Demonstration and Evaluation of SUPERPAVE Technologies

Final Evaluation Report for CT Route 2

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Connecticut Department of Transportation
Bureau of Engineering and Highway Operations
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16. Abstract

Connecticut’s first full-scale Superpave project was constructed in 1997 on CT State Route 2 in Colchester, Lebanon, and Bozrah. Six 2-mile sections, four Superpave and two ConnDOT Class 1 overlays, were placed between May and September 1997. Two of the Superpave mixes and one Class 1 mix utilized 20% RAP obtained by milling the existing surface layer from Route 2. This was also Connecticut's first HMA project where Quality Control was the responsibility of the contractor.

This final report covers the period following construction where field evaluations were performed from September 1997 through March 2003. During this time frame, pavement cores were removed, friction tests obtained, and condition surveys were performed. ConnDOT research staff collected information on the full 2-mile sections using manual surveys and automated equipment; and, since this project is also part of FHWA's LTPP SPS 9A study for "Verification of SHRP Asphalt Specification and Mix Design," 1000 ft sections within each of the six pavements were monitored, sampled and tested by the North Atlantic FHWA LTPP regional contractor using SHRP-LTPP protocols.

The pavements are perfoming as would be expected for 5-year old overlays on a 27-year old base. Thus far, there are only minor differences between the conventional ConnDOT class 1 overlays and the Superpave mixes. The mixes containing 20 percent RAP appear to be performing slightly worse than the virgin mixes, possibly due to excessive air voids, higher permeability, lower asphalt content, and a high ground-water table.

17. Key Words
Superpave, Quality Assurance, SHRP, LTPP-SPS9A, recycled asphalt pavement, RAP, hot-mix asphalt, pavement performance, condition survey

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Special thanks are offered to Dr. Charles E. Dougan, P.E., former Director of Research and Materials, whose vision based on his many years of experience leading ConnDOT's Division of Research, led to the establishment of this SUPERPAVE research study.

ConnDOT Maintenance personnel from the Colchester Maintenance Garage provided lane closures and traffic protection for the duration of the field data collection.

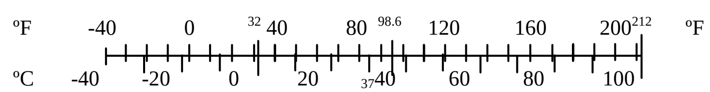
METRIC CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
mi^2	square miles	2.59	square kilometers	km^2
ac	Acres	0.405	hectares	ha
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb.)	0.907	Megagrams	Mg
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		TEMPERATURE (exact)		
ºF	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	ºC

APPROXIMATE CONVERSIONS FROM METRIC MEASURES

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
km^2	square kilometers	0.386	square miles	mi^2
ha	hectares (10,000 m^2)	2.47	acres	ac
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	Megagrams (1000 kg)	1.103	short tons	T
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		TEMPERATURE (exact)		
ºC	Celsius temperature	9/5 (then add 32)	Farenheit temperature	ºF



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INTRODUCTION

Background

A new method of mix design for Hot-Mix Asphalt (HMA) paving materials was the focus of a major effort under the Strategic Highway Research Program (SHRP) beginning in the 1980's. To provide better estimates of field performance of HMA, new laboratory testing equipment was also designed and developed as part of the national-level SHRP research. These activities, as well as the development of performance-graded (PG) binder specifications and tests, were collectively designated as Superpave, an acronym for superior performing asphalt pavements.

Engineers and technicians at The Connecticut Department of Transportation (ConnDOT), as well as other state DOTs, expressed some concern about quality control (QC) procedures that are inherently part of Superpave specifications. There was also concern about the design and use of recycled asphalt pavement (RAP) within the Superpave system, which was not addressed during the SHRP research.

A recommended procedure to adjust the Superpave mix process for inclusion of RAP entitled, "Guidelines for the Design of Superpave Mixtures Containing RAP," was prepared by the FHWA Expert Task Group on Mix Design, and distributed in early 1997. Another NCHRP study, Project 9-12, "Incorporation of RAP in the SUPERPAVE System," was also started in 1997 at the North Central SUPERPAVE Center, Purdue University. The National Cooperative Highway Research Program (NCHRP) studied QC issues under Project 9-7, "Field Procedures and Equipment to Implement SHRP Asphalt Specifications."

The results from the above-cited studies were not available in 1996 when ConnDOT proposed its first full-scale field application of Superpave. However, limited field trials had been conducted previously in Connecticut to identify potential placement problems. The early work on a few lower volume secondary roadways demonstrated that HMA producers could formulate and place Superpave mixes using local resources.

Armed with knowledge of design and construction techniques, but with concern about RAP and QC, Connecticut's first full-scale Superpave project was constructed on CT Route 2 between April and September 1997. A construction report [1] published in December 1997 documents various aspects of the Superpave design, production, quality control testing, acceptance testing, and placement.

The Route 2 project was, and continues to be, part of the FHWA Long-Term Pavement Performance (LTPP) Special Pavement Study #9A "Verification of SHRP Asphalt Specification and Mix Design." As such, it is anticipated that the FHWA North Atlantic Region Contractor (Stantec) will continue to collect field monitoring and performance data through approximately 2007. These data are entered into a national database and once having gone through quality checks are distributed via the DataPave program for use by various interested research parties.

Study Objectives

The objectives of the ConnDOT research study as published in the study proposal [2] dated February 1996 were:

1. to assess the quality assurance procedures set forth in NCHRP Project 9-7; and,

2. to evaluate the performance of Superpave mixes using both virgin and recycled aggregates.

Additional benefits accrued from the study included:

1. local contractor experience with design, placement, and lab testing of Superpave;

2. participation in FHWA’s LTPP Special Pavement Study #9A;

3. provision of a showcase project for the New England states in conjunction with FHWA Demonstration Project #90, “SUPERPAVE Asphalt Mix Design and Field Management;”

4. development and assessment of Superpave special provisions for ConnDOT; and,

5. ConnDOT’s first experience with contractor-provided quality control (QC).

It should be noted that the NCHRP Project 9-7 report was not available until after the Special Provisions for the Route 2 project were published. Therefore, objective #1 could not be met in its entirety during this study.

PROJECT LOCATION AND DESCRIPTION

Figure 1 shows the location of the project on Route 2 in eastern Connecticut. At this location, Route 2 is a four-lane, median-divided highway, functionally classified as a principal arterial. It is also designated as part of the National Highway System (NHS). This location was selected because it met FHWA criteria established for participation

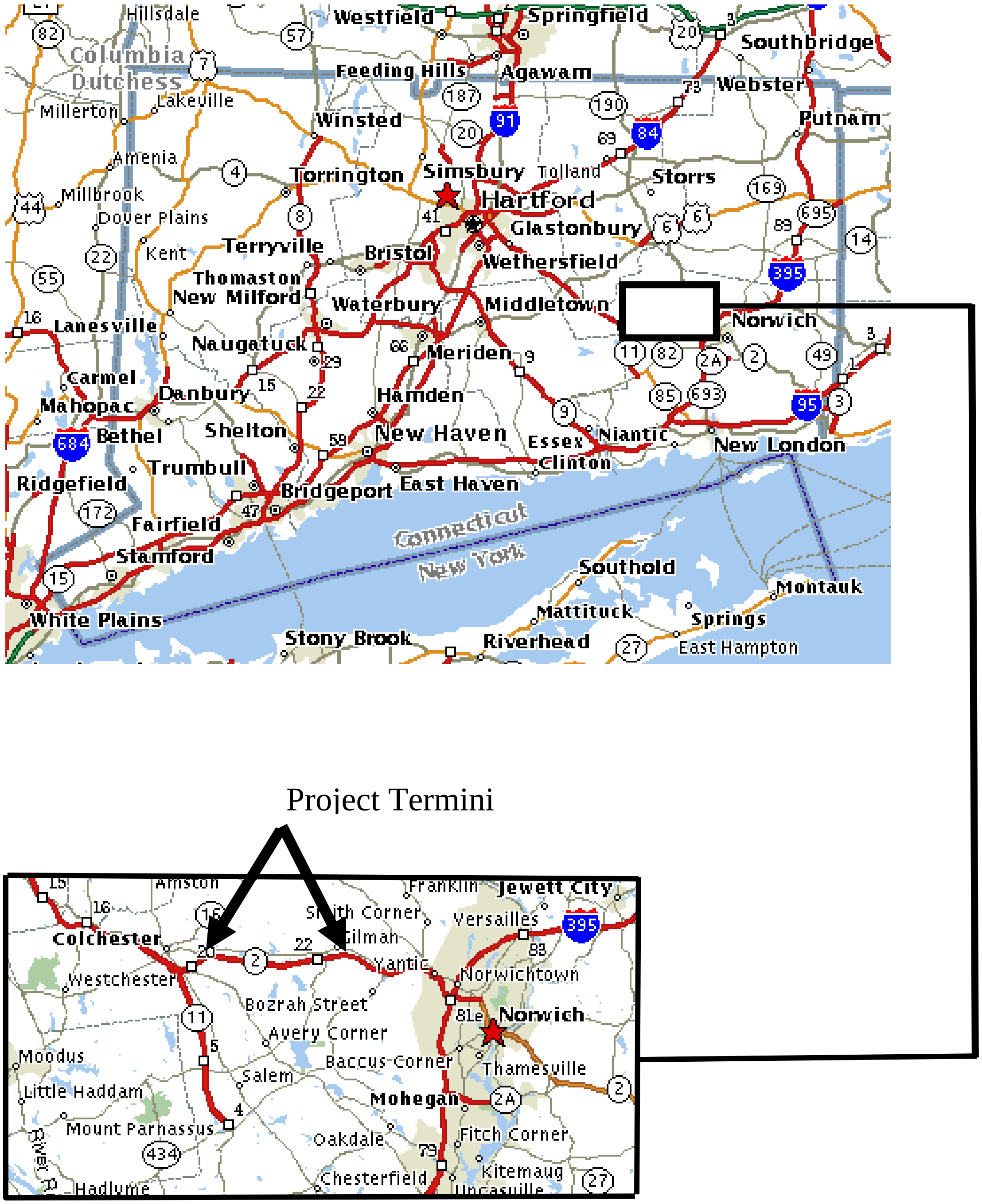

FIGURE 1. PROJECT LOCATION

in the FHWA LTPP Special Pavement Study 9A. The purpose of the SPS 9A study is to verify the performance of the SUPERPAVE System. The criteria used to select the site included projected traffic and 18,000 lb. equivalent single axle loads, horizontal and vertical curvature, grade, consistency within cuts and fills, absence of major drainage structures within test sections, and other LTPP specified requirements. According to the LTPP study design, each participating project, of which there were supposed to be 35 studied in North America, must have at least three sections: a control section containing a standard agency mix design, a SUPERPAVE design using a performance graded (PG) asphalt with 98 percent reliability, and a SUPERPAVE section with PG asphalt offering 50 percent reliability.

Route 2 was originally constructed in 1970 as a full-depth hot-mix asphalt (HMA) pavement. It was composed of a 10-inch subbase, a 4-inch calcium chloride stabilized base, six inches of plant mix HMA base, and four inches of surface course containing ConnDOT Class 1 HMA. Class 1 is a mix with 100 percent of the aggregate passing the ¾ in. sieve. In 1986, Route 2 was overlaid with ConnDOT Class 114 (a semi open-graded HMA mix) to a depth of two inches.

The section of Route 2 selected for the SUPERPAVE study begins in the town of Colchester and runs easterly for a distance of six miles through the town of Lebanon and into Bozrah. When originally constructed in 1970, Route 2 consisted of two 12-ft wide travel lanes, a 12-ft wide outside shoulder, and a 3-ft wide inside shoulder. Truck climbing lanes existed throughout the westbound direction and for the final 2.5 miles of the eastbound direction. All the truck climbing lanes were eliminated during the SUPERPAVE construction project in 1997. The 1997 project design (see Figure 2) for Route 2 called for removal of the two inches of Class 114 surface, placement of a one-inch layer of ConnDOT Class 2 leveling course, and placement of a top course of 2.5 inches of various HMA designs. The top course was placed as a single lift to meet the testing requirements of SPS 9A.

MAINLINE

PRE-CONSRUCTION **POST-CONSTRUCTION**

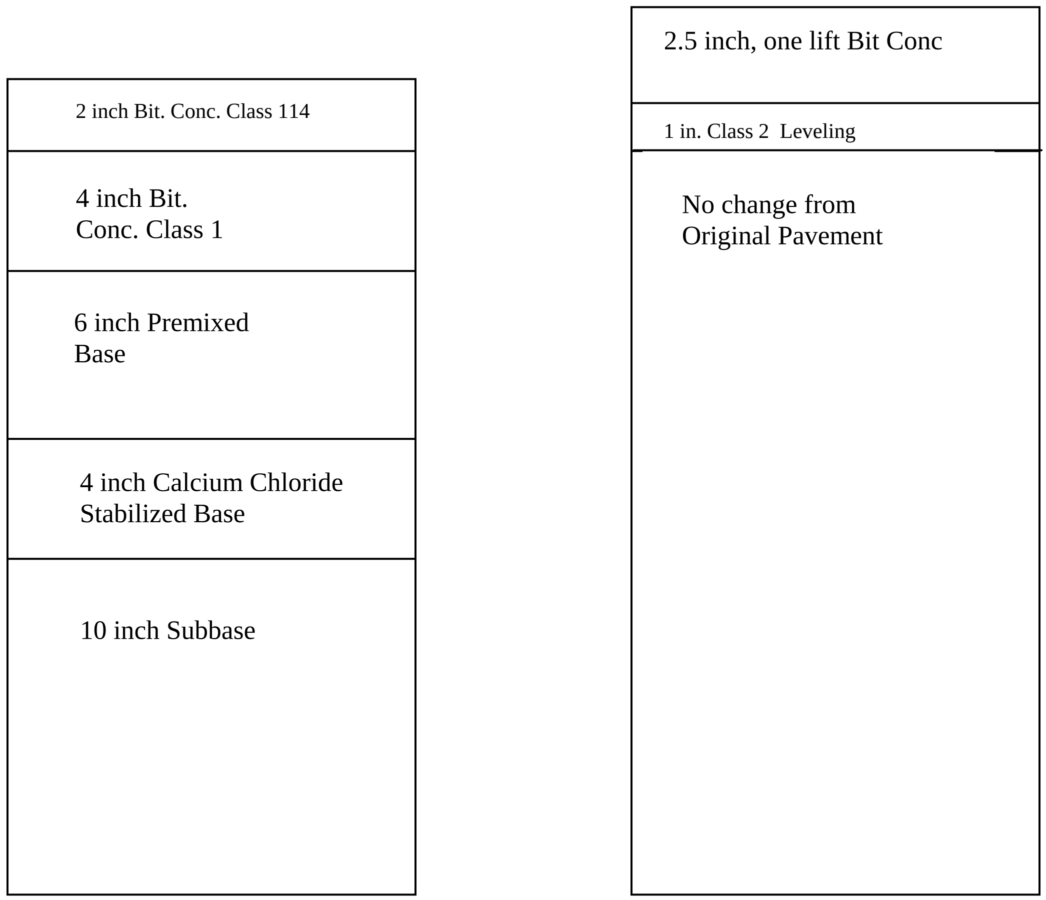

FIGURE 2 – TYPICAL CROSS SECTIONS FOR ROUTE 2, PRE- AND POST-CONSTRUCTION

Study Sites

For purposes of the research evaluation, the six-mile project was divided into three sections in each direction (eastbound and westbound), six sections total, each approximately two miles long. Figure 3 shows the layout of the project using ConnDOT official highway log mileage. In theory, each of the six sections was to be overlaid with a different mix. These mixes were bid for the project as the following: 1) Class 1 Virgin; 2) SUPERPAVE Virgin; 3) SUPERPAVE Alternate Virgin; 4) Class 1 RAP; 5) SUPERPAVE RAP; and, 6) SUPERPAVE Alternate RAP. In reality, the SUPERPAVE Virgin and SUPERPAVE Alternate Virgin were the same mix design with only the asphalt PG grade changed. The same was true with the SUPERPAVE RAP and the SUPERPAVE Alternate RAP. The eastbound direction contained the three virgin mixes. The westbound direction contained all mixes with 20 percent RAP. In addition to requiring the above section parameters, the asphalt grade was also specified for each section in the project special provisions. The specified asphalt grades are given in Table 1.

Image #1 shows Route 2 after completion of the project in September 1997. Image #2 shows the same roadway in June 2003.

FHWA LTPP Study Criteria

Upon approval by FHWA LTPP of the Route 2 test site in 1995, ConnDOT agreed to provide certain measurements and data, and to provide traffic protection at the test sites for other vendors under contract to FHWA. The first phase of field data collection occurred in the summer and fall of 1996. During the summer, ConnDOT research personnel met with FHWA and representatives from Stantec, to select 1000-ft length monitoring sections within the low-speed lane of each of the six two-mile sections. These 1000-ft sections are used to monitor the performance of the pavement for the life of the LTPP test study (through 2007). Each 1000 feet is divided into three segments; 500 feet in the

