60.3	59.0	47.3
6/23/95*	33.0	45.0	48.0	46.7
10/6/95	30.3	47.0	48.0	43.0
6/7/96	34.0	49.3	55.0	46.7
6/27/97	27.7	47.7	53.0	38.0

* Pavement surface contaminated with soil from a nearby construction project

Skid resistance in these PCC pavement sections with natural and manufactured sand was probably about the same when the pavement was new. From the data presented in Table 4.15, friction levels decreased steadily with traffic passes, but at a faster rate on the section with manufactured sand. By 1997, the section with natural sand had significantly higher skid numbers in the driving and passing lanes than the section with manufactured sand. In conclusion,

1. The use of natural sand in PCC pavement provides higher skid resistance than manufactured sand during the first few years of service. Later, the final level of skid resistance will depend upon the friction characteristics of both the fine and coarse aggregates.
2. The rate at which surface friction on PCC pavement decreases over time is a function of the traffic volume, as evidenced by the difference in skid numbers between the driving lane and the passing lane. To maintain adequate skid resistance on PCC pavements, only natural sand should be used for fine aggregate. Manufactured sand should be disallowed.

4.12 CONCLUSIONS

4.12.1 Laboratory Testing of Base Materials

1. Samples of 307 NJ base prepared for the triaxial and resilient modulus tests had a lower density than the 307 IA and 304 bases, probably due to a lack of smaller particles in the mix. The 307 NJ material was difficult to compact in the molds.
2. Axial strain at failure in the triaxial tests was lower for the 307NJ base than the 307 IA and 304 bases. As traffic loads pass over PCC pavement with the 307 NJ base, heavy loads and vibrations may cause the base to densify, thereby resulting in a loss of support for the PCC pavement. This can have a significant impact on field performance.

4.12.2 Field – Cracking Observations

1. PCC test sections with a 25-foot slab length all averaged one or two transverse cracks per slab when constructed on 304, 307 NJ, 307 IA, 310 and CTFD base. Slabs on ATFD base had minimal cracking.
2. PCC test sections with 13-foot slab lengths had minimal cracking when constructed on 304, 307 IA, 310 and either ATFD or CTFD bases, while 13-foot slabs on 307 NJ base had significant transverse cracking. This cracking occurred after the sections were opened to traffic and was not believed to be associated with construction.
3. Both sections on the 307 NJ base had significant transverse and some longitudinal cracking. This may be caused by densification of this free draining material under heavy traffic loads and high stresses being induced in the slabs from the resulting loss of support. When slab ends are curled upward, significant tensile stresses from the weight of the slab and heavy traffic passing over the cantilevered slab ends will be generated away from the joints. Cores taken at the cracks show how the size of the crack opening decreases from top to bottom of the slab, which supports this hypothesis.
4. The section with 25-foot long slabs and CTFD base was the most extensively cracked section in the experiment. The rigidity of the CTFD base resists deformation during curling and warping. As a result, the separation between the slab and base becomes more pronounced, the length of unsupported slab length becomes larger and higher

stresses are introduced into the slab. The high FWD measurements on the 13-foot long slabs with CTFD base are indicative of this phenomenon.

4.12.3 Field – FWD Measurements

1. Slabs on ATFD base had the lowest normalized FWD deflection in June 1999 at midslab, indicating the highest pavement stiffness. Slabs on 307 NJ base had the highest midslab deflection, indicating the lowest pavement stiffness. This may be partially due to the presence of transverse cracks in the 307 NJ sections. While the subgrade can affect overall pavement stiffness, it is interesting that midslab deflections in the 13 and 25-foot long sections with the same base were similar. Higher deflections in the CTFDB section may be due to a lack of support resulting from curling and warping on a very rigid base layer.
2. The 13 and 25-foot slabs on 307 NJ base, and the 13-foot slabs on CTFD base had very high FWD deflection and very low load transfer at the joints, indicating poor joint performance. The CTFD base section with 13-foot slabs, though largely uncracked at this time, is likely to show some transverse cracking soon.
3. With high joint deflections, and joint leave readings being significantly higher than the joint approach readings, faulting is likely to occur in the 307 NJ section with 13-foot slabs.

4.12.4 Summary – Overall Base Performance

1. All transverse cracking and FWD parameters used to measure performance on both sections with 307 NJ base were rated poor to very poor in Table 4.6.
2. Wheel path cores taken from the sections with ATFD base showed severe stripping of the asphalt cement from aggregate in the base.

In summary, the preponderance of data collected in the laboratory and at the ERI/LOR 2 site suggests the 307 NJ and CTFD bases are performing poorly at this time. All sections with 25-foot slabs, except those with ATFD base, and the section with 13-foot slabs on 307 NJ base have significant transverse cracking. The 13-foot long slabs with 307 NJ base also have some longitudinal cracking. Considering the relatively short time these pavement sections have been in service, this level of performance is unacceptable. The ATFD base appears to be performing the best at this time.

High FWD joint deflections in the wheel path of the CTFD base section may be caused by a lack of support as the pavement slabs curl and warp on a rigid base. Higher than expected midslab readings may be caused by curling and warping in the transverse direction. This may result in some slab faulting or cracking in the CTFD base section in the near future.

4.13 RECOMMENDATIONS

Based upon results obtained thus far in this project, the following recommendations are presented for consideration at this time:

1. Limited results observed thus far indicate poor performance of 307 NJ and CTFD bases under this PCC pavement evaluated on SR 2. It appears the 307 NJ base may densify under traffic loading and lose needed support around the joints. The rigidity of CTFD base may cause high tensile stresses in PCC slabs as they curl and warp, which will likely result in premature slab cracking. Since other satisfactory materials are available for use as a base under PCC pavement, the use of 307NJ and CTFD base should be discontinued for this application.
2. The SR 2 test sections have been in service for a limited period of time. There should be continued monitoring to determine the long term performance of 304 and 310 dense graded aggregate bases, 307 IA base and ATFD base.
3. The original objective of this installation was to observe the effect of base type on the development of D-cracking in PCC pavement constructed with D-cracking susceptible and D-cracking resistant aggregate. This is an important consideration in the design of PCC pavement and the site should continue to be monitored for early signs of D-cracking.
4. To maintain adequate skid resistance on PCC pavements, only natural sand should be used for fine aggregate. Manufactured sand should be disallowed.

5.0 EVALUATION OF PAVEMENT JOINT PERFORMANCE; JAC-35 AND GAL-35

5.1 INTRODUCTION

The field performance of steel and fiberglass dowel rods used for load transfer in rigid pavement repairs was evaluated with strain gauges cemented to the rods for the determination of shear forces, moments, torques and axial loads. The concrete in these repaired sections was also instrumented with gauges for the calculation of internal stress. Loads were applied with a Falling Weight Deflectometer (FWD), and a single and tandem-axle dump truck. Truck speeds varied between 5 and 65 mph. Response data obtained from the gauges was used to investigate variations in force due to truck speed, size and material of the dowels, and the type of repair joint. Dominating forces in the dowel rods were moments and vertical shear forces. Field performance data were compared to analytical solutions using modified versions of ILLI-SLAB.

One-inch diameter fiberglass dowels are not recommended for rigid pavement, and there are not sufficient benefits to warrant use of the undercut (YU) joint. ILLI-SLAB did not accurately predict measured joint response. Recommendations are included for the repair of rigid pavement dowels and joints.

The scope of this study encompassed data collected from two sites. The first site, located on State Route 35 between Rio Grande and Jackson in Jackson County, was instrumented in June 1990. The second site, located on State Route 35 between Rio Grande and Gallipolis in Gallia County, was instrumented in July 1991. Sites I and II contained 12 and 4 sections, respectively, of instrumented concrete pavement repairs. Similar types of repairs, dowel rods and loading were used at both sites.

5.2 SITE 1

This location consisted of approximately five miles of rehabilitated pavement, in which twelve sections were selected for instrumentation. Each section consisted of two Y type joints and two YU type joints shown in Figure 5.1. Since natural traffic patterns place the right wheel path, or zone of maximum stress, approximately three feet from the edge of the pavement, the third reinforcing bar from the edge of the pavement was instrumented with strain gauges. Steel dowel rods were used in the first six sections and fiberglass dowel rods were used in the remaining six sections. Two sizes of each dowel rod were installed; 1-inch diameter by 12 inches long and 1.5-inch diameter by 18 inches long. The type of dowel rod used in each instrumented section is summarized in Table 5.1.

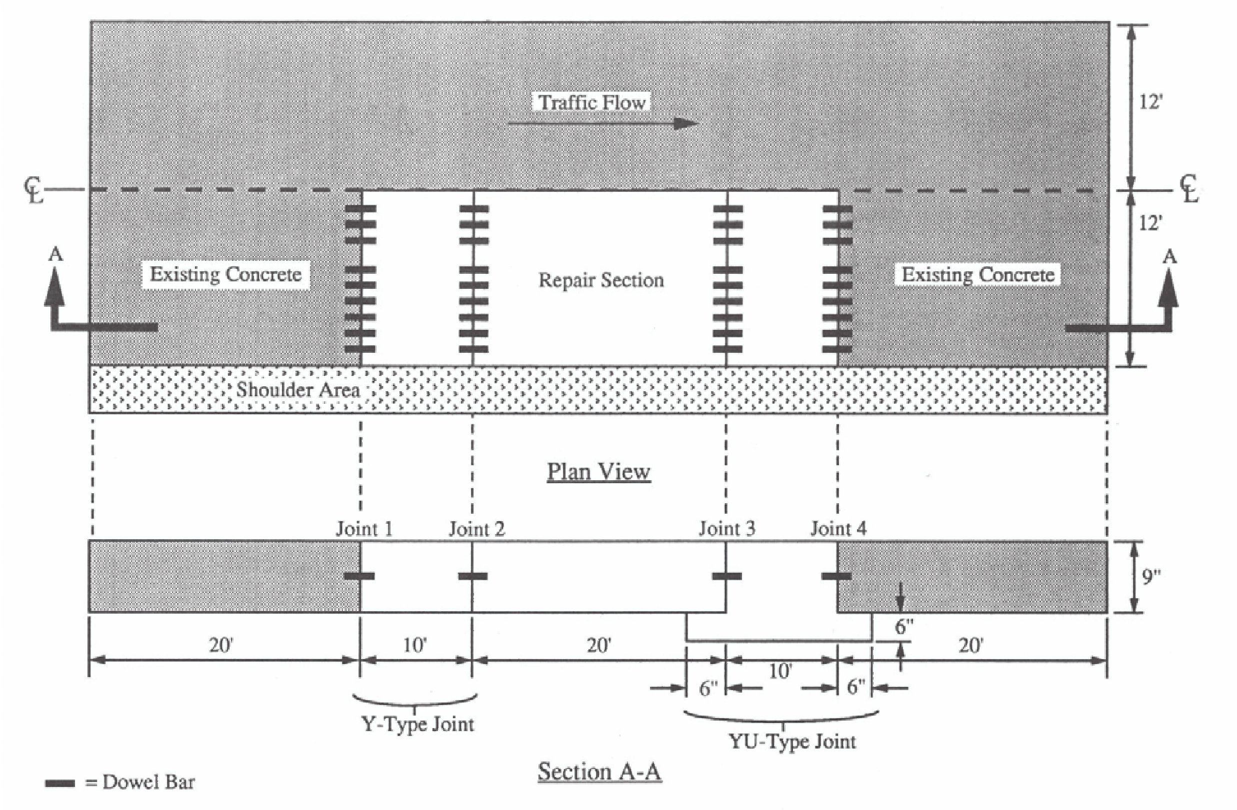


Figure 5.1 Types of Joint Repairs

Table 5.1 Dowel Rod Identification

Section	Dowel Bar
1	1.5" Ø x 18" long steel
2	1" Ø x 12" long steel
3	1" Ø x 12" long steel
4	1" Ø x 12" long steel
5	1.5" Ø x 18" long steel
6	1.5" Ø x 18" long steel
7	1.5" Ø x 18" long fiberglass
8	1.5" Ø x 18" long fiberglass
9	1.5" Ø x 18" long fiberglass
10	1" Ø x 12" long fiberglass
11	1" Ø x 12" long fiberglass
12	1" Ø x 12" long fiberglass

Rosettes and embedment strain gauges were used at each instrumented position in the repairs. Four, 45-degree rosettes were placed around the dowel bars, Joints 1 and 4 each contained four embedment gauges placed vertically along the dowel rod, and Joints 2 and 3 each included four embedment gauges placed vertically along the dowel rod.

A total of 1200 strain gauges were incorporated into Site 1 as follows:

Joint 1	Y-type joint	22(12+4+6)* strain gauges
Joint 2	Y-type joint	28(12+4+12) strain gauges
Joint 3	YU-type joint	28(12+4+12) strain gauges
Joint 4	YU-type joint	22(12+4+6) strain gauges

* (Rosette gauges + embedment gauges at the concrete surface + embedment gauges along the bar)

The placement of rosettes on the dowel rods and most wire connections were completed in the laboratory. The installation procedure for each repair section consisted of removing concrete in the designated repair section, removing the pavement shoulder and drilling holes for dowel bars in the existing concrete, and digging two small wire trenches for Joints 1 and 2. When the trenches were dug, the individual dowel bars for Joints 1 and 2 were placed near their respective locations. Using a 1.5-inch diameter PVC pipe as a protective conduit, all gauge wires from the instrumented bar were routed to a safe location. At the end of the PVC pipe, a bell type enlarger protected four electrical connectors for each joint. The instrumented dowel bar for Joint 1 was grouted into the predrilled hole.

A dowel bar basket containing the necessary number of dowel rods was used at Joint 2. To create a joint, a groove was shaped in the fresh concrete to control shrinkage cracking. At Joint 3, a temporary concrete form was placed to hold dowel rods at the specified location. This assembly consisted of a wooden bulkhead with drilled holes allowing dowel rods to be placed before the concrete was placed. The undercut for this particular joint was not made initially. Gauge wires were guided through a hole in the bulkhead. With dowels in place, the repair section was ready to be partially constructed. Metal shields protected the instrumented bars as concrete was placed into the repair and vibrated. After placement of the first part of each section was complete and the concrete had set, the last ten feet of the repair section was prepared in a similar manner as discussed above. The wood form was removed, and the existing concrete was undercut for the YU-type section. Following the placement of concrete for the YU-type section, the section was prepared for testing.

5.3 SITE 2

By November 1990, approximately 16% of the gauges used at Site 1 became non-functional. The largest percent loss occurred in Joints 1 and 2 where the approximate loss rate was 30%. Approximately 22% of all the gauges eventually failed to operate properly. Upon investigation, it was found that the most significant factor contributing to this failure was movement of the concrete slab, which cut the silicon epoxy protective coating on the wires and allowed the wires to be exposed to the chemical action of concrete and deicing salts. With these observations, it was decided that certain changes would be made at the second site.

A four-mile long section of SR 35 in Gallia County was being widened to three lanes. Four sections of the existing pavement scheduled for repair opposite the ODOT Garage were selected for instrumentation. Each section contained Y-type joints with two instrumented dowel rods located 3 and 6 feet from the edge of the pavement. Steel dowel rods were used in two sections and fiberglass dowel rods in the other two sections. One-inch diameter rods by 12 inches long and 1.5-inch diameter rods by 18 inches long were incorporated into the sections. The type of dowel rod used in each instrumented section was as follows:

Section 1	1.5” Ø x 18” long fiberglass
Section 2	1.5” Ø x 18” long steel
Section 3	1.0” Ø x 12” long fiberglass
Section 4	1.0” Ø x 12” long steel

To measure stress distributions and shear loading in the joints with more accuracy, three strip HBM 120-ohm strain gauge rosettes were cemented to the dowel bars. To protect these rosettes, an area the size of the rosettes was ground flat on the dowel rods. A small channel was cut from these areas to the end of the rods to protect the gauge wires. Wires from the three rosettes were then bundled at the end of the dowel rods, where they were soldered to the Alpha 35206 cable. This protected the wires during movement of the joints.

