Evaluation of a Cold In-Place Recycled Rehabilitation Treatment

Final Report

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16. Abstract

In 1998, a state highway with fewer than 5,000 vehicles per day of traffic received a three-inch cold in-place recycled (CIR) base treatment to rehabilitate the pavement, which had developed extensive reflective cracking in a previous overlay. The CIR treatment was followed by a two-inch overlay to complete the preservation project. Adjacent pavement on this highway received a conventional HMA overlay, and served as the experimental control for this research. This report presents results of an evaluation of the CIR treatment after ten years of service under light traffic. It includes results of testing of drilled cores and manual distress surveys. It also includes a state-of-the-art SPSS™ statistical analysis of data collected by ConnDOT's Photolog personnel. These data include rut depths determined from full-width and partial-width transverse profiling equipment, international roughness index (IRI) values, and WiseCrax pavement distress values. The WiseCrax analysis shows that the CIR treatment was an effective preservation technique that mitigated reflective cracking, as a 65% reduction in pavement cracking was observed for the CIR versus the control pavement. The density of drilled cores taken from the CIR base ranged from 80% to 90% of the maximum theoretical density (MTD). Accordingly, rutting is a concern when using a CIR treatment. Overall, rut depths were 10% less severe for the CIR rehabilitated pavement than for the control pavement; however, where longitudinal joints were located in the wheel path, CIR treated pavement rut depths were 83% more severe than control pavement rut depths. Also, CIR pavement rut depths were 60% to 183% more severe on uphill grades >=4% than downhill grades >=4%. IRI values were comparable between the CIR pavement and the control. As a result of this research, ConnDOT has established a goal to select four new construction projects, one from each District on low volume roadways, to receive CIR treatment of the base. The Department's Pavement Management section will determine which pavements are most suitable for this application, following the 2005 guidelines developed by the pavement preservation work group.

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Cold In-Place Recycling, CIR, CIP, Rut Depth, Full-Width Transverse Profiles, Partial-Width Transverse Profiles, WiseCrax, Reflective Cracking, Reflective Crack Mitigation, Pavement Treatment, Pavement LCC Analysis	No restrictions. Hard copy of this document is available through the National Technical Information Service, Springfield, VA 22161. The report is available on-line from the National Transportation Library at http://ntl.bts.gov

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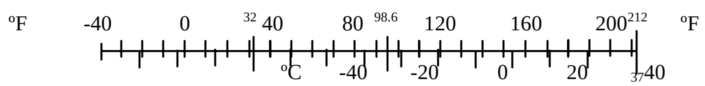
METRIC CONVERSION FACTORS

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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EXECUTIVE SUMMARY

Results of this study demonstrated that the use of cold-in-place recycling (CIR) of 3 or 4 inches of existing pavement on S.R. 695 immediately followed by a 2-inch HMA Class 1 overlay was a successful rehabilitation choice. The estimated lifespan for this alternative was 7-12 years. Now, 11 years later, the pavement is still performing and has not reached its terminal serviceability, although Connecticut Department of Transportation (ConnDOT) Pavement Management (PM) personnel have indicated that S.R. 695 is slated for rehabilitation for this or next year. At this time, PM personnel are suggesting paving over the existing pavement with a 1-inch leveling course, followed by a 2-inch finish course. Their design is still preliminary however.

Cores were obtained and tested to evaluate material characteristics. The density of the CIR material ranged from a low of 80.8% to a high of 89.2% of the maximum theoretical density (MTD). These results validate testing performed 1-year after paving operations were completed. Densities ranged from a low of 83.1% to a high of 89.9% back in October 1999, so it appears the density did not change much over the years. In February 2000, Division of Materials Testing personnel recommended that "with the combination of a fine gradation, high liquid content and relatively low density," the pavement should be monitored for evidence of rutting.

This research was conducted subsequent to their recommendation. The most severe rutting was measured in sections with positive grades (uphill). There was a strong correspondence between the simple line charts for rut depth versus mileage and grade versus mileage. Sections of positive (uphill) grade tended to have greater rut depths, especially for the steeper sections (4%). This may be a result of heavy trucks losing speed as they climb these hills, and also their increased traction forces. Overall, combining both wheel paths and both directions, the average rut depth on positive grades (uphill) was 0.413 inches, and the average rut depth on negative grades (downhill) was 0.225 inches. Therefore, rut depths were 84% deeper for positive grades than for negative grades for pavement over CIR base. This phenomenon may have been exacerbated by a general tendency for longitudinal construction joints to be located in the right wheel path more frequently on uphill grades, but rut depths were also more severe in the left wheel path, where no longitudinal joints were located.

Thus, longitudinal joints located in the wheel path do not completely explain the problem.

The rideability of the pavement with CIR base was comparable to that of the control pavement. The average of the right and left wheel path International Roughness Indexes (IRIs) for the pavement over CIR base was 96 inches/mile in the eastbound direction and 85 inches/mile in the westbound direction after 10-years of service (measured in 2008). After approximately the same number of years (11-years) of service, the control pavement's IRIs were 99 inches/mile in the eastbound direction and 85 inches/mile in the westbound direction. Thus, there were no appreciable differences in rideability between the pavement over CIR base and the control pavement.

A WiseCrax pavement distress survey was performed with data collected by ConnDOT Photolog personnel. The system measured total crack lengths for every 5 or 10 meter segment of pavement. Graphs of total crack lengths per 5 or 10 meter segment versus mileage were synchronized and plotted for the CIR and control pavements after 10 years of service. These demonstrated that reflective cracking was mitigated by the CIR treatment, as total crack lengths were less severe for the pavement over CIR base than they were for the control pavement. This was quantified, as 2,290 and 2,068 ft of cracks per lane-mile were observed in the eastbound and westbound directions, respectively, of the control pavement; while only 499 and 1,037 ft of cracks per lane-mile were observed in the eastbound and westbound directions, respectively, of the CIR pavement. Therefore, a 65% reduction in pavement cracking was observed for the CIR pavement.

Finally, a life-cycle cost (LCC) pavement analysis was performed. Results of the LCC suggest 37% cost savings for CIR treated pavements versus traditionally treated pavements over a 48-year analysis period.

INTRODUCTION

Background

During the mid-1990's, engineers from ConnDOT's Pavement Management (PM) section investigated the pavement design requirements for a resurfacing and safety improvements project on S.R. 695 (Governor John Davis Lodge Turnpike) in Killingly, CT (see Figure 1) from Ross Road easterly to U.S. Route 6 (Milepost 0.96 to Milepost 4.49). A memorandum on the subject is presented in Appendix A.