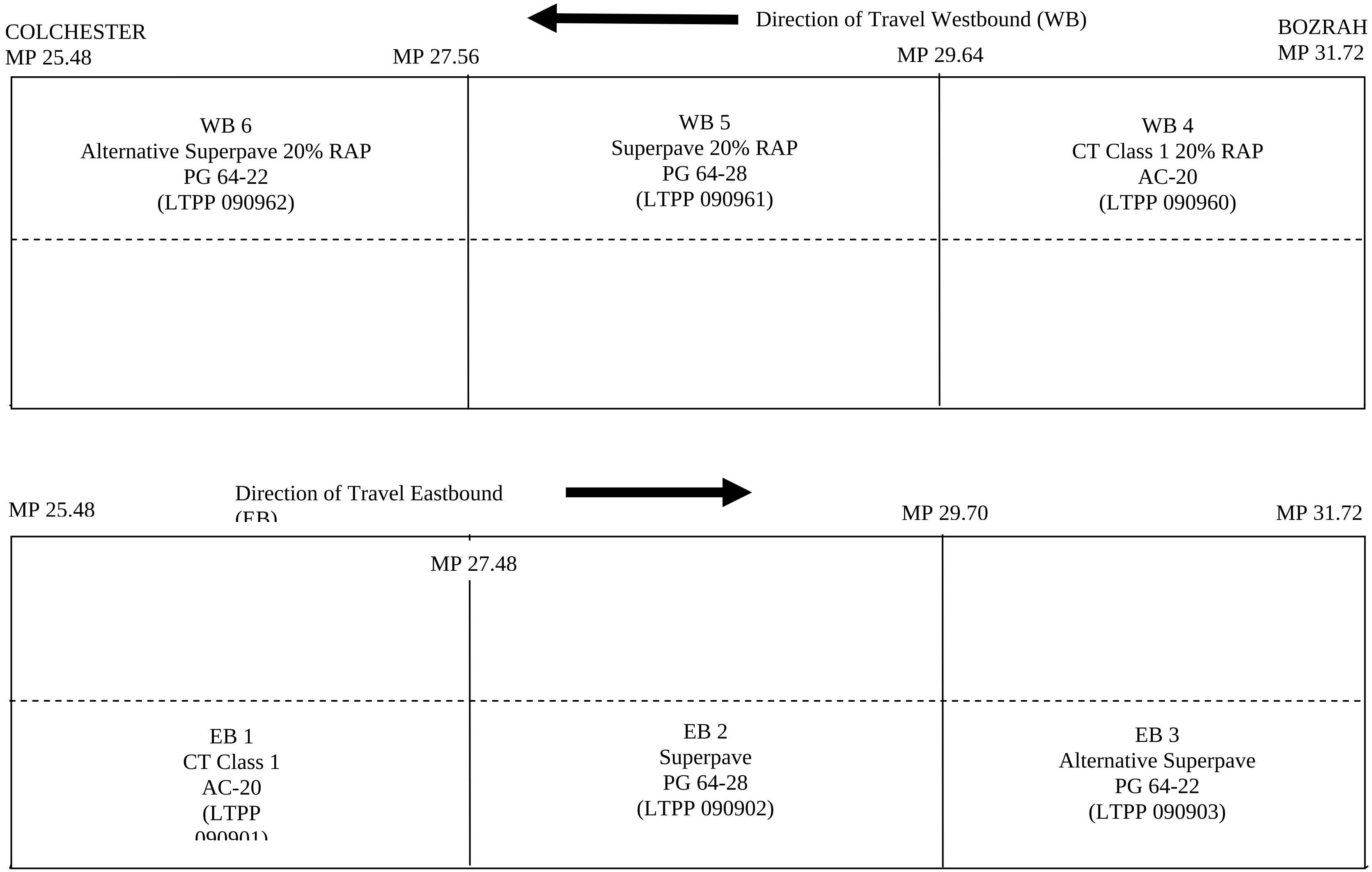

FIGURE 3
SPS 9A TEST SECTION LAYOUT
CONNECTICUT ROUTE 2, LOG MILE 25.48-31.72
TOWNS OF COLCHESTER, LEBANON, BOZRAH

TABLE 1
STUDY SECTION PARAMETERS

STUDY SECTION DESIGNATION	LTPP SECTION DESIGNATION	TYPE OF PAVEMENT	CONNDOT LOG MILEAGE	SECTION LENGTH (MI)	FINAL ASPHALT GRADE DESIRED
EB 1	090901	Class 1 Virgin	25.48-27.48	2.00	AC-20
EB 2	090902	SUPERPAVE Virgin	27.48-29.70	2.22	PG 64-28
EB 3	090903	SUPERPAVE Alternate Virgin	29.70-31.72	2.02	PG 64-22
WB 4	090960	Class 1 RAP	31.72-29.64	2.08	AC-20
WB 5	090961	SUPERPAVE RAP	29.64-27.56	2.08	PG 64-28
WB 6	090962	SUPERPAVE Alternate RAP	27.56-25.48	2.08	PG 64-22

IMAGE #1 - ROUTE 2, BOZRAH/LEBANON TOWN LINE, FACING EAST IN WESTBOUND DIRECTION, SHORTLY AFTER PAVING (SEPTEMBER 1997)

IMAGE #2 - ROUTE 2, BOZRAH/LEBANON TOWN LINE, FACING EAST IN EASTBOUND DIRECTION, PAVEMENT AGE 5-1/2 YEARS (JUNE 2003)

middle for monitoring, and 250 feet on each end for extraction of core samples. The identifiers assigned to the LTPP sections are given in Table 1 along with the designations used throughout this report for the research study sections. For example, LTPP section 090901 falls within section EB 1; LTPP section 090960 within section WB 4.

PAVEMENT MIX DESIGNS

The construction paving contractor, SONECO/Northeastern Inc., provided all Quality Control (QC) during production and placement of the pavement mixes. ConnDOT was responsible for Acceptance (A).

Table 2 shows the master range, aggregate, and void requirements for the SUPERPAVE mixtures, given in metric units as required at the time of the contract.

TABLE 2
SUPERPAVE MASTER RANGE AND AGGREGATE REQUIREMENTS

SIEVE	**CONTROL POINTS**		**RESTRICTED ZONE**	
mm	Min.	Max.	Min.	Max.
19.00	-	100	-	-
12.50	90	100	-	-
9.50	-	-	-	-
4.75	-	-	-	-
2.36	28.0	58.0	39.1	39.1
1.18	-	-	25.6	31.6
0.600	-	-	19.1	23.1
0.300	-	-	15.5	15.5
0.150	-	-	-	-
0.075	2.0	10.0	-	-
VMA 14% Min.	VFA 65-78%	Dust to Asphalt Ratio (%) 0.6 to 1.2 (1)	Air Voids at N_{des} 4%+/-1.2	Tensile Strength Ratio 80% Min. (2)

TRAFFIC LEVEL	**COARSE AGGREGATE ANGULARITY** ***PENN.DOT. TEST METHOD NO. 621***		**FINE AGGREGATE ANGULARITY** ***AASHTO TP33(ASTM C1252)***	
(80 kN) ESALs	DEPTH FROM SURFACE <100 mm	DEPTH FROM SURFACE >100 mm	DEPTH FROM SURFACE <100 mm	DEPTH FROM SURFACE >100 mm
<3 x 10^6	75/--	50/--	40	40
	Note:"75/--" denotes that a minimum of 75% of the coarse aggregate has one fractured face.		Note: Criteria are presented as minimum percent air voids in loosely compacted fine aggregate passing the 2.36 mm sieve.	

FLAT, ELONGATED PARTICLES ASTM D4791	**CLAY CONTENT** **AASHTO T-176**	**Gyratory % of compaction at (N) number of gyrations in a geographic location where the 7 day average temperature is < 39^{o}C AASHTO TP 4**		
>4.75 mm	SAND EQUIVALENT	(≤89%) Ni	Nd	(≤98%) Nm
10	40	7	86	134
Note: Criteria presented as maximum percent by weight of flat and elongated particles of materials retained on 4.75 mm sieve.	Note: Criteria is presented as a minimum for fine aggregate passing the 2.36 mm sieve.	N_i = Initial number of Gyrations; N_d = Design number of Gyrations; N_m = Maximum number of Gyrations.		

(1) Dust is considered to be the percent of material passing the 0.075-mm sieve
(2) Tensile Strength ratio: AASHTO T-283
Compiled from SHRP-A-407: The SUPERPAVE Mix Design Manual for New Construction and Overlays

The Connecticut Advanced Pavement Laboratory (CAP Lab) at the University of Connecticut, under contract to SONECO, was responsible for the pavement mix design. Their designs were based upon a maximum average 7-day air temperature of less than 102°F, a traffic level of less than 3 million 80-kN ESALs, and Gyratory Mix Compaction of N_i = 7, N_d = 86, and N_m = 134 as defined in Table 2. The weather data used for the design was obtained from National Weather Service records for Colchester, Connecticut. The selected PG asphalts of 64-28 and 64-22 provided 98 percent and 50 percent reliability for the low temperatures. For the high temperatures, the PG 64 asphalt approaches 100 percent reliability, while PG 58 would have provided 98 percent reliability. For the virgin sections, the requested binders, namely AC-20, PG 64-28, and PG 64-22 were used. However, for the RAP designs, the binders used were AC-10 for the Class 1 mix, PG 58-34 for the SUPERPAVE RAP section, and PG 58-28 for the Alternate SUPERPAVE RAP section. These PG binders were selected to offset the aging effects of the existing asphalt binder already attached to the RAP. All asphalt cement was supplied and certified by Hudson Companies of Providence, Rhode Island. Table 3 lists the asphalts used and the final-mix asphalt grade for the RAP mixes.

TABLE 3
ASPHALTS USED IN RECYCLED MIXES

SECTION DESIGNATION	SPECIFIED GRADE OF THE ASPHALT IN THE FINAL MIX	ASPHALT GRADE ADDED TO RECYCLED MIXES (IN COMBINATION WITH RAP)	THE ACTUAL MEASURED GRADE OF THE ASPHALT IN THE FINAL MIX
WB 4	AC-20	AC-10	AC-20
WB 5	PG 64-28	PG 58-34	Not Graded
WB 6	PG 64-22	PG 58-28	PG 76-22

The PG 58-34 could not be obtained as a neat (unmodified) asphalt. Therefore, a modifier was used for this one asphalt only. The modifier used was Styrelf/Styrene and the asphalt was obtained by Hudson

Companies from Petro-Canada in Toronto, Ontario. It was determined that this product would increase the cost of the two miles of pavement in Section WB 5 by as much as $60,000(US). This cost increase was not anticipated or approved by ConnDOT. Therefore, a compromise was reached to pave only one day with the more expensive asphalt, and to use the PG 58-28, as in Section WB 6, for the rest of Section WB 5. The construction project stations where PG 58-34 was used were from station 160+20 to 192+60 (log mile 28.69-28.07) westbound, which is a length of 3240 feet. The 1000-ft monitoring section for WB 5 (LTPP section 090961) is located within this area.

For the virgin mix designs, the CAP Lab tried three different gradations: coarse, medium, and fine. The medium gradation worked the best and resulted in a mix that was above the 'restricted zone.' For the RAP mixes, thirteen trial gradations were performed. A mix with the gradation below the 'restricted zone' was selected. The gradations for the virgin and RAP mixes plotted on a 0.45-power gradation chart are given in Figures 5 and 6.

The contractor provided aggregates from its Montville plant and quarry for use in the mix design. The aggregates were composed of ½-inch and 3/8-inch crushed stone, natural sand, and washed manufactured sand. Specific gravity, gradation, and angularity tests were performed on both the fine and coarse aggregates. In addition, the coarse aggregate underwent tests for flat and elongated particles, abrasion resistance, and soundness. The fine aggregates were tested for sand equivalence and fine aggregate angularity. The RAP materials were tested for asphalt content, specific gravity of aggregate, and gradation.

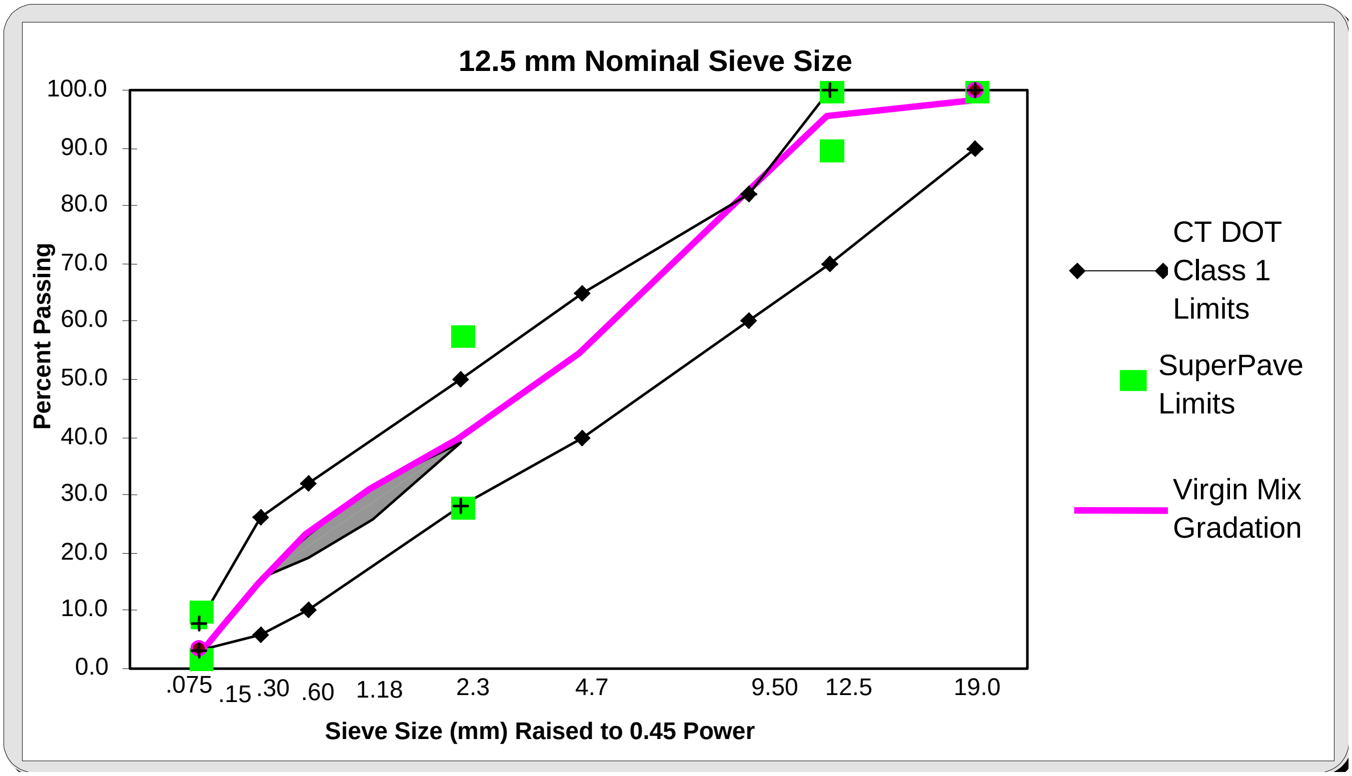

FIGURE 5. SUPERPAVE VIRGIN MIX GRADATION

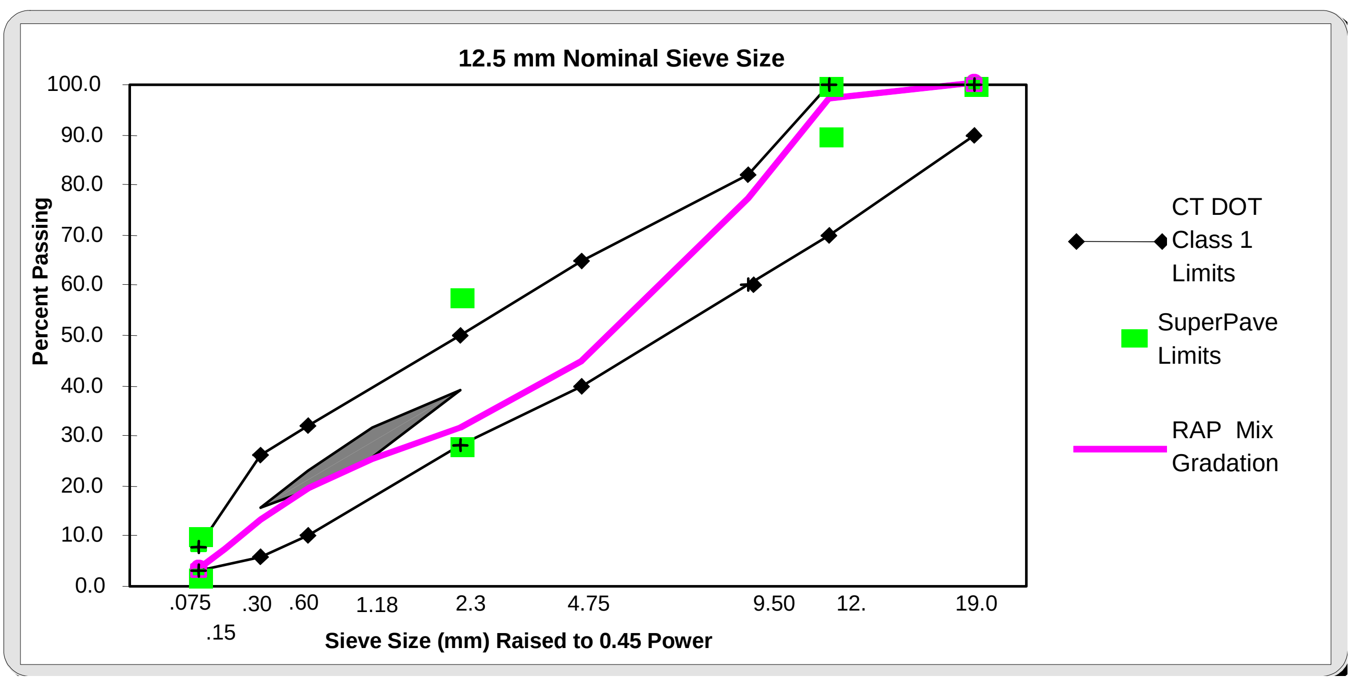

FIGURE 6. SUPERPAVE RAP MIX GRADATION

The designs for the two virgin SUPERPAVE mixes and the two SUPERPAVE mixes with RAP were provided in the construction report [1]. The design procedure used was SHRP A-407, “The SUPERPAVE Mix Design Manual for New Construction and Overlays, Level 1, Volumetric Design.” The Moisture Susceptibility Test, AASHTO T-283 was performed after the mixes were designed. The results of this test for tensile strength ratio did not produce the required 80 percent. As a result, an anti-strip agent was added to the binder. The anti-strip agent used was Kling Beta LV. It was introduced by Hudson at their facility in Rhode Island. The asphalt grade had to be re-certified as a result of the addition of the anti-strip agent.

When the Kling Beta was added to the PG 64-22 asphalt at 0.5 percent by weight of the binder, it was found that the PG asphalt grading changed slightly. The change was enough so that the asphalt could not be certified as a PG 64-22. Thus, additional moisture susceptibility tests were performed with 0.25 percent and 0.33 percent anti-strip agent. These amounts did not affect the grading of the asphalt. One-quarter of one percent was ultimately selected, as it produced a mix that met the T-283 test requirements. Table 4 shows the amount of anti-strip agent used in each of the six sections. The anti-strip agent was not used in the ConnDOT class 1 mixes since ConnDOT does not require moisture susceptibility testing, and stripping has traditionally not been an issue of concern with Connecticut’s pavements.

PAVEMENT CONSTRUCTION

Construction involved three phases: removal of the existing surface course of Class 114; placement of a leveling course of Class 2; and, placement of the surface layer for the six sections utilizing the various Class 1 and SUPERPAVE mixes. Paving of the surface layers for the entire project, excluding the ramps, occurred between June 17, 1997

TABLE 4
PERCENTAGE OF ANTI-STRIP AGENT USED IN EACH MIX

SECTION DESIGNATION	TYPE OF PAVEMENT	PERCENT OF ANTI-STRIP AGENT USED
EB 1	Class 1 Virgin *	0
EB 2	SUPERPAVE Virgin	0.25
EB 3	SUPERPAVE Alternate Virgin	0.25
WB 4	Class 1 RAP *	0
WB 5**	SUPERPAVE RAP	0.375
WB 6	SUPERPAVE Alternate RAP	0.375

*AASHTO T-283 not required by ConnDOT for HMA mixes.
** Most of Section WB 5 used the same mix as section WB 6; only 3240 ft, including the 1000 ft LTPP monitoring section, contained the PG 58-34 with modifier.

and September 9, 1997. Paving of the 1000-ft monitoring sections was performed on the dates indicated in Table 5. Paving was generally performed from left to right (i.e., left shoulder and high speed lane in the first pass, low-speed lane in the second pass, and right shoulder in the third pass.) This resulted in paving widths of approximately 14 ft, 13 ft, and 11-15 ft, for each pass, respectively.

TABLE 5
DATE AND WEATHER CONDITIONS AT TIME OF PAVING
THE 1000-FT LTPP MONITORING SECTIONS

SPS9A LTPP SECTION DESIGNATION	TYPE OF SURFACE PAVEMENT	DATE PAVING OCCURRED	AIR TEMPERATURE AND WEATHER
090901	Class 1 Virgin	06/23/97	80°F, Sunny
090902	SUPERPAVE Virgin	07/15/97	90°F, Sunny
090903	SUPERPAVE Alternate Virgin	06/28/97	80°F, Sunny
090960	Class 1 RAP	08/07/97	73°F, Sunny
090961	SUPERPAVE RAP	09/08/97	65°F, Cloudy
090962	SUPERPAVE Alternate RAP	08/12/97	73°F, Cloudy

Table 6 contains other pertinent information about the surface-layer paving for each 1000-ft monitoring section. Included in the Table, is pavement temperature immediately behind the paver, type of rollers used, asphalt modifiers (other than anti-strip agents), asphalt content, air voids, voids in the mineral aggregate (VMA), and percent maximum theoretical density achieved in the field.