The gauges and wires were protected with M-coat B protective coating using a procedure recommended by Measurement Group Inc. The procedure consisted of applying a Nitrile Rubber Coating compound over the gauges and wires in layers until grooves in the dowel rods were completely filled, covering the rubber coating with aluminum tape, and applying another coating to ensure a seal between the tape and the dowel rod.

In addition to the gauges used on the dowel rods, each section was instrumented with two embedment gauges, one at the concrete surface and the other below the first dowel rod. Wires coming from the embedment gauges were extended underground with the other cables to the berm. The soldered connections were protected rubber as described above. Construction procedures were similar to those used at Site 1. These precautions proved to be very effective in protecting the gauges and wiring. All gauges were functional one year after installation.

5.4 MOVING LOAD TEST

Ohio Department of Transportation dump trucks loaded with gravel were used to load the PCC repairs. The rear axles weighed approximately 9900 lbs. The trucks passed over each joint in the following sequence:

Single Axle Truck 45 mph

Single Axle Truck 55 mph

Single Axle Truck 65 mph

Tandem Axle Truck 55 mph

As trucks passed over the joints, the entire response cycles were recorded with a data acquisition system and stored for analysis.

5.5 FALLING WEIGHT DEFLECTOMETER

Impulse loads were applied with a Dynatest Model 8000 Falling Weight Deflectometer to the approach and leave sides of each joint near the instrumented dowel rods. Tests were conducted with at applied pressure of 230 psi. Another series of tests was conducted in Section 3 where the load was placed at four different locations along Joints 1 and 2. Data obtained in June 1998 for drops approximating 9000 lb. ft. are shown in Appendix H.

5.6 FIELD DATA

Data from the dowel rods and concrete embedment gauges were collected at the rate of 2000 points/gauge for a period of 1.2 seconds in the form of voltages and converted to strain using a computer program written for this project. The high speed data acquisition system provided the capability of simultaneously recording strains from all gauges during the application of load. This created a “strain-image” at the joint.

Computations on the output data provide the magnitude of forces on dowel rods at specified times as dynamic load was applied over the joint. Through the use of comparative loops, a computer program was able to analyze all forces at the sensors during a single event, and identify maximum positive or negative values. These maximum positive and negative values are of most interest, since they represent the most severe conditions experienced by the repair during the loading. Computer software was used to plot maximum forces on a single dowel rod under different loading conditions.

The output from embedment gauges was analyzed using the same procedure. Each load application produced one maximum reading per gauge. Since large variations were noted in the data, average values were used for all sections containing similar dowel rods and subjected to similar loading conditions. Three tests were performed with trucks carrying approximately the same load. Concrete stress was determined from average strain measurements in the embedment gauges for a particular joint, load and bar geometry.

5.7 CONCLUSIONS

Based upon the results of this study, and finite element analyses, the following conclusions can be drawn:

1. The two dominant forces in dowel bars were bending moment around the X-axis and vertical shear. Moment around the Y-axis, axial force and torque did not contribute significantly to dowel bar response.
2. Subgrade stiffness had a significant effect on dowel bar response.
3. Looseness around dowel bars affected their response and load transfer capability.
4. The most effective dowel bar in these tests was the 1.5” diameter steel bar. One-inch diameter fiberglass dowel bars are not recommended for rigid pavement.
5. Larger diameter and stiffer dowel bars transferred more load across PCC joints.
6. The performance of the 1” steel dowel bars were similar to the 1.5” fiberglass bars.
7. The finite element model ILLI-SLAB did not predict dynamic response accurately.
8. Bending stresses in the concrete and bearing stresses around the dowel bars were small.
9. Undercutting a PCC joint repair initially reduced the forces in dowel bars. The effectiveness of the undercut diminishes over time.
10. Dowel bar forces were about the same in the Y and YU type of joint repairs.
11. Shear forces generated at speeds of 45-65 mph were about the same, and less than shear forces generated at speeds below 30 mph.
12. Measured dowel bar forces were lower than theoretical forces.

6.0 DOWEL BAR EVALUATION ON ATH 33

6.1 OVERVIEW

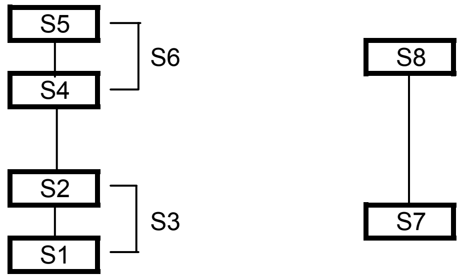
In July 1994, four types of dowel bars were installed on the eastbound side of ATH 33 near the State St. exit in Athens, Ohio. These bars included round and I-beam shaped steel and fiberglass. The round dowels were 1 ½” in diameter and the I-beam shaped dowels were approximately 1 ½” high and wide. On August 15, 2001, the ODOT Falling Weight Deflectometer was used to evaluate the effectiveness of these bars. Table 6.1 shows the normalized maximum deflection and the percent of load transfer measured across the joints in the approach position. The joint containing fiberglass I-beam dowels had the highest deflection and the lowest load transfer of the five joints in this test. As of the date of this report, no visible distress was apparent at any of the five joints.

Table 6.1 FWD Measurements on ATH 33

Station	Type of Dowel Bar	Normalized Df1 in JA (mils/kip)	Load Transfer (%)
828+03	Fiberglass I-Beam	0.63	68.3
828+23	Steel Round	0.45	94.9
828+43	Steel I-Beam	0.51	86.4
828+70	Steel Round	0.42	98.4
828+91	Fiberglass Round	0.53	85.1

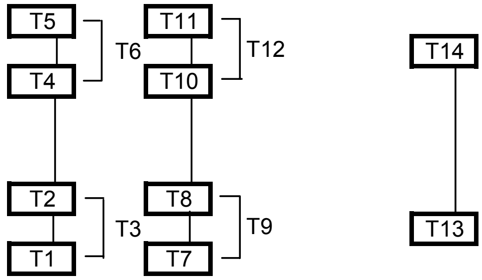
APPENDIX A WHEEL GEOMETRY, TIRE PRESSURES AND WEIGHTS OF TEST TRUCKS ON OHIO SHRP TEST ROAD

Table A.1 Single-Axle Truck Weights



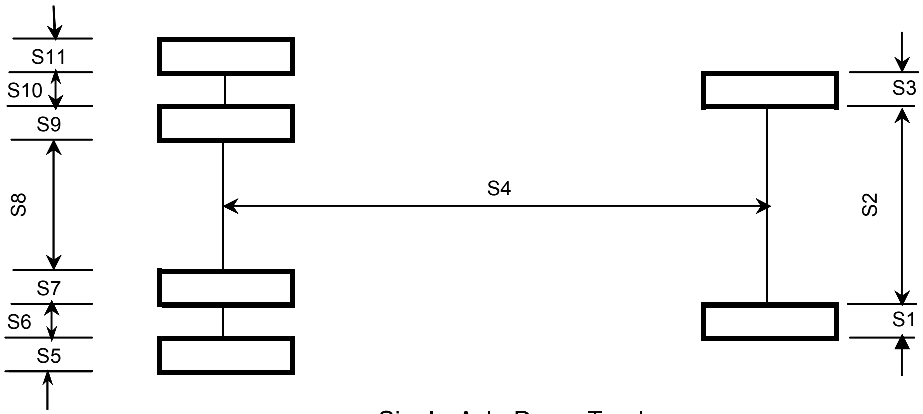
Test Series	Test Date	Load I.D.	Wheel Load (lbs.)									
			S1	S2	S3	S4	S5	S6	R. Axle	S7	S8	F. Axle
2	8/2,3/96	A			3770			3840	7610			
	8/5,6/96	B			9150			9335	18485	4690	4660	9350
	8/6,7/96	C			8870			9580	18450	4760	4850	9610
	8/9/1996	D			10680			11550	22230	4760	4850	9610
2	8/12/1996	A			10680			11550	22230	4760	4850	9610
	8/13/1996	A			10930			10160	21090			
	8/14/1996	B			9290			8810	18100	4690	4820	9510
4	7/2/1997	K	3300	5400	8700			8650	17350	4250	4300	8550
	7/3/1997	L	5350	7750	13100			11850	24950	4450	4450	8900
	7/29,30/97	M,N	4950	6350	11300			10150	21450	3650	3600	7250
	7/30,8/6/97	O,P	5700	7550	13250			12100	25350	3950	3750	7700
5	10/9,14/98	98A,B	4150	5300	9450	4850	4100	8950	18400	4750	4650	9400
	10/14,15/98	98C,D	5300	6750	12050	6700	5250	11950	24000	4800	4600	9400
	10/19/98	98E	5300	6750	12050	6700	5250	11950	24000	4800	4600	9400
	10/20/98	98F	4650	5800	10450	6000	4200	10200	20650	4900	4750	9650
6	9/27/99	99A			10550			9600	20150	4900	4600	9500
	9/28/99	99B			8500			7800	16300	5350	4850	10200
	10/1/99	99C			11050			9600	20650	5150	4600	9750
	10/5/99	99D			8800			8150	16950	5350	4800	10150
7	10/7,12,13/99	99E,F,G			10700			9950	20650	5250	4800	10050
8	4/27/01	01A	5350	5450	10800	5650	5200	10850	18400	4700	4700	9400
	4/30/01	01B	4850	4600	9450	4800	4150	8950	18400	4750	4400	9150
	5/1/01	01C	4850	4600	9450	4800	4150	8950	18400	4750	4400	9150
	5/2/01	01D	5500	5600	11100	6150	5400	11550	22700	4800	4500	9300

Table A.2 Tandem-Axle Truck Weights

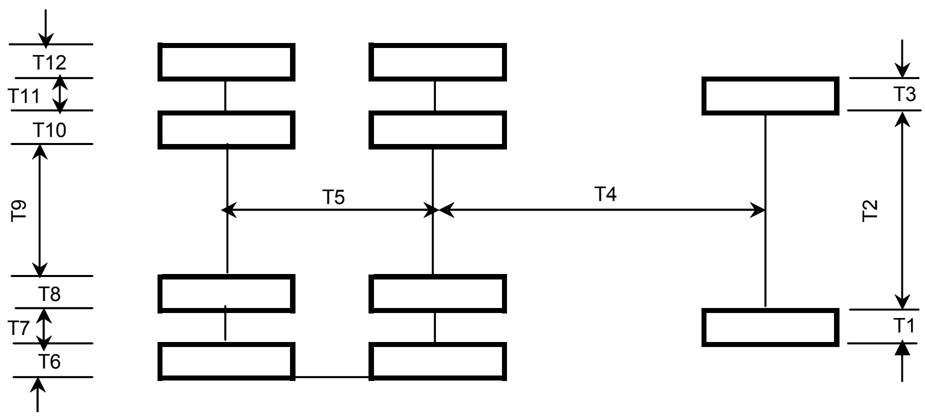


Test Series	Test Date	Load I.D.	Wheel Load (lbs.)													
			T1	T2	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12	T13	T14
2	8/2,3/96	A			8050			8180			8120			8500	7360	7850
	8/5,6/96	B			10220			11160			10550			10590	8220	8770
	8/12,13/96	A			10220			11160			10550			10590	8220	8770
	8/14/1996	B			7750			8250			8010			8530	7030	7680
					3159			3158			3050			3030		
					11350			10700			11800			10300		
3	6/4,5/97	A	6700	3250	9950			9500	4650	6450	11100			9700	8150	8050
	6/9,10,19/97	B,BA,Y	4000	4350	8350			7800	4250	4600	8850			8000	6600	6450
	6/20,23,24/97	Z,C,D	3800	4500	8300			7800	3950	5150	9100			7800	6700	6500
	6/24,25/97	E,F	2200	2700	4900			4550	2400	3400	5800			4200	6000	5800
	6/25,26/97	G,H	1200	1750	2950			3000	1550	2150	3700			2900	5500	5500
4	7/2/1997	K	3900	4950	8850			7250	4200	5250	9450			7450	7300	7200
	7/3/1997	L	5500	7100	12600			11700	5700	7050	12750			12400	8400	8600
	7/29,30/97	M,N	4050	5200	9250			8250	4350	5400	9750			8600	7550	7550
	7/30,8/6/97	O,P	5300	6000	11300			10750	5900	6350	12250			10800	8350	8250
5	10/9,14/98	A,B	3750	3650	7400	5600	2750	8350	3100	5300	8400	5150	3100	8250	6700	6850
	10/14,15/98	C,D	4600	4550	9150	6200	3400	9600	3650	5850	9500	6100	4000	10100	7500	7500
	10/19/1998	E	4600	4550	9150	6200	3400	9600	3650	5850	9500	6100	4000	10100	7500	7500
6	9/27/99	99A			9250			9100			9700			8900	7420	7150
	9/28/99	99B			7250			7800			7700			7700	7150	7050
	10/1/99	99C			9850			9100			9750			9200	7100	7050
	10/5/99	99D			7300			8050			7900			7600	7450	7350
7	10/7,12,13/99	99E,F,G			10000			9450			9800			9500	7300	7250
8	4/27/01	01A			8400			8950			8700			8800	7500	7500
	4/30/01	01B			7350			7200			7800			7050	6750	6550
	5/1/01	01C			7350			7200			7800			7050	6750	6550
	5/2/01	01D			11550			11500			11050			11700	8400	8050

Table A.3 ODOT Dump Truck Dimensions



Single-Axle Dump Truck



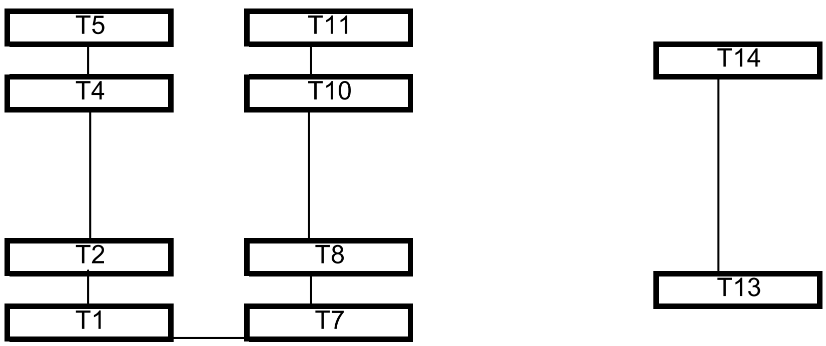
Tandem-Axle Dump Truck

Test Series	Dimensions on Single-Axle Dump Truck (in.)										
	S1	S2	S3	S4	S5	S6	S7	S8	S9	S10	S11
2-4	8.5	70.3	8.5	137.0	9.8	3.0	9.8	50.0	9.8	3.0	9.8
5	9.3	70.0	9.3		8.0	4.8	8.0	51.0	8.0	4.8	8.0
6-7	8.3	71.8	8.3	140.3	8.0	5.3	8.0	51.3	8.0	5.3	8.0
8											

Test Series	Dimensions on Tandem-Axle Dump Truck (in.)											
	T1	T2	T3	T4	T5	T6	T7	T8	T9	T10	T11	T12
1-4	10.5	69.3	10.5	178.8	53.5	9.5	3.0	9.5	49.3	9.5	3.0	9.5
5	11.0	70.0	11.0			9.0	3.6	9.4	49.1	9.3	4.1	8.9
6-7	13.0	67.5	13.0	180.0	54.0	8.5	4.8	8.5	50.5	8.5	4.8	8.5
8												

Table A.4 Tire Pressure in Controlled Vehicles 1999, 2001

Single-Axle Dump Truck



Tandem-Axle Dump Truck

Date	Pressure in Single-Axle Dump Truck Tires (psi)					
	S1	S2	S4	S5	S7	S8
9/28/99 10/1/99	85.0	115.0	110.0	110.0	150.0	130.0
5/1,2/01	79.0	77.0	82.0	82.5	99.5	98.5

Date	Pressure in Tandem-Axle Dump Truck Tires (psi)									
	T1	T2	T4	T5	T7	T8	T10	T11	T13	T14
9/28/99	140.0	30.0	130.0	145.0	140.0	145.0	150.0	140.0	150.0	150.0
10/1/99	140.0	105.0	130.0	145.0	140.0	145.0	150.0	140.0	150.0	150.0
5/1,2/01	101.0	98.0	105.0	103.5	102.5	85.5	105.5	104.0	112.5	111.0

APPENDIX B REPORT SYNOPSIS "FINAL REPORT ON FORENSIC STUDY FOR SECTION 390101 OF OHIO SHRP U.S. 23 TEST PAVEMENT"

Introduction

A forensic study for Section 390101 of the Ohio SHRP Test Road was conducted from July 8 to 10, 1997. The main objective of the investigation was to obtain critical data relevant to the performance and cause of excessive rutting at a limited number of locations of this section. Most of the parameters monitored during this forensic study were essential for an in-depth evaluation of the performance of this section at a future date. For instance, rutting could have occurred in the asphalt concrete, DGAB or subgrade, or more than one layer might have contributed to the rutting. This study was designed to determine the possible causes of rutting and any other distress that occurred in the pavement system.