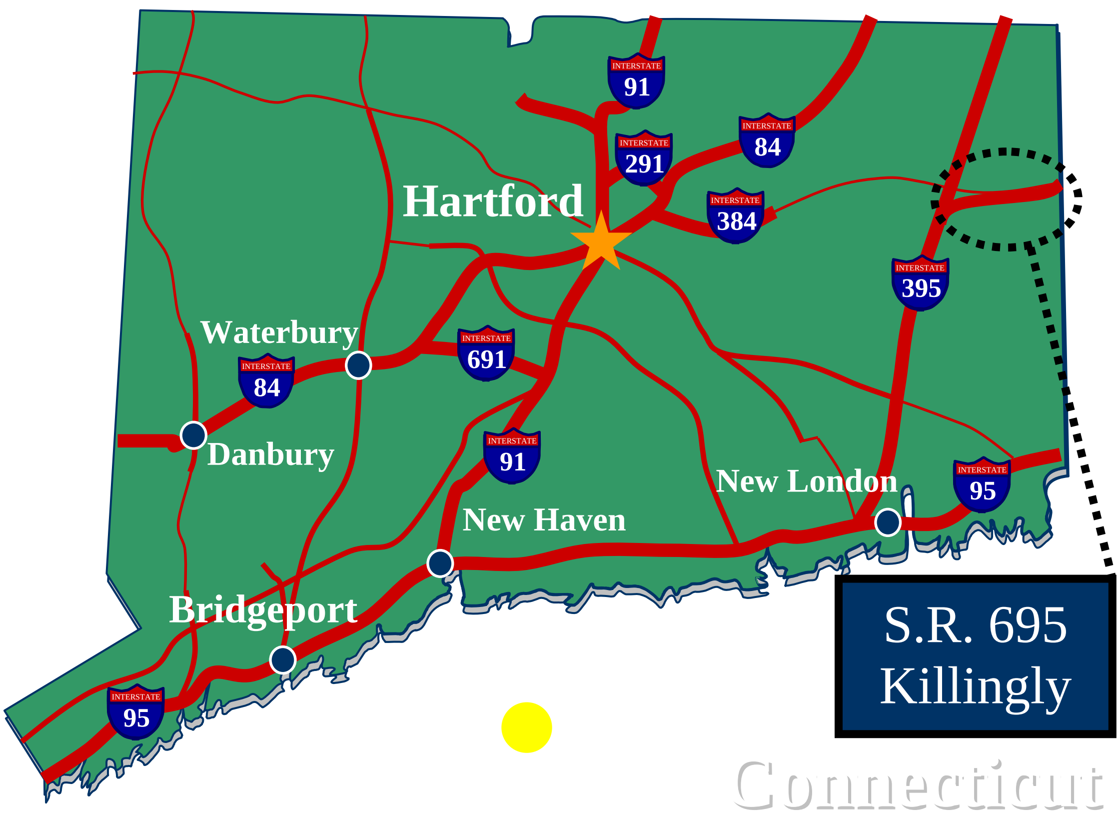


FIGURE 1 Map illustrating location of S.R. 695 in Killingly, Connecticut.

The existing pavement was composed of 1.5-inches of Class 1 HMA overlay (1987) on 1-inch Class 138 HMA surface treatment (1967) on the original 1958 flexible pavement structure, which includes a macadam layer on the bottom. A full-depth core taken prior to construction is presented in Figure 2.

FIGURE 2 Full-depth core 10-inches deep drilled prior to cold in-place recycling construction.

The existing pavement was distressed in the form of block cracking. These cracks reflected up through the underlying pavement.

The PM section reviewed 1986 Photolog images and observed severe block/alligator cracking (see Figure 3). The pavement below the 1987 overlay was very brittle. It appeared in these historical Photolog images that the initial cracking reflected through the 1987 overlay before 1990. Sometime between 1990 and 1991, State Maintenance forces sealed all of the cracks to preserve the pavement structure (see Figure 4 of 1991 photo taken at same location as Figure 3 photo).

In 1994, the block cracks reached a moderate level of severity. Comparisons of Photolog images indicated that the cracking had increased by approximately 30% since it was sealed.

The PM section recommended cold-in-place recycling (CIR) of 4-inches of the existing pavement followed by a 2-inch Class 1 HMA overlay. This recommendation was made considering the following reasons:

1. Based on the past overlay performance, standard overlaying of the existing pavement (2-inch Class 1 HMA surface lift on top of a 1-inch Class 2 HMA

leveling lift) would only provide a minimal service life before requiring additional maintenance,

2. It was believed that CIR would rejuvenate the existing brittle pavement as a means of extending the service life,
3. The Average Daily Traffic (ADT) in 1993 was 2,515 vehicles, which was believed to make this site an excellent candidate for CIR, and
4. It was anticipated that this combination of CIR/overlay would provide the longest service life for the cost.

FIGURE 3 1986 Photolog image of S.R. 695 EB at 4.41 miles showing block cracking.

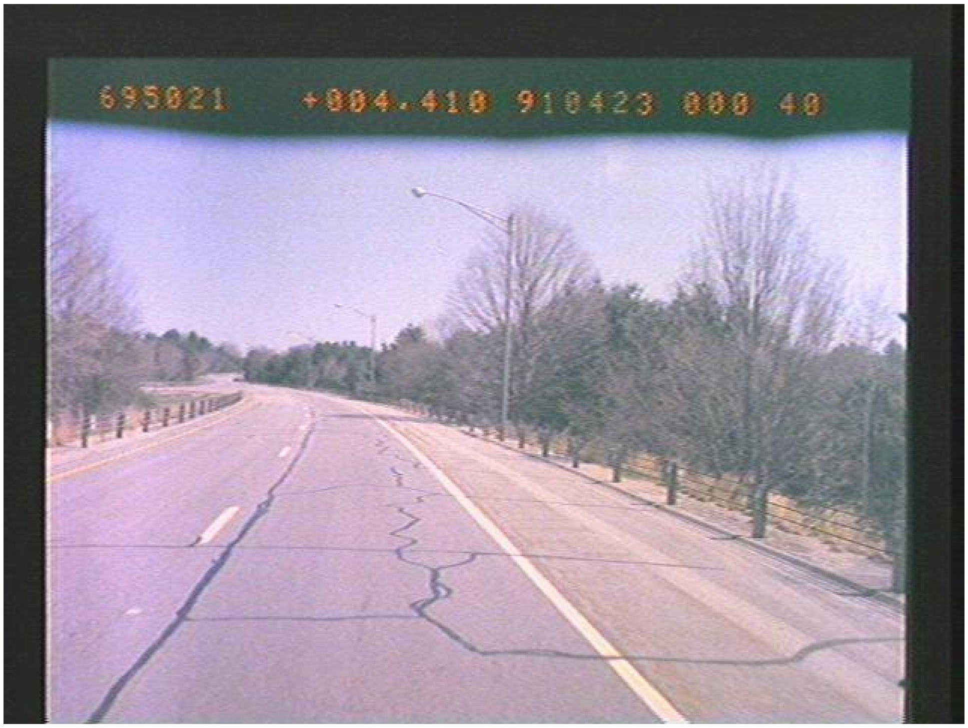


FIGURE 4 (a) 1991 Photolog image of S.R. 695 EB at 4.41 miles showing reflective cracking in only 4 years following construction in 1987.

The PM recommendation was taken and CIR resurfacing was performed in 1998 as part of Project 68-184. The purpose of Project 68-184 was to improve safety and extend the service life of this section of S.R. 695. S.R. 695 was classified as a Rural Principal Arterial (Freeway) in the Preliminary Design Statement. The Preliminary Design Statement includes design elements, such as maximum grade (4%) and minimum radius (2,291 feet) that were proposed for the project. It also includes the Preliminary Cost Estimate. Note: the Cold-In-Place Recycling estimate was $3.00 per square yard (S.Y) at an estimated quantity of 184,800 S.Y. for a total estimated cost of $554,400. For more details, see the Preliminary Design Statement in Appendix B. See Appendix C for the Location Plan.

Cardi Corporation, Inc. (Cardi) in Warwick, Rhode Island was the contractor for the project, and Gorman Brothers, Inc. (Gorman) from Albany, New York was the subcontractor that did the cold in-place recycling.

Gorman used HFMS-2 emulsion. They performed a lab mix design by crushing 12 cores representing 200,000 s.y. of pavement. While their lab tests indicated 1.5% was the best application rate, they started at 2.0% "to assure that mixing, paving and compaction were acceptable." They indicated that it was better to start at a safe percentage of emulsion and cut back if the recycled mix paved and compacted well.

Regarding moisture content, Gorman indicated that they "always do their mix design at 4.0% total fluids (emulsion plus water)." They estimated that the in-place pavement contained 1% moisture and added "enough water plus the emulsion to get at least 4% total fluids."

Gorman indicated that the air voids in the cold in-place mix would be higher than what is normally seen in HMA mixes because the RAP itself contains 5 to 8% air voids. They estimated that the cold in-place mix would contain 8 to 15% air voids. More details are presented in a copy of a July 27, 1998 letter from Gorman to Cardi in Appendix D.