TABLE 6
SUMMARY OF MIXTURE LAYDOWN DATA

LTPP SECTION	MODIFIER	PERCENT ASPHALT	AIR VOIDS	VMA	FIELD DENSITY (PERCENT MAXIMUM THEORETICAL)	MAT TEMP. ^{0}F BEHIND PAVER	BREAKDOWN ROLLER	INTERMEDIATE ROLLER	FINAL ROLLER
090901	None	5.4	4.4	16.8	92.8	270-315	Hyster C766A	None	Hyster C350C
090902	None	5.3	3.6	14.4	93.3	240-295	Hyster C766A	None	Hyster C350C
090903	None	5.3	3.3	13.7	92.7	270-285	Hyster C766A	None	Hyster C350C
090960	None	5.0	2.8	13.9	92.9	265-280	Hyster C766A	None	Hyster C350C
090961	Styrelf	4.8	4.8	14.9	92.1	300-325	Hyster C766A	Caterpiller CB-614	Hyster C350C
090962	None	5.0	4.8	15.5	91.0	275-280	Hyster C766A	None	Hyster C350C

In Section WB 4, there were a couple of areas where the surface layer was removed and replaced within a few days of paving. These occurred between construction project stations 63+90 to 64+68 (log mile 30.51-30.49) in the low-speed lane and between station 89+38 to 90+38 (log mile 30.03-30.01) in the high speed lane. These areas were not within the 1000-ft LTPP monitoring sections. The reason for removal was lack of density and immediate raveling. There appeared to be uncoated material and/or RAP material not homogeneously mixed with virgin material, delivered from two trucks.

PERFORMANCE DATA COLLECTION

“Pre-construction” data, i.e., data collected prior to the Superpave construction, (material samples, tests, and road condition measurements) were collected during September 1996 and April 1997, as indicated in Table 7. These data were collected primarily for the LTPP study for entry into the LTPP database.

TABLE 7
PRE-CONSTRUCTION DATA COLLECTION REQUIRED
FOR CONNDOT RESEARCH AND/OR FHWA LTPP SPS 9A

ACTIVITY	DATA OBTAINED	FREQUENCY	WORK CONDUCTED BY
4-in Auger Probe	Distance to Subsurface Bedrock	Preconstruction; One time: Sept 1996	ConnDOT
12-in dia. Pavement Cores	Moisture Induced Damage & Layer Thickness	Preconstruction; One time: Sept 1996	ConnDOT
Bulk Soil Samples	Particle Size Distribution, Soil Classification, Moisture Content	Preconstruction; One Time: Sept 1996	ConnDOT
Friction Testing	ASTM Locked – wheel Friction Number	Preconstruction; One Time: April 1997	ConnDOT
ARAN Survey	Roughness, Rutting, Geometry, Images	Preconstruction; One Time: April 1997	Roadware Corp.
Profilometer	Longitudinal Profile and Roughness	Preconstruction; One Time: April 1997	Stantec
Falling Weight Deflectometer	Pavement Deflections	Preconstruction; One Time: April 1997	Stantec
Distress Surveys	Cracks, Patches, Ravelling, etc	Preconstruction; One Time: April 1997	Stantec
Manual Transverse Profile (Dipstick)	Rut Depths	Preconstruction; One Time: April 1997	Stantec

There was a considerable amount of sealed longitudinal and transverse cracking observed in the existing pavement surface, prior to construction in 1997. Occasional areas of raveling and patching, possibly as a result of isolated areas of segregation in the class 114 mix, were also noted (see Images #3 and #4). Twelve-inch diameter pavement cores removed in September 1996 provided no indication of stripping within the 1986 or underlying 1970 pavement layers.

The “during-construction” data indicated in Table 8, (densities, and tests on asphalt, aggregate, and mixes), were obtained during the construction project, and were presented and discussed in the

IMAGE #3. ROUTE 2 EASTBOUND PRIOR TO CONSTRUCTION (AUGUST 1996)

IMAGE #4. ROUTE 2 WESTBOUND AT THE COLCHESTER/LEBANON TOWN LINE, JUST PRIOR TO MILLING (MAY 1997)

TABLE 8
DURING-CONSTRUCTION DATA COLLECTION REQUIRED
FOR CONNDOT RESEARCH AND/OR FHWA LTPP SPS 9A

ACTIVITY	DATA OBTAINED	FREQUENCY	WORK CONDUCTED BY
Nuclear Density	Pavement Density	During Construction; (3 per section)	ConnDOT
Bulk Asphalt Cement	Ship to LTPP MRL Sparks, Nevada	During Construction; (1 per section)	ConnDOT
Bulk Combined Aggregate	Ship to LTPP MRL Sparks, Nevada	During Construction; (1 per section)	ConnDOT
Bulk Surface Course Sample	Make Gyratory Molds in Lab & Run SHRP Protocol Tests	During Construction; (1 per section)	ConnDOT

TABLE 9
POST-CONSTRUCTION DATA COLLECTION REQUIRED
FOR CONNDOT RESEARCH AND/OR FHWA LTPP SPS 9A

ACTIVITY	DATA OBTAINED	FREQUENCY	WORK CONDUCTED BY
Friction Testing	ASTM Locked – wheel Friction Number	Semi-Annual: Every Spring and Fall	ConnDOT
ARAN Survey	Roughness, Rutting, Geometry, Images	Annual	ConnDOT, Photolog Inventory
Profilometer	Longitudinal Profile and Roughness	Occasional	Stantec
Falling Weight Deflectometer	Pavement Deflections	Occasional	Stantec
Distress Surveys	Cracks, Patches, Raveling, etc	Annual	Stantec & ConnDOT
Manual Transverse Profile (Dipstick)	Rut depths	Occasional with Profilometer Survey	Stantec
150 mm Pavement Cores	Laboratory Tests	Post Construction; (0, 6, 12, 18, 24, & 48 months)	ConnDOT & Stantec
Traffic Data	Volume, Classification & Weights (ESALs)	Post Construction; (Continuous)	ConnDOT

construction report.[1] “Post-construction” testing began at the completion of the paving, in late September 1997. Data collected and/or tests run and their approximate frequency are presented in Table 9.

The ConnDOT Research Division staff has performed friction testing of all six sections in the low-speed travel lane. The first data set following construction was collected during the fall of 1997. The friction testing continued to be performed semi-annually, in the spring and fall. All of these data are presented in Table 10, and discussed in a later section of this report. ConnDOT’s Data Services Section staff from the Division of Research ran ConnDOT’s Automatic Road Analyzer (ARAN) vehicle over the slow-speed lane of the sections in September 1997 to collect roughness, cross slope, grade, curvature, transverse profile (rut depths), and GPS data. These measurements were repeated on an annual basis from 1998 through 2002, as part of normal photolog inventories of Connecticut’s state maintained highways. It must be noted that the ARAN vehicles were upgraded, i.e., modified in April 2000, which had an impact on the roughness values reported after 1999. This impact is explained below under detailed discussion of the results shown in Table 18.

Stantec performed measurements of pavement deflection, transverse and longitudinal profile, and distress, periodically, beginning in September 1997. These surveys were performed within the 1000-ft LTPP monitoring sections only. The profile and distress data are presented in Tables 12, 17 and 19 for these surveys, and discussed later in this report in conjunction with ConnDOT’s collection of similar data. ConnDOT does not own deflection testing equipment, nor does it currently use deflection data for pavement design or pavement management. For these reasons, the falling weight deflectometer (FWD) data collected by Stantec is not presented or discussed in this report. These data should be available on the FHWA DataPave web site at www.datapave.com.

Six-inch diameter pavement cores were obtained by ConnDOT on a predetermined schedule, at times of 0, 6, 12, 18, 24, and 48 months after construction. These sampling intervals were prescribed by the FHWA LTPP program. ConnDOT's Division of Materials Testing staff tested the cores for pavement thickness, bulk specific gravity, maximum specific gravity, asphalt content, aggregate gradation, air voids, voids filled with asphalt (VFA), and VMA. The recovered asphalt was tested for penetration, viscosity, dynamic shear, and creep stiffness. All tests performed were required (and specified) by the LTPP program. All test results were submitted to the LTPP contractor on their designated LTPP forms.

Shortly after completion of construction, a weigh-in-motion (WIM) system was installed in both directions of Route 2, in all lanes, just west of the overpass with Camp Moween Road in Lebanon (milepoint 29.61), in the vicinity of the junctions of sections EB 2 and EB 3 eastbound, and sections WB 4 and WB 5 westbound. The automated WIM station was installed to collect continuous traffic volumes, classifications, and axle and gross vehicle weights. These data were and continue to be reported to LTPP on a quarterly basis. They have been collected, calibrated, and validated according to LTPP protocols for traffic data collection.

At the same time as the WIM installation, a roadway weather information system (RWIS) was installed in the median of Route 2 at the location of the WIM system. Orginally, the weather station was a requirement for the LTPP SPS9A Superpave study. However, at about the time of installation the requirement was changed, as the FHWA decided to go to a virtual weather system where multiple weather station sites are averaged together. The RWIS then became part of a network of RWIS sites for winter storm operations, maintained by ConnDOT’s Office of Maintenance.

In addition to the distress surveys performed by Stantec for the LTPP sections, ConnDOT research staff performed manual surveys on three occasions. In addition, the Wisecrax software from the ARAN survey was used on another occasion, for the full 24 lane-miles of the study sections. The data collected during the final survey in March 2003 are presented in Tables 13 to 15 and discussed later.

Friction Tests

Friction testing was performed semi-annually each spring and fall, from September 1997 through October 2002. The ASTM E-501 ribbed and ASTM E-524 smooth tires were used for these measurements. All sections were tested at ¼-mile increments in the low-speed lane, resulting in approximately eight tests per section, per tire. The average values reported as FN_{40} are presented in Table 10. All data were collected at 40 mph, in accordance with ASTM E-274 test method, using the ConnDOT pavement friction tester, which meets the same ASTM standard. The data represent the coefficient of friction multiplied by 100, measured under wet conditions. The ribbed tire measurements are indicative of the microtexture of the pavement surface, whereas the smooth tire measurements are representative of the macrotexture.

All data presented in Table 10 indicate an acceptable friction level has existed throughout the five years of study. Overall, the reduction in friction number over time has been minimal. These data are also plotted in Figures 7 through 10 to show the trends over time. The smooth-tire friction numbers indicate that during the five years there

TABLE 10
HISTORICAL PAVEMENT FRICTION DATA (FN_{40})
ROUTE 2 EB

Section EB 1

Time of Measurement	Ribbed Tire	Smooth Tire
Fall 2002	50.0	36.3
Spring 2002	53.4	42.9
Fall 2001	48.9	41.6
Spring 2001	49.0	40.3
Fall 2000	49.9	37.1
Spring 2000	53.9	42.9
Fall 1999	53.5	40.5
Spring 1999	57.7	41.7
Fall 1998	52.4	36.9
Spring 1998	56.5	37.5
Fall 1997	54.9	38.2
Overall Average	**52.7**	**39.6**

Section EB 2

Time of Measurement	Ribbed Tire	Smooth Tire
Fall 2002	52.0	36.9
Spring 2002	54.8	44.2
Fall 2001	50.7	42.4
Spring 2001	52.2	42.1
Fall 2000	51.4	40.2
Spring 2000	55.2	44.1
Fall 1999	53.6	41.3
Spring 1999	58.3	43.2
Fall 1998	54.5	39.5
Spring 1998	58.7	42.4
Fall 1997	57.8	45.0
Overall Average	**54.5**	**41.9**

Section EB 3

Time of Measurement	Ribbed Tire	Smooth Tire
Fall 2002	50.4	37.3
Spring 2002	55.4	42.6
Fall 2001	50.7	41.2
Spring 2001	52.5	40.9
Fall 2000	51.8	37.8
Spring 2000	55.7	43.4
Fall 1999	54.5	41.1
Spring 1999	59.4	42.1
Fall 1998	55.2	38.8
Spring 1998	59.1	40.5
Fall 1997	57.2	40.4
Overall Average	**54.7**	**40.6**

TABLE 10 (Continued)
HISTORICAL PAVEMENT FRICTION DATA (FN_{40})
ROUTE 2 WB

Section WB 4

Time of Measurement	Ribbed Tire	Smooth Tire
Fall 2002	50.9	37.7
Spring 2002	53.5	42.9
Fall 2001	49.3	41.3
Spring 2001	51.7	41.0
Fall 2000	49.0	39.0
Spring 2000	53.6	42.8
Fall 1999	52.5	39.8
Spring 1999	57.4	42.6
Fall 1998	53.2	37.9
Spring 1998	57.6	39.6
Fall 1997	52.7	39.9
Overall Average	**52.9**	**40.4**

Section WB 5

Time of Measurement	Ribbed Tire	Smooth Tire
Fall 2002	51.8	42.7
Spring 2002	52.9	48.3
Fall 2001	50.1	45.8
Spring 2001	51.3	47.1
Fall 2000	49.2	44.3
Spring 2000	53.6	49.4
Fall 1999	52.5	43.9
Spring 1999	56.9	48.5
Fall 1998	54.2	45.1
Spring 1998	58.6	50.4
Fall 1997	52.9	47.3
Overall Average	**53.1**	**46.6**

Section WB 6

Time of Measurement	Ribbed Tire	Smooth Tire
Fall 2002	50.2	42.5
Spring 2002	51.7	46.3
Fall 2001	48.8	45.6
Spring 2001	52.0	46.7
Fall 2000	48.3	43.1
Spring 2000	53.0	47.2
Fall 1999	51.2	46.2
Spring 1999	57.1	48.3
Fall 1998	53.3	45.2
Spring 1998	58.5	49.7
Fall 1997	53.5	47.5
Overall Average	**52.5**	**46.2**

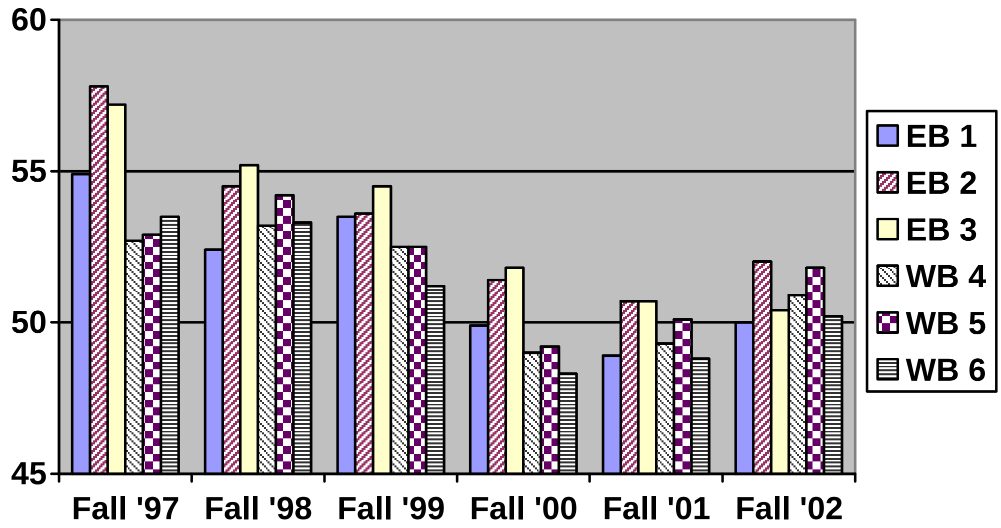

FIGURE 7
RIBBED TIRE FRICTION NUMBERS MEASURED DURING EACH FALL,
1997 THROUGH 2002

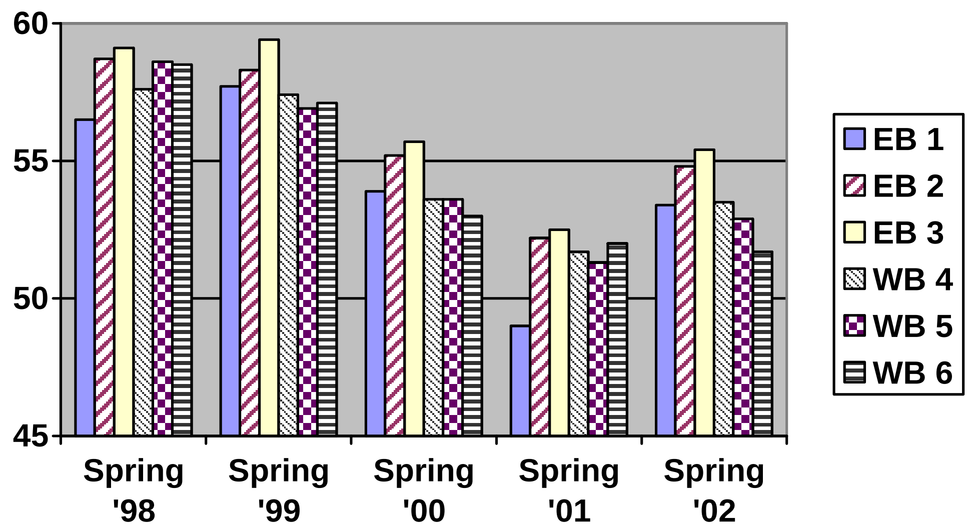

FIGURE 8
RIBBED TIRE FRICTION NUMBERS MEASURED DURING EACH SPRING,
1998 THROUGH 2002

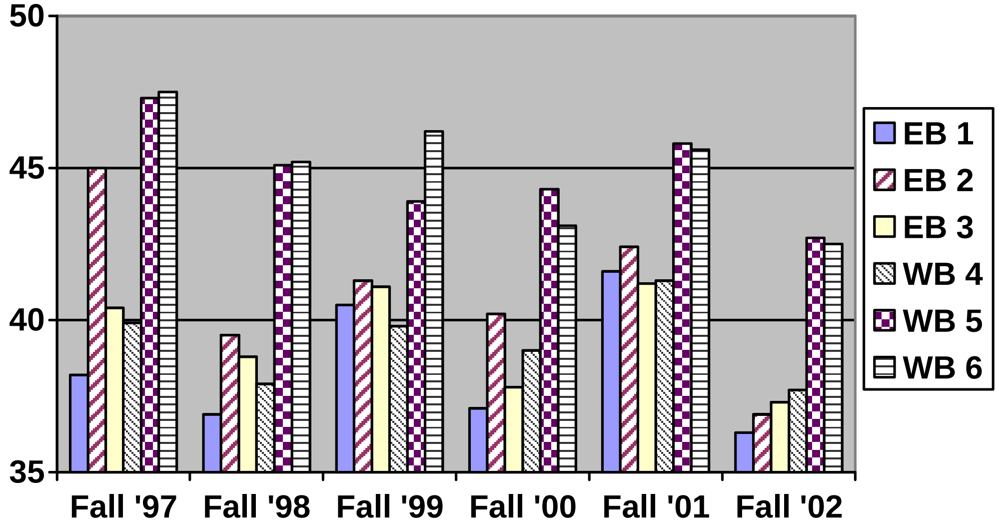

FIGURE 9
SMOOTH TIRE FRICTION NUMBERS MEASURED DURING EACH FALL,
1997 THROUGH 2002

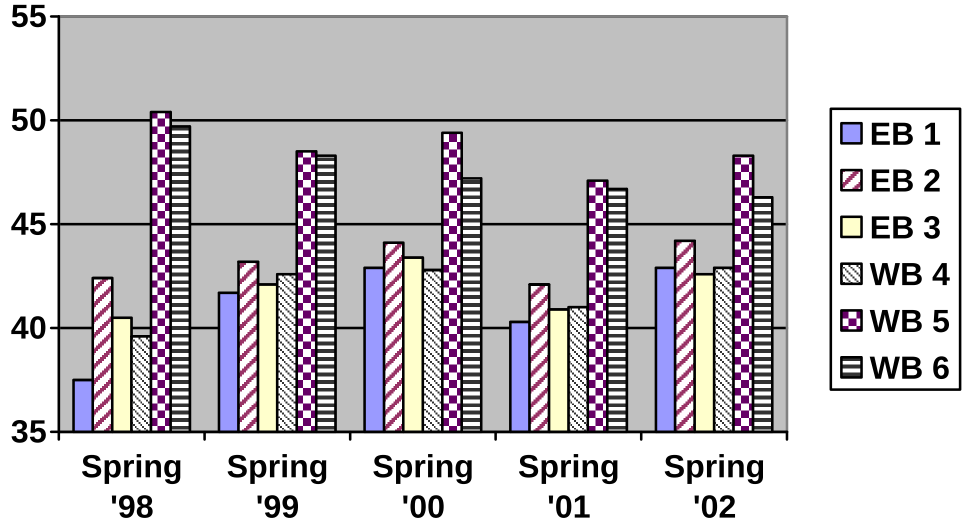

FIGURE 10
SMOOTH TIRE FRICTION NUMBERS MEASURED DURING EACH SPRING,
1998 THROUGH 2002

should have been no potential for hydroplaning in any of the sections. Thus far, an approximate variation of 8-9 FN_{40} in five years has been measured on average for ribbed-tire tests, and 5-8 FN_{40}, for smooth-tire tests. Much of this variation has been seasonal, possibly due to winter weather variations and the resultant varying amounts of abrasives applied to the pavements from year to year.