Summary of Non-Contact Profilometer

Results obtained with the non-contact profilometer are shown below. The data show a significant loss of pavement serviceability (PSI) over time.

Date	Time	Pavement Serviceability Index (PSI)
8-13-1996	After construction	3.89
6-17-1997	Prior to forensic study	2.78

Elevation of Water Table

Elevation of the water table was higher than anticipated under the pavement and probably was the source of high moisture contents observed under the pavement sections throughout the year.

Load History

The Ohio SHRP Test Road was opened for traffic on 8/14/96, and closed on 9/3/96 to repair Sections 390102 and 390107, which had both failed. The road was reopened for traffic on 9/11/96 and closed on 12/3/96 to avoid other failures during the winter and to preserve instrumentation until testing could be completed in the spring. A load cell based weigh-in-motion (WIM) system was used to continuously monitor traffic loading while the sections were in service. Table B-1 enumerates the total number of ESAL's monitored during the first two weeks the road was opened to traffic.

Table B.1 Total Number of ESAL's For First Two Weeks

DATE	DAILY NO. OF TRUCKS	DAILY NO. OF ESALs
8-14-96	1906	1887
8-15-96	1614	2009
8-16-96	2100	2443
8-17-96	910	900
8-18-96	784	1213
8-19-96	2062	2715
8-20-96	2329	3105
8-21-96	2307	3033
8-22-96	2054	1728
8-23-96	2100	2500
8-24-96	910	1000
8-25-96	785	1200
8-26-96	2062	2700
8-27-96	2329	3000
8-28-96	2310	3000
TOTALS	26562	32433

Forensic Procedure

To accomplish the objective of the forensic study, the following steps were followed:

1. Videotape the entire section. Photos were taken with a digital camera of selected areas and referenced by station.

2. Conduct distress surveys according to SHRP-P-338 "Distress Identification Manual for the Long-Term Pavement Performance Project."

3. Conduct Falling Weight Deflectometer tests.

4. Determine longitudinal and transverse profiles with a dipstick.

5. Determine surface elevations with a rod and level.

6. Conduct Dynamic Cone Penetration (DCP) tests in the right wheelpath and centerline of the test lane.

7. Cut three lateral trenches 3-4 feet wide at selected locations where the dipstick and Falling Weight Deflectometer indicated minimum, average, and maximum pavement distress.

Asphalt Concrete Pavement

a. Determine the change in thickness of each lift across the entire 12 ft. traffic lane (including the wheel paths and center of lane) by measuring downward from the AC surface to the lift lines at 12 inch intervals. Elevations of the lift surfaces, as determined at the time of construction, also were used to obtain these results.

b. Obtain six cores of AC for laboratory testing to determine the density and basic mix properties (by ODOT), and verify the construction mix design.

c. Cut a 12-inch wide transverse sample of AC from the pavement for laboratory analysis.

Dense Graded Aggregate Base

a. Determine the thickness profile using the same procedure as for AC.

b. Measure moisture content and gradation of aggregate in the base.

Subgrade

a. Determine the subgrade moisture at every 6" interval up to a 4' depth.

b. Determine the subgrade density at every 6" interval up to a 4' depth with a nuclear gage.

c. Determine surface profile and compare to construction elevations.

8. Perform Cone Penetration Tests (CPT)

    a. Determine the elevation of bedrock.

    b. Determine the profile of the tip resistance, skin resistance, and water pressure of subgrade.

    c. Identify weak zones in the subgrade.

9. Measure the elevation of the water table.

10. Determine the ESAL's.

11. Obtain climatic data as available over traffic loading period

Trenches

Based on results from the Falling Weight Deflectometer and the dipstick, three trenches were excavated to a depth of four feet based upon the level of distress present. The pavement at trench one exhibited average rutting and FWD deflection with no cracks (Station 1 + 50). Trench two was excavated where the most rutting and cracking, and the highest deflection occurred (Station 2 + 65). Trench three was situated at the least distressed region (Station 4 + 00). To maintain natural water content in the base and subgrade layers during removal of the asphalt concrete layers, no water was used in sawing the AC layers.

Variability in Stiffness

Results of the Falling Weight Deflectometer on the subgrade, base, and AC indicated that the stiffness of the pavement system between Stations 2+00 and 2+50 and at Station 4+00 was less than the rest of this 500 foot long section. At these two locations, deflections were almost twice as high for a given load. When the falling weight tests were conducted at the surface of the base, lower deflections were exhibited at Station 2 + 50. A series of dynamic cone penetration were performed. In each test, the AC layer was removed without using water. Results of the dynamic cone penetration tests indicated non-uniformity throughout the section. The lowest index was measured in the trench at Station 2 + 65, which agreed with results obtained with the FWD on the day the trenches were excavated. Nuclear density tests were conducted in the trenches as they were excavated

Rutting

The transverse profile of the driving lane as measured with a dipstick, and a rod and level indicate some significant variations in the profile from the proposed design. There was a discrepancy between results obtained with the dipstick and the survey. This could be due to a difference in accuracy between the two systems, and also in referencing the survey to a bench mark outside of the pavement. It is important to note that the dipstick is more accurate than standard surveying tools, but it measures relative to some starting point on the pavement.

Severe damage occurred at the bottom layer of the asphalt. This distress probably occurred from the presence of excess water and high cyclic stresses induced by traffic loading. Debonding of the asphalt layers was also noted in trenches at Station 1 + 50 and 2 + 65. Moisture was present between the Type 1 and Type 2 mixes, and at the bottom of the asphalt concrete where the 304 aggregate adhered to the bottom of the Type 2 mix.

Several asphalt cores were obtained from the right wheelpath and shoulder. Careful observation of these cores indicated that the bottom layer at the wheelpath had disintegrated to the point where it could be broken apart by hand; whereas, the same layer at the shoulder was in excellent condition. It appears that most of the damage to the bottom layer was caused by traffic loading. Debonding could be due to the presence of dust on the existing AC as new lifts were added to the structure and to high shear stresses induced by traffic.

Examination of the intermediate and surface asphalt courses at the trenches showed no changes from the as-placed thickness. Similar observations were noted from cores. Based upon these results, it was concluded that the rutting occurred below the asphalt concrete. Base thickness was significantly less than the design thickness of eight inches across the entire lane, indicating some shortage at the time of construction. Base thickness was even less in the wheel paths, indicating some rutting in this layer.

Moisture

Laboratory samples were obtained from different depths in the trenches and transferred to the Ohio Research Institute for Transportation and the Environment (ORITE) laboratory in sealed containers for testing. Subgrade moisture was highest Trench 2. For this trench near the surface of the subgrade, the high water content could be caused by rain infiltrating through pavement cracks. Moisture in all three trenches was high at a depth of two feet. Results from the TDRs indicated that subgrade moisture was relatively constant during the short life of Section 390101. Thus, average moisture data obtained from the trenches or the TDRs could be used for the determination of resilient modulus.

Resilient Modulus of Subgrade

In this investigation, a detailed laboratory study was conducted to determine properties of the subgrade. The resilient modulus of the subgrade was calculated from the Dynamic Cone Penetration (DCP) tests. Here CBR was determined from DCP data and the following equations were used to determine resilient modulus (M_R) from the DCP (Livneh 1987).

$$\text{Log (CBR)} = 2.20 - 0.71 \text{ Log (PI)}^{1.5} \pm 0.075 \qquad (1)$$

where PI = DCP penetration index (mm/blow) and

$$M_R = 1200 \times \text{CBR} \qquad (2)$$

Although this technique yielded higher values of resilient modulus, it should be noted that there is a need for more research to determine the proper relation between DCP and the resilient modulus of soil.

Analysis of Pavement System

Initially, the service life of Section 390101 was predicted to be 2½ years implementing the AASHTO design equations with the following parameters:

Structural Numbers (SN): AC 0.35 DGAB 0.14
$P_o = 4.5$
$P_t = 2.5$
R= 50%
$S_o = 0.49$

In this study, the performance of this section was reevaluated using all parameters in the initial prediction of pavement life, except for resilient modulus, which was modified for the in-situ water content. The new predicted pavement life was only 4 to 5 months, which agreed well with field experience.

Fatigue Cracking

Trench 2 was located in an area 3 feet wide by 23 feet long where fatigue cracking was quite evident. These cracks were classified from medium to high severity. When looking at the profile after the asphalt had been removed from the trench, one of these cracks was noted to extend only through the upper lift, thus suggesting initiation from the surface and delamination of the lifts. No other cracks were visible along the saw cut.

Cone Penetration Tests

The cone penetration test (CPT) system utilized in the investigation was mounted on a heavy semi-truck and fully enclosed within the trailer body. The gross weight of the CPT truck was about 22 tons. The CPT system was completely self-sufficient, providing both electrical and hydraulic powers internally. Major components of the system included thrust machine/reaction frame, universal head clamps, system control panel, piezo-electric cone penetrometer, electronic sensors, extension rods, and computerized data acquisition units.

The cone had an outside diameter of 1.75 in. and was advanced hydraulically into the ground at a rate of 2 cm/sec. The data collected during each CPT consisted of tip resistance (q_c), sleeve friction (f_c), instantaneous pore water pressure (p), and friction ration (R_f) which is equal to f_c divided by q_c. Overall, data from the CPT showed that:

1. Relatively hard material was detected at an average depth of 2 ft. below the top of the base.

2. High pore water pressure commonly recorded during the CPT investigation indicated elevated moisture content in the subgrade.

3. A significant weak zone was encountered in Trench 3 just below the base.

Conclusions

An in-depth forensic study of Section 390101 in the Ohio SHRP Test Road was performed to determine the cause of excessive rutting at three locations. This investigation revealed substantial variability in the stiffness of the base and subgrade throughout the 500 ft. test section. In the region with the worst distress, the asphalt layers were debonded and fatigue cracks were visible on the surface. Rutting within the AC layer was insignificant. Most of the rutting could be attributed to the base and subgrade. It appears that the constructed thickness of the base was less than the design requirements. There was no significant change in moisture in the subgrade throughout the seasons. No stripping of AC binder from the aggregate was noted in the surface and the intermediate layers. The base adhered to the Type 2 mix. Subgrade moisture was higher than expected throughout the short life of this section. Utilizing resilient modulus obtained from laboratory data for the in-situ field conditions (moisture of soil at trenches) and employing the AASHTO equation for predicting the test section performance, the life expectancy for this section was 4 to 5 months. It is clear that the actual life of this section on the Ohio SHRP Test Road was reasonably close to the predicted life.

References

Livneh, M. “The Correlation Between Dynamic Cone Penetrometer (DCP) Values and CBR Values.” Transportation Research Institute, Technion-Israel Institute of Technology, Publication No.87-065, 1987.

APPENDIX C REPORT SYNOPSIS “EVALUATION OF SUBGRADE VARIABILITY ON OHIO SHRP TEST ROAD”

Introduction

Construction of the Ohio SHRP Test Road was initiated in 1994. One of the early priorities was to build a uniform subgrade for the forty, 500-foot long test sections included in the project and, thereby, permit a more direct comparison of section performance. Preliminary borings indicated a relatively consistent, predominantly AASHTO A-6 soil, along the three mile long site and the topography was flat. To avoid localized pockets of weakness, provisions were made in the construction specifications and plan notes to replace unsuitable material with A-6 soil from a suitable borrow pit. Fortunately, acceptable borrow was available from a field adjoining the project. This summary documents the extent to which subgrade uniformity was achieved on the Ohio SHRP Test Road.

As construction proceeded, more subgrade undercutting was required than originally anticipated due to the presence of old basements, wells, septic fields, etc. left when this section of U.S. 23 north of Delaware was upgraded from a two-lane pavement to a four-lane divided facility in the 1960s. Undercutting depths varied from a few inches to over ten feet. Moisture and density were monitored with a nuclear density gauge to ensure proper compaction as the subgrade was brought up to its final elevation. It was then proof rolled to identify localized areas of weakness. Once the subgrade in each test section was approved, the Falling Weight Deflectometer (FWD) was used to measure in-situ stiffness at 15.2 m (50 ft.) intervals in the centerline and right wheel path of each SHRP test section. With the exception of Sections 390159 and 390264, the subgrade in all mainline sections was finished by September 1995, and the bases were completed before winter set in that year. The subgrades in Sections 390159 and 390264 were finished in June 1996, and final paving was completed on all mainline sections before being opened to traffic on August 14-15, 1996.

Testing

Instrumentation was placed in the sampling and testing portion of 18 test sections to monitor subsurface environmental conditions throughout the project. Sensors were installed to measure temperature, moisture and frost to a depth of six feet in accordance with SHRP protocol. Piezometer wells were added along the edge of nine test sections to record the elevation of the water table. Bulk samples of subgrade soil were obtained from 12 sections and tested in the laboratory for various mechanical properties. Also, a series of Dynamic Cone Penetrometer (DCP) tests were performed in Section 390101 during a forensic study to determine the cause of early rutting in the wheel paths. The following summarizes the results of these investigations:

AASHTO Soil Classification: Of ten sections sampled on the mainline pavement, six were identified as A-6, three were identified as A-4/A-4a, and one was identified as A-7-6. Two samples obtained from the ramp where the SPS-8 experiment was located were identified as A-4/A-4a.

Subgrade Density and Moisture: After the subgrade in each test section was approved, nuclear density measurements were typically taken at Stations 1+00, 2+50 and 4+00 in the centerline of the test lane and submitted for inclusion in the LTPP database. Averages of these readings are shown in the Table C-1.