As construction proceeded, the design was revised from 4-inches of CIR to 3-inches due to the lack of bituminous material in the shoulder pavement, where the contractor began to reach a Macadam layer during recycling. Note: it is desirable to have a uniform layer across the entire travel way. A 2-inch ConnDOT Class 1 surface layer was placed on top of the CIR by Cardi and pavement construction was completed by November 1998. Figure 5 presents a photo of the CIR base prior to being overlaid with HMA.

FIGURE 5 1998 Photolog image of S.R. 695 during construction. Note: this is the CIR base pavement prior to being overlaid with hot-mix asphalt.

The section of S.R. 695 from Ross Road easterly to U.S. Route 6 is referred to as the CIR pavement in this report. The section west of Ross Road is referred to as the control pavement. A map depicting these two sections of S.R. 695 are presented in Figure 6.

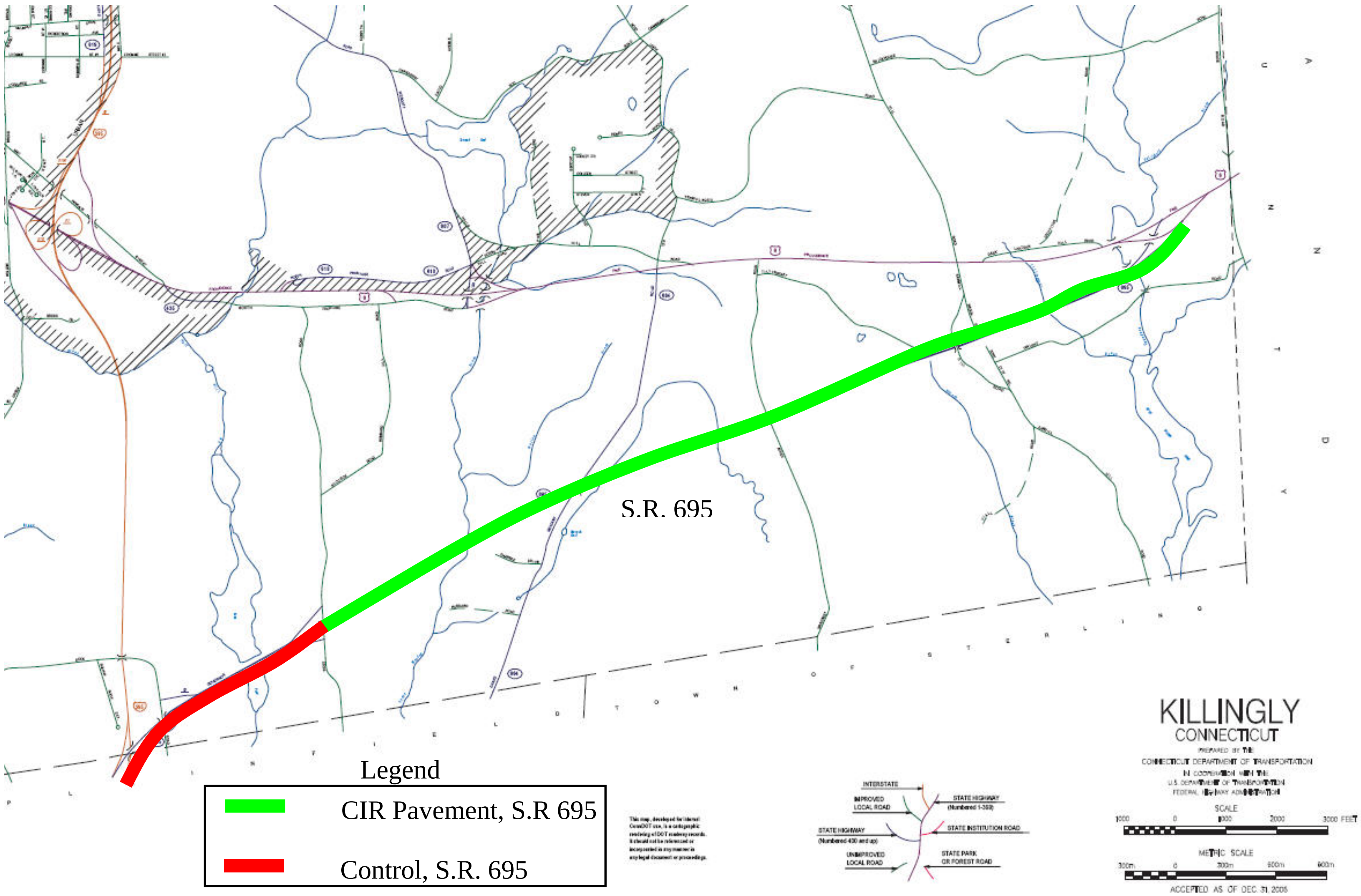


FIGURE 6 Map of S.R. 695 showing locations of CIR and Control pavements.

Core samples were obtained by Division of Materials Testing (DMT) personnel from this section of pavement approximately 1-year after paving operations were completed. They tested the cores in accordance with AASHTO T 166, "Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens." In a memorandum presenting the results of their findings, DMT personnel indicated "the density of the material has always been and is still of concern to us, ranging from a low of 83.1% to a high of 89.9%. With the combination of a fine gradation, high liquid content and relatively low density, we would recommend that this road be monitored in the future for evidence of rutting." A copy of this memorandum is presented in Appendix E. Note: these densities correspond to approximately 10 to 17% air voids, which is close to the range estimated by Gorman Brothers, Inc. prior to construction (8 to 15%). Therefore, it may be stated that high air voids are inherent in CIR, and the low densities measured by DMT personnel were not caused by poor construction methods.

Literature Review

The Maine Department of Transportation defines CIR as "a process in which a portion of an existing bituminous pavement is pulverized or milled, and then the reclaimed material is mixed with new binder and, when needed, virgin aggregates" ***(1).*** Emulsified asphalt is usually used as the new binder (it was used for this study on S.R. 695), although more recently, expanded (foamed) asphalts are being used to bind CIR mixes.

Foamed asphalts are produced by pumping hot asphalt cement through an expansion chamber on the CIR unit, and then injecting cold water at about 1% the volume of asphalt. The water quickly vaporizes, which creates thousands of tiny bubbles in the asphalt. This has an expansive effect, which appears to foam the asphalt. When foamed asphalt is used as the binder in CIR, the resultant material is referred to as cold in-place recycled expanded asphalt mix (CIREAM) ***(1).*** The advantage to using CIREAM instead of conventional emulsified CIR is that it cures faster. An HMA overlay can be placed on top of a CIREAM base following a 2-day curing period, whereas conventional CIR requires a minimum of 14 days to cure.

During the past twenty years, several states have used CIR as a pavement rehabilitation strategy. These states include but are not limited to Arizona, Iowa, Montana, Nevada, and Pennsylvania.

The Arizona Department of Transportation has used CIR in conjunction with both HMA overlays and with double applications of seal treatments. Mallela et al. (2006) ***(2)*** recently conducted a study to evaluate and document the performance of selected CIR projects in Arizona. They compiled and summarized details and data from 17 CIR projects. They indicated that "the overall performance of the CIR projects was found to be good, with most projects showing low to moderate levels of distress and roughness after many years".***(2)***

The Nevada Department of Transportation (NDOT) has used CIR and full-depth reclamation (FDR) for more than 20 years. Bermanian et al. (2006) recently had a peer reviewed state of the practice paper published by the Transportation Research Board (TRB) on CIR and FDR ***(3)***. They concluded that these treatments are high quality cost-effective rehabilitation strategies. They indicated that NDOT saved over 600 million dollars during a 20-year period by using these treatments compared with complete reconstruction costs. They recommended that other agencies use CIR and FDR rehabilitation strategies on lower volume roadways, and then on higher volume roadways as the agency gains experience.