Some variation in surface texture has been observed among the various pavements, which can affect the friction values. In 1999, the pavement texture was measured in all six sections using an ARAN texture device, which was on loan from the ARAN manufacturer at that time. The mean texture depth (MTD) in mils measured for each section is given in Table 11. The two Superpave recycled sections (WB 5 and WB 6) showed a significantly higher surface texture compared to the four other sections. The smooth-tire friction numbers reflect the more open and coarse texture of sections WB 5 and WB 6. These two sections have somewhat higher smooth-tire friction numbers than the other four sections. This can be seen in Figures 9 and 10. It was also observed in 1997 and 1998 that water appeared to seep through the pavement and drain onto the surface near longitudinal paving joints, at a number of locations within Section WB 6.

The 5-1/2 year overall average friction numbers for every section (smooth and ribbed tires) are also given in Table 10. These values are plotted in Figure 11. They represent an average of all data collected between fall 1997 and fall 2002. There is a relatively insignificant difference between any of the sections for the ribbed-tire numbers. As mentioned previously, the smooth-tire friction numbers are higher for the two recycled sections (WB 5 and WB 6) with the higher texture, and somewhat larger aggregate (by design.)

TABLE 11
PAVEMENT TEXTURE MEASUREMENTS

Study Section	Average Mean Texture Depth (mils)	Standard deviation (+/-) (mils)
EB 1	31	3
EB 2	32	2
EB 3	32	2
WB 4	34	3
WB 5	39	4
WB 6	41	6

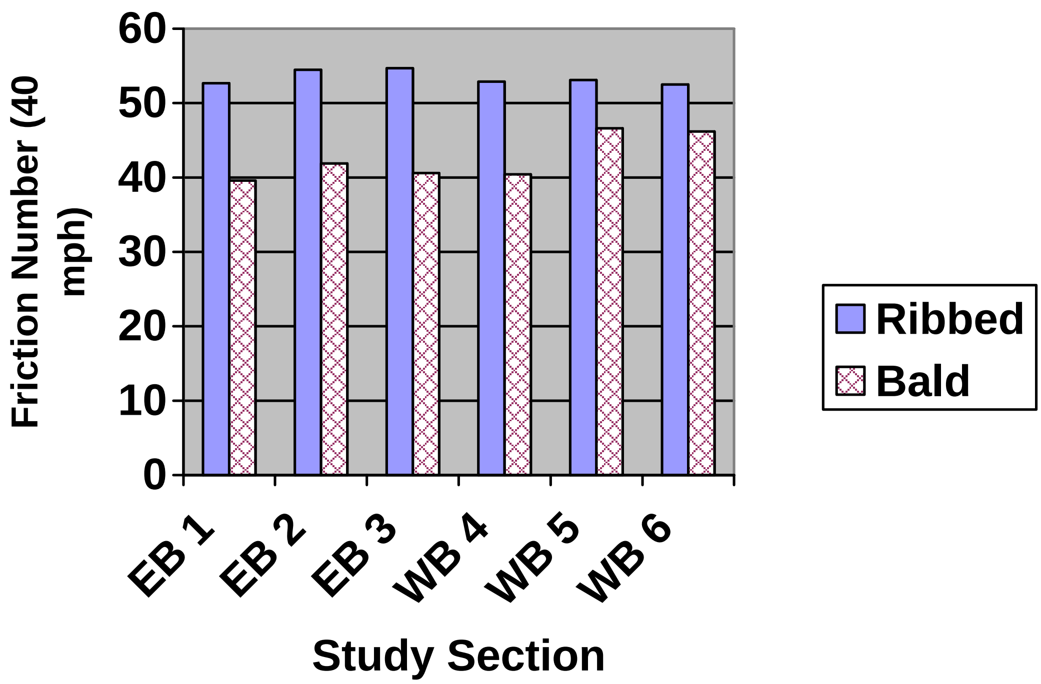

FIGURE 11
FIVE-AND-ONE-HALF-YEAR AVERAGE FRICTION NUMBER MEASURED AT 40 MPH
FOR THE SIX STUDY SECTIONS

Condition Surveys

Cracking   Condition surveys were performed by both Stantec personnel and ConnDOT Research personnel. Stantec performed surveys of only 500 ft of the sections used for the LTPP Study. A summary of cracking and other distress found by Stantec on September 17, 2002 is given in Table 12. Since the LTPP sections monitored for distress are only 500 feet long, the distress data reported in Table 12 are not necessarily representative of each entire 2-mile section.

TABLE 12
DISTRESS OBSERVED WITHIN 500-FT LTPP SECTIONS (SEPTEMBER 2002)

LTPP Section	Transverse Cracks (ft)	Longitudinal Cracks (ft)	Percent Longitudinal Joint cracked*	Percent of Longitudinal Edge Joint Cracked**
090901	7.2 (2 cracks)	0	0	55
090902	3.0 (2 cracks)	0	0	0
090903	0	2.0 (1 crack)	100	0
090960	0	0	61	0
090961	0	29.2 (2 cracks)	90	35
090962	0	0	42	0

* cold construction joint between lanes
** cold construction joint between shoulder and right lane
The longitudinal cracks listed in Table 12 were not in the wheel paths.

Research staff performed manual field driveover surveys of the entire project three times during the five year evaluation: August 1998, August 1999 and March of 2003. There was also one set of automated images that were summarized using Wisecrax software, in September 2000. The cracking data for the final field survey of March 2003 are included in Tables 13 through 15 and discussed below.

Table 13 presents a summary of transverse cracking in all six sections, including both cracks and transverse construction paving joints that have cracked (TCJC). The data are presented separately for each lane (low-speed and high-speed). It should be noted that information provided for section WB 6 on transverse cracks is presented as both the total for the section and as divided into two segments, by the first and second mile. This is due to the spatial distribution of the transverse cracks observed in this section. The first mile of section WB 6 contains almost no cracks, whereas the second mile contains significantly more cracking than any of the other five test sections. For all sections, data is provided as number (#) of transverse cracks identified during the field survey, and total length of transverse cracks in feet.

In the low-speed lane, sections EB 1, EB 2, EB 3, and WB 4 have similar amunts of transverse cracking. Whereas section WB 5 has significantly less cracking, and WB 6 significantly more cracking than the other four sections. In the high-speed lane, the results are different. Section EB 1 and WB 4 have significantly less cracking than sections EB 2, EB 3, WB 5 and WB 6. In both lanes, WB 6 contains more than double the amount of transverse cracking of any other section. The spatial distribution and total amount of cracking within section WB 6 is very difficult to explain.

One theory is that the underlying condition is having a much more significant influence on the performance of the surface than are the mix designs. All of the sections were milled, prior to paving, but only the top two-inch layer of material was removed. Therefore, whatever cracks existed in 1986 in the original pavement (which was placed in 1970), may be reflecting through the surface today.

TABLE 13
TRANSVERSE CRACKING (TWO-MILE SECTIONS)
MARCH 2003

	Low Speed Lane		High Speed Lane		Low Speed Lane TCJC*			High Speed Lane TCJC*		
Section	**Feet**	**# of cracks**	**Feet**	**# of cracks**	**Feet**	**# of joints**	**% length cracked**	**Feet**	**# of joints**	**% length cracked**
EB 1	224	33	96	15	33	4	63	39	5	60
EB 2	227	45	187	21	43	4	83	24	5	37
EB 3	206	44	266	33	46	4	88	42	5	65
WB 4	201	46	93	15	61	6	78	77	7	85
WB 5	32	4	110	13	70	8	67	70	7	77
First Mile of WB 6	0	0	44	5						
Second Mile of WB 6	710	77	425	58						
WB 6	710	77	469	63	43	5	66	44	5	68

* TCJC = transverse construction-joint cracks

After the pavement was milled in 1997, it was difficult to see any cracks, since dust had filled in the cracks during the milling operation. One reflection crack can be seen, however, in Image #5 taken during the milling operation. There is also evidence from viewing 1986 ConnDOT photolog images of Route 2 that prior to the 1986 overlay, the old pavement (placed in 1970) contained considerable distress, primarily in the form of block cracking at a moderate level. This appeared to be fairly uniform throughout the project, but was more prevalent in the high-speed lane than in the low-speed lane (see Figure 12). Also, the final mile of section WB 6 contained much more severe transverse cracking, with some areas of patching (see Figure 13).

IMAGE #5 CRACKS EVIDENT THROUGH BOTH SURFACE AND MILLED PAVEMENT (MAY 1997)

Theoretically, if the underlying cracks from 1986 (or actually what existed in 1997 during construction) were uniformly distributed throughout the six miles in both directions, then a comparison of

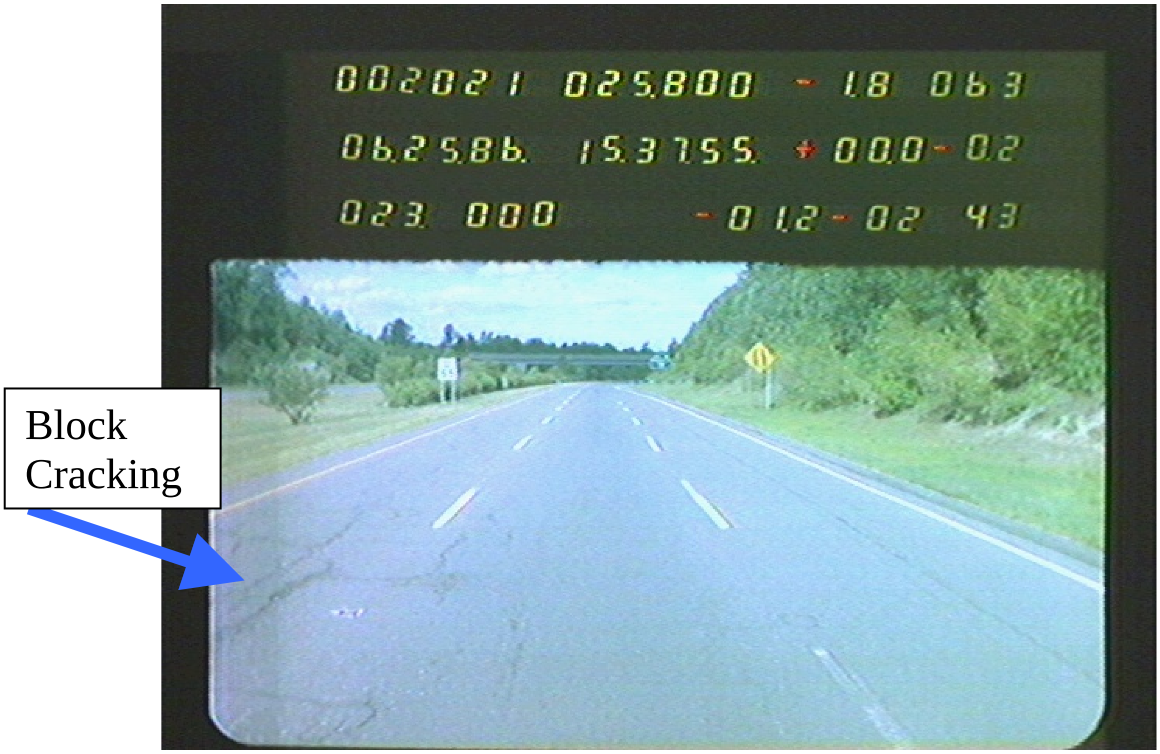

FIGURE 12. ROUTE 2, SECTION EB 1, JUNE 25, 1986 (TYPICAL BLOCK CRACKING PREVALENT THROUGHOUT MOST OF THE PROJECT)

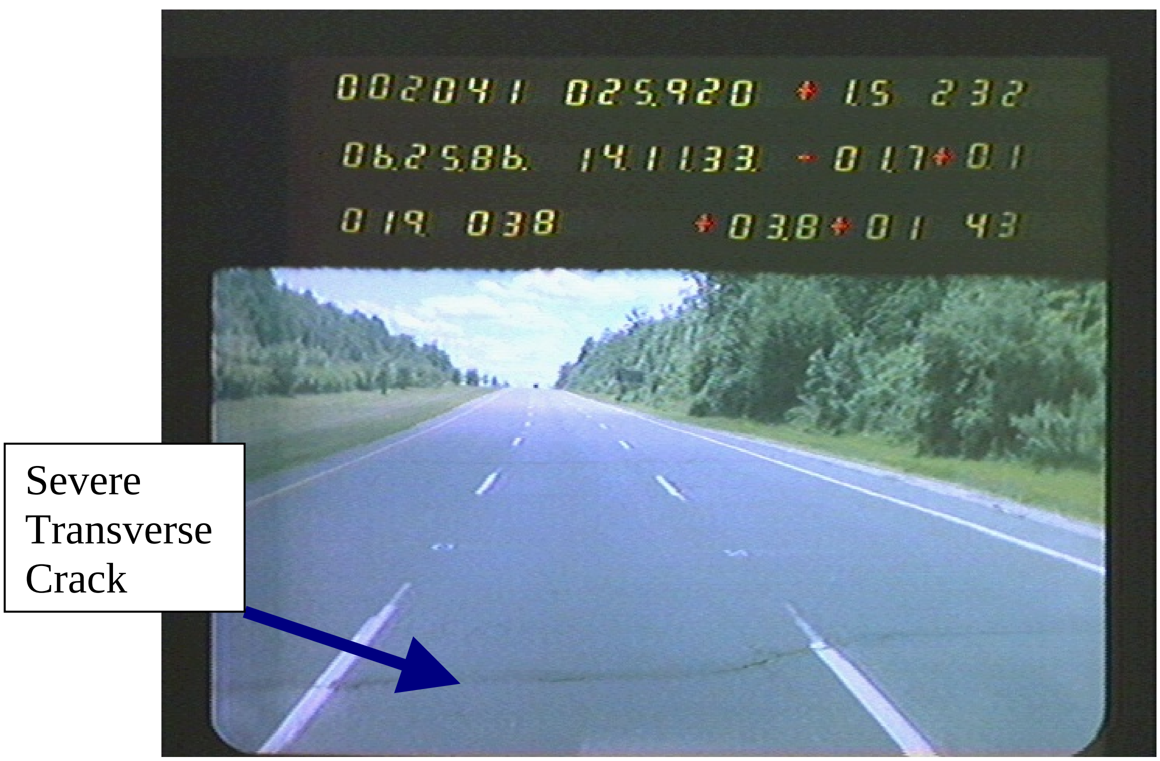

FIGURE 13. ROUTE 2, SECTION WB 6, JUNE 25, 1986. (SEVERE CRACKING THAT EXISTED ONLY AT THE END OF SECTION WB 6)

current surface cracking between the sections could be done, for example, using the data in Table 13 as previously discussed, to determine the difference in five-year performance of the various surface mixes. Transverse cracks are generally shrinkage-induced tension cracks that occur during cold weather. It is very difficult to prevent these cracks from re-occurring above a layer that already contains transverse cracks. However, block cracking in the size of less than 5 ft by 5 ft can be more easily contained, as total movement at the cracks is less during expansion and contraction. This could explain why the second mile of section WB 6, which contained the severe transverse cracks in 1986, contains more cracking (presumably reflection) than the rest of the project.

On the assumption that the rest of the project (i.e., sections EB 1, EB 2, EB 3, WB 4, and WB 5) had fairly uniform block cracking in 1986, then the differences in observed surface cracking in 2003, as presented in Table 13 for these sections, might be attributed more to variations in the pavement mix. If this assumption is made, then overall, based upon transverse cracking alone, sections EB 1 and WB 5 and the first mile of WB 6 are performing the best. However, due to the unknown reliability of the assumption, very little confidence can be given to this conclusion.

The quantity of cracks located at transverse construction paving joints is also given in Table 13 (see TCJC) for each lane of each section. In these sections, the column labeled as # represents the total number of construction joints within each section. The percentage of length of these joints with cracks is also given in Table 13. There does not appear to be a significant difference between any of the section’s transverse construction joint performance. More than half of the joints are cracked in nearly all of the sections. Quality of construction joints is highly dependent upon the paving crew. Therefore, conclusions about performance of mixes would not be justified solely on the basis of transverse paving joint conditions after five years.

Cracks along longitudinal construction-paving joints were also observed during field surveys. Since this project was paved in three passes, (left lane, right lane and shoulder,) there are two longitudinal paving joints that run parallel throughout the six miles of roadway in both directions. Cracking at the joint between the two travel lanes and cracking at the joint between the shoulder and right lane are summarized separately in Table 14. The percent length with cracking and the actual length of cracking are given in Table 14 for each section.

TABLE 14
LONGITUDINAL CONSTRUCTION-JOINT CRACKS
MARCH 2003

	Longitudinal Construction Joint Between Travel Lanes		Longitudinal Construction Joint Between Right Lane and Shoulder		Sum of Both Longitudinal Joints
Section	**Total Length of Cracks (mi)**	**Percent Joint Length Cracked**	**Total Length of Cracks (mi)**	**Percent Joint Length Cracked**	**Total Length of Cracks (mi.)**
EB 1	1.25	63	0.58	29	1.83
EB 2	1.13	50	0.08	3	1.21
EB 3	1.71	87	0.24	12	1.95
WB 4	1.66	83	0.83	42	2.49
WB 5	1.65	77	0.26	12	1.91
WB 6	1.24	60	0.50	24	1.74

It can be stated that the Superpave Virgin 98% reliability (Section EB 2) pavement provided the longest lived cold joints to date. Fifty percent of the middle joint and only 3 percent of the right joint were cracked after 5 years (six winters). The other five sections have considerably more cracks at longitudinal construction paving joints than section EB 2. The section with the most longitudinal joint cracks is WB 4, recycled conventional class 1 mix (non-Superpave).