Gravimetric Moisture Content: During the first two years of service, subsurface time-domain reflectometry (TDR) probes located in the upper portion of the subgrade did not detect any significant seasonal moisture effects. Probes placed 0.15 m (6-18 in.) below the surface of the subgrade indicated moisture contents of about 20% for all sections. Moisture near the surface of the subgrade varied according to the type of base used, as follows: 12-15% for dense graded aggregate bases (DGAB), around 10% for free-draining bases, and 20-40% for stabilized bases. Water table elevations are shown in Table C-2. While the water table dropped in the fall and early winter of 1997, it remained stable during the remainder of the year and relatively constant throughout the project.

FWD: Subgrade stiffness was determined with an FWD through the application of a haversine load pulse to a pavement surface and measurement of vertical deflection at seven radial distances within the basin generated by the load. For subgrade, deflection under the center of the 300 mm (11.8 in.) diameter load plate was used with the Boussinesq equation to calculate the composite modulus of elasticity. Subgrade moduli calculated in this manner for test sections in the Ohio SHRP Test Road were highly variable, as indicated in Table C-1. FWD profiles obtained in each section were used to identify specific locations with low stiffness.

DCP: This device drives a steel rod into unstabilized bases and soil with a known amount of energy. By continuously monitoring the rate of penetration, the stiffness of various layers can be measured accurately with depth. Three locations (Stations 1+50, 2+65 and 4+00), judged from visual distress and FWD measurements to be an average, the most and the least distressed areas respectively in Section 390101after it had failed, were selected for DCP investigation. These data show the zones of least resistance to DCP penetration to be between 0.60 and 1.00 m (24-39 in.) below the top of the base at Station 1+50 (15-40 mm/blow) and between 0.20 and 0.60 m (8-24 in.) below the top of the base at Station 2+65 (25-150 mm/blow). At Station 4+00, the rates of penetration increased steadily from 10-60 mm/blow over a depth of 1.00 m (39 in.) below the top of the base. The top of the subgrade was 0.20 m (8 in.) below the top of the base in this section.

Table C.1 Summary of In-Situ Subgrade Tests

Section No.	Soil Classification	Nuclear Density Readings Dry Unit pcf	Dry Unit Kg/m^3	Moisture Content (%)	In-Situ Modulus - FWD Average Mpa	Average ksi	Standard Mpa	Standard ksi	CV
		SPS-1							
390101		116.8	1870.4	8.9	80.6	11.69	40.1	5.81	0.50
390102		124.6	1995.9	8.3	140.5	20.37	58.3	8.45	0.41
390103		119.8	1919.0	7.7	108.2	15.69	30.2	4.38	0.28
390104		119.7	1918.0	9.2	116.2	16.85	48.7	7.06	0.42
390105		117.6	1883.8	9.7	107.2	15.54	22.8	3.31	0.21
390106		123.4	1976.2	10.0	123.3	17.88	40.9	5.93	0.33
390107	A-7-6	121.3	1942.5	6.8	115.6	16.76	39.4	5.71	0.34
390108		117.4	1881.1	8.5	130.7	18.95	44.0	6.38	0.34
390109		119.7	1917.9	9.7	79.4	11.51	39.2	5.68	0.49
390110	A-4/A-4a	118.0	1889.7	9.7	89.3	12.95	37.5	5.44	0.42
390111	A-6	121.3	1943.6	9.7	124.7	18.08	62.0	8.99	0.50
390112		121.9	1953.2	8.7	95.3	13.82	43.3	6.28	0.45
390159		118.9	1905.1	11.3	39.8	5.77	22.0	3.19	0.55
390160	A-4/A-4a	123.1	1971.8	8.5	128.5	18.63	38.6	5.60	0.30
		SPS-9							
390901		126.2	2021.5	9.7	186.0	26.97	99.6	14.44	0.54
390902	A-4/A-4a	122.2	1958.0	10.7	106.9	15.50	47.8	6.93	0.45
390903		126.1	2020.4	8.8	98.8	14.33	41.1	5.96	0.42
		SPS-2							
390201		119.6	1916.3	11.1	62.4	9.05	28.6	4.15	0.46
390202	A-6	124.6	1995.4	10.4	123.4	17.89	70.0	10.15	0.57
390203		120.4	1928.6	8.4	103.0	14.94	28.2	4.09	0.27
390204		124.5	1994.3	9.8	205.3	29.77	95.4	13.83	0.46
390205	A-6	118.6	1899.3	11.0	64.3	9.32	37.1	5.38	0.58
390206		120.0	1921.7	10.1	87.8	12.73	46.1	6.68	0.53
390207	A-6	120.9	1936.1	8.2	117.8	17.08	36.2	5.25	0.31
390208		115.2	1845.3	9.3	112.7	16.34	39.0	5.66	0.35
390209		118.1	1891.8	11.7	71.6	10.38	54.1	7.84	0.76
390210		116.0	1858.7	8.8	71.1	10.31	31.4	4.55	0.44
390211	A-6	119.7	1917.4	9.4	109.3	15.85	21.2	3.07	0.19
390212		126.0	2017.8	9.2	140.9	20.43	49.0	7.11	0.35
390259		115.0	1842.1	8.7	79.0	11.46	33.9	4.92	0.43
390260		121.4	1945.2	11.6	101.5	14.72	41.6	6.03	0.41
390261		120.7	1933.9	9.0	124.1	17.99	43.9	6.37	0.35
390262	A-6	120.4	1929.7	8.9	107.8	15.63	42.6	6.18	0.40
390263		119.4	1912.6	11.3	93.7	13.59	42.7	6.19	0.46
390264		112.4	1799.9	13.4	34.3	4.97	15.8	2.29	0.46
390265		121.9	1953.2	8.6	88.7	12.86	18.3	2.65	0.21
Average		120.7	1930.8	9.6	104.7	15.18	42.5	6.16	0.41
Standard. Deviation		4.3	68.6	1.8	56.8	8.24			
Coeff. Of Variation		0.04	0.04	0.18	0.54	0.54			

Table C.2 Water Table Elevations

Depth of water table below top of pavement 12/17/96 to 1/22/99, in meters (feet)

Section No.	Average		Maximum		Minimum	
	Depth	Elevation	Depth	Elevation	Depth	Elevation
390103	2.60(8.52)	(946.85)	3.71(12.17)	(943.20)	1.96(6.43)	(948.94)
390108	2.00(6.56)	(946.79)	2.87(9.42)	(943.93)	1.57(5.15)	(948.20)
390102*	1.58(5.18)	(948.51)	1.90(6.23)	(947.46)	1.26(4.13)	(949.56)
390104	1.20(3.94)	(952.06)	1.71(5.61)	(950.39)	0.80(2.62)	(953.38)
390901	2.53(8.30)	(947.22)	3.48(11.42)	(944.10)	1.70(5.58)	(949.94)
390204	2.77(9.09)	(946.47)	3.30(10.83)	(944.73)	2.39(7.84)	(947.72)
390212	1.73(5.68)	(951.47)	2.12(6.96)	(950.19)	1.47(4.82)	(952.33)
390201	1.60(5.25)	(949.62)	1.77(5.81)	(949.06)	1.38(4.53)	(950.34)
390208	2.56(8.40)	(945.96)	3.60(11.81)	(942.55)	2.02(6.63)	(947.73)

*Sensor destroyed after 3/12/97 reading

Laboratory Testing: The results of laboratory tests for moisture content, dry unit weight, and resilient modulus are shown in Table C-3. Resilient moduli (not the same as in-situ moduli calculated from FWD data) measured with a triaxial pressure chamber varied little with confining pressure, but decreased dramatically with increased deviator stress. The higher the clay content, the more sensitive resilient modulus was to moisture content. The following table summarizes these laboratory data at optimum moisture content:

Table C.3 Laboratory Tests

Soil Type	Moisture (%)	Unit Dry Weight		Deviator Stress		Resilient Modulus	
		kN/m3	pcf	kPa	psi	Mpa	ksi
				13.8	2.0	130	18.85
A-4	17	17.2	109.5	27.6	4.0	90	13.05
				41.4	6.0	75	10.88
				13.8	2.0	125	18.13
A-6	12	18.3	116.6	27.6	4.0	90	13.05
				41.4	6.0	70	10.15
				13.8	2.0	160	23.20
A-7-6	18.1	18.1	115.3	27.6	4.0	120	17.40
				41.4	6.0	100	14.50

<u>Summary</u>

Coefficients of variation calculated for various subgrade parameters indicate much smaller variations in density and moisture for the 36 mainline test sections (0.04 and 0.18 respectively) than for stiffness (modulus) measured with the FWD (0.54). Stiffness variations within the individual test sections were also rather high. While in-situ stiffness, which is the mode of support in a pavement structure, is related to density, moisture and soil type, there is no clear correlation between these parameters. Data in the table show stiffness and density to be somewhat related, with the effects of moisture being unclear.

In addition to the inherent complexity of relating subgrade moisture and density to stiffness, is the manner in which these parameters are measured. Two or three nuclear density measurements were obtained at widely spaced locations in each test section for subgrade approval. These measurements were quite localized and only evaluated material in the top 0.30 m (12 in.) of the layer being tested. The FWD applies a full-scale load and measures composite stiffness throughout the total depth of subgrade supporting the load. The FWD also requires much less time per reading, thus allowing for a more comprehensive assessment of the surface in question. Typically, a total of 21 FWD measurements were taken along the centerline and right wheel path of each test section.

Average subgrade moduli in the 36 mainline test sections, as determined with the FWD and the Boussinesq equation, varied from 34.3-205.3 Mpa (4.97-29.77 ksi) with an average of 104.7 Mpa (15.8 ksi). The standard deviation was 56.8 Mpa (8.24 ksi) and the coefficient of variation was 0.54. This six-fold difference in average moduli can have a dramatic effect on the performance of highway pavements, especially those designed for limited service or those exposed to heavy volumes of truck traffic. Add to this range in moduli the large coefficients of variation observed within test sections, and even larger differences in moduli become apparent throughout the project. Two examples of poor subgrade are in Sections 390159 and 390264, which were constructed a year later than the other mainline sections. Section 390264 had the lowest density and stiffness, and the highest moisture content of all the sections in the mainline pavement. Variations in standard pavement construction are likely to be even more dramatic.

It is interesting to note the order in which test sections have failed to date on the Ohio SHRP Test Road. By the summer of 1998, four sections on the mainline SPS-1 pavement had been removed and replaced. Sections 390102 and 390107 rutted badly within days of being opened to traffic. Section 390101 displayed similar distress a few weeks later. After less than a year of service, Section 390105 had a dramatic localized failure at the specific location where FWD measurements taken three weeks earlier showed significantly reduced stiffness in the pavement structure. These four sections displayed the lowest average composite stiffness of all sections on the mainline pavement when they were new and they failed in order of increasing stiffness. Likewise, an FWD profile of Section 390101 after it had been closed to traffic indicated that the lowest stiffness was in the most severely distressed area.

Conclusions

Based upon results obtained thus far on the Ohio SHRP Test Road, it appears that stiffness measured on the base and subgrade with the FWD is a much better representation of load carrying capacity than density and moisture measured with the nuclear density gauge. Also, this stiffness is a composite of the entire pavement structure in place at the time the measurements were taken, rather than just the top lift of material. Because FWD measurements are quite sensitive, they may also be used on in-service pavements to assess overall structural integrity, to identify localized areas of weakness which may require special attention prior to or during a major rehabilitation, and to design overlay thickness. Governmental agencies responsible for maintaining highway infrastructure should consider the measurement of stiffness to evaluate and monitor pavement condition.

APPENDIX D TECHNICAL NOTE "EARLY SPS-1 PERFORMANCE ON THE OHIO SHRP TEST ROAD"

Introduction

Governmental agencies responsible for providing a safe and serviceable pavement infrastructure utilize construction and material specifications to maintain some minimum level of quality and uniformity throughout their system. These specifications evolve over time and, in general, are written to achieve the best overall results with currently available resources in terms of materials, technology, and funding. Since entire construction projects cannot be tested for compliance, sampling techniques have been established to control quality. Despite these efforts, localized areas in the completed pavement structure can exhibit premature distress resulting from material deficiencies, construction oversights, excess moisture, or variability within the underlying support layers. Preliminary results from the Ohio SHRP Test Road point to the Falling Weight Deflectometer (FWD) as being an effective tool for monitoring the stiffness of individual layers in the pavement structure during construction and, thereby, providing the opportunity for repairing areas of reduced support prior to completion of the pavement. By eliminating these potential problems before the pavement is opened to traffic, performance will be greatly enhanced.

Variability in subgrade and base stiffness is a major contributor to premature distress on asphalt concrete pavements, as evidenced by localized failures where heavy traffic loads either punch through the pavement or cause severe wheel path rutting and cracking associated with poor support. To illustrate the amount of variability that can occur on a given project, FWD data obtained on four experimental sections on the Ohio SHRP Test Road which failed by the summer of 1998 were examined in detail. Design parameters included in these sections are as shown in the following table.

Table D.1 Design Parameters of Distressed SPS-1 Sections

Section No.	Thickness (in.)		Base Type	Drainage Present
	AC	Base		
390101	7	8	Dense Graded Aggregate	No
390102	4	12	Dense Graded Aggregate	No
390105	4	8	4" ATB/4" DGAB	No
390107	4	8	4" PATB/4"DGAB	Yes

Test sections in this particular pavement, constructed for the Specific Pavement Studies (SPS) experiment in the Long Term Pavement Performance (LTPP) Program, should have exhibited excellent uniformity because of the high profile of this project and because of the following conditions surrounding it:

1. The project was located in an area of very flat topography.
2. Preliminary borings suggested a relatively uniform soil along the three-mile project length.
3. The project was part of a national experiment, and the ODOT and LTPP placed a strong emphasis on the importance of having uniform test sections. Localized areas of weakness resulting in premature failure would skew the results of the experiment.
4. Provisions were made to replace any subgrade material that failed to meet ODOT specifications.
5. Extensive sampling and testing were performed throughout each construction phase.

Construction

During excavation for the U.S. 23 test pavement, any wet, organic or otherwise unsuitable subgrade material was removed and replaced with borrow from a pit adjoining the project. Under specifications used on the project, moisture and density were monitored with a nuclear density gauge as the excavated areas were built up to grade. The subgrade was then proof rolled to identify areas of weakness where corrective action might be required. Proof rolling certainly is a more comprehensive test of the stiffness and uniformity of the subgrade surface than widely spaced nuclear density tests, but the results are subjective, it is unreliable as indicated by variations noted later with the FWD, and it is not economical on small projects. Final acceptance of the subgrade in each 152.4 m (500 ft.) long test section was typically based on two or three randomly spaced nuclear density measurements obtained in the middle of the test lane and 0.30 m (12 in.) below the finished surface.

FWD Testing

Because the SPS-1 experiment was designed to evaluate the structural effectiveness of various design parameters in asphalt concrete (AC) pavement, the FWD was used to monitor in-situ composite stiffness as individual layers within the test sections were completed. The FWD applies a haversine load to the surface being tested through a 300mm (11.8 in.) diameter plate, and vertical deflections of the surface are measured at seven radial distances within the resulting basin generated by the load. These deflections reflect the stiffness of the pavement structure under the load, with lower deflections indicating a stiffer pavement. When more than one layer is present, back calculation techniques can be used to quantify the stiffness of individual layers within the pavement structure at the time of testing in terms of moduli of elasticity.