In a 2008 TRB publication, Loria et al. ***(4)*** reported on an evaluation of various reflective cracking mitigation techniques, which included CIR treatments in Nevada. They found, in general, that the performance of the treatment was largely dependent upon preexisting pavement conditions. Nevertheless, they did evaluate and rank three different CIR treatments: CIR-A (CIR 2″ and overlay 2.5″), CIR-B (CIR 3.0″ and overlay 3.0″), and CIR-C (CIR 2″ and overlay 2″). They found that treatments CIR-A and CIR-B stopped reflective cracking for 3 years and retarded them for 5 years for cases where alligator cracking was not observed in the existing pavement. Conversely, they indicated that treatment CIR-C was ineffective in resisting reflective cracking.

Jungyong et al. (2007) submitted a TRB Paper (07-1259) that documented an effort to collect CIR performance data along with Falling Weight Deflectometer (FWD) data to develop performance models. They selected a total 26 test sections to evaluate from an inventory of CIR roads in Iowa. They concluded that these pavements performed very well and predicted that they would last up to 25 years before being rehabilitated. They predicted that CIR roads with a good subgrade will last up to 35 years ***(5)***.

The Pennsylvania Department of Transportation (PennDOT) uses CIR as a treatment typically on rehabilitation projects of roadways with average daily traffic (ADT) levels of 8,000 or less but has used it on projects with up to 13,000 ADT. The TRB published a paper by Morian et al. (2004) regarding PennDOT's experience with CIR as a reflective crack control technique. They evaluated the performance of several CIR pavement sections in Pennsylvania and concluded that CIR provided two and three times the resistance against reflective cracking as conventionally resurfaced control sections ***(6).***

The Ministry of Transportation Ontario (MTO) has used CIR on more than 50 contracts since the 1980's, and recently placed CIREAM as a base overlayed with HMA. Lane and Kazmierowski ***(7)*** indicated that the CIREAM base provided a "fairly smooth, hard, uniform surface suitable for temporary traffic and provided an excellent platform for HMA paving operations." These paving operations consisted of a 50-mm HMA overlay. One year after construction, they found "no discernible distortion, rutting, or cracking."

Research is in progress in New York State to establish an expected service life for CIR projects and to recommend improvements to New York State Department of Transportation standards for CIR construction. Their goal is to expand the use of CIR, thereby decreasing the usage of limited natural resources and associated consumption of energy. The projected end date of the research is June 2009 ***(8)***.

Objectives

Evaluate and document the performance, consistency and durability of the S.R. 695 section of CIR pavement.

ANALYSIS

The 2007 ConnDOT Traffic Log ***(9)*** reported that the 2007 ADT for S.R. 695 ranged between 4,000 and 4,700 for its different segment lengths. The CIR segment 2007 ADT was 4,000. This is a significant increase from 1993, when the ADT was 2,515, but compared to the interstate highways, which have ADTs of over 100,000, it is still relatively light traffic. Table 1 presents ConnDOT Traffic Log ADTs from 1997 to 2007 for all of the segments of S.R. 695.

TABLE 1 S.R. 695 ADTs from 1997 to 2007

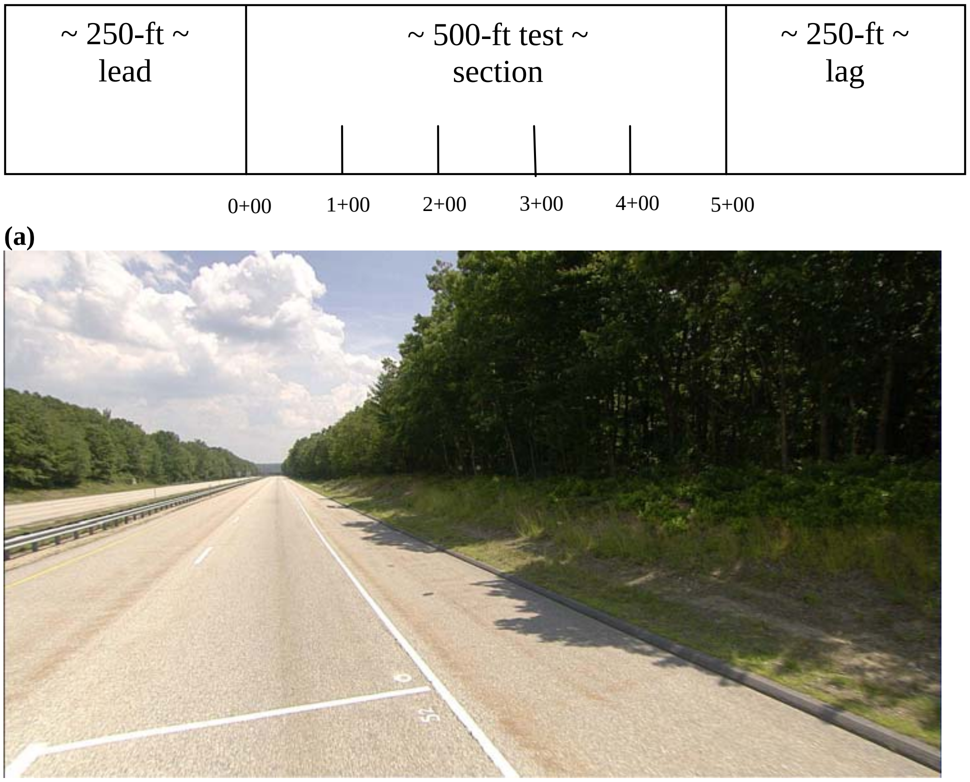
From Cum. Miles	To Cum. Miles	97' ADT	98' ADT	99' ADT	00' ADT	01' ADT	02' ADT	03' ADT	04' ADT	05' ADT	06' ADT	07' ADT
.00	.11	3400	3500	4300	4400	3400	3400	3400	3400	4300	4200	4300
.11	.44	---	---	4300	4400	3400	3400	3400	3400	4300	4200	4300
.11	.50	3400	3500	---	---	---	---	---	---	---	---	---
.44	.50	---	---	4500	4600	3600	3600	3600	3600	4500	4500	4500
.50	.73	3800	3900	4700	4800	3800	3800	3800	3800	4700	4700	4700
.73	4.49	3400	3500	4300	4400	3300	3300	3300	3300	4000	4000	4000

Pavement Management personnel calculated the Equivalent Single Axle Loads (ESALs) to be 904,689, using the maximum 2007 ADT of 4,700 for a "Rural other Principal Arterial" classification. Using this classification, the sum of the estimated percentages of trucks and other heavy vehicles was 5.2%. Because the Design ESALs are greater than 0.3 million and less than 3.0 million, the Superpave Design Level for S.R. 695 would be Level 2 if it were paved today.

Two 500-ft test sections on S.R. 695, where CIR base was used, were randomly selected. This was accomplished by entering the starting and ending mileages of the CIR base pavement on S.R. 695 into the RANDBETWEEN() function in Microsoft Excel, which output starting mileages for the test sections. Next, these starting mileage points were visually verified as to whether they were acceptable test sections using ConnDOT's DigitalHIWAY application. Sections including bridges or culverts were discarded and replaced with another random selection. The process was repeated until two acceptable sections were located. A third test section on S.R. 695, where conventional base pavement was used, was also randomly selected as a control.

The CIR test sections were labeled S1 and S2, and the control test section was labeled C. All three test sections were located in the low-speed lane on S.R. 695 EB.