An objective method to measure total length of longitudinal cracking was not employed during the manual field surveys. Table 15 gives a rough percentage of the roadway within each section that contains longitudinal cracks within travel lanes. The cracked longitudinal construction joints reported in Table 14, and discussed earlier, are not included with the cracks in Table 15. Most of the longitudinal cracks observed within the travel lanes are outside the wheel paths. The cracks appear to be somewhat random, and are possibly reflection cracks of portions of the underlying block cracking discussed previously. It can be seen in Table 15 that the recycled pavements contain more longitudinal cracking than the virgin mixes, regardless of whether they are Superpave or conventional mixes. However, the quantity of longitudinal cracking observed after five years is relatively small for all sections. No section contains longitudinal cracking for more than 12 percent of the section length.

TABLE 15
APPROXIMATE PERCENT OF PROJECT SECTION CONTAINING LONGITUDINAL CRACKS WITHIN THE TRAVEL LANES (MARCH 2003)*

Section	Percent Length
EB 1	5
EB 2	1
EB 3	1
WB 4	12
WB 5	10
WB 6	12

*Cracks at Longitudinal Joints are not included in these quantities. They are reported in Table 14.

<u>Rutting</u> - Pavement rut depths were measured for all sections at the times that the ARAN vehicles performed annual inventory surveys. The LTPP contractor also measured rut depths within the 500-ft LTPP sections on three occasions, at the same time that deflection testing was performed. The LTPP contractor used a Face Technologies Dipstick

Profiler to measure elevations across the transverse width of the low-speed lane, which were then plotted and converted to rut depths. The average rut depths for each wheel path reported by the LTPP contractor for the 500-ft sections are listed in Table 16. The average rut depths measured by ConnDOT on the full two miles of each section are reported in Table 17. (No rut depth data was collected in 2001 by ConnDOT.) All reported values are for the low-speed lane only. It is obvious from either table that rutting is not yet significant in any of the study sections. The maximum rut depths measured thus far are 1/8 inch.

A plot of the rut depths for 2002, (see Figure 14) shows that the westbound recycled sections have somewhat higher ruts than the virgin sections in the eastbound direction (particularly in the right wheel path.)

TABLE 16
WHEEL PATH RUT DEPTHS (INCHES) BY YEAR, WITHIN THE 500-FT LTPP SECTIONS, MEASURED BY LTPP CONTRACTOR

LTPP Section*	**1997**	**1998**	**2001**
090901 LWP	0.083	0.071	0.122
090901 RWP	0.004	0.0	0.012
090902 LWP	0.063	0.043	0.091
090902 RWP	0.008	0.004	0.012
090903 LWP	0.106	0.098	0.150
090903 RWP	0.008	0.008	0.031
090960 LWP	0.008	0.004	0.043
090960 RWP	0.004	0.004	0.031
090961 LWP	0.079	0.087	0.177
090961 RWP	0.004	0.004	0.051
090962 LWP	0.028	0.039	0.075
090962 RWP	0.004	0.0	0.020

Testing Dates: October 28, 29 & 30, 1997; April 1, 2, & 3, 1998; Otober 16, 17, & 18, 2001.
* LWP=left wheel path; RWP=right wheel path

TABLE 17
WHEEL PATH RUT DEPTHS (INCHES) BY YEAR, MEASURED BY CONNDOT

Section *	**1998**	**1999**	**2000**	**2002****
EB 1 LWP	0.068	0.076	0.107	0.104
EB 1 RWP	0.041	0.039	0.065	0.055
EB 2 LWP	0.068	0.077	0.120	0.103
EB 2 RWP	0.044	0.035	0.052	0.059
EB 3 LWP	0.061	0.070	0.116	0.089
EB 3 RWP	0.045	0.040	0.054	0.066
WB 4 LWP	0.080	0.091	0.134	0.121
WB 4 RWP	0.047	0.041	0.064	0.087
WB 5 LWP	0.104	0.113	0.151	0.112
WB 5 RWP	0.075	0.059	0.086	0.119
WB 6 LWP	0.103	0.095	0.133	0.106
WB 6 RWP	0.079	0.057	0.075	0.105

* Note: EX. EB 1 LWP = Eastbound, Section 01, left wheel path
** Rut data not collected in 2001: Test Dates: August 18, 1998; June 30, 1999; June 16, 2000; May 4, 2002.

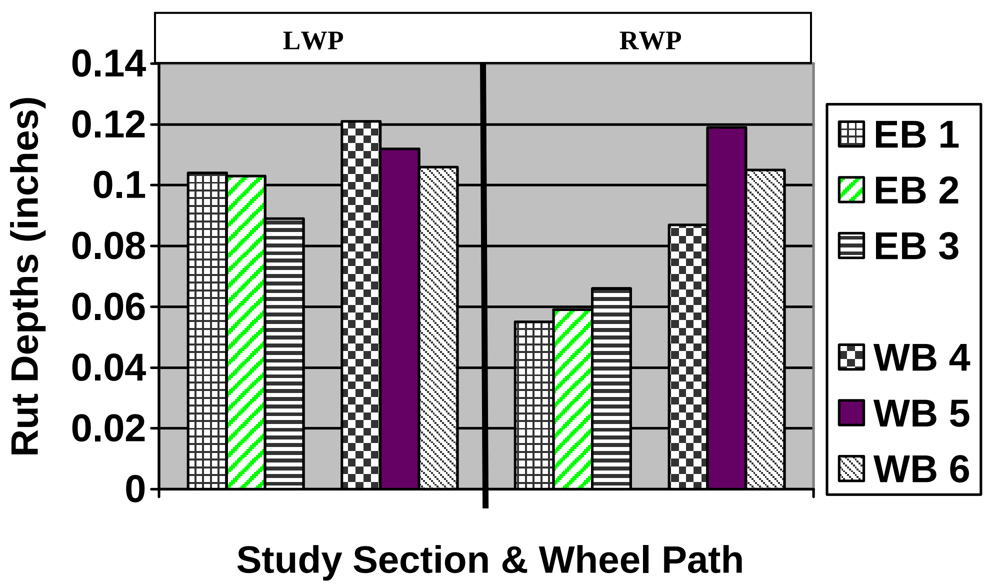

FIGURE 14, AVERAGE RUT DEPTH (INCHES) FOR LEFT (LWP) AND RIGHT (RWP) WHEEL PATHS OF ALL SIX SECTIONS

<u>Roughness</u> - Values for roughness, measured as International Roughness Index (IRI) in inches per mile, are listed in Table 18. It

can be seen in this table and in Figure 15 that there was a reported significant reduction in roughness on all six sections between 1999 and 2000. The overall average of this reduction for all sections is 4.80 inches per mile. The ConnDOT ARAN vehicle equipment was upgraded in early 2000, which can provide an explanation for the sudden reduction in roughness for all sections during one year. When an adjustment of 4.80 is added to all the values for year 2000 and later years, the revised data indicate that the roughness was very consistent over the five year period. In other words, the roughness values measured in May 2002 were nearly identical to the values measured in October 1997, once the 4.80 adjustment is included (see Table 18b). The real unknown is whether the 4.80 adjustment should be made to the roughness values after year 2000, or by subtracting the value to the years prior to 2000.

TABLE 18
INTERNATIONAL ROUGHNESS INDEX (IRI) FOR ALL SECTIONS, BOTH WHEEL PATHS, MEASURED BY CONNDOT

Section	**1997**	**1998**	**1999**	**2000**	**2001**	**2002**
EB 1 LWP	81.81	83.24	83.05	78.66	78.06	75.81
EB 1 RWP	73.92	75.42	75.11	70.49	69.66	70.90
EB 2 LWP	68.48	69.13	69.40	65.29	65.59	65.25
EB 2 RWP	70.78	70.40	70.80	66.46	67.13	69.21
EB 3 LWP	78.44	79.09	79.22	73.92	74.97	72.60
EB 3 RWP	70.10	69.50	69.73	65.12	64.91	64.67
WB 4 LWP	73.17	71.22	71.47	66.88	67.41	67.88
WB 4 RWP	78.88	75.81	76.00	74.24	72.24	75.68
WB 5 LWP	72.86	74.44	74.30	66.06	66.900	66.91
WB 5 RWP	74.78	74.27	74.01	68.68	68.38	67.44
WB 6 LWP	72.58	72.66	72.14	66.25	67.86	67.01
WB 6 RWP	70.54	70.3	69.72	65.34	64.76	65.74
Overall Average	**73.86**	**73.79**	**73.75**	**68.95**	**68.99**	**69.09**

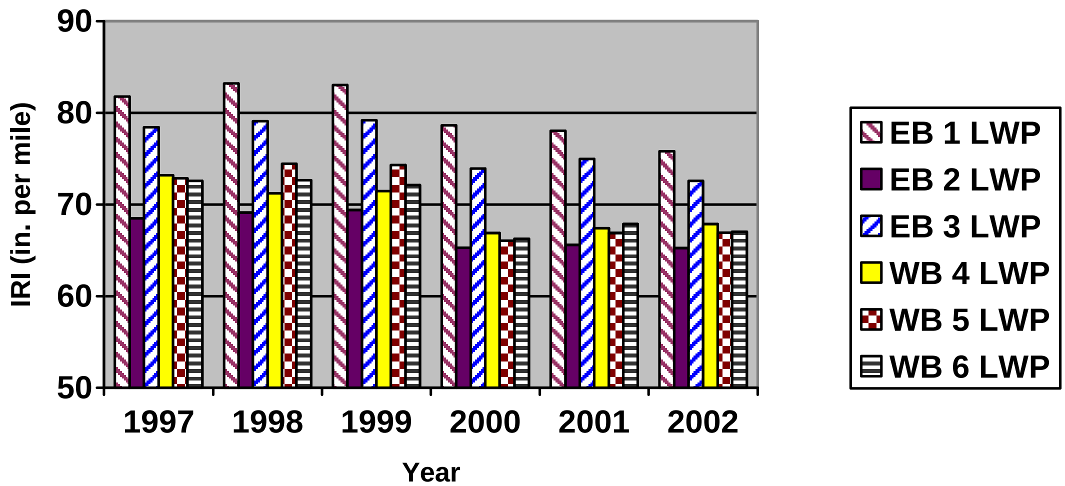

FIGURE 15, ROUGHNESS FOR LEFT WHEEL PATH ONLY FOR THE SIX SECTIONS FROM 1997 THROUGH 2002

Roughness data (IRI) measured by the LTPP contractor for the 500-ft LTPP sections, using an inertial profiler from K.J. Law Engineers, are given in Table 19. A comparison of these data with ConnDOT's measurements in Table 18 indicates that prior to the equipment upgrade in 2000 ConnDOT's measurements were much higher than the LTPP contractor's. From Table 19, it can be shown that the average overall roughness, measured by the LTPP contractor, for the combined sections (i.e., average of all six sections) in 2001 and 1997 is 61.01 and 62.85 in/mi., respectively. Whereas, from Table 18, (ConnDOT's measurements) the average overall roughness for 2001 and 1997 (without adjustment) is 68.99 and 73.86 in/mi., respectively. If 4.80 is subtracted from the 1997, 1998, and 1999 data rather than added onto later years, the difference between the overall average from 1997 to 2001 as indicated in Table 18b becomes -0.27 in/mi. for the ConnDOT-collected data. This value (difference) is -1.84 in/mi. for the LTPP-collected data set. These are similar differentials, even though the magnitude of the values measured by the two devices differs significantly. [As an aside, this also demonstrates the lack of reproducibility between various manufacturers of profilers, while at the same time the repeatability of

TABLE 18B
COMPARISON OF DIFFERENCES IN ROUGHNESS BY SECTION BEFORE AND AFTER ADJUSTMENT

Section	Difference Between 2000 and 1999 Data (in/mi)	Difference Between 2002 Data & 1997 Data after Adjustment by 4.80 (in/mi)
EB 1 LWP	4.39	-1.20
EB 1 RWP	4.62	-1.78
EB 2 LWP	4.11	+1.57
EB 2 RWP	4.34	+3.23
EB 3 LWP	5.30	-1.04
EB 3 RWP	4.61	-0.63
WB 4 LWP	4.59	-0.49
WB 4 RWP	1.76	+1.60
WB 5 LWP	8.24	-1.15
WB 5 RWP	5.33	-2.54
WB 6 LWP	5.89	-0.77
WB 6 RWP	4.38	0.0
Average Difference	**4.80**	**-0.27**

TABLE 19
ROUGHNESS AT 500-FT LTPP SECTIONS
MEASURED BY LTPP CONTRACTOR (INCHES/MILE)

LTPP Section	**Oct 1997**	**Jun 1998**	**Jul 1999**	**Aug 2001**
090901 LWP	60.34	62.57	64.17	64.74
090901 RWP	68.08	64.42	64.61	61.91
090902 LWP	52.37	53.88	54.08	53.04
090902 RWP	77.73	69.88	68.89	66.06
090903 LWP	70.47	71.10	72.78	71.78
090903 RWP	61.19	56.39	60.54	58.33
090960 LWP	59.57	58.48	64.52	64.11
090960 RWP	65.06	62.14	64.22	61.48
090961 LWP	57.20	57.17	56.59	55.84
090961 RWP	63.26	59.25	57.87	60.47
090962 LWP	54.94	54.65	55.28	55.21
090962 RWP	64.04	62.96	62.37	59.12
Overall Average	**62.85**	**61.07**	**62.16**	**61.01**

the same ARAN vehicle over time, as long as equipment upgrades do not occur. [3]

A comparison of the difference in roughness between the various study sections can be observed in Figure 16. The roughness in inches per mile (IRI) measured in 2002 with the ARAN (values taken from Table 18) is plotted for each wheel path and each section in Figure 16. The difference in roughness between the left and right wheel paths appears to be more significant than any differences between the various sections. However, when the left and right wheel paths are averaged together, the two non-Superpave sections, EB 1 and WB 4 have the highest roughness compared to the four Superpave sections. This can be seen in the plot of Figure 17.

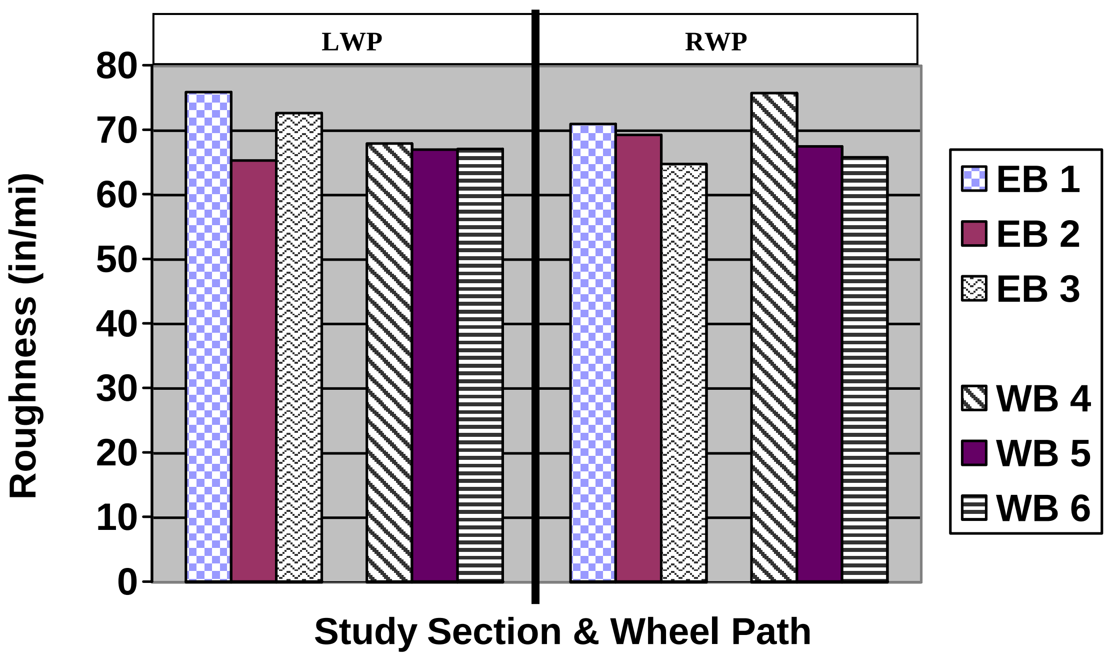

FIGURE 16 - INTERNATIONAL ROUGHNESS INDEX (IRI) IN INCHES PER MILE FOR EACH WHEELPATH OF EACH STUDY SECTION MEASURED IN YEAR 2002

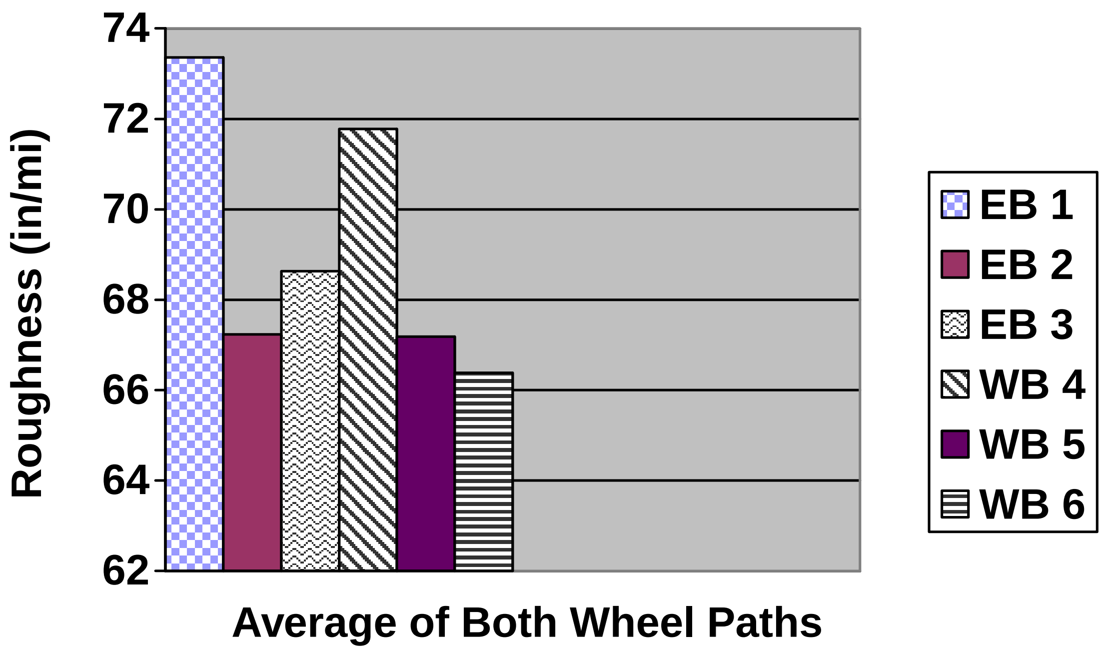

FIGURE 17 - INTERNATIONAL ROUGHNESS INDEX (IRI) IN INCHES PER MILE FOR EACH STUDY SECTION MEASURED IN YEAR 2002

Climate

Weather - Tables 20, 21, and 22 provide data on temperature and precipitation. These data were obtained from National Oceanographic and Atmospheric Administration (NOAA) records of either a weather station at Norwich, CT., (a location that is about 10 miles from the study sites), or Hartford, CT. Even though weather data was collected at a RWIS on-site location, in this case due to software, format, and program issues it was easier to obtain summarized data from NOAA records. Also, some of the data stored from the RWIS was erroneously deleted so that historical data prior to 2000 is no longer available.