Different FWD load packages are used on the various layers within a pavement structure during construction. Lighter loads are applied to the subgrade than on the base and finished pavement. Even when testing with a single load package on any one layer, some differences in applied load will occur due to variations in pavement stiffness and variations inherent within the FWD system itself. For this reason, it is often convenient to normalize measured deflections to a standard load of 450 kg (1000 lbs.) in order to simplify data analysis or, perhaps, to compare FWD data with Dynaflect deflection data which are obtained with a sinusoidal load of 450 kg (1000 lbs.). All FWD data discussed herein have been normalized in this manner.

Performance

The following graphs show normalized FWD deflection profiles measured along the right wheelpath in the four failed SPS-1 test sections at various points in time, i.e. completion of the subgrade, completion of the base layer(s) and completion of the finished pavement prior to being opened to traffic on June 11, 1996. Additional profiles are provided on Sections 390101 and 390105 to show normalized Df1 deflections after failure and just before failure, respectively.

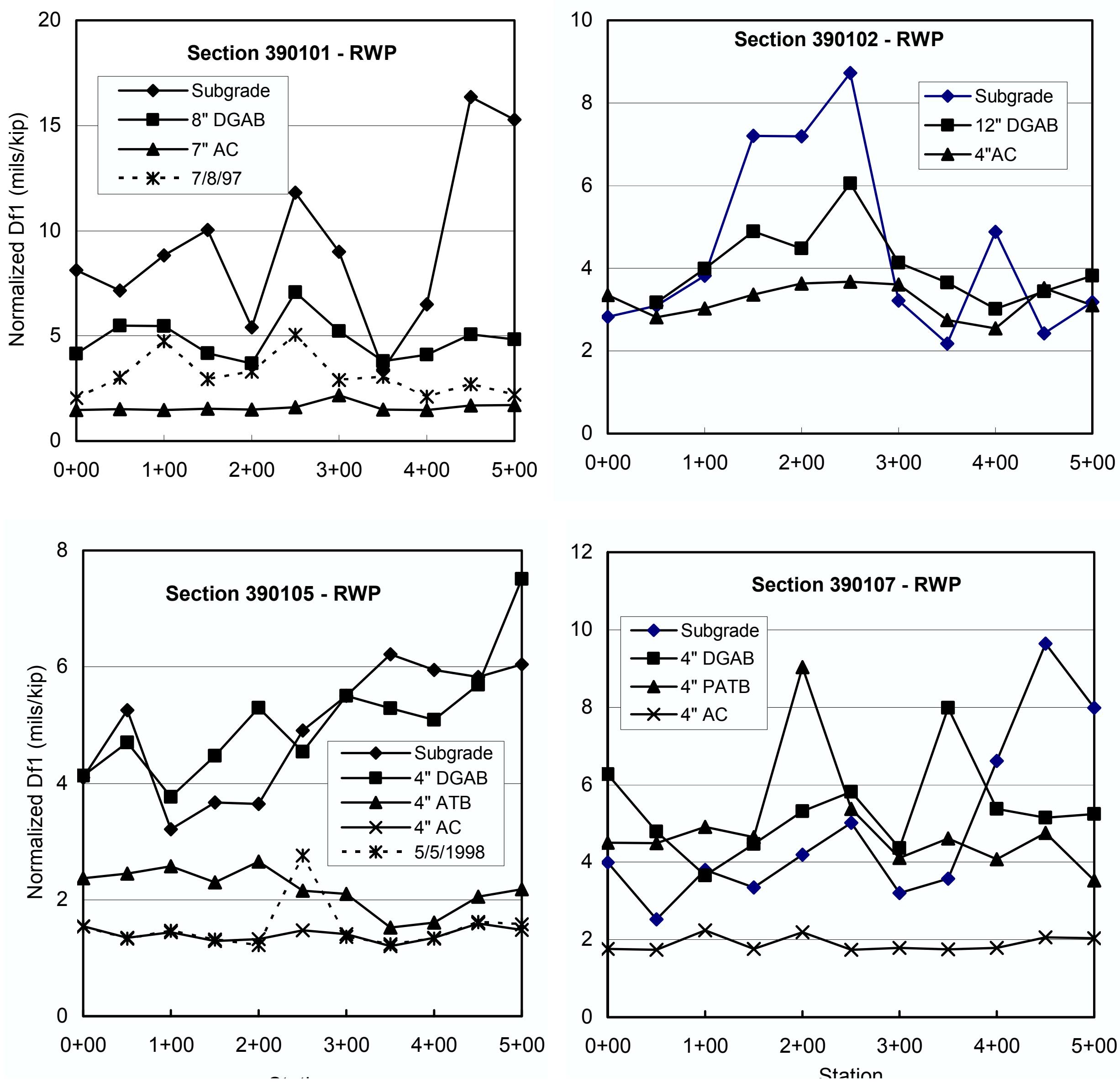


Figure D.1 Normalized Maximum FWD Df1 Profiles of Distressed SPS-1 Sections

Sections 390102 and 390107 were rutted throughout their length within a few weeks after being opened to traffic on August 14, 1996. Section 390101 showed severe rutting a short time later. The entire SPS-1 pavement was closed on December 3, 1996 to allow for the passage of winter, the reconstruction of the three distressed sections, and the completion of a third set of controlled vehicle tests in 1997. It was reopened on November 11, 1997. Section 390105 experienced a rather dramatic localized failure at Station 2+30 on May 29, 1998, about three weeks after FWD measurements indicated a localized weakness in that area. Several observations can be made from these graphs, including:

1. Despite efforts made to provide uniform support on this test pavement using ODOT and SHRP specifications, and nuclear density tests, subgrade stiffness was highly variable within and between the four failed 152.4-meter (500-ft.) long test sections. It

is likely, therefore, that subgrade stiffness on typical pavement projects is also highly variable.

2. While FWD measurements in the right wheel path were offset approximately three feet laterally from the middle of the lane where nuclear density measurements were taken for approval of the subgrade, satisfactory moisture/density readings were not indicative of uniform subgrade stiffness.

3. As new layers were added to the pavement structures, the magnitude and uniformity of stiffness in the total pavement structure generally improved in accordance with the stiffness of these new layers.

4. The addition of dense graded aggregate base on the subgrade did not increase the total composite stiffness of the structure at every location. This was especially true in areas where FWD-Df1 measurements on the subgrade alone were less than about 4 mils/kip.

5. Sections 390102 and 390107, which had the highest average initial deflections of any of the 36 mainline sections on the test road when they were newly completed, failed first.

6. Section 390101, with the third highest initial deflection, failed soon after Sections 390102 and 390107 failed. During a forensic investigation, the most severely distressed location in this section was Station 2+65, which was between the highest deflection measured on the DGAB (Station 2+50) and the highest initial deflection measured on the completed pavement (Station 3+00).

7. The fourth highest average initial deflection on the project was measured in Section 390105. This section failed at Station 2+30, near where FWD readings taken three weeks earlier indicated severe localized weakness in the pavement structure. Measurements obtained elsewhere in the section were very similar to those recorded two years earlier when the pavement was new. There were no obvious indications from earlier FWD data of unusually low stiffness anywhere in Section 390105.

8. Based upon FWD measurements obtained in these four AC test sections designed for limited service, severe pavement distress occurred when normalized deflection under the load plate (Df1) approached 2 mils/kip on the completed pavement.

9. Failure did not always occur at specific locations where high FWD deflections were measured on the subgrade or base. This may, in part, be due to other weaker areas not being detected between these test points, which were spaced 15.2 m (50 ft.) apart. While FWD sampling on these sections was much more comprehensive than the nuclear density sampling, it still represented a small percentage of the surface being evaluated.

Conclusions

FWD measurements are an early indicator of the structural integrity of AC pavements and it appears they may be used to predict future performance. As of June 1998, failures had occurred on the four SPS-1 sections with the highest average deflection measured on the newly completed pavement and in the order of increasing stiffness. In Sections 390101 and 390105, the earliest and most severe distresses were located in specific areas with the highest individual FWD measurements at the approximate time of the failures. These early section failures cannot be attributed solely to any particular pavement layer, but to the combination of parameters in the SPS-1 matrix which, by design, limited performance. Distress appears to be imminent, at least on thin section in-service pavements, when normalized deflection under the FWD load plate approaches 2 mils/kip with a 4050 kg (9000 lb.) load.

Although somewhat related, soil density is not a reliable indicator of in-situ subgrade stiffness. While the addition of moisture may increase soil density, it may at the same time reduce in-situ stiffness. Also, nuclear density measurements are labor intensive, the depth of sampling is limited to 0.30 m (12 in.), and tests taken every 45-75 m (150 – 250 ft.) constitute a very small sample of the total subgrade area being evaluated. FWD measurements can be taken rapidly, thereby allowing a broader sampling of the subgrade surface and, on subsequent layers, stiffness is integrated over the total depth of the pavement structure supporting the applied load. FWD measurements also provide a better representation of how pavement structures actually carry traffic loads.

Items of equipment other than the FWD that offer some potential benefits in measuring subgrade, base and pavement stiffness are the Dynaflect trailer, which operates very much like the FWD, the Humboldt Soil Stiffness Gauge (HSSG) and the Dynamic Cone Penetrometer (DCP). The HSSG is a hand-held device, which measures stiffness at the rate of about one test per minute. Because the HSSG only measures stiffness in the upper six inches of the subgrade, measurements need to made in individual layers as the subgrade is built up to grade. The DCP applies a standard amount of energy to a rod as it is driven into unstabilized base or subgrade. The rate of penetration is continuously monitored such that specific layers of weakness within the structure which permit the rod to pass through easily can be identified for corrective action. It requires approximately five minutes to test the subgrade to a depth of four feet at each location.

APPENDIX E 1998 FWD DEFLECTION PROFILES ON SHRP TEST SECTIONS OHIO SHRP TEST ROAD

Table E.1 Ohio SHRP Test Road – Normalized FWD Df1 Profiles in AC Sections – May 1998

Section No.	Test Date	Pvt. Surf. Temp. (°F)	Test Path	Avg. Load (K)	Normalized Df1 Measurements (mils/kip) at Station												
					0+00	0+50	1+00	1+50	2+00	2+50	3+00	3+50	4+00	4+50	5+00	Average Midlane	Average RWP
Southbound SPS-1 (AC)																	
390101	Section failed - Removed from service																
390102	Section failed - Removed from service																
390103	5/5/98	60	Midlane	9.13	0.87	0.70	0.82	0.84	0.83	0.75	0.74	0.64	0.72	0.65	0.63	0.74	
			RWP	9.07	0.84	0.83	0.90	0.85	0.77	0.83	0.75	0.71	0.91	0.69	0.70		0.80
390104	5/5/98	85	Midlane														
			RWP	9.57	0.43	0.35	0.40	0.43	0.36	0.46	0.38	0.40	0.47	0.37	0.37		0.40
390105	5/5/98	71	Midlane														
			RWP	9.44	1.54	1.33	1.47	1.32	1.22	2.76	1.36	1.24	1.33	1.62	1.58		1.52
390106	5/5/98	85	Midlane														
			RWP	9.72	0.53	0.49	0.55	0.49	0.49	0.49	0.51	0.44	0.46	0.49	0.49		0.49
390107	Section failed - Removed from service																
390108	5/5/98	60	Midlane	9.40	0.85	0.86	0.86	0.89	0.81	0.79	0.84	0.93	0.98	0.74	0.90	0.86	
			RWP	9.53	0.99	0.96	0.94	1.02	0.96	0.80	0.74	1.00	0.90	0.83	1.06		0.93
390109	5/5/98	60	Midlane	9.49	0.99	0.93	0.84	0.85	0.94	0.86	0.82	0.81	0.77	0.70	0.95	0.86	
			RWP	9.50	0.80	0.80	0.71	0.73	0.87	0.79	0.80	0.67	0.71	0.68	0.96		0.77
390110	5/5/98	60	Midlane	9.18	0.55	0.53	0.49	0.45	0.59	0.59	0.57	0.56	0.63	0.65	0.76	0.58	
			RWP	9.26	0.56	0.48	0.51	0.50	0.57	0.64	0.61	0.63	0.55	0.56	0.64		0.57
390111	5/5/98	88	Midlane														
			RWP	9.42	0.56	0.65	0.55	0.54	0.55	0.58	0.59	0.72	0.73	0.76	0.66		0.63
390112	5/5/98	88	Midlane														
			RWP	8.93	0.43	0.47	0.47	0.45	0.52	0.53	0.47	0.54	0.51	0.44	0.46		0.48
390159	5/5/98	60	Midlane	9.30	0.24	0.20	0.17	0.20	0.17	0.17	0.22	0.17	0.20	0.18	0.21	0.19	
			RWP	9.16	0.21	0.23	0.18	0.24	0.19	0.19	0.20	0.18		0.21	0.24		0.21
390160	5/5/98	71	Midlane														
			RWP	9.59	0.47	0.50	0.54	0.50	0.50	0.50	0.50	0.36	0.50	0.55	0.55		0.50
390161 (390102)	5/5/98	71	Midlane														
			RWP	9.45	0.46	0.51	0.47	0.53	0.53	0.50	0.57	0.52	0.47	0.51	0.57		0.51
390162 (390107)	5/5/98	80	Midlane														
			RWP	9.71	0.40	0.45	0.43		0.38	0.35	0.43	0.49	0.34	0.35	0.43		0.41
390163 (390101)	5/5/98	85	Midlane														
			RWP	9.63	0.35	0.41	0.39	0.41	0.41	0.39	0.50	0.51	0.56	0.45	0.51		0.44
Ramp SPS-8 (AC)																	
39A803 (390803)	7/14/98	76	Midlane	9.93	1.25	1.28	1.25	1.27	1.31	1.31	1.26	1.36	1.38	1.43	1.40	1.32	
			RWP														
39A804 (390804)	7/13/98	75	Midlane	9.27	1.10	1.07	1.09	1.03	1.01	0.98	0.98	0.88	0.89	0.94	0.92	0.99	
			RWP														
Southbound SPS-9 (AC)																	
390901	5/5/98	80	Midlane														
			RWP	9.11	0.37	0.36	0.47	0.37	0.38	0.36	0.37	0.32	0.32	0.40	0.36		0.37
390902	5/5/98	88	Midlane														
			RWP	9.19	0.27	0.26	0.35	0.43	0.42	0.40	0.42	0.32	0.33	0.31	0.35		0.35
390903	5/5/98	88	Midlane														
			RWP	9.24	0.35	0.44	0.46	0.49	0.49	0.43	0.50	0.45	0.40	0.35	0.43		0.44

Table E.2 Ohio SHRP Test Road – Normalized FWD Df1 Profiles in PCC Sections – May 1998