Site S1 was located at approximately 1.80 miles on a 4% uphill grade, Site S2 at approximately 3.10 miles (see Figure 4(b)) on a 2% downhill grade, and Site C at approximately 0.85 miles on a 2% to 3% uphill grade. Once the test section locations were determined, they were identified with marking tape and spray paint. Each test section also included a 250-ft lead and lag section before and after the 500-ft test section, for a total length of 1000 feet. All of the cores were obtained from these lead and lag sections. A typical layout for these test sections is shown below in Figure 7(a), and typical marking at the start of the section is shown in a photo in Figure 7(b).



(a)

(b)

FIGURE 7 (a) Typical test section layout. (b) Test section S2 at its start @ 0-ft, Photologged in 2008.

Next, manual distress surveys were performed on May 14, 2008. Research personnel documented various distress types, including transverse cracking, longitudinal cracking, fatigue cracking and rutting. These are presented in Appendix F.

Site S1 Distress Survey

Low to moderate levels of fatigue cracking were observed in the right wheel path throughout the length of Site S1 (see Figure 8). The right wheel path was more susceptible to fatigue cracking than the left wheel path because of the existence of a longitudinal joint in the right wheel path. The left wheel path was virtually free of fatigue cracking, except for a short stretch approximately 50-ft long.

FIGURE 8 Site S1 @ 400 feet from start of test section. Evidence of fatigue cracking in the right wheel path is visible.

The average rut depth, measured with the rut bars fully extended (3.6 meter transverse profile) on the Photolog van, was also significantly greater in the right wheel path (0.775 inches) than the left wheel path (0.255 inches) for Site S1. Perhaps this owes to fatigue cracking observed in the right wheel path. Water may have infiltrated the base layer, exacerbating freeze-thaw conditions, causing ruts to develop. Areas of rutting in the right wheel path were also physically measured with a straightedge and ruler, and were observed to be as much as 1-inch deep in some instances. Figures 6-11 present full-width transverse profiles for Site S1 at 100-ft increments, from beginning to end.

TABLE 2 Descriptive Statistics for Site S1, Rut Depths (inches) Measured with Rut Bars Extended for a 3.6 Meter (approximately 12-ft) Transverse Profile

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	32	.118	.354	.255	.0678
Right Wheel Path Rut Depth (inches)	32	.512	1.378	.775	.1834

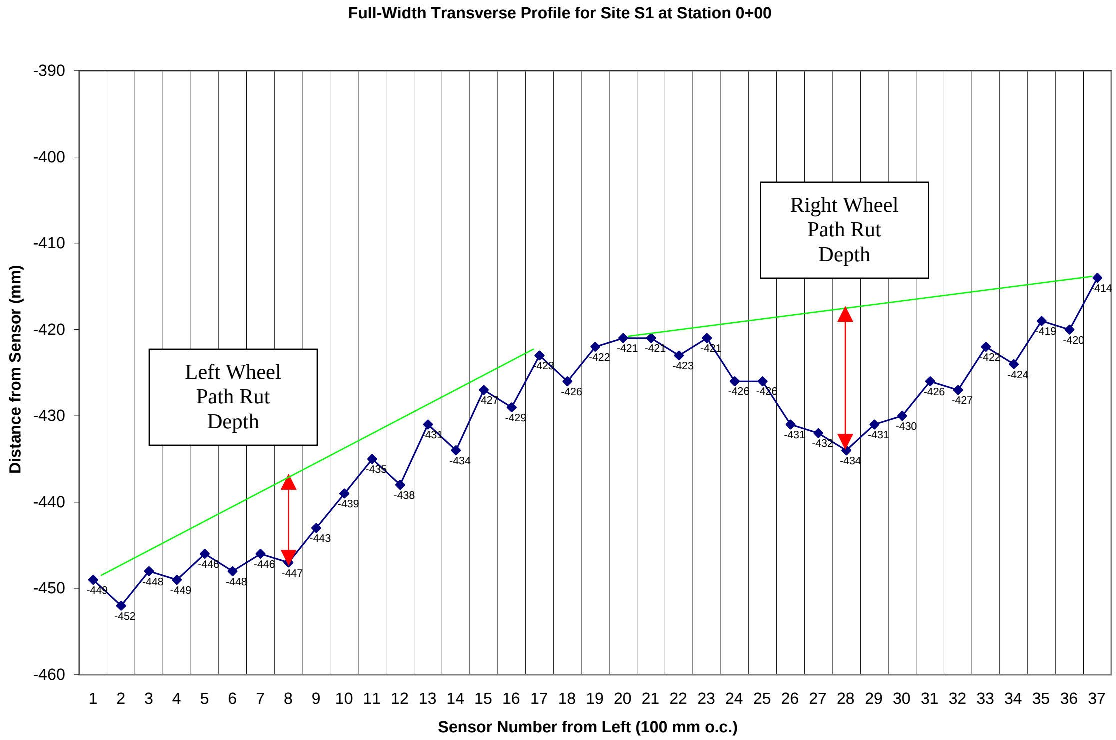


FIGURE 9 Full-width transverse profile for Site S1 at Station 0+00 of 500-ft test section. The green lines represent imaginary string lines extended. The red arrows represent the wheel path rut depths.

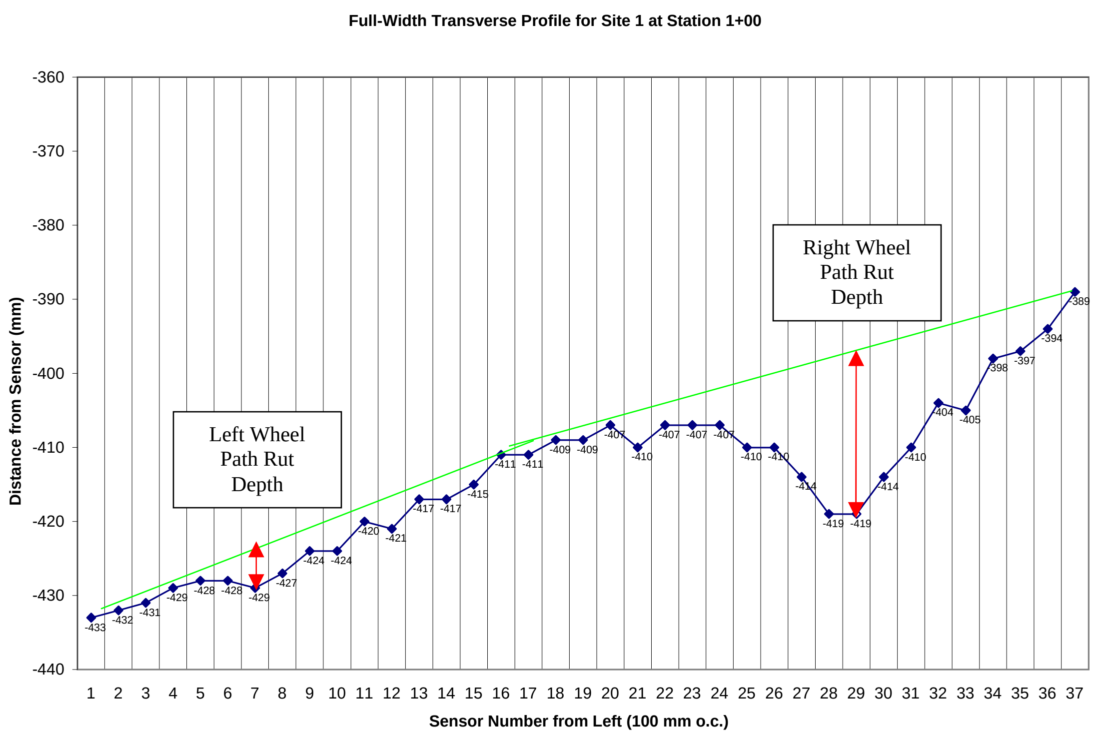


FIGURE 10 Full-width transverse profile for Site S1 at Station 1+00 of 500-ft test section. The green lines represent imaginary string lines extended. The red arrows represent the right wheel path rut depths.