The deviations for temperature in Table 20 (row 4) indicate that all years during the period 1997-2001 (with the exception of year 2000) were warmer than the previous 30-year average. Year 2001 was the only year with higher than average degree-days (Table 20, row 6), indicating that this one winter period was cooler than normal. It is almost impossible to draw any conclusions regarding distress development and weather conditions that existed during the study.

TABLE 20
WEATHER DATA: TEMPERATURES (NORWICH, CT)

	1997	1998	1999	2000	2001	2002
Minimum (lowest temp of year)	Jan 19 **0 ^{0}F**	Jan 1 **8 ^{0}F**	Jan 2 **5 ^{0}F**	Jan 17 **2 ^{0}F**	Jan 4 **7 ^{0}F**	Jan 2 **12 ^{0}F**
Maximum (highest temp of year)	July 18 **98 ^{0}F**	June 26 **94 ^{0}F**	July 5 **101 ^{0}F**	June 11 **93 ^{0}F**	Aug 10 **102 ^{0}F**	July 5 **99 ^{0}F**
Average annual temp (^{0}F)	50.7	53.3	52.9	50.1	51.9	
Deviation from long-term average, 50.3^{0}F*	+0.4	+3.0	+2.6	-0.2	+1.5	
# of heating degree-days**	5673	5192	5230	5560	6041	
Heating Degree-day Deviation and percent of long-term average 5869**	-196, 97%	-677, 88%	-639, 89%	-309, 95%	+172, 103%	

* = average from 1961-1990
** = Heating Degree-days = each whole degree that the daily mean temp. is below 65^{0}F
Measurements: Norwich Public Utility Plant, New London County, Elevation 20 ft
Source: Climatological Data-New England, National Climatic Data Center, Department of Commerce, Asheville, North Carolina, Volumes 109, #1 through Volume 114, #7. (1997-2002)

More detailed information about the summer and winter periods is presented in Table 21. These data are obtained from NOAA records for Hartford, CT, which is approximately 25 miles from the study location. Table 21 provides six sets of monthly information for the period 1997 through 2002 as follows: For the summer months of June, July and August; comparison of normal (average) and actual data for average daily temperatures, # of days with air temperature above 90^{0}F, and average maximum daily temperatures. For the winter months of December, January, and February; comparison of normal (average) and actual data for average daily temperatures, # of days with air temperature below 0^{0}F, and average minimum daily temperatures is provided.

All of the shaded cells represent months with above normal values during the summer months or below normal values during the winter months. As can be seen, the above normal data significantly surpass the below normal data. Compared to the 98-year average for Hartford,

TABLE 21
TEMPERATURE DATA FOR HARTFORD, CT (BRADLEY INTERNATIONAL AIRPORT) 1997 THROUGH 2002

SUMMER	**June**	**July**	**August**	***WINTER***	**December**	**January**	**February**
Average Monthly Daily Temperature (^{0}F)				**Average Monthly Daily Temperature (^{0}F)**			
98 Year Average	67.9	72.4	71.0	98 Year Average	30.7	26.8	28.1
1997	68.7	72.3	70.1	1997-1998	31.4	32.6	36.2
1998	67.0	72.9	73.6	1998-1999	36.8	25.9	31.8
1999	71.0	76.5	72.1	1999-2000	34.6	24.0	31.0
2000	67.7	69.6	69.6	2000-2001	25.4	25.4	28.4
2001	69.6	69.7	75.2	2001-2002	36.9	34.0	34.6
2002	67.3	75.2	73.9				
Number of Days with Maximum Temperature Above 90^{0}F				**Number of Days with Minimum Temperature Below 0^{0}F**			
30 Year Average	3.6	8.0	4.8	30 Year Average	1.1	3.2	1.7
1997	8	5	3	1997-1998	0	0	0
1998	1	4	2	1998-1999	0	1	0
1999	8	14	4	1999-2000	0	7	0
2000	4	1	1	2000-2001	0	2	0
2001	2	4	8	2001-2002	0	0	0
2002	3	13	14				
Average Monthly Maximum Daily Temperature (^{0}F)				**Average Monthly Minimum Daily Temperature (^{0}F)**			
30 Year Average	80	85	82.7	30 Year Average	21.3	15.8	18.6
1997	81.5	84.1	80.3	1997-1998	23.5	26.4	28.0
1998	75.6	83.5	84.6	1998-1999	27.6	17.4	23.1
1999	83.7	88.5	82.2	1999-2000	26.2	15.0	22.8
2000	78.4	79.7	79.6	2000-2001	16.9	16.1	20.3
2001	80.6	82.2	86.2	2001-2002	28.4	25.8	23.8
2002	79.2	86.8	86.0				

Source: Department of Commerce, NOAA-National Climatic Data Center, Local Climatologocal Data – Annual Summary with Comparative Data, Hartford, CT (BDL), 1997-2002.

between 1997 and 2002, four summers averaged above normal and four winters averaged milder than normal for two or more months. This pattern would lead to speculation that weather conditions were favorable for the development of ruts. The summer of 2002 was a particularly hot period with more than double the average of days with high temperatures greater than 90^0F. However, as discussed previously, significant rutting has not been found to-date on Route 2.

In conclusion, overall the summers have been somewhat warmer than normal, whereas the winters have been exceptionally warmer than normal for this five year period. Warmer than normal winters would not be conducive to development of cold-temperature cracking for the designs used at this project.

It can be inferred from Table 22 that eastern Connecticut does not have a "rainy season" or "dry season." Precipitation is evenly distributed over the year. The average number of rainy days per month is also relatively consistent throughout the year.

TABLE 22
PRECIPITATION DATA: NUMBER OF DAYS PER MONTH WITH 0.10 IN. OR MORE OF PRECIPITATION (NORWICH, CT)

Month	**1997**	**1998**	**1999**	**2000**	**2001**	**2002**
January	8	8	11	6	8	8
February	4	8	8	5	4	5
March	11	6	10	8	10	5
April	7	8	3	11	7	7
May	8	13	7	9	6	7
June	6	11	0	7	5	8
July	3	4	5	7	6	4
August	8	4	7	9	7	4
September	3	4	7	8	4	6
October	4	6	8	2	3	4
November	7	4	4	5	2	11
December	8	4	6	6	5	NA
Total # of days with 0.10 or more precip.	77	80	76	83	67	NA
Ave/mo.	6.4	6.7	6.3	6.9	5.6	NA
Total Annual Precip (in.)	43.62	62.15	52.05	51.21	47.63	NA
Deviation from average 50.01 (in)**	-6.39	+12.14	+2.04	+1.20	-2.38	NA

Source: Department of Commerce, NOAA-National Climatic Data Center, Climatologocal Data – New England Volume 109 #1 through Volume 114, #11

TRAFFIC LOADS

The Superpave design on Route 2 was based on a projected accumulation of between 1 and 3 million 18,000-lb equivalent single axle loads (18K ESALs) per travel lane over the design life of the pavement. Typically, the accumulated traffic loads would have a direct effect on pavement performance life. An automated WIM system installed in Fall 1997, which became fully operational in September 1998, counts and classifies all vehicles traveling along each of the four lanes of Route 2, at milepoint 29.61 in Lebanon. It also measures the weight of each axle and summarizes the data per vehicle. Using the weights, the WIM software calculates the 18K ESAL per vehicle. Determining the accumulated ESALs is thus a summation of the vehicles recorded for the period of time under consideration, with some extrapolation required for any periods of time when data are not collected or considered invalid.

The measured traffic-volume counts (ADTs) and ESALs for route 2 are summarized in Table 23. From the Table, it can be calculated that the average traffic distribution (ADT) over the five years is evenly split at 50 percent by direction (east and west). Roughly 75 percent of the traffic travels in the outside (right) lanes. The distribution by travel lane of ESALs is also shown in Table 23. Due either to the travel pattern of loaded trucks or the weights measured by the WIM system, it is indicated in table 23 that the ESALs are considerably higher in the westbound direction. Approximately 57 percent of total accumulated ESALs have occurred in the westbound direction (43 percent eastbound). The accumulated ESALs for the five-year period as a percentage of maximum design lane ESALs (3 million) are also given in the bottom row of Table 23. Based upon these percentages, the maximum design ESALs would not be expected to be reached for at least another ten years. Since only five years have passed, with only two to 25 percent of the design ESALs accumulated per lane thus far, it is obvious that any distress developed to date is not likely to have been caused by accumulated loads. As indicated in other portions of this report, almost no load-related (fatigue) cracking has been observed to-date. This, along with the absence of significant rutting, suggests that the pavement is not deteriorating at an unreasonable or unexpected rate. The only exceptions are three short segments of the high-speed lane in the WB 5 and WB 6 sections, which are a special situation discussed in a later section of this report.

TABLE 23
FIVE-YEAR TRAFFIC VOLUMES AND ESALS FOR ROUTE 2, LEBANON, CT, 1998-2002

	WB SLOW LANE		WB PASS LANE		EB PASS LANE		EB SLOW LANE		
YEAR	**Veh/day**	**ESALs**	**Veh/day**	**ESALs**	**Veh/day**	**ESALs**	**Veh/day**	**ESALs**	**ADT**
1998	8,078	102,750	2,109	8,342	2,413	9,310	7,842	95,015	20,442
1999	8,560	172,484	2,605	11,977	2,889	9,668	8,373	112,630	22,427
2000	8,731	152,388	2,904	9,820	2,209	13,158	8,652	115,805	22,496
2001	8,759	145,343	2,887	9,141	3,150	12,185	8,617	115,733	23,413
2002	8,465	131,731	2,928	9,388	3,223	13,406	8,321	79,478	22,937
TOTAL		704,696		48,668		57,727		518,661	
% ADT Change 1998-2002	4.8		38.8		33.6		6.1		12.2
% of Design ESALs Accumulated		23.5*		1.6*		1.9*		17.3*	

* Total ESALs divided by 3,000,000 multiplied by 100

MATERIALS SAMPLING AND TESTING

Tests were performed by ConnDOT on cores that were removed after completion of the paving, in September 1997, according to LTPP specification. These tests are listed in Table 24. Additional cores were taken at time intervals of 6, 12, 18, 24, and 48 months (i.e., between the spring of 1998 and fall 2001). The dates that field cores were removed for each section and for each time interval are given in Table 25.

Stantec assisted with and observed the removal of the cores. They ensured that the cores were removed from the correct locations and properly labeled. ConnDOT Materials Testing personnel performed the actual coring of the 6-inch diameter cores using an Acker Model T22 Core Rig with a water-cooled diamond tip core barrel. Except for the first interval 0, eight cores were removed from each 1000-ft section, each time. This resulted in 48 cores per section. Section EB 2 had an additional 10 cores removed during time interval 0, only. Over the complete period of four and one-half years (fall 1997 through fall 2001); this resulted in a total of 298 six-inch cores being removed and tested.

TABLE 24
SPS 9A PROJECT – CONNDOT LABORATORY TESTS ON PAVEMENT CORES
AT TIME=0, 6, 12, 18, 24, & 48 MONTHS

LABORATORY TEST	LTPP TEST/PROTOCOL	NUMBER OF TESTS
Core Examination/Thickness	AC01/P01	48, (58)*
Bulk Specific Gravity	AC02/P02	48
Maximum Specific Gravity	AC03/P03	12
Asphalt Content (Extraction)	AC04/P04	48
Aggregate Gradation (Extracted Aggregate)	AG04/P14	12
Volumetrics		
Volume Percent of Air Voids	AASHTO PP19	12
Percent Voids in Mineral Aggregate	AASHTO PP19	12
Voids Filled with Asphalt	AASHTO PP19	12
Recovered Asphalt Cement		
Abson Recovery	AE01/P21	48
Penetration at 5 deg. C	AASHTO T49	6
Penetration at 25 deg., 46 deg. C	AE02/P22	12
Viscosity at 60 deg., 135 deg. C	AE05/P25	24
Specific Gravity at 16 deg. C	AE03/P23	12
Dynamic Shear at 3 temps	AASHTO TP5	12
Creep Stiffness at 2 temps.	AASHTO TP1	12
Direct Tension at 2 temps,	AASHTO TP3	12

* Additional cores taken at section EB 2 only during time=0

TABLE 25
DATES OF CORE REMOVAL BY SECTION

Time Interval (months after construction)	Study Section	Number of Cores Removed	Date of Core Removal
0	EB 1	8	9/15/97
	EB 2	18	9/15/97 & 9/16/97
	EB 3	8	9/16/97
	WB 4	8	9/16/97
	WB 5	8	9/17/97
	WB 6	8	9/17/97
6	EB 1	8	4/1/98
	EB 2	8	4/1/98
	EB 3	8	4/2/98
	WB 4	8	4/2/98
	WB 5	8	4/3/98
	WB 6	8	4/3/98
12	EB 1	8	10/6/98
	EB 2	8	10/6/98
	EB 3	8	10/7/98
	WB 4	8	10/7/98
	WB 5	8	10/8/98
	WB 6	8	10/8/98
18	EB 1	8	4/6/99
	EB 2	8	4/6/99
	EB 3	8	4/7/99
	WB 4	8	4/7/99
	WB 5	8	4/8/99
	WB 6	8	4/8/99
24	EB 1	8	9/14/99
	EB 2	8	9/14/99
	EB 3	8	9/14/99
	WB 4	8	9/15/99
	WB 5	8	9/15/99
	WB 6	8	9/15/99
48	EB 1	8	10/16/01
	EB 2	8	10/16/01
	EB 3	8	10/17/01
	WB 4	8	10/17/01
	WB 5	8	10/18/01
	WB 6	8	10/18/01

Laboratory Test Results

Except for the core thickness measurement and visual examination, which were done on-site in the field, all the tests listed in Table 24 were performed at the ConnDOT Materials Testing Laboratory in Rocky Hill, Connecticut. The results of all tests were reported on LTPP forms and submitted to Stantec for review. Upon acceptance by Stantec the data are entered by the contractor into the LTPP national database.

One of the issues discovered during coring was evidence of moisture-induced stripping in section WB 6. Stripping was visually evident during core removal at months 6 and 12, but not during later core intervals. This suggests that the stripping damage may be isolated or sporadic in nature. [NOTE: Every core removed is spaced five feet longitudinally from the previous core. Therefore, at time interval 18 months, the first core removed is at least 40 feet away from the cores removed at interval six months.]

Moisture-induced asphalt stripping was unexpected, because traditional pavement mixes in Connecticut have no history of a stripping problem. Also, in section WB 6 the anti-strip agent Kling Beta was used at 0.375 percent. However, this is the same section where water has been observed seeping through the surface of the pavement near longitudinal joints. A poor drainage situation, high water table, or excess of pavement permeability have likely contributed to the asphalt stripping from the aggregate. [NOTE: Auger soil samples obtained in September 1996 indicated that in section WB 6 water was encountered at 10 to 12 feet below the surface. Auger samples from the other five sections did not locate water within 20 feet of the surface.]

As indicated in Table 24, laboratory tests were performed on cores and the extracted asphalt. Some of the data measured for asphalt content, penetration and viscosity, and volumetrics, are plotted in Figures 18 through 25. To simplify the analysis, each figure contains data for only three of the time intervals when cores were obtained: 6 months, 18 months and 48 months. The trends over time for each measured property can be observed within these Figures.

Percent asphalt content is plotted in Figure 18. Theoretically, asphalt content should not deviate over time within a pavement. Figure 18 shows that asphalt content for three of the sections dropped by 0.5 percent at time 18 months, but rose somewhat again at month 48. This can only be explained as unexpected variability in the extraction testing. Regardless, the higher asphalt contents were obtained in the virgin mixes (EB 1, EB 2, and EB 3), with average asphalt contents between 4.8 and 5.2 percent. The three recycled mixes contain asphalt contents between 4.3 and 4.8 percent.

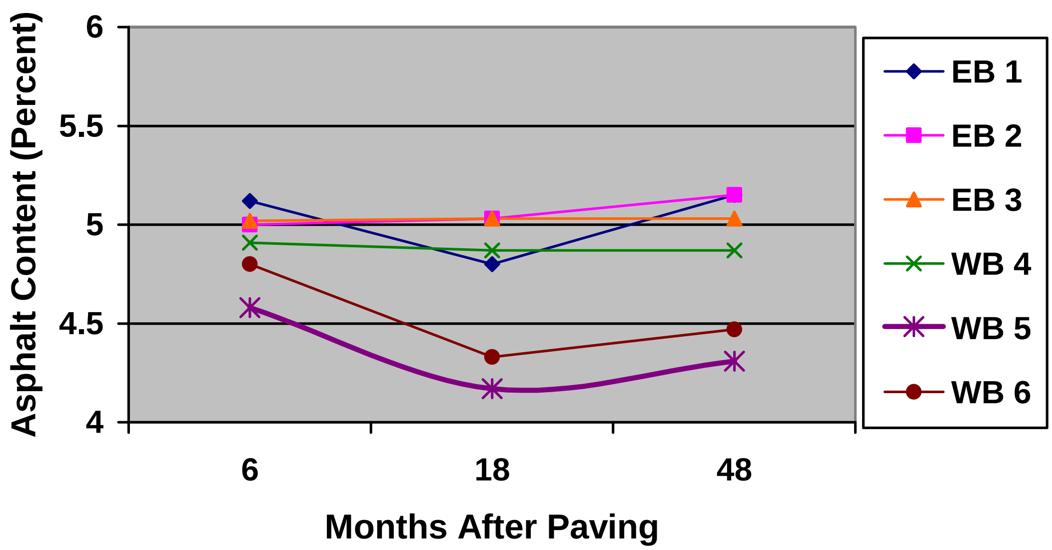

FIGURE 18 - ASPHALT CONTENT MEASURED AT 6, 18, AND 48 MONTHS

Figures 19 and 20 are graphs of Kinematic Viscosity at 275^{0}F and Absolute Viscosity at 140^{0}F, respectively. Figures 21 and 22 show Penetration at 77^{0}F and 115^{0}F as a function of time. As pavement ages, these asphalt properties change. It is expected that viscosity will increase and penetration decrease over time.

The most noteworthy feature of the four graphs is that all sections are behaving as expected, except section EB 3. The section EB

3 asphalt has decreased in viscosity and increased in penetration. The reason for these results is unknown.