Section No.	Test Date	Pvt. Surf. Temp. (^oF)	Test Path	Avg. Load (K)	Normalized Df1 Measurements (mils/kip) at Station											Average Midlane	Joint Average RWP
					0+00	0+50	1+00	1+50	2+00	2+50	3+00	3+50	4+00	4+50	5+00		
Northbound SPS-2 (PCC)																	
				Normalized Df2 Measurements (mils/kip)													
390201	5/4/98	65	RWP	11.82	0.68		0.54	0.67	0.62		0.55	0.59	0.63	0.53	0.58		0.60
			RWP-Jt**	11.76	.66/.56					.68/.56				.60/.50			.65/.54
390202	5/4/98	65	Midlane	11.77	0.53	0.60	0.61	0.63	0.42	0.50	0.63	0.56	0.53	0.50	0.70	0.56	
			RWP-Jt**	11.89	.88/.78		.69/.64			1.00/.85		.77/.69		.82/.67			.83/.73
390203	5/4/98	71	Midlane	11.59	0.37	0.32	0.39	0.35	0.34	0.36	0.31	0.34	0.36	0.30	0.29	0.34	
			RWP-Jt**														
390204	5/4/98	58	Midlane	12.94	0.23	0.24	0.23	0.26	0.24	0.23	0.22	0.23	0.24	0.28	0.26	0.24	
			RWP-Jt**	12.39	.53/.44		.48/.40		.45/.44		.60/.48		.56/.44		.53/.48		.53/.45
390205	5/4/98	65	Midlane	12.09	0.49	0.38	0.43	0.44	0.44	0.46					0.42	0.44	
			Mdln-Jt**	12.17						.47/.44		.47/.45		.58/.52		.51/.47	
390206	5/4/98	65	Midlane	12.07	0.42	0.50	0.45	0.41	0.56	0.48	0.46	0.48	0.52	0.35	0.37	0.45	
			RWP-Jt**														
390207	5/4/98	75	Midlane	11.87	0.26	0.26	0.25	0.26	0.29	0.26	0.28	0.29	0.30	0.29	0.31	0.28	
			RWP-Jt**														
390208	5/4/98	75	Midlane	11.94	0.26	0.26	0.25	0.30	0.30	0.30	0.28	0.26	0.28	0.32	0.29	0.28	
			RWP-Jt**														
390209	5/4/98	71	RWP	11.99	0.53	0.53	0.52	0.43	0.48		0.33	0.43	0.53	0.46	0.48		0.47
			RWP-Jt**	11.71	.76/.61					.62/.51							.69/.56
390210	5/4/98	62	Midlane	12.11	0.43	0.35	0.38	0.36	0.44	0.35	0.34	0.36	0.39	0.36	0.34	0.37	
			RWP-Jt**	12.01	.75/.61		.59/.49		.68/.53			.56/.44		.55/.41			.63/.50
390211	5/4/98	71	Midlane	11-98	0.26	0.28	0.26	0.31	0.30	0.30	0.27	0.29	0.29	0.27	0.26	0.28	
			RWP-Jt**														
390212	5/4/98	62	Midlane	12.20	0.27	0.25	0.25	0.24	0.24	0.23	0.27	0.28	0.28	0.27	0.25	0.26	
			RWP-Jt**	11.86	.53/.46		.53/.39		.52/.45			.60/.53		.51/.43			.54/.45
390259	5/4/98	58	RWP	12.93			0.37				0.43						0.40
			RWP-Jt**	12.88	.69/.41		.52/.43		.58/.52		.60/.52		.57/.49		.53/.45		.58/.47
390260	5/4/98	65	Midlane	11.88	0.25	0.24	0.27	0.22	0.22	0.30	0.29	0.23	0.24	0.25	0.23	0.25	
			RWP-Jt**	11.65	.63/.50		.57/.49		.38/.32		.47/.38				.47/.38		.50/.41
390261	5/4/98	71	Midlane	11.88	0.23	0.23	0.24	0.24	0.25	0.24	0.22	0.22	0.24	0.22	0.23	0.23	
			RWP-Jt**														
390262	5/4/98	75	Midlane	12.09	0.23	0.19	0.20	0.21	0.24	0.24	0.20	0.24	0.23	0.23	0.19	0.22	
			RWP-Jt**														
390263	5/4/98	82	Midlane	11.67	0.35	0.38	0.44	0.41	0.42	0.42	0.59	0.43	0.40	0.34	0.33	0.41	
			RWP-Jt**														
390264	5/4/98	82	Midlane	11.35	0.37	0.37	0.41		0.36	0.43	0.35	0.36	0.38	0.39	0.36	0.38	
			RWP-Jt**														
390265	5/4/98	71	RWP	11.58	0.27	0.28	0.27	0.25	0.34	0.31	0.26	0.30	0.29	0.28	0.30	0.29	
			RWP-Jt**														

* Df2 was used for these SPS-2 readings. Df1 was erratic and often less than Df2. Df1 is typically 5-10% greater than Df2.

** Df2A/Df2L at joint closest to station

APPENDIX F 2001 FWD AND DYNATEST MEASUREMENTS OHIO SHRP TEST ROAD

Table F.1 Ohio SHRP Test Road – Normalized FWD Df1 Profiles in AC Sections – April 2001

Section No.	Test Date	Pvt. Surf. Temp. (^oF)	Test Path	Normalized FWD Df1 in mils/kip at Station 0+00	0+50	1+00	1+50	2+00	2+50	3+00	3+50	4+00	4+50	5+00	Average Midlane (mils/kip)	Average RWP (mils/kip)	Section Average (mils/kip)
Southbound SPS-1 (AC)																	
390101	Section Replaced																
390102	Section Replaced																
390103	4/11	59	Midlane	1.43	1.15	1.40	1.30	1.17	1.14	1.13	1.12	1.35	1.17	1.23	1.24		1.25
			RWP	1.43	1.12	1.29	1.44	1.12	1.39	1.18	1.16	1.51	1.08	1.21		1.27	
390104	4/11	70	Midlane	0.63	0.53	0.49	0.50	0.55	0.55	0.54	0.52	0.57	0.51	0.52	0.54		0.52
			RWP	0.52	0.47	0.49	0.50	0.51	0.54	0.53	0.50	0.53	0.52	0.50		0.51	
390105	Section Replaced																
390106	4/11	67	Midlane	0.78	0.68	0.72	0.67	0.63	0.61	0.68	0.61	0.65	0.80	0.64	0.68		0.67
			RWP	0.70	0.73	0.71	0.63	0.62	0.56	0.65	0.61	0.63	0.68	0.65		0.65	
390107	Section Replaced																
390108	4/11	64	Midlane	1.37	1.40	1.42	1.23	1.25	1.05	1.10	1.15	1.18	1.01	1.26	1.22		1.27
			RWP	1.67	1.48	1.52	1.26	1.28	1.11	1.20	1.17	1.22	1.07	1.48		1.31	
390109	4/11	61	Midlane	0.97	1.08	0.87	0.85	0.97	0.92	0.94	0.86	0.92	1.09	1.35	0.98		1.01
			RWP	1.08	1.15	0.97	0.89	1.03	1.03	1.01	0.90	0.82	1.21	1.26		1.03	
390110	4/11	59	Midlane	0.77	0.74	0.71	0.71	0.72	0.78	0.76	0.83	0.75	0.82	0.89	0.77		0.81
			RWP	0.83	0.84	0.74	0.75	0.83	0.90	0.77	0.92	0.80	0.93	0.93		0.84	
390111	4/11	61	Midlane	0.73	0.78	0.75	0.70	0.67	0.68	0.73	0.82	0.88	0.89	0.81	0.77		0.77
			RWP	0.70	0.77	0.72	0.69	0.61	0.69	0.72	0.79	0.88	0.99	0.94		0.77	
390112	4/12	68	Midlane	0.46	0.56	0.54	0.50	0.57	0.53	0.53	0.53	0.50	0.48	0.49	0.52		0.52
			RWP	0.45	0.52	0.53	0.50	0.60	0.51	0.52	0.54	0.53	0.50	0.50		0.52	
390159			Midlane														
			RWP														
390160	4/11	59	Midlane	0.60	0.68	0.66	0.60	0.55	0.54	0.57	0.51	0.68	0.60	0.69	0.61		0.59
			RWP	0.66	0.67	0.56	0.53	0.60	0.50	0.51	0.45	0.57	0.58	0.64		0.57	
390161 (390102)	4/11	59	Midlane	0.44	0.41	0.44	0.43	0.48	0.49	0.48	0.46	0.52	0.46	0.54	0.47		0.46
			RWP	0.47	0.43	0.45	0.41	0.47	0.47	0.47	0.45	0.48	0.48	0.49		0.46	
390162 (390107)	4/11	59	Midlane	0.35	0.32	0.35	0.31	0.31	0.33	0.30	0.35	0.28	0.27	0.33	0.32		0.30
			RWP	0.33	0.28	0.29	0.28	0.30	0.26	0.27	0.32	0.29	0.25	0.30		0.29	
390163 (390101)	4/11	61	Midlane	0.35	0.31	0.30	0.40	0.34	0.34	0.37	0.36	0.33	0.37	0.40	0.35		0.35
			RWP	0.38	0.29	0.32	0.38	0.28	0.36	0.38	0.32	0.30	0.40	0.37		0.34	
390164 (390105)	4/11	59	Midlane	1.01	1.02	1.04	1.12	0.88	0.84	0.83	0.96	0.90	0.92	0.87	0.94		0.94
			RWP	1.15	1.07	0.94	1.00	0.83	0.83	0.83	0.99	0.92	0.90	0.89		0.94	
Ramp SPS-8 (AC)																	
390803	Section Replaced																
390804	Section Replaced																
39A803 (390803)	4/5	44	Midlane	1.49	1.60	1.60	1.09	1.28	0.95	1.10	1.07	0.99	1.24	1.29	1.25		1.17
			RWP	1.14	1.53	1.31	0.94	1.15	0.90	1.01	1.02	0.94	0.98	1.07		1.09	
39A804 (390804)	4/5	44	Midlane	0.60	0.55	0.61	0.57	0.64	0.54	0.55	0.55	0.44	0.40	0.62	0.55		0.57
			RWP	0.64	0.61	0.65	0.66	0.68	0.59	0.61	0.54	0.45	0.42	0.57		0.58	
Southbound SPS-9 (AC)																	
390901	4/12	70	Midlane	0.44	0.41	0.45	0.41	0.41	0.38	0.38	0.36	0.40	0.38	0.40	0.40		0.41
			RWP	0.41	0.42	0.45	0.38	0.42	0.38	0.38	0.38	0.47	0.43	0.47		0.42	
390902	4/12	68	Midlane	0.26	0.28	0.30	0.31	0.31	0.33	0.32	0.29	0.30	0.29	0.30	0.30		0.30
			RWP	0.29	0.28	0.30	0.32	0.32	0.34	0.34	0.29	0.29	0.29	0.30		0.31	
390903	4/12	70	Midlane	0.45	0.48	0.52	0.51	0.51	0.46	0.46	0.45	0.40	0.39	0.43	0.46		0.45
			RWP	0.36	0.45	0.47	0.48	0.49	0.48	0.47	0.47	0.41	0.40	0.41		0.44	

Table F.2 Ohio SHRP Test Road – Normalized FWD Df1 Profiles in PCC Sections – April 2001

Section No.	Test Date	Pvt. Surf. Temp. (^oF)	Test Path	Normalized FWD Df1 in mils/kip at Station											Average Midlane (mils/kip)	Average Df1A/Df1L (mils/kip) Load Transfer (%)
				0+00	0+50	1+00	1+50	2+00	2+50	3+00	3+50	4+00	4+50	5+00		
Northbound SPS-2 (PCC)																
390201	4/3	54	Midlane	0.47	0.49	0.56	0.64	0.60	0.48	0.57	0.54	0.53	0.51	0.59	0.54	
			RWP-Df1*	.75/.82		1.04/.94			.87/.82		.91/.91		.82/.73			.88/.84
			RWP-LT**	92.2		89.0			69.9		85.8		92.6			85.9
390202	4/3	53	Midlane	0.48	0.98	0.83	0.67	0.60	0.63	0.42	0.47	0.53	0.53	0.67	0.62	
			RWP-Df1*	.72/.75		.80/.76			.73/.68		.73/.64		.72/.67			.74/.70
			RWP-LT**	98.1		95.7			92.1		96.0		98.2			96.0
390203	4/4	39	Midlane	0.34	0.35	0.36	0.30	0.33	0.33	0.30	0.32	0.34	0.32	0.29	0.33	
			RWP-Df1*	.64/.65		.57/.56			.73/.65		.60/.56		.76/.66			.66/.62
			RWP-LT**	94.8		91.8			72.5		91.9		66.8			83.6
390204	4/3	39	Midlane	0.27	0.26	0.28	0.30	0.26	0.25	0.29	0.25	0.25	0.28	0.27	0.27	
			RWP-Df1*	.99/.93		.57/.57			.53/.53		.63/.60		.62/.59			.67/.64
			RWP-LT**	98.5		99.4			98.4		88.9		94.7			96.0
390205	4/3	53	Midlane	0.60	0.70	0.52	0.50	0.47	0.57	0.44	0.47	0.44	0.47	0.43	0.51	
			RWP-Df1*	.70/.69		.54/.51			.60/.57		.56/.57		.62/.56			.60/.58
			RWP-LT**	91.4		88.7			87.6		91.5		86.8			89.2
390206	4/4	53	Midlane	0.48	0.47	0.50	0.43	0.56	0.55	0.52	0.54	0.57	0.41	0.46	0.50	
			RWP-Df1*	.58/.57		.55/.51			.65/.61		.83/.80		.63/.62			.65/62
			RWP-LT**	88.9		90.8			89.4		91.1		89.9			90.0
390207	4/4	36	Midlane	0.26	0.26	0.28	0.23	0.24	0.23	0.29	0.28	0.29	0.32	0.31	0.27	
			RWP-Df1*	.53/.47		.40/.36			.39/.36		.44/.39		.53/.48			.46/.41
			RWP-LT**	86.5		83.1			83.0		84.0		83.2			84.0
390208	4/4	51	Midlane	0.30	0.32	0.30	0.29	0.30	0.31	0.29	0.28	0.29	0.28	0.32	0.30	
			RWP-Df1*	.47/.45		.47/.43			.48/.47		.50/.45		.45/.40			.47/.44
			RWP-LT**	81.4		90.6			92.9		87.3		95.3			89.5
390209	4/4	54	Midlane	0.50	0.50	0.54	0.47	0.53	0.39	0.36	0.44	0.46	0.44	0.45	0.46	
			RWP-Df1*	.77/.70		.68/.64			.62/.56		.69/.62		.68/.61			.69/.63
			RWP-LT**	89.6		93.7			93.9		91.1		95.2			92.7
390210	4/3	42	Midlane	0.40	0.41	0.39	0.40	0.47	0.39	0.38	0.40	0.41	0.38	0.35	0.40	
			RWP-Df1*	.67/.63		.62/.59			.63/.63		.64/.58		.58/.55			.63/.60
			RWP-LT**	91.3		87.7			93.1		89.5		90.2			90.4
390211	4/4	35	Midlane	0.27	0.26	0.27	0.28	0.30	0.28	0.26	0.28	0.26	0.25	0.26	0.27	
			RWP-Df1*	.49/.44		.55/.53			.47/.45		.50/.48		.49/.46			.50/.47
			RWP-LT**	89.1		94.6			91.2		87.9		89.3			90.4
390212	4/3	43	Midlane	0.28	0.25	0.25	0.24	0.24	0.24	0.26	0.28	0.28	0.28	0.26	0.26	
			RWP-Df1*	.57/.58		.55/.55			.58/.57		.58/.59		.58/.55			.57/.57
			RWP-LT**	99.4		96.7			97.0		98.0		95.7			97.4
390259	4/3	39	Midlane	0.30	0.30	0.28	0.29	0.30	0.35	0.31	0.35	0.31	0.30	0.28	0.31	
			RWP-Df1*	.60/.63		.56/.54			.69/.63		.60/.61		.62/.60			.61/.60
			RWP-LT**	94.5		94.6			95.2		96.2		96.9			95.5
390260	4/3	42	Midlane	0.25	0.24	0.25	0.23	0.23	0.25	0.27	0.23	0.25	0.26	0.25	0.25	
			RWP-Df1*	.50/.46		.43/.40			.39/.37		.34/.30		.50/.44			.43/.39
			RWP-LT**	88.0		94.3			90.8		87.0		91.3			90.3
390261	4/4	35	Midlane	0.24	0.23	0.23	0.23	0.23	0.24	0.21	0.22	0.23	0.21	0.22	0.23	
			RWP-Df1*	.44/.41		.48/.48			.40/.39		.47/.46		.45/.43			.45/.43
			RWP-LT**	89.2		93.0			86.4		91.0		90.9			90.1
390262	4/4	53	Midlane	0.28	0.23	0.22	0.22	0.23	0.24	0.25	0.26	0.25	0.24	0.24	0.24	
			RWP-Df1*	.45/.41		.45/.41			.46/.43		.49/.46		.44/.42			.46/.43
			RWP-LT**	91.8		88.1			90.1		91.4		91.9			90.7
390263	4/4	53	Midlane	0.40	0.38	0.45	0.45	0.58	0.38	0.45	0.43	0.39	0.36	0.32	0.42	
			RWP-Df1*	.58/.55		.62/.53			.51/.49		.52/.49		.57/.52			.55/.52
			RWP-LT**	69.0		63.7			83.4		87.7		90.4			78.8
390264	4/4	53	Midlane	0.31	0.36	0.43	0.39	0.34	0.34	0.33					0.36	
			RWP-Df1*													
			RWP-LT**													
390265	4/4	35	Midlane	0.26	0.28	0.26	0.22	0.29	0.24	0.28	0.28	0.25	0.26	0.27	0.26	
			RWP-Df1*	.50/.46		.48/.41			.43/.42		.59/.58		.57/.54			.51/.48
			RWP-LT**	91.2		84.6			92.2		95.7		92.0			91.1
Ramp SPS-8 (PCC)																
390809	4/5	38	Midlane	0.72	0.72	0.74	0.71	0.74	2.02	0.63	0.62	0.68	0.62	0.63	0.80	
			RWP-Df1*	1.51/1.76		1.44/1.45			1.25/1.61		1.37/1.59		1.48/1.43			1.41/1.57
			RWP-LT**	99.5		90.6			102.2		91.3		82.2			93.2
390810	4/5	38	Midlane	0.58	0.38	0.47	0.35	0.37	0.39	0.34	0.39	0.39	0.43	0.40	0.41	
			RWP-Df1*	1.22/1.34		.94/.99			.94/.91		.96/.86		1.08/1.15			1.03/1.05
			RWP-LT**	94.3		101.1			95.6		82.0		91.0			92.8