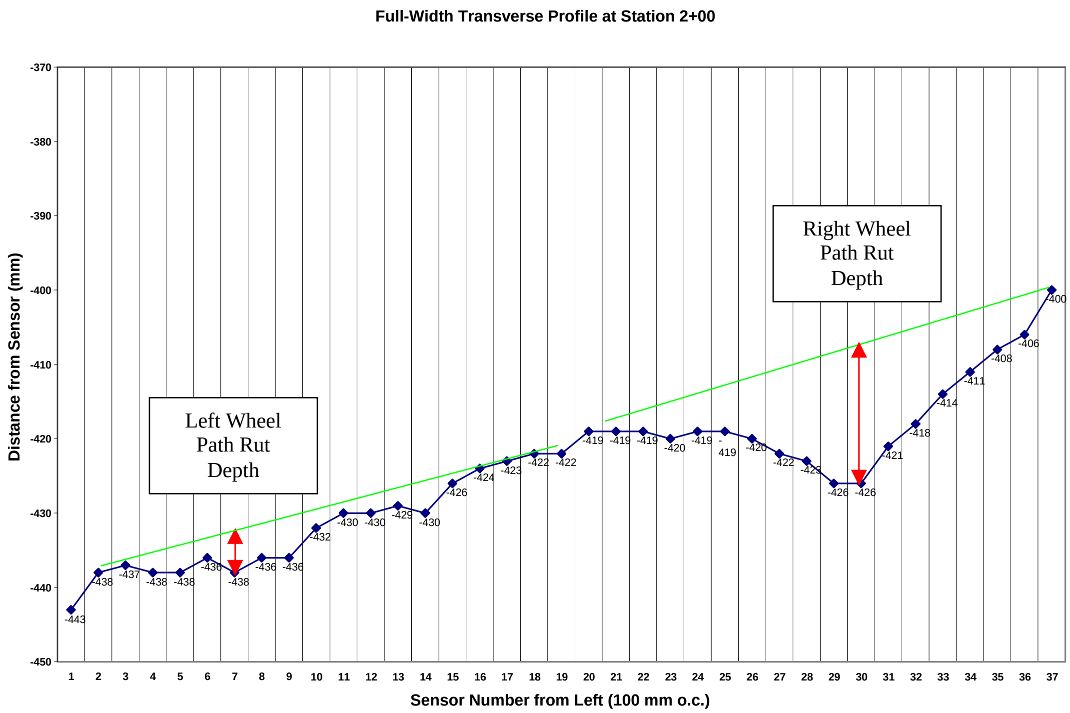


FIGURE 11 Full-width transverse profile for Site S1 at Station 2+00 of 500-ft test section. The green lines represent imaginary string lines extended. The red arrows represent the wheel path rut depths.

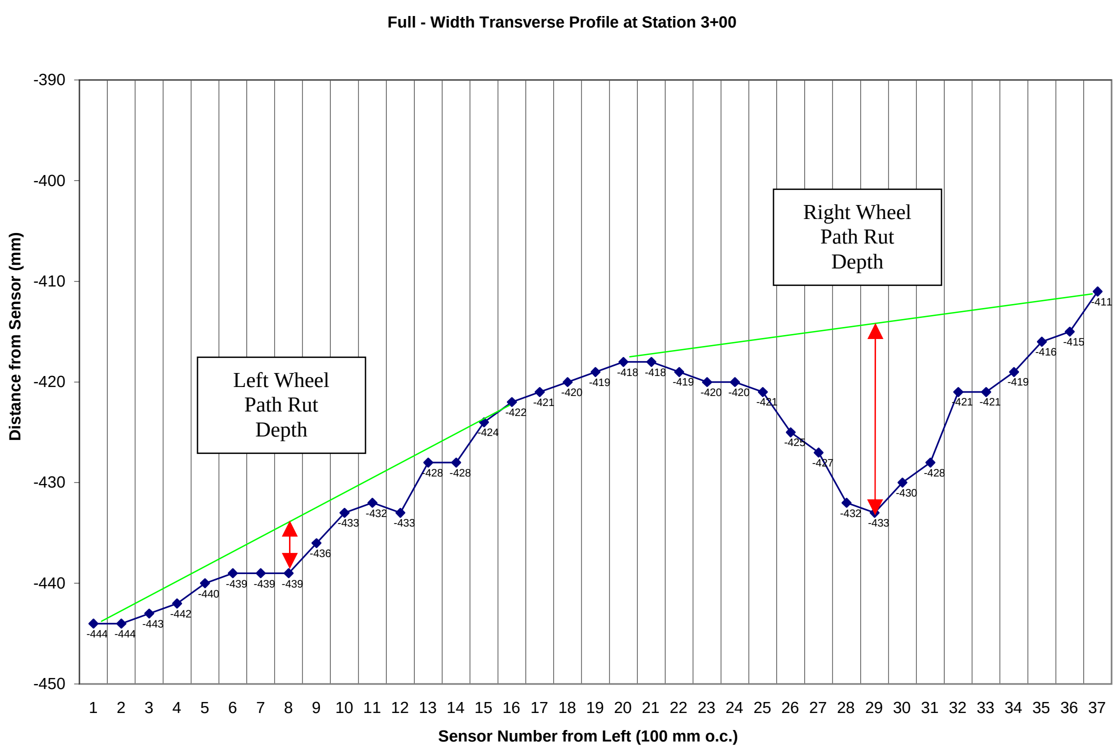


FIGURE 12 Full-width transverse profile for Site S1 at Station 3+00 of 500-ft test section.

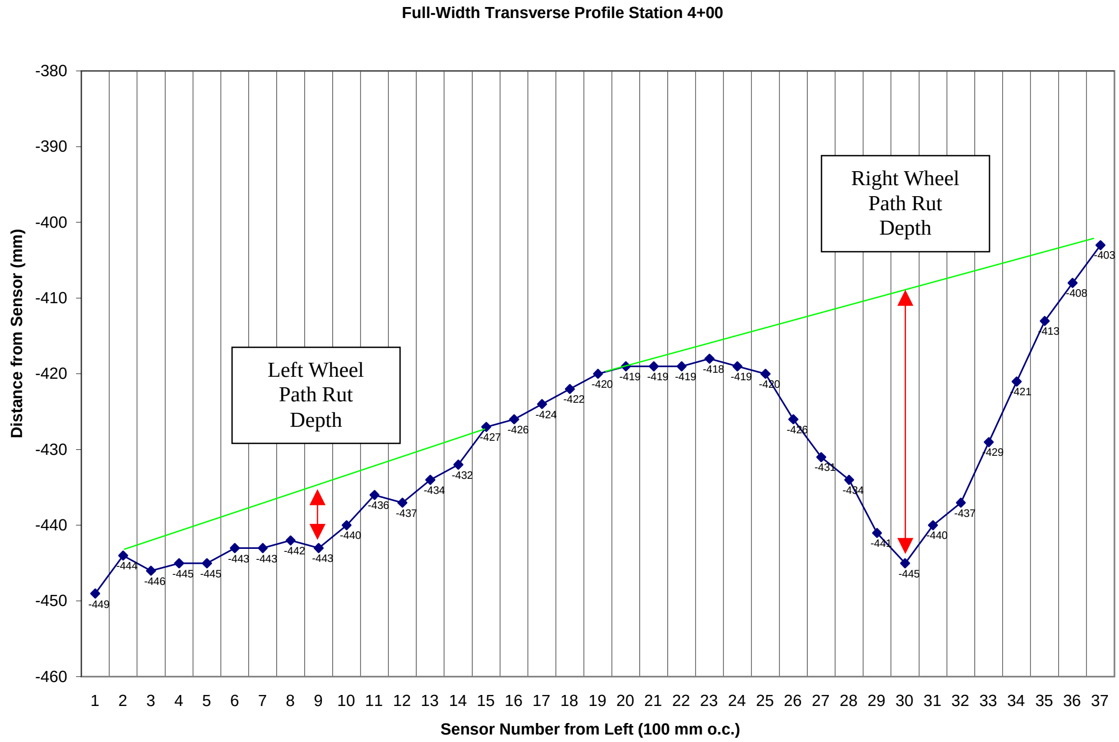


FIGURE 13 Full-width transverse profile for Site S1 at Station 4+00 of 500-ft test section.