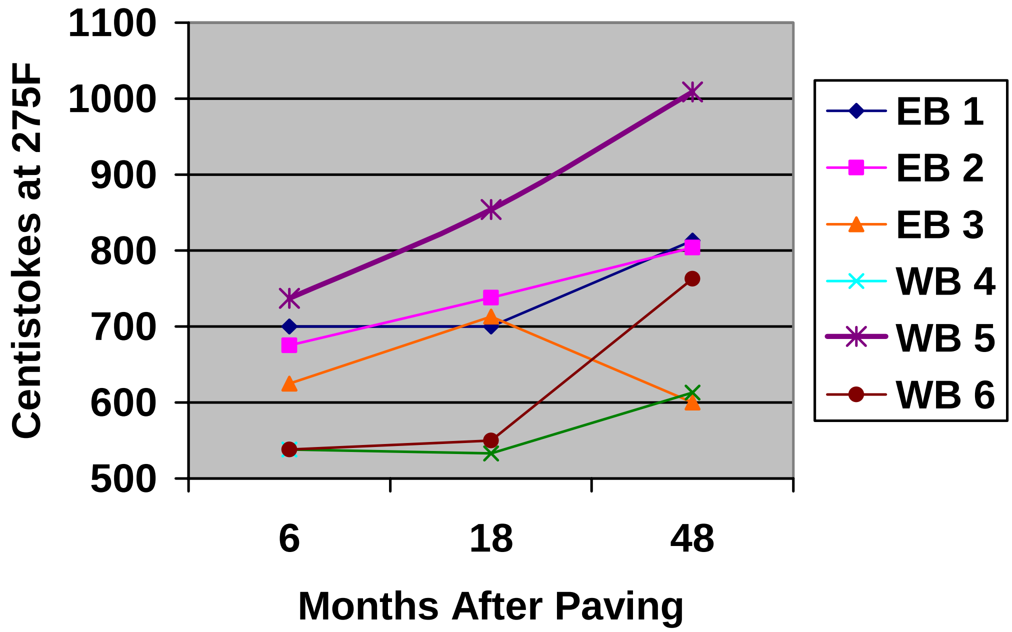

FIGURE 19 - KINEMATIC VISCOSITY MEASURED AT 6, 18, AND 48 MONTHS

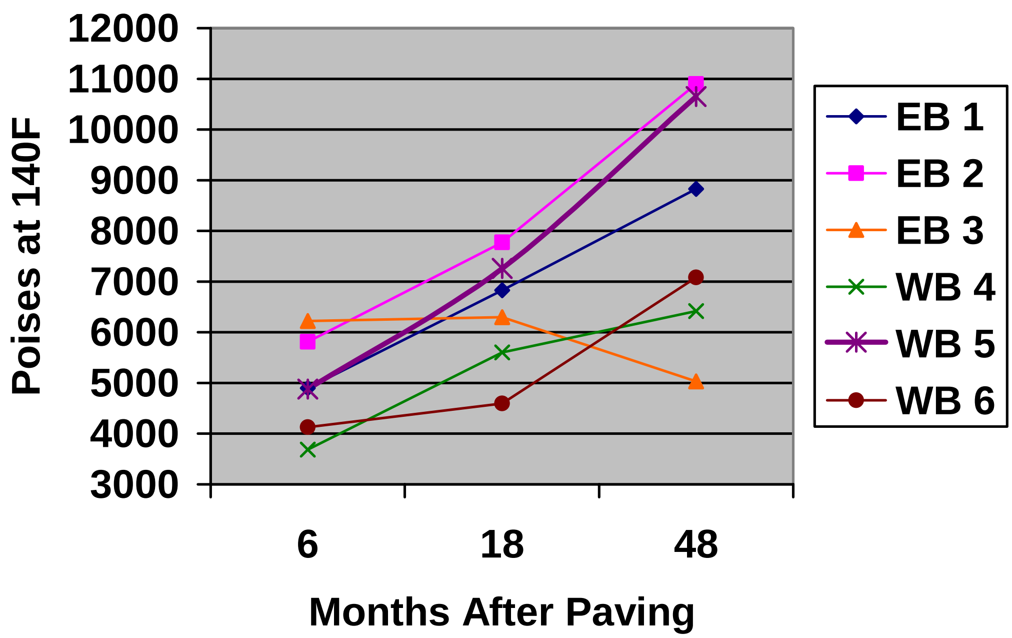

FIGURE 20 - ABSOLUTE VISCOSITY MEASURED AT 6, 18, AND 48 MONTHS

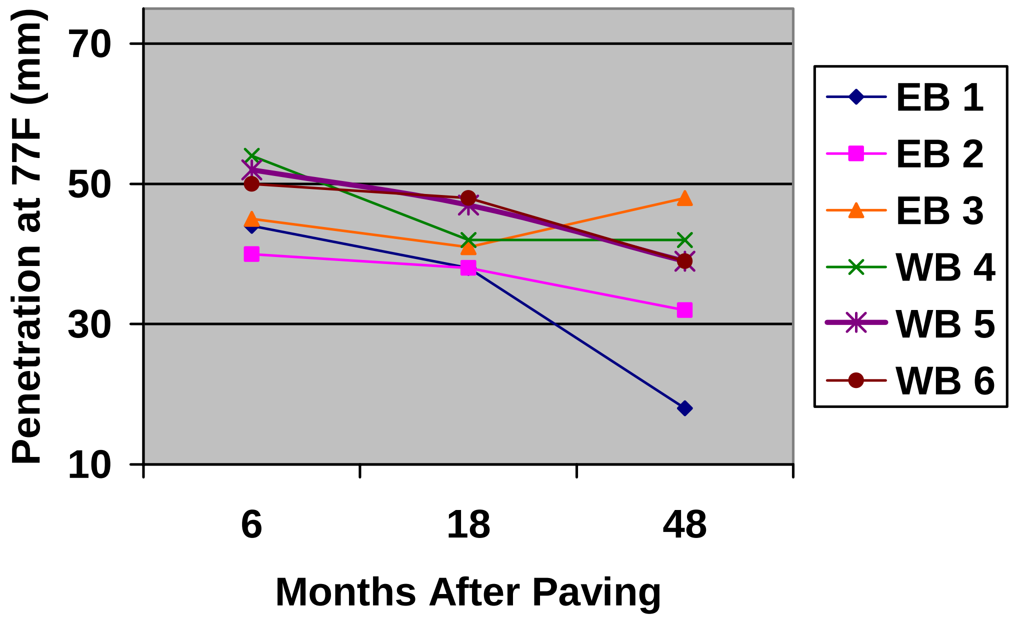

FIGURE 21 - PENETRATION (77^{0}F) MEASURED AT 6, 18, AND 48 MONTHS

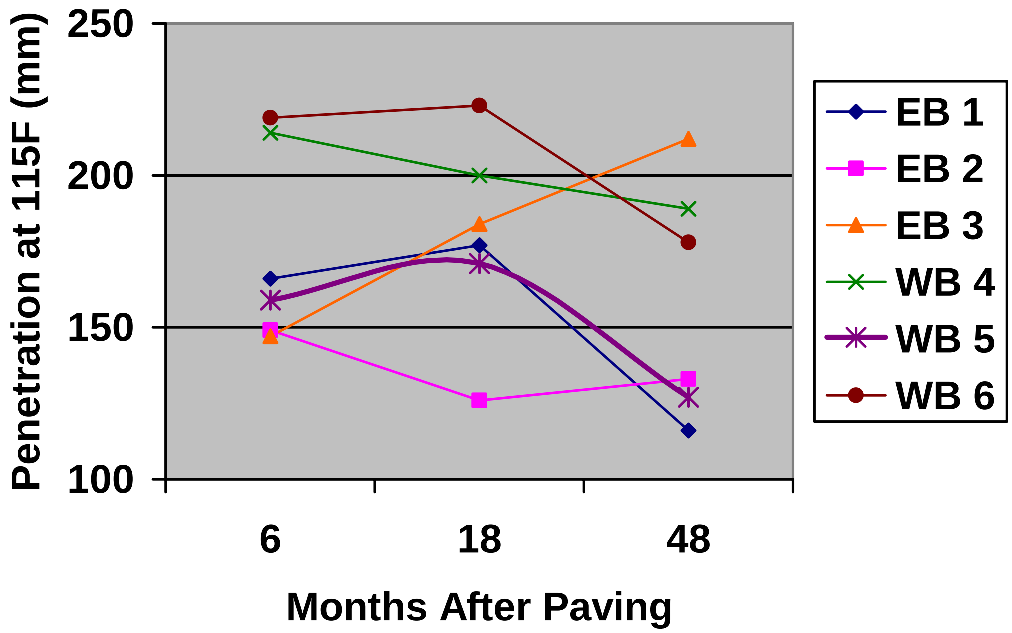

FIGURE 22 - PENETRATION (115^{0}F) MEASURED AT 6, 18, AND 48 MONTHS

Figures 23 through 25 give the total air voids, voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA), as determined from cores taken at 6 months, 18 months, and 48 months. The design tolerance limits for these attributes are also included as dashed lines in these figures. There is considerable variation between the six sections. Section EB 1 (the class 1 conventional mix) initially has the lowest voids, lowest VMA, and highest VFA, all of which are near or just outside the specified Superpave tolerance limits. At the opposite extreme (highest voids, highest VMA, and lowest VFA) is section WB 5 (Superpave with RAP, 98 percent reliability). All other sections have values that are scattered between these two sections. Not surprisingly, the two recycled sections (WB 5 and WB 6) with the highest texture (see Table 11) have the highest voids. These two sections contain isolated segments with premature failures (cracking and raveling), which brought about additional coring and testing that is discussed in the next section in this report.

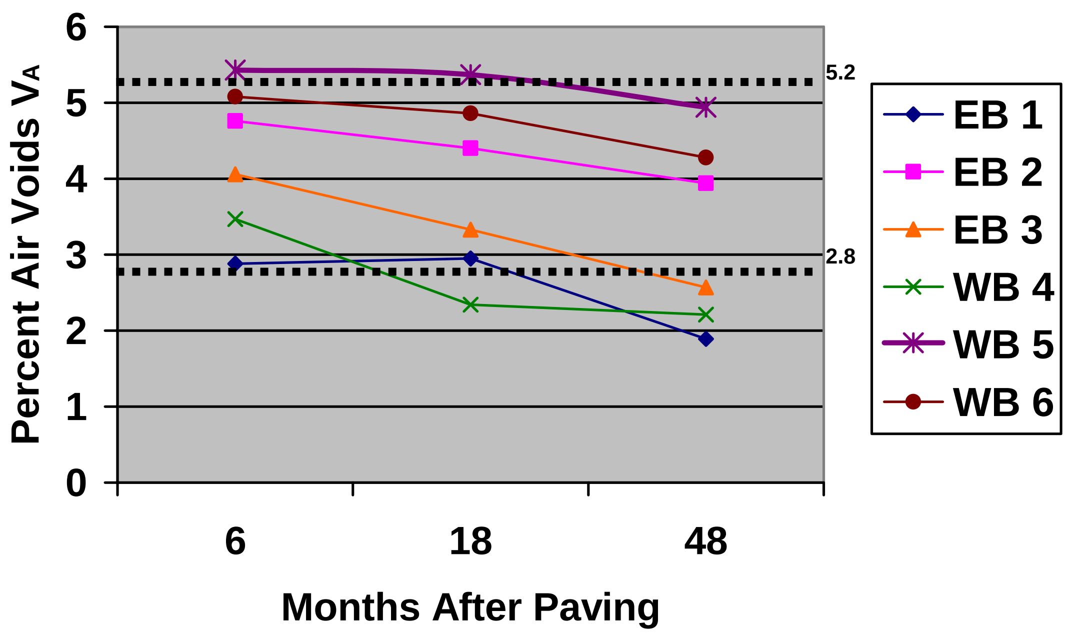

FIGURE 23 - AIR VOIDS MEASURED AT 6, 18, AND 48 MONTHS (SPECIFICATION LIMITS INDICATED)

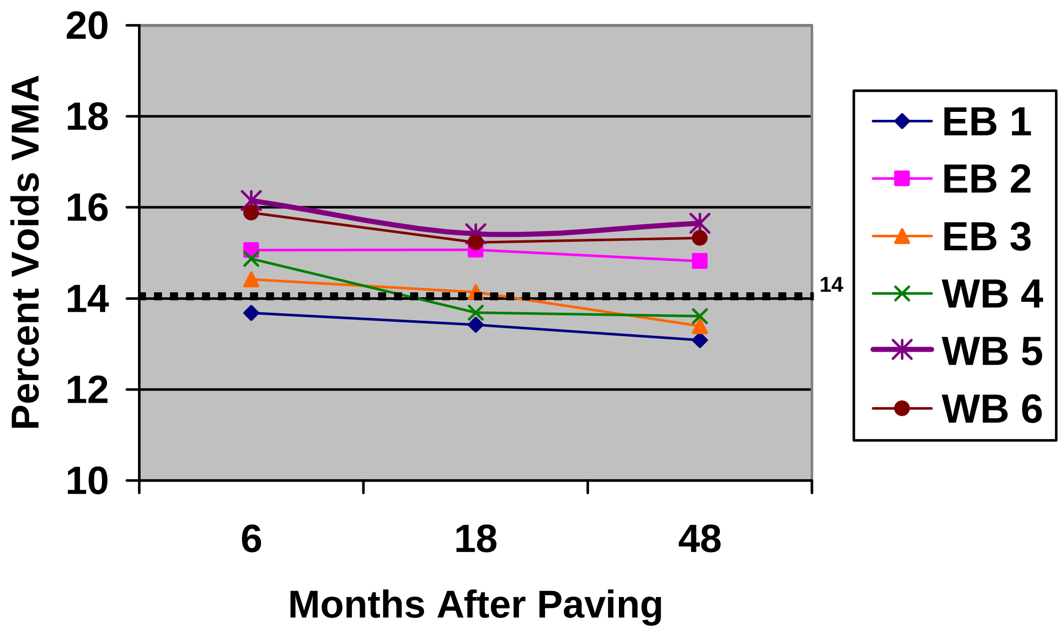

FIGURE 24 - VOIDS IN THE MINERAL AGGREGATE MEASURED AT 6, 18, AND 48 MONTHS (LOWER SPECIFICATION LIMIT INDICATED)

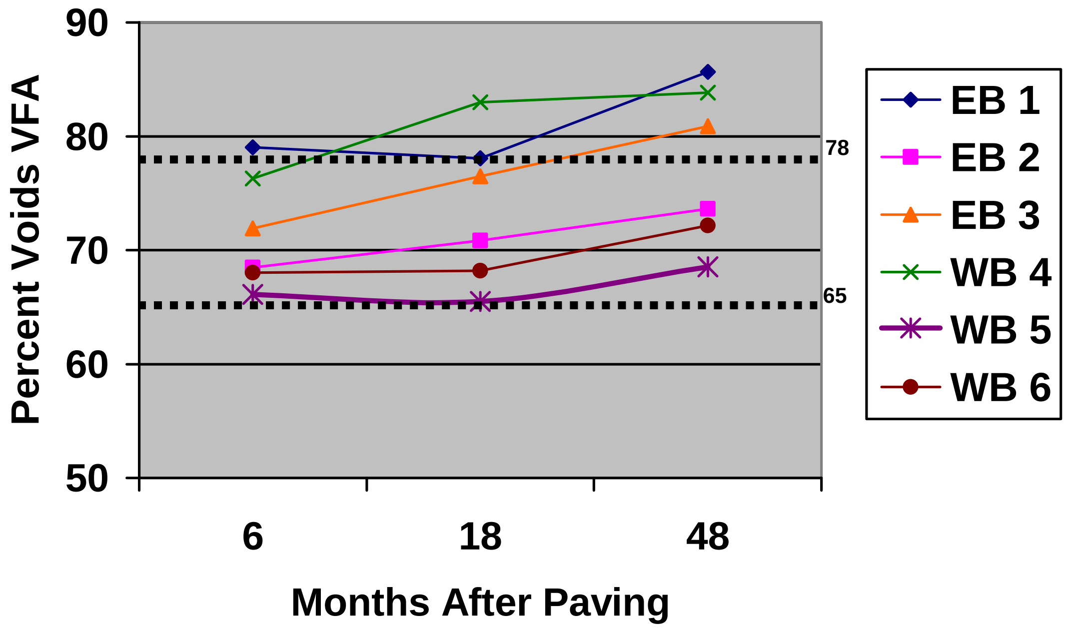

FIGURE 25 - VOIDS FILLED WITH ASPHALT MEASURED AT 6, 18, AND 48 MONTHS (SPECIFICATION LIMITS INDICATED)

Additional Cores and Testing Performed After 67 Months

At the end of five years and seven months, three short sections, (each 100 to 500 ft long) in the westbound direction, high-speed lane, contained considerably more cracking than the overall average for the sections. Each of the deteriorated areas begins at a transverse paving joint representing the start of a new day’s pavement placement. An example of a deteriorated portion of the high-speed lane of Section WB 5 can be seen in Image #6. Sealed cracks are visible, with almost no cracking apparent in the adjacent low-speed lane, or in the foreground of the high-speed lane. The adjacent low-speed lane was paved the same day as the distressed high-speed lane, approximately six hours later. A closeup view showing raveling and lack of fine aggregate at the surface can be seen in Image #7.

IMAGE #6 – SURFACE CONDITION IN SECTION WB 5, WESTBOUND MILEPOINT 29, (JUNE 2003)

IMAGE #7 - SECTION WB 5, CLOSE-UP OF DETERIORATED SECTION WITH CRACKING AND RAVELING

In an attempt to identify what went wrong in these areas, six-inch pavement cores were removed from two of them during June 2003, as an added task to this study. The cores were obtained from two of the distressed areas and two nearby undistressed areas in the same lane. These were all paved on different days. A total of 24 cores were removed, and tested in the ConnDOT laboratory.

The difference between the average or typical portion of WB 5 and the deteriorated area in WB 5 is illustrated in Images 8 and 9. These closeup images are of the cores after removal. The fine aggregate is visible at the surface in Image #8, but coarse aggregate is predominant in Image #9. Moisture damage, or stripping, was found at the bottom of the cores. The aggregate was not well-coated with asphalt, and the cores could be easily broken by hand. It should also be noted that, as previously indicated in Figures 18 and 23, the recycled sections (WB 5

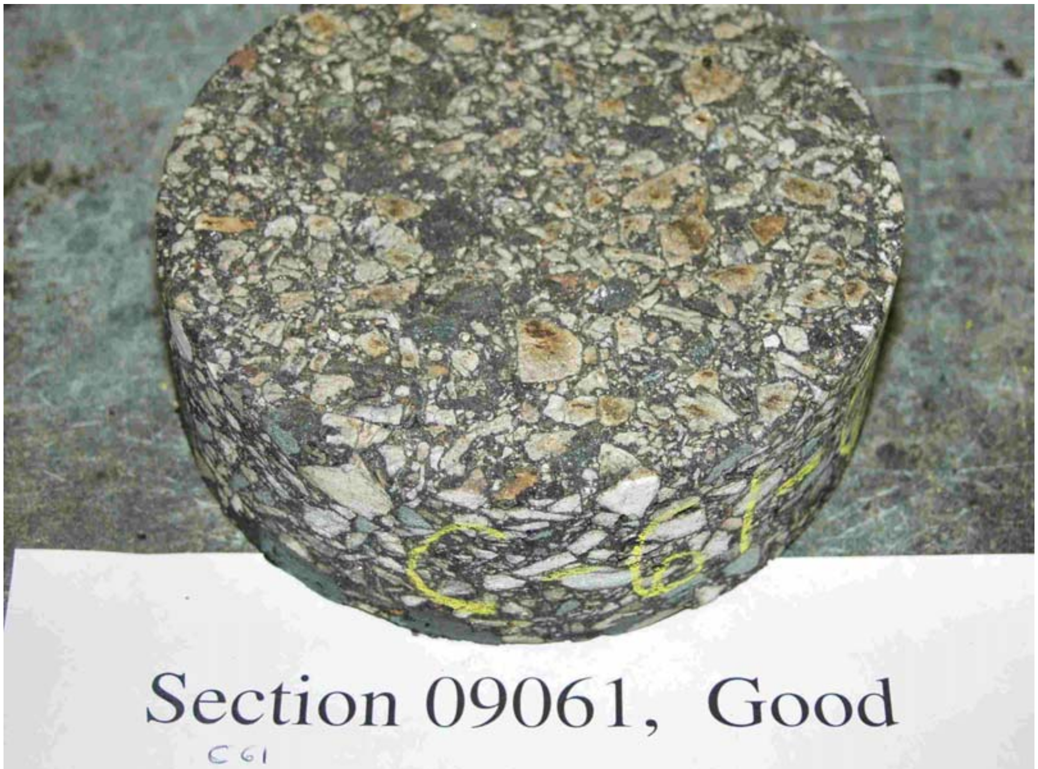

IMAGE #8 – SIX-INCH PAVEMENT CORE FROM TYPICAL PORTION OF SECTION WB 5.