Table F.3 Ohio SHRP Test Road – Dynaflect Profiles in AC Sections - April 2001

Section No.	Test Date	Pvt. Surf. Temp. (^oF)	Test Path	Dynaflect W1 in mils/kip at Station											Average Midlane (mils/kip)	Average RWP (mils/kip)	Section Average (mils/kip)
				0+00	0+50	1+00	1+50	2+00	2+50	3+00	3+50	4+00	4+50	5+00			
Southbound SPS-1 (AC)																	
390101	Section Replaced																
390102	Section Replaced																
390103	4/4	45	Midlane	1.08	1.03	1.27	1.12	1.00	0.91	0.89	1.03	1.04	0.98	0.82	1.02		1.01
			RWP	1.20	0.98	1.13	1.14	0.94	1.10	0.97	0.87	1.10	0.87	0.84		1.01	
390104	4/10	59	Midlane	0.49	0.41	0.45	0.45	0.50	0.55	0.47	0.47	0.49	0.48	0.44	0.47		0.47
			RWP	0.45	0.41	0.46	0.47	0.49	0.56	0.48	0.45	0.47	0.47	0.44		0.47	
390105	Section Replaced																
390106	4/10	59	Midlane	0.57	0.65	0.71	0.59	0.58	0.60	0.61	0.53	0.53	0.68	0.60	0.60		0.59
			RWP	0.60	0.64	0.66	0.57	0.54	0.55	0.61	0.53	0.50	0.60	0.56		0.58	
390107	Section Replaced																
390108	4/4	58	Midlane	1.15	1.00	0.99	0.95	0.90	0.79	0.81	0.81	1.03	0.79	0.90	0.92		0.94
			RWP	1.13	1.02	1.02	1.00	0.91	0.84	0.89	0.97	0.96	0.82	0.95		0.96	
390109	4/4	53	Midlane	0.71	0.79	0.72	0.66	0.77	0.69	0.73	0.60	0.63	0.70	0.81	0.71		0.73
			RWP	0.83	0.87	0.76	0.71	0.77	0.75	0.75	0.64	0.68	0.74	0.83		0.76	
390110	4/4	52	Midlane	0.69	0.59	0.58	0.55	0.71	0.71	0.68	0.74	0.65	0.69	0.81	0.67		0.69
			RWP	0.67	0.60	0.57	0.57	0.78	0.78	0.72	0.80	0.72	0.71	0.87		0.71	
390111	4/10	59	Midlane	0.60	0.71	0.59	0.55	0.57	0.59	0.66	0.76	0.77	0.78	0.77	0.67		0.66
			RWP	0.60	0.69	0.57	0.55	0.53	0.61	0.64	0.72	0.75	0.83	0.78		0.66	
390112	4/10	61	Midlane	0.44	0.51	0.51	0.47	0.54	0.53	0.48	0.46	0.47	0.45	0.46	0.48		0.48
			RWP	0.43	0.45	0.51	0.48	0.55	0.52	0.50	0.47	0.48	0.45	0.46		0.48	
390159			Midlane														
			RWP														
390160	4/10	52	Midlane	0.61	0.68	0.59	0.56	0.51	0.50	0.51	0.42	0.65	0.51	0.62	0.56		0.55
			RWP	0.66	0.68	0.55	0.53	0.51	0.48	0.47	0.40	0.54	0.52	0.62		0.54	
390161 (390102)	4/10	54	Midlane	0.39	0.38	0.39	0.37	0.41	0.42	0.42	0.42	0.45	0.43	0.52	0.42		0.42
			RWP	0.39	0.37	0.39	0.38	0.42	0.42	0.43	0.42	0.43	0.42	0.52		0.42	
390162 (390107)	4/10	58	Midlane	0.29	0.28	0.28	0.28	0.26	0.26	0.26	0.29	0.24	0.24	0.27	0.27		0.26
			RWP	0.27	0.26	0.26	0.26	0.25	0.25	0.24	0.27	0.23	0.22	0.26		0.25	
390163 (390101)	4/10	59	Midlane	0.26	0.23	0.23	0.28	0.24	0.26	0.27	0.29	0.28	0.32	0.32	0.27		0.26
			RWP	0.25	0.22	0.23	0.27	0.21	0.24	0.26	0.27	0.26	0.32	0.31		0.26	
390164 (390105)	4/4	65	Midlane	0.75	0.79	0.83	0.80	0.61	0.61	0.58	0.67	0.67	0.62	0.75	0.70		0.72
			RWP	0.80	0.83	0.83	0.86	0.65	0.68	0.65	0.70	0.71	0.66	0.77		0.74	
Ramp SPS-8 (AC)																	
390803	Section Replaced																
390804	Section Replaced																
39A803 (390803)	4/4	41	Midlane	0.75	0.98	0.88	0.62	0.76	0.67	0.66	0.63	0.59	0.66	0.74	0.72		0.70
			RWP	0.83	0.78	0.70	0.59	0.68	0.52	0.61	0.68	0.59	0.80	0.67		0.68	
39A804 (390804)	4/4	45	Midlane	0.44	0.40	0.43	0.43	0.41	0.36	0.37	0.35	0.30	0.32	0.40	0.38		0.39
			RWP	0.51	0.42	0.44	0.44	0.42	0.40	0.38	0.36	0.31	0.32	0.36		0.40	
Southbound SPS-9 (AC)																	
390901	4/10	72	Midlane	0.35	0.39	0.48	0.43	0.45	0.42	0.36	0.33	0.35	0.34	0.37	0.39		0.39
			RWP	0.38	0.39	0.47	0.41	0.45	0.41	0.37	0.34	0.38	0.36	0.41		0.40	
390902	4/10	63	Midlane	0.23	0.23	0.30	0.29	0.31	0.36	0.30	0.26	0.28	0.27	0.28	0.28		0.28
			RWP	0.22	0.23	0.27	0.30	0.32	0.36	0.31	0.25	0.26	0.26	0.29		0.28	
390903	4/10	70	Midlane	0.32	0.42	0.48	0.50	0.47	0.43	0.48	0.40	0.34	0.33	0.37	0.41		0.41
			RWP	0.31	0.42	0.45	0.50	0.47	0.46	0.47	0.43	0.34	0.36	0.37		0.42	

Table F.4 Ohio SHRP Test Road – Dynaflect Profiles in PCC Sections – April 2001

Section No.	Test Date	Pvt. Surf. Temp. (°F)	Test Path	Normalized FWD Df1 in mils/kip at Station											Average Midlane	Average Df1A/Df1L (mils/kip)
				0+00	0+50	1+00	1+50	2+00	2+50	3+00	3+50	4+00	4+50	5+00	(mils/kip)	Load Transfer (%)
Northbound SPS-2 (PCC)																
390201	4/3	48	Midlane	0.58	0.49	0.58	0.61	0.60	0.48	0.65	0.66	0.61	0.54	0.56	0.58	
			RWP-W1*	1.17/.90		1.21/1.34			1.00/1.30		1.48/1.67		.83/1.14			1.14/1.27
			RWP-LT**	84.6		81.8			89.0		71.6		90.4			83.5
390202	4/3	42	Midlane	0.55	0.56	0.58	0.47	0.42	0.55	0.61	0.59	0.56	0.68	0.73	0.57	
			RWP-W1*	1.13/1.14		.84/.78			.84/.76		1.72/2.04		1.47/1.53			1.20/1.25
			RWP-LT**	86.7		84.5			83.3		90.1		87.1			86.3
390203	4/3	60	Midlane	0.41	0.42	0.44	0.38	0.42	0.41	0.37	0.45	0.40	0.41	0.36	0.41	
			RWP-W1*	.51/.51		.54/.62			.60/.67		.53/.57		.53/.54			0.54/0.58
			RWP-LT**	76.5		85.2			86.7		88.7		86.8			84.8
390204	4/3	39	Midlane	0.30	0.32	0.41	0.41	0.33	0.31	0.41	0.47	0.42	0.37	0.43	0.38	
			RWP-W1*	.81/.75		.73/.80			.76/.75		1.21/1.21		1.47/1.62			1.00/1.03
			RWP-LT**	87.7		87.7			86.8		86.8		89.8			87.8
390205	4/3	45	Midlane	0.52	0.53	0.50	0.48	0.48	0.51	0.42	0.48	0.49	0.42	0.58	0.49	
			RWP-W1*	.82/.79		.86/.73			.67/.83		.84/1.04		.71/.74			0.78/0.83
			RWP-LT**	82.9		77.9			86.6		83.3		83.1			82.8
390206	4/3	43	Midlane	0.55	0.48	0.45	0.42	0.63	0.45	0.68	0.56	0.54	0.48	0.57	0.53	
			RWP-W1*	.83/.71		.69/.85			.69/.71		.78/.97		.77/.97			0.75/0.84
			RWP-LT**	80.7		84.1			87.0		87.2		85.7			84.9
390207	4/3	62	Midlane	0.30	0.31	0.31	0.28	0.30	0.31	0.35	0.38	0.40	0.40	0.42	0.34	
			RWP-W1*	.39/.43		.37/.37			.42/.45		.45/.43		.49/.48			0.42/0.43
			RWP-LT**	89.7		83.8			88.1		86.7		83.7			86.4
390208	4/4	34	Midlane	0.34	0.34	0.37	0.35	0.34	0.34	0.32	0.28	0.32	0.43	0.35	0.34	
			RWP-W1*	.58/.48		.54/.58			.65/.64		1.12/1.11		.80/.84			0.74/0.73
			RWP-LT**	82.8		88.9			86.2		87.5		90.0			87.1
390209	4/3	54	Midlane	0.57	0.58	0.60	0.48	0.54	0.42	0.38	0.44	0.55	0.49	0.48	0.50	
			RWP-W1*	1.11/.90		.82/.59			.82/.85		1.12/1.34		.73/.74			0.92/0.88
			RWP-LT**	84.7		80.5			80.5		84.8		82.2			82.5
390210	4/3	41	Midlane	0.50	0.43	0.42	0.41	0.43	0.43	0.40	0.42	0.47	0.41	0.36	0.43	
			RWP-W1*	.97/.87		.92/.83			1.01/.89		.82/.96		.67/.75			0.88/0.86
			RWP-LT**	83.5		80.4			79.2		84.1		86.6			82.8
390211	4/3	56	Midlane	0.32	0.32	0.34	0.36	0.35	0.35	0.35	0.35	0.33	0.33	0.35	0.34	
			RWP-W1*	.44/.36		.52/.53			.55/.59		.50/.56		.51/.54			0.50/0.52
			RWP-LT**	86.4		88.5			83.6		88.0		90.2			87.3
390212	4/3	39	Midlane	0.35	0.29	0.32	0.25	0.28	0.29	0.36	0.40	0.41	0.41	0.36	0.34	
			RWP-W1*	.73/.65		.73/.64			1.07/1.00		1.13/1.02		1.08/1.19			0.95/0.90
			RWP-LT**	84.9		82.2			86.0		80.5		85.2			83.8
390259	4/3	39	Midlane	0.38	0.35	0.37	0.37	0.40	0.42	0.41	0.48	0.44	0.42	0.41	0.40	
			RWP-W1*	1.20/1.02		1.04/1.10			1.33/1.23		1.76/1.60		1.05/.91			1.28/1.17
			RWP-LT**	88.3		87.5			85.0		85.8		88.6			87.0
390260	4/3	41	Midlane	0.31	0.29	0.37	0.27	0.24	0.34	0.32	0.25	0.26	0.30	0.30	0.30	
			RWP-W1*	1.02/1.00		1.21/1.27			.66/.63		.57/.50		.49/.46			0.79/0.77
			RWP-LT**	82.4		90.1			87.9		84.2		91.8			87.3
390261	4/3	56	Midlane	0.32	0.29	0.29	0.28	0.31	0.33	0.30	0.31	0.32	0.32	0.36	0.31	
			RWP-W1*	.47/.52		.45/.48			.47/.46		.47/.42		.44/.49			0.46/0.47
			RWP-LT**	85.1		80.0			85.1		87.2		86.4			84.8
390262	4/4	35	Midlane	0.30	0.29	0.30	0.29	0.32	0.33	0.33	0.30	0.31	0.29	0.26	0.30	
			RWP-W1*	.99/.95		.78/.76			.65/.67		.82/.88		.67/.57			0.78/0.77
			RWP-LT**	85.9		85.9			86.2		84.1		80.6			84.5
390263	4/4	36	Midlane	0.38	0.39	0.42	0.40	0.45	0.42	0.47	0.39	0.42	0.52	0.41	0.42	
			RWP-W1*	1.38/1.20		1.03/1.18			1.14/1.19		.93/1.05		1.22/1.03			1.14/1.13
			RWP-LT**	77.5		72.8			87.7		88.2		84.4			82.1
390264	4/4	36	Midlane	0.42	0.47	0.80	0.42	0.46	0.45	0.42	0.48				0.49	
			RWP-W1*	1.30/1.42		1.05/1.24			1.14/1.39		.88/.95					1.09/1.25
			RWP-LT**	93.1		85.7			91.2		86.4					89.1
390265	4/3	57	Midlane	0.31	0.34	0.31	0.26	0.36	0.30	0.33	0.34	0.35	0.31	0.36	0.32	
			RWP-W1*	.46/.44		.46/.39			.53/.59		.52/.57		.56/.67			0.51/0.53
			RWP-LT**	84.8		73.9			88.7		80.8		92.9			84.2
Ramp SPS-8 (PCC)																
390809	4/4	37	Midlane	0.62	0.79	0.86	0.79	0.76	2.29	0.61	0.65	0.60	0.62	0.60	0.84	
			RWP-W1*	1.47/1.84		2.50/2.95			1.83/2.35		1.65/1.55		1.01/1.32			1.69/2.00
			RWP-LT**	81.6		90.0			91.8		86.1		93.1			88.5
390810	4/4	36	Midlane	0.51	0.42	0.46	0.36	0.43	0.45	0.43	0.42	0.42	0.55	0.40	0.44	
			RWP-W1*	1.31/1.49		1.09/1.27			1.03/1.02		1.07/1.05		1.05/1.07			1.11/1.18
			RWP-LT**	88.5		86.2			82.5		83.2		84.8			85.0