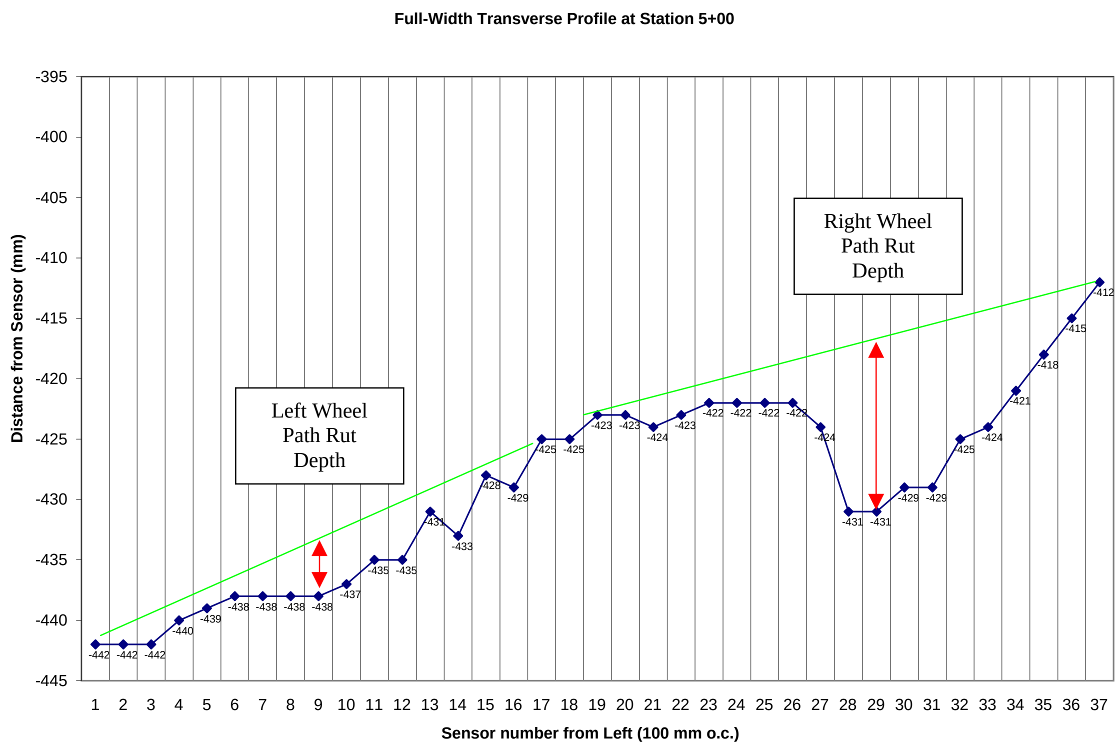


FIGURE 14 Full-width transverse profile for Site S1 at Station 5+00 of 500-ft test section.

Site S1 also had a few longitudinal and transverse cracks, but they were not significant. Overall, for a 10-year old pavement, the condition of Site S1 insofar as cracking is concerned was good; however, moderate levels of rutting existed.

Site S2 Distress Survey

At Site S2, less fatigue cracks were observed in the right wheel path, but there was more longitudinal cracking. Perhaps the longitudinal cracking observed was a precursor to fatigue cracking, and the condition at Site S2 was basically a less severe condition than that observed at Site S1, due to the location of the longitudinal joint in the right wheel path. Site S2 was also located on a 2% downhill grade, whereas Site S1 was located on a 4% uphill grade. The development of fatigue cracking from longitudinal cracking may have been expedited at Site S1 in comparison to Site S2 due to the increased rutting observed there, which may have been worsened by the fact that it was located on a steep uphill grade. Some low severity transverse cracking was also observed at Site S2.

The average rut depth in the right wheel path at Site S2, measured with the Photolog van, was substantially less severe (0.214 inches) than it was for Site S1. The left wheel path rut depth was 0.170 inches. Perhaps less rutting existed at Site S2 because it was located on a 2% downhill grade, whereas Site S1 was located on a 4% uphill grade. The overall condition of Site S2, for a ten-year old pavement, was very good. Rutting was not a problem.

TABLE 3 Descriptive Statistics for Site S2, Rut Depths (inches) Measured with Rut Bars Extended for a 3.6 Meter (approximately 12 ft) Transverse Profile

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	31	.079	.276	.170	.0470
Right Wheel Path Rut Depth (inches)	31	.118	.472	.215	.0874

Site C Distress Survey

Site C, the control section, had low and moderate severity longitudinal and transverse cracking throughout its length. Fatigue cracking was not observed. This section of S.R. 695 was paved in 1993, and then crack sealed in 2005. Fatigue cracking may have been prevented from developing by sealing the cracks. They had not yet developed after 12 years of service, when they were crack sealed, and did not develop in the three years after crack sealing.

Table 4 presents rut depths determined from full-width 3.6 meter transverse profiles for Site C. The average left wheel path rut depth was 0.324 inches, and the average right wheel path rut depth was 0.299 inches. Note that the control section was paved 5 years before Sites S1 and S2, so more severe rutting should be expected. The control section rut depths were more severe than they were for Site S2, but the right wheel path rut depths for Site S1 were considerably more severe (0.775 inches on average) than those for the control site (0.299 inches), in spite of it being 5 years newer.

TABLE 4 Descriptive Statistics for Site C, Rut Depths (inches) Measured with Rut Bars Extended for a 3.6 Meter (approximately 12 ft) Transverse Profile

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	32	.197	.472	.324	.0676
Right Wheel Path Rut Depth (inches)	32	.197	.472	.299	.0805

Tests of Cores Obtained from S.R. 695

Four (4) sets of cores were drilled at each test site. In order to obtain the required amount of material for testing, each set consisted of two cores, which were combined into one sample. Results of testing for these core samples are presented in Table 5. The tests were performed in accordance with AASTHTO T 331, "Bulk Specific Gravity and Density of Compacted Asphalt Mixtures Using Automatic Vacuum Sealing Method."

At Sites S1 and S2, where CIR base was used, the density of the CIR layer ranged from a low of 80.8 percent to a high of 89.2 percent compaction. These results validate the densities measured approximately one year after construction, which were mentioned in the Introduction of this report. Again, the densities measured at that time ranged from a low of 83.1% to a high of 89.9%. It should be noted that those tests were performed in accordance with another method, AASHTO T 166, "Bulk Specific Gravity of Compacted Asphalt Mixtures using Saturated Surface-Dry Specimens." Some differences between results using these different methods should be expected. Poor compaction in the CIR layer was apparent upon visual inspection of the cores, as can be seen in Figure 15. Note the open texture in the bottom CIR layer in Figure 15, in contrast to the top HMA layer.