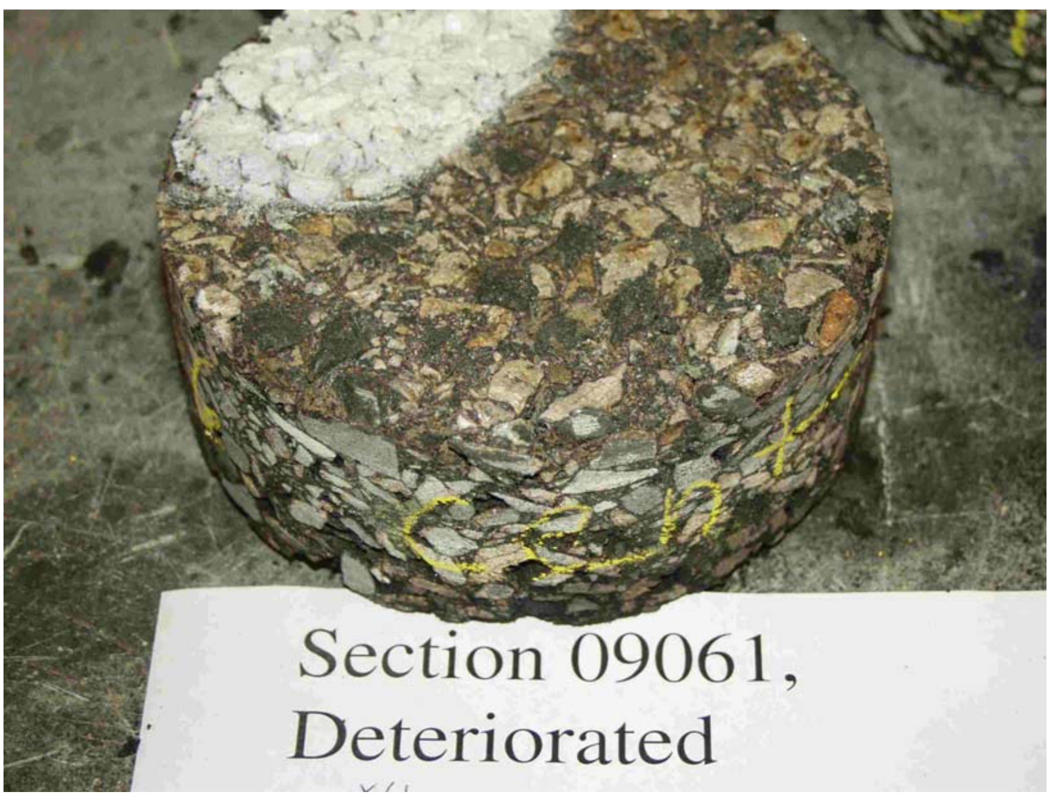

IMAGE #9 - SIX-INCH PAVEMENT CORE FROM DETERIORATED PORTION OF SECTION WB 5 and WB 6) had the lowest asphalt contents and highest air voids of the six LTPP sections.

In Table 26, the results are given for laboratory tests of the cores. Included are: gradation (% passing the various sieves #200 to ¾ in.), binder content (% binder), air voids (% voids), maximum theoretical gravity (Gmm), relative density, viscosity at 140 ^{0}F, penetration at 77^{0}F, and brittleness (BBR) for the control (average typical area) and the deteriorated areas.

Based upon the similarity in binder content, gradation, and Gmm between the control areas and the deteriorated areas within each section, it can be stated that the HMA materials are essentially the same. Some significant differences among the control areas versus the deteriorated areas are found when comparing the void content, relative density and binder age. If the HMA components are relatively consistent, but the voids (or their complement, the in-place density) varies substantially, as demonstrated in Table 26, it may be assumed that lack of compaction during placement and rolling would be the primary cause.

The lack of compaction premise may also be supported by the fact that the binder from the deteriorated areas is substantially harder or more aged, as indicated by lower penetration and higher viscosity. This may be due to an accelerated rate of oxidation that would occur at an area of lower density (higher voids). Another possible reason for the harder binder in the deteriorated areas may be inadequate mixing of RAP and virgin materials such that a portion of pavement with substantially greater than 20 percent RAP material could have been placed in the deteriorated areas.

TABLE 26
RESULTS OF LABORATORY TESTS ON CORE EXTRACTIONS FOR
FORENSIC STUDY OF DETERIORATED SECTIONS WITHIN WB 5 AND WB 6

Summary Data for Section WB 5

	CONTROL AREA	DETERIORATED AREA	Variation of Det. Area Relative to Control
200	4.0	3.4	-0.60
100	7.6	7.2	-0.40
50	13.4	13.7	0.30
30	18.7	19.2	0.50
16	24.3	25.1	0.80
8	33.6	34.2	0.60
4	49.4	48.5	-0.90
3/8	79.9	78.5	-1.40
1/2	96.1	96.9	0.80
3/4	100.0	100.0	0.00
% binder	5.1	4.4	-0.70
% Voids	5.91	9.93	4.02
Gmm	2.509	2.513	0.004
Relative Density	94.09	90.07	-4.02
Viscosity @ 140^{o}F	69,040	117,420	48,380
Penetration @ 77^{o}F	18	8	-10
BBR m	0.282	0.255	-.027
BBR Stf	268	428	160

Summary Data for Section WB 6

	CONTROL AREA	DETERIORATED AREA	Variation of Det. Area Relative to Control
200	4.1	4.2	0.1
100	7.7	7.7	0.0
50	14.3	13.4	-0.9
30	20.2	17.1	-3.1
16	25.8	21.8	-4.0
8	33.6	30.2	-3.4
4	46.0	45.8	-0.2
3/8	78.0	78.5	0.5
1/2	97.9	97.7	-0.2
3/4	100.0	100.0	0.0
% binder	4.0	4.5	0.5
% Voids	5.96	9.38	3.42
Gmm	2.514	2.520	0.006
Relative Density	94.04	90.62	-3.42
Viscosity @ 140^{o}F	7,852	103,570	95,718
Penetration @ 77^{o}F	39	15	-24
BBR m	0.371	0.270	-.101
BBR Stf	222	414	192

The fact that 20 percent RAP was used in a batch plant production is a concern unto itself because of the possibility that moisture content can be too high, and the HMA plant dry/wet mixing time too short. Unknown at the time (1997), but more thoroughly researched since, is that this amount of RAP combined with high moisture and potentially inadequate dry/wet mixing times could be a significant source of thermal segregation during placement of a HMA mixture.

In summary, both the physical (high voids) and the symptomatic effects (aged binder) on the pavements indicate that the primary trigger for these isolated premature distressed areas was inadequate compaction. The principle reason for the lack of compaction, whether it was caused by inadequate rolling, poor RAP mixing, or thermal segregation due to moisture and short dry/wet mixing times, is very difficult to isolate or determine five years after placement.

SUMMARY AND CONCLUSIONS

Connecticut’s first full-scale Superpave project was constructed on CT Route 2 between April and September 1997. Construction involved three phases: removal of the existing surface course of ConnDOT Class 114 surface mix; placement of a leveling course of ConnDOT Class 2; and, placement of the surface layer for the six sections. A construction report published in December 1997 [1] documents various aspects of the Superpave design, production, quality control testing, acceptance testing, and placement.

The section of Route 2 selected for the SUPERPAVE study begins in the town of Colchester and runs easterly for a distance of six miles through the town of Lebanon and into Bozrah. Six two-mile sections (both directions) were constructed during 1997: a control section containing a standard agency mix design (EB 1), a SUPERPAVE design using a PG asphalt with 98 percent reliability (EB 2), and a SUPERPAVE section with PG asphalt offering 50 percent reliability (EB 3). All were placed in the eastbound direction. Each of these three designs, but with 20 percent RAP, were also placed in the westbound direction, designated WB 1, WB 2, and WB 3 respectively.

“Post-construction” monitoring and testing began at the completion of the paving, in late September 1997. Six-inch diameter pavement cores were obtained by ConnDOT at multiple times, after project completion, of 0, 6, 12, 18, 24, and 48 months. The ConnDOT Research Division staff performed friction testing of all six sections semi-annually, in the spring and fall, from September 1997 through fall 2002. ConnDOT’s Data Services Section staff from the Division of Research ran ConnDOT’s Automatic Road Analyzer (ARAN) vehicle over the sections in September 1997 to collect roughness, cross slope, grade, curvature, transverse profile (rut depths), and GPS data. These measurements were repeated on an annual basis from 1998 through 2002, as part of normal photolog inventories of Connecticut’s state maintained highways. Stantec (the FHWA North Atlantic Region Long Term Pavement Performance (LTPP) contractor) performed measurements of pavement deflection, transverse and longitudinal profile, and distress, beginning in September 1997. They will continue these measurements periodically through 2007 for the LTPP program. These surveys were performed within specially designated 1000-ft LTPP monitoring sections. In addition to the distress surveys performed by Stantec for the LTPP sections, ConnDOT research staff performed manual surveys on three occasions, and used the Wisecrax software from the ARAN survey on another occasion, for the full 24 lane-miles of the study sections.

An acceptable friction level was measured throughout the five years of study in the slow-speed lane. The ASTM smooth-tire friction numbers indicate that during the five years there should have been no potential for hydroplaning in any of the sections. Overall, the reduction in friction number over the five-year period has been minimal; a total average drop of only 6-9 FN_{40}. There is a relatively insignificant difference between any of the sections for the ASTM ribbed-tire friction numbers. The two Superpave recycled sections (WB 5 and WB 6) showed a significantly higher surface texture compared to the four other sections. The smooth tire numbers are higher for the two recycled sections with the greater texture.

It appears that the condition of the underlying pavement is having a much more significant influence on the performance of the surface than is the mix design. All of the sections were milled, prior to paving, but only the top layer of two inches placed in 1986 was removed. Therefore, whatever cracks existed in 1986 in the original pavement (placed in 1970), may be reflecting through the surface again in 2003.

Four types of cracking, transverse construction-joint cracking, plain transverse cracking (shrinkage), longitudinal cracking (shrinkage or load-related), and longitudinal construction joints were monitored and summarized separately through the five years of study. By the end in March 2003, it appears that there was very little difference between any of the sections regarding quantity of transverse construction joint cracks that had formed. More than half of the joints had developed cracks. Construction joint quality is highly dependent upon the paving crew and construction practices, and therefore, conclusions about performance of mixes might not be justified on the basis of transverse construction joint condition.

Transverse cracks not associated with construction joints were also observed for the study. By the end of the study in March 2003, in the low-speed lane, sections EB 1, EB 2, EB 3, and WB 4 had similar amounts of transverse cracking. Section WB 5 (Superpave with 20 percent RAP, 98 percent reliability design) had significantly less cracking, and WB 6 (Superpave with 20 percent RAP, 50 percent reliability design) had significantly more cracking than the other four sections. In the high-speed lane, the results were different. Section EB 1 and WB 4 had significantly less cracking than sections EB 2, EB 3, WB 5 and WB 6. In both lanes, WB 6 contained more than double the amount of transverse cracking of any other section. Although difficult to explain, this is likely the result of reflections of existing conditions, where the underlying conditions were much worse in the last mile of section WB 6 than anywhere else in the project.

From analysis of longitudinal construction-joint cracking it could be concluded that the Superpave Virgin 98% reliability (Section EB 2) pavement provided the longest lived cold joints thus far. Fifty percent of the middle joint and only three percent of the right joint were cracked after five years (six winters). The section with the most longitudinal joint cracking was WB 4, recycled pavement conventional class 1 mix (non-Superpave).

Most of the longitudinal cracks (i.e., non-joint associated) observed within the travel lanes were outside of the wheel paths. They appear to be somewhat random and were possibly reflection cracks from underlying block cracking that existed in the 1970 pavement. The recycled pavements contained more longitudinal cracking than the virgin mixes, regardless of whether they are Superpave or conventional mixes. However, the quantity of longitudinal cracking observed after five years was relatively small for all of the six study sections. No section contained longitudinal cracking for more than 12 percent of the section length.

Rutting was not yet significant in any of the study sections. The maximum rut depths measured after five years were 1/8 inch.

The variation in roughness between the left and right wheel paths appears to be as significant as any differences between the various sections. However, when data for the two wheel paths is averaged together, it was obvious that the two roughest sections at approximately 73.5 and 71.9 in/mi. were EB 1 and WB 4, respectively. These are the two conventional (non Superpave) mixes. The four Superpave mixes contained roughness values of between 66.5 and 68.5 in/mi.

Since only five years had passed with only 25 percent of the design ESALs accumulated in the worst case lane, it is expected that any distress development should not be the result of fatigue. Since nearly no load-related cracking had been observed, along with the absence of significant rutting, it appears that the pavements are not deteriorating at an unreasonable or unexpected rate. The only exception is for three short segments of the high-speed lane in the WB 5 and WB 6 sections, where short sections (100 to 500 feet long) of what may have been out-of-specification or inadequately-rolled material had been placed.

From weather data obtained from NOAA stations in Connecticut, it can be stated that the summers were somewhat warmer than normal, and the winters were exceptionally milder than normal for the period 1997 - 2002. Warmer than normal winters would not be conducive to the development of cold-temperature cracking, unless there were isolated days with extremely low temperatures. The weather data does not show this. In fact, the coldest temperature recorded in eastern Connecticut between June 1997 and December 2002 was +2 ^{0}F in January 2000. The PG64-22 asphalt is designated to resist cracking down to a minimum temperature of -7.6 ^{0}F.

One item discovered during various coring operations was evidence of moisture-induced asphalt stripping in section WB 6. Stripping was visually evident during core removal at months 6 and 12, but not during later core intervals. This suggests that the stripping damage may be isolated or sporadic in nature. Some asphalt stripping was also observed from cores taken at locations other than the 1000-ft LTPP sections, within sections WB 5 and WB 6, in early June 2003.

In conclusion, the pavements are performing as would be expected for overlays on a 27-year-old pavement structure. Thus far, there is no notable difference between the conventional ConnDOT class 1 overlays and the Superpave mixes. Most cracking that has occurred is assumed to be reflection of underlying conditions. However, the mixes containing 20 percent RAP appear to be performing at a somewhat reduced level, possibly due to higher air voids, increased permeability and water seepage in areas with a high water table. Testing indicates these sections also contain lower asphalt content.

IMPLEMENTATION OF FINDINGS

As a result of what was learned during design and construction of Route 2, a number of changes were made to the ConnDOT Superpave Special Provisions. ConnDOT continues the evolution toward implementation of Quality Assurance specifications for HMA, where the contractor is responsible for quality control.

Construction tolerances for gradation of the Superpave mixes used on Route 2 were considerably tighter than those allowed with conventional ConnDOT class 1 mixes. These were discussed in the construction report [1]. It was concluded that the contractor had no difficulty meeting the tighter gradation tolerances. As a result, the special provisions developed for Superpave placed in years 2000 and later continue to specify the stricter tolerances for material passing the various sieve sizes. In addition, for year 2000 and later, the tolerance for asphalt content has been reduced from +/- 1.5% to +/- 1.0%.

The control test strip used on Route 2 was 300 feet in length. This was Connecticut’s first experience with test strips for HMA. It was found that 300 feet was too short to be able to establish an effective rolling pattern that consistently produced desired pavement density. The test strip was changed to 600 ft in later special provisions.

Although 20 percent RAP was used on Route 2, the use of RAP in Superpave is now limited to a maximum of 10 percent. It must now be crushed after milling so that 100 percent passes the 0.75-inch sieve. The RAP must also be free from crack or joint sealant to the greatest extent possible.

The Route 2 Superpave, and later Superpave mixes placed since 1999, indicated that permeability is a concern with some of the mix designs. This has also been an issue in other states. Emphasis during 2003 is being placed on ways to reduce permeability, especially for mixes that will be placed on bridges.

The latest Superpave special provisions allow for the pavement designs to be within the previously defined “restricted zone” for gradations, as long as good experience can be demonstrated.

Full implementation of Superpave in Connecticut continues according to the schedule contained in Appendix A. After January 1, 2004, almost all paving on state-maintained roadways will be with Superpave specifications.

REFERENCES

1. Larsen, D.A. and Rodrigues, N., “Demonstration and Evaluation of SUPERPAVE Technologies: Construction report for Route 2,” December 1997, ConnDOT.
2. Dougan, C.E., “Demonstration and Evaluation of SUPERPAVE Technologies Project Proposal,” February 1996, ConnDOT.
3. Larsen, D.A., “Evaluation of Lightweight Non-Contact Profilers for Use in Quality Assurance Specifications on Pavement Smoothness,” December 1999, ConnDOT.

APPENDIX

Superpave Implementation Schedule

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION

memorandum

COM-09A REV. 2/91 Printed on Recycled or Recovered Paper

subject Superpave Implementation

date May 30, 2002

to Mr. Walter H. Coughlin
Mr. Louis R. Malerba

from Michael W. Lonergan
Construction Administrator
Bureau of Engineering and
Highway Operations

The implementation schedule for the Superpave Design system was last revised per a May 11, 2001 memorandum from the Construction Administrator. As part of that memorandum, the Department was to re-evaluate this schedule after one year.

As a result of the April 26, 2002 Joint HMA Task Force - Superpave Subcommittee meeting held at the Rocky Hill Laboratory, this task was undertaken with the following results:

a. All Limited Access Highways shall contain Superpave specifications and this includes VIP structural overlays. *(No change)*

b. All other roadways, where the total tonnage of HMA for the projects exceed 10,000 tons, including VIP structural overlay jobs, shall include Superpave specifications. *(No change)*

c. All other roadways containing between 5,000 to 10,000 tons shall require the Superpave specifications with the exclusion of the plant testing requirement by the Contractor.

d. All projects containing less than 5,000 tons will continue to specify the Marshall Design system until the Final Design Plan (FDP) date of January 1, 2004, after which these projects will require the Superpave specifications also with the exclusion of the plant testing requirement by the Contractor.

The Department is also moving toward further implementation of QA specifications and the exact pace of that implementation will be subject to review as our experience grows as well as the results of our current Research project.

If you have any questions, or need additional information, please contact Mr. Keith R. Lane, Director of Research and Materials. He may be reached at 860-258-0371.

Nelio J. Rodrigues/lr/gp/M_Superpave implementation_5-2-02

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DMT – Files

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