APPENDIX G DISSERTATION SYNOPSIS FROM LOG 33 “PERFORMANCE ANALYSIS OF BASES FOR FLEXIBLE PAVEMENT”

Doctorial Dissertation by

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Abstract from the Dissertation

Six test sections of roadway with five different base types and three different base thicknesses were constructed on U.S. 33 in Logan County with embedded transducers to measure pavement responses to structural, environmental and loading effects. Asphalt concrete samples were obtained and tested in the laboratory using the same environmental conditions found at the test site. The laboratory analysis and FWD data were used to analyze material properties and estimated resilient modulus by backcalculation using MODCOMP3. After obtaining material properties of each layer, all parameters were used in the OUPAVE and KENLAYER computer programs to model the pavement responses. The horizontal tensile strain at the bottom of the asphalt layer and the vertical compressive strain at the surface of the subgrade were utilized as the input parameters in distress mode models. Damage analyses for each section were made and compared using fatigue and rutting models. Long term performance and expected service life of each section were estimated using actual ESAL data obtained from ODOT. Finally, environmental data were evaluated for their effects on the pavement. Permeability and performance data of base material obtained from previous researches were also utilized. Cement treated base and asphalt treated base are superior in draining water. New Jersey untreated base and cement treated base performed well compared to other base types and are good bases for flexible pavement.

General

Five different base materials, including: asphalt treated base, cement treated base, New Jersey untreated base, Iowa untreated base, and standard 304 aggregate base were analyzed for performance under a range of load and environmental conditions. The Falling Weight Deflectometer was used to evaluate pavement stiffness in the field and the resilient modulus test was used to determine material stiffness in the laboratory. Next, backcalculation software (MODCOMP3) was used to determine material properties from FWD deflections, and OUPAVE and KENLAYER computer software was used to evaluate pavement response. Horizontal tensile strains at the bottom of the asphalt layer and vertical compressive strains at the surface of the subgrade were compared as a measure of performance of the various base materials. Permeability and performance of each base material obtained from previous researches were also considered. Damage analyses were performed on each section and compared with fatigue and rutting data at the site. The performance of all sections was evaluated and summarized using the criteria mentioned above. The following sections briefly describe the individual test sections.

Section 1

Section 1 contained four inches of asphalt treated free draining base over four inches of dense-graded aggregate base. From backcalculation data, the modulus of the asphalt treated base was not much higher than the New Jersey untreated base. Tensile strain at the bottom of the asphalt concrete layer was similar on both bases. Permanent deformation at the site was not quite large as that predicted by theoretical calculations. Cracking was observed at the site, and occurred at construction joint located in the middle of right lane. Hydraulic conductivity of the asphalt treated free-draining was very high compared to the untreated base materials. Using asphalt as a binder for granular material may not improve performance because the asphalt coating can separate from the aggregate when exposed to high moisture contents over a period of time. Dynamic loads can produce large tensile strains at the bottom of the asphalt concrete layer and large compressive strains at the top of the subgrade layer. The modulus of asphalt concrete is highly susceptible to temperature, and large permanent deformations can be observed during the summer months. Asphalt stripping in the stabilized base layer caused by excessive moisture resulted in the material falling apart while samples were being collected at the site. Therefore, designing permeable bases with asphalt binders may not increase the performance of flexible pavements. Using distress mode criteria, this section should carry ordinary traffic until the year 2004 without any maintenance requirements.

Section 2

The cement treated base in Section 2 provided the best performance. The modulus of cement treated base is higher than the other treated and untreated base materials. No cracks have been detected at the site. Tensile strain at the bottom of the asphalt concrete layer and compressive strain at the top of the subgrade layer were lowest of the materials tested. Hydraulic conductivity is highest among the other base materials. Cement binders can improve stiffness and reduce strain under the pavement. The properties of cement treated base will not vary much even with high temperatures in summer and high moisture in spring. A good drainage system in this base allows water to drain off before water can reach the subgrade layer. From distress models, this section should last until the year 2009 without any maintenance. Due to the limited time this section has been in service, the long-term performance of cement treated bases cannot be predicted. However, cement treated base is one of the most durably bases for asphalt concrete pavement.

Section 3

Four inches of New Jersey base were placed under the asphalt concrete pavement layer. The performance of this section was similar to Section 1 (asphalt treated base). The modulus of the base was quite high when compared to other untreated aggregate bases. Tensile strain at the bottom of the asphalt concrete layer and compressive strain at the top of the subgrade layer were similar to that observed in Section 1. Fatigue and rutting distress in this section were comparable to that observed in Section 1. The hydraulic conductivity of this base material was much less than the cement treated and asphalt treated bases but similar to the Iowa base. The New Jersey base permits water to drain away quickly and is stiff enough to withstand traffic loads. This section is expected to remain in service until 2005, which is slightly longer than Section 1. The performance of this base was comparable to the asphalt treated base, but construction costs were lower making the New Jersey base one of the best alternatives for AC pavements.

Section 4

This section contained Iowa base, an untreated drainable aggregate base similar to New Jersey base, but differing in gradation. Iowa base was designed to drain water quickly to avoid water infiltrating the subgrade layer. The modulus of this base was quite low compared to the others. The tensile strain at the bottom of the asphalt concrete was quite high, which can accelerate cracking in that layer. Compressive strain at the top of the subgrade layer was quite small, because of the high subbase modulus. From FWD data, average deflections in the vicinity of the load plate were quite high and low farther out because of the low stiffness of the Iowa base material. This base was designed and mixed by different sizes of granular materials and was expected to have high stiffness and permeability. Unfortunately, it was not able to provide the expected performance. The modulus was lowest of the bases tested and large permanent deformations were observed at the site. Hydraulic conductivity and the resilient modulus in the laboratory were similar to the New Jersey base. This section is expected to last until 2003 without any maintenance. From overall performance data at this site, this base is not appropriate for flexible pavement. New gradations may improve performance.

Section 5

Eight inches of standard 304 dense-graded aggregate base was placed under the asphalt concrete pavement layer. This base was widely utilized throughout the state and was designed to obtain maximum density and high stiffness. Unfortunately, it has low hydraulic conductivity. Water cannot drain quickly, and this can diminish pavement performance when high moisture contents are present in the base layer. The modulus of this base is not very high, and the tensile strain at the bottom of the asphalt concrete layer and the compressive strain at the top of the subgrade layer are quite large. There were no cracks observed on the pavement surface and the amount of permanent deformation in this section was substantially lower than predicted by theory, since there were no gaps between the aggregate particles, which allowed the particles to move. The distress model predicts this section will require maintenance by the year 2002.

Section 6

The materials in this section were identical to those in Section 5, but the thickness of asphalt concrete pavement and base layers were different. Increasing the thickness of the asphalt concrete can diminish tensile strain at the bottom of the asphalt concrete layer, and compressive strain and deflection at the top of the subgrade layer. Permanent deformations observed in this section were similar to those measured in Section 5. No cracking was observed in this section. The estimated service life of this section was similar to Section 3, in that maintenance will be required in the year 2003. Based upon the similarities observed thus far between Sections 5 and 6, it is concluded that replacing dense-grade aggregate base thickness with asphalt concrete thickness may not significantly improve pavement performance.

Conclusions

The following conclusions are presented from this study:

1. Untreated New Jersey base is one of the best materials to use with flexible pavement. It is durable, it provides good drainage, and it has a long service life.

2. The performance of free-draining asphalt treated base can be enhanced by increasing the percentage of asphalt binder or by using a modified asphalt binder to reduce stripping.

3. Iowa base is not recommended for flexible pavement due to its marginal performance in these tests. New gradations should be examined to improve this performance.

4. Standard 304 dense-graded aggregate base is the most frequently used base for flexible pavement in Ohio but, in situations where it is used, better drainage should be provided because of its low permeability.

5. Slightly increasing the thickness of asphalt concrete does not significantly affect short-term pavement performance.

6. To prevent fine-grained particles from migrating into coarser material, geotextiles that allow water to move freely, but restrict particle movement, are recommended between unstabilized layers of substantially differing gradations.

APPENDIX H JUNE 1998 FWD DATA GAL 35

Table H.1 GAL 35 FWD Joint Measurements – June 1998, Load ~ 9000 lb. Ft.

Section No.	Dowel Bars		Date	Joint No.	Joint Approach		Joint Leave		JSR
	Diameter	Length/Material			Norm. $Df1_A$ (mils/kip)	LT_A (%) ($Df3_A/Df1_A$)	Norm. $Df1_L$ (mils/kip)	LT_L (%) ($Df2_L/Df1_L$)	($Df1_L/Df1_A$)
1	1.5"	18" steel	6/8/98	1	1.11	62.4	0.95	73.7	0.86
				2	0.47	81.8	0.37	96.0	0.79
				3	0.61	99.3	0.62	101.0	1.02
				4	0.53	93.9	0.56	87.4	1.06
				Average	0.68	84.4	0.63	89.5	0.93
2	1"	12" steel	6/8/98	5	0.98	76.0	0.79	99.2	0.81
				6	1.20	85.8	1.30	90.0	1.08
				7	0.72	107.7	0.97	88.9	1.35
				8	0.83	96.3	1.07	78.4	1.29
				Average	0.93	91.5	1.03	89.1	1.13
3	1"	12" steel	6/8/98	9	0.75	69.7	0.63	92.3	0.84
				10	0.80	86.8	0.80	88.8	1.00
				11	0.87	81.6	0.86	84.7	0.99
				12	NR	NR	0.95	90.5	NR
				Average	0.81	79.4	0.81	89.1	0.94
4	1"	12" steel	6/8/98	13	1.06	85.8	1.69	51.4	1.59
				14	1.08	70.8	1.28	70.9	1.19
				15	0.59	94.0	0.71	85.4	1.20
				16	0.65	98.3	0.77	86.8	1.18
				Average	0.85	87.2	1.11	73.6	1.29
5	1.5"	18" steel	6/8/98	17	0.58	97.3	0.69	96.4	1.19
				18	0.49	99.2	0.56	96.6	1.14
				19	0.46	96.9	0.53	96.8	1.15
				20	0.67	100.0	0.83	78.7	1.24
				Average	0.55	98.4	0.65	92.1	1.18
6	1.5"	18" steel	6/8/98	21	0.91	63.8	0.60	105.1	0.66
				22	0.70	82.4	0.75	73.0	1.07
				23	0.48	85.3	0.42	94.9	0.88
				24	0.36	84.0	0.41	77.5	1.14
				Average	0.61	78.9	0.55	87.6	0.94
7	1.5"	18" fiberglas	6/8/98	25	0.78	73.0	0.63	90.1	0.81
				26	0.53	95.0	0.52	96.0	0.98
				27	0.54	67.7	0.38	105.9	0.70
				28	0.49	71.5	0.47	76.7	0.96
				Average	0.59	76.8	0.50	92.2	0.86
8	1.5"	18" fiberglas	6/8/98	29	0.54	90.8	0.48	98.1	0.89
				30	0.52	78.6	0.53	84.9	1.02
				31	0.43	100.0	0.49	94.2	1.14
				32	0.41	96.5	0.36	102.4	0.88
				Average	0.48	91.5	0.47	94.9	0.98
9	1.5"	18" fiberglas	6/8/98						
				Average					
10	1"	12" fiberglas	6/8/98	33	0.75	75.7	0.48	123.5	0.64
				34	0.98	53.7	0.68	110.1	0.69
				35	0.68	75.0	0.54	75.9	0.79
				36	0.69	70.1	0.82	57.8	1.19
				Average	0.78	68.6	0.63	91.8	0.83
11	1"	12" fiberglas	6/8/98	37	0.61	94.8	0.72	78.5	1.18
				38	0.83	76.9	0.74	83.2	0.89
				39	0.70	57.6	0.45	103.5	0.64
				40	0.49	91.5	0.62	65.2	1.27
				Average	0.66	80.2	0.63	82.6	1.00
12	1"	12" fiberglas	6/8/98	41	0.72	54.1	0.85	46.6	1.18
				42	0.62	90.4	0.61	97.7	0.98
				43	0.57	65.0	0.42	105.5	0.74
				44	0.55	66.7	0.62	68.9	1.13
				Average	0.62	69.1	0.63	79.7	1.01
			6/9/98	1	0.48	85.9	0.47	92.5	0.98
				2	0.36	87.1	0.37	81.9	1.03
				3	0.85	63.0	0.64	80.1	0.75
				4	0.62	80.5	0.67	77.0	1.08
				Average	0.58	79.1	0.54	82.9	0.96
C/L			6/9/98	1A	0.81	69.6	0.80	70.9	0.99
				2A	0.68	63.3	0.69	65.2	1.01
				3A	1.34*	59.4	2.07*	55.6	1.54*
				4A	2.32*	74.6	3.06*	75.7	1.32*
				Average	0.75	66.7	0.75	66.9	1.00

NR - No reading
* Water affected load cell reading

Table H.2 GAL 35 Joint Response Summary – June 1998

Section No.	Joint Approach		Joint Leave		JSR ($Df1_L/Df1_A$)
	Norm. $Df1_A$ (mils/kip)	LT_A (%) ($Df3_A/Df1_A$)	Norm. $Df1_L$ (mils/kip)	LT_L (%) ($Df2_L/Df1_L$)	
1" Ø x 12" Long Steel Dowels					
2	0.93	91.5	1.03	89.1	1.13
3	0.81	79.4	0.81	89.1	0.94
4	0.85	87.2	1.11	73.6	1.29
Avg.	0.86	86.0	0.99	83.9	1.12
1.5" Ø x 18" Long Steel Dowels					
1	0.68	84.4	0.63	89.5	0.93
5	0.55	98.4	0.65	92.1	1.18
6	0.61	78.9	0.55	87.6	0.94
Avg.	0.61	87.2	0.61	89.8	1.02
1.5" Ø x 18" Long Fiberglass Dowels					
7	0.59	76.8	0.50	92.2	0.86
8	0.48	91.5	0.47	94.9	0.98
9					
Avg.	0.53	84.1	0.48	93.5	0.92
1" Ø x 12" Long Fiberglass Dowels					
10	0.78	68.6	0.63	91.8	0.83
11	0.66	80.2	0.63	82.6	1.00
12	0.62	69.1	0.63	79.7	1.01
Avg.	0.68	72.6	0.63	84.7	0.94

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