TABLE 5 Results of Tests Performed on Cores in Accordance with AASHTO T 331

Sample ID	Bag Weight (g)	Sample Weight before Sealing (g)	Sealed Sample Weight in Water (g)	Sample Weight after Water Submersion (g)	Density of Water for Temp. Correction (g/cm^3)	Maximum Specific Gravity	Bulk Specific Gravity (g/cm^3)	Air Voids (%)	Percent Compaction (%)
C1	28.2	911.2	495.1	910.6	1	2.385	2.232	6.4	93.6
C2	28.2	1186.0	638.6	1186.2	1	2.385	2.196	7.9	92.1
C3	28.3	954.9	513.7	954.9	1	2.385	2.201	7.7	92.3
C4	28.2	1175.7	634.5	1175.6	1	2.385	2.204	7.6	92.4
S1-1	28.3	1437.7	737.7	1436.9	1	2.464	2.079	15.6	84.4
S1-2	28.4	1952.2	993.9	1951.2	1	2.464	2.057	16.5	83.5
S1-3	28.2	2712.1	1466.4	2708.9	1	2.464	2.199	10.8	89.2
S1-4	28.3	2675.5	1431.6	2674.8	1	2.464	2.168	12.0	88.0
S2-1	28.4	3060.8	1605.3	3059.9	1	2.512	2.118	15.7	84.3
S2-2	28.4	2008.1	1029.8	2007.1	1	2.512	2.073	17.5	82.5
S2-3	28.4	2024.4	1019.9	2025.5	1	2.512	2.030	19.2	80.8
S2-4	28.3	1788.3	943.4	1787.3	1	2.512	2.140	14.8	85.2

Full-depth cores were drilled for these tests. Table 6 presents the total full-depth thickness of the pavement at Sites S1 and S2. At Site S1, the full-depth ranged between 7 and 10.5 inches. At Site S2, the full-depth ranged between 9.5 and 11 inches. The proposed thickness of the CIR base was 3 inches. Table 6 also presents the actual measured thicknesses of the CIR base layer. At Site S1, the CIR thickness range between 2 and 3 inches. At Site S2, it ranged between 2 and 2.75 inches.

TABLE 6 CIR base layer thicknesses and full-depths of cores

Site	Sample	Total Thickness of CIR Base Layer (in.)	Full-depth of Core (in.)
S1	1A	2.0	7.0
S1	1B	3.0	7.5
S1	2A	2.0	10.0
S1	2B	2.0	8.5
S1	3A	2.0	10.5
S1	3B	2.5	10.0
S1	4A	2.5	9.0
S1	4B	2.0	10.0
S2	1A	2.75	10.0
S2	1B	2.0	9.5
S2	2A	2.5	11.0
S2	2B	2.5	10.0
S2	3A	2.25	10.0
S2	3B	2.5	10.0
S2	4A	2.0	9.0
S2	4B	2.0	6.5

Sieve analyses of the base layer immediately beneath the wearing surface layer were performed on all of the extracted core samples. The sieve analyses were performed in accordance with ASTM C 136, "Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates." The base layer refers to the CIR material at Sites S1 and S2. These grain size distributions are tabulated in Appendix G.

Overall, CIR grain size distributions were similar to the ConnDOT Class 2 distribution specified in ConnDOT *Standard Specifications for Roads, Bridges, and Incidental Construction.* However, they were somewhat coarser on the larger sieves, similar to Class 1 mix. The pavement prior to recycling met Class 1 gradation requirements. The CIR process resulted in a slightly finer mix.

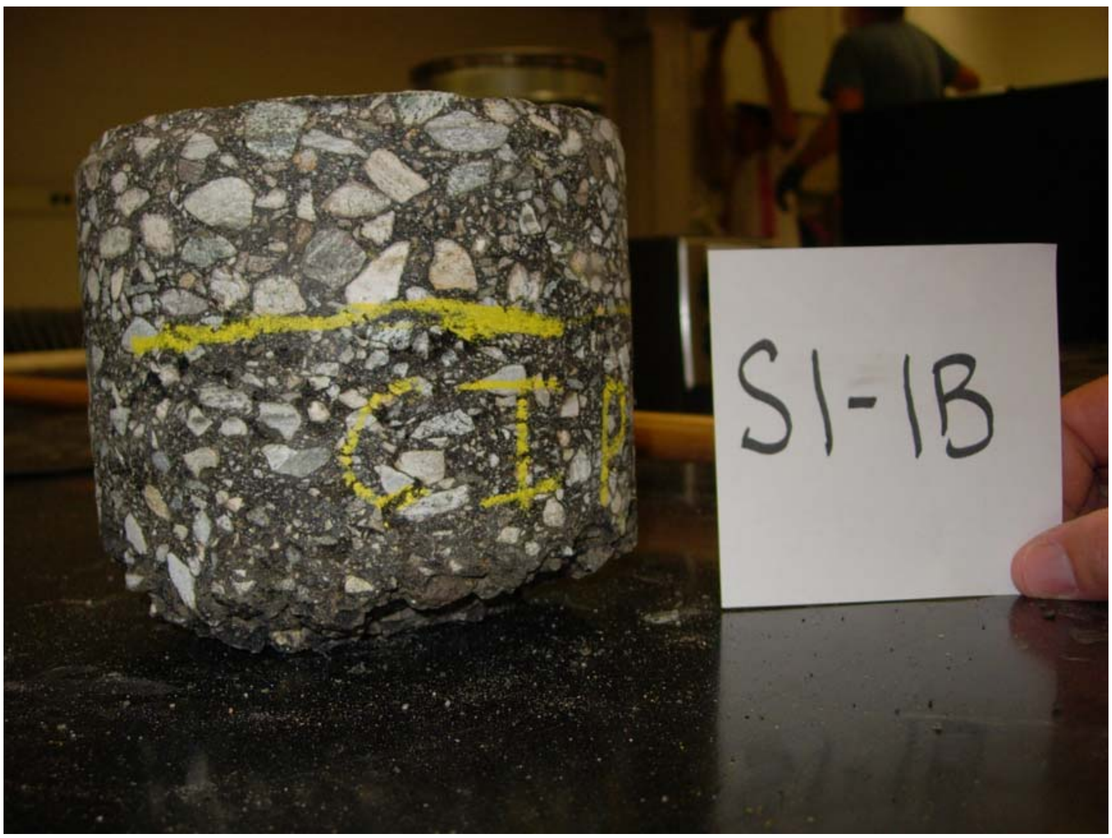


FIGURE 15 Core S1-1B obtained from Site S1. Note open texture in bottom CIR layer.

Photolog Rut Depths

The ConnDOT Photolog van measures rut depths with infrared sensors mounted across a front bumper assembly, which includes rut bars that can be extended as far as 3.6 meters (approximately 12 feet). The sensors are located 100 mm on center; therefore, when the bars are fully extended to 3.6 meters, there are 37 sensors which measure the distance from the sensor to the pavement surface. When the rut bars are fully extended, a full transverse profile of a pavement lane can be measured. Photos of the van are shown below in Figure 16 with the rut bars fully extended.

(a) (b)

FIGURE 16 (a) ConnDOT Photolog van with rut bars extended fully to 3.6 meters. (b) view showing sensors located at 100 mm on center.

Due to safety considerations, annual rut inventory measurements are carried out with the rut bars extended 300 mm (approximately 1 ft) on either side, which provides partial transverse profile surveys 2.4 meters wide (approximately 8 ft). Rut data collected and discussed below between 1999 and 2008 represent values determined from partial transverse profiles of 2.4 meters. Subsequent to analyzing these data, it was decided to perform a full-width transverse profile of S.R. 695 for comparison. This was carried out in February 2009. These data are presented following presentation of the 1999 to 2008 results.

Partial Transverse Profiles of S.R.695 EB (1999-2008)

2008 Photolog rut data were analyzed for S.R. 695 eastbound and westbound. In the eastbound direction, the mean rut depth in the left wheel path was 0.118 inches and in the right wheel path it was 0.238 inches (see Table 7). Right and left rut depths were plotted versus mileage in a simple line chart shown in Figure 17, and it was observed that for the right wheel path, the rut depths were substantially higher for the section between 1.663 miles and 3.048 miles. Accordingly, data were split between these mileposts and the rest of S.R. 695 EB. Between these mileposts, the mean rut depth in the right wheel path was 0.423 inches, while the left wheel path rut depth stayed virtually the same at 0.119 inches (see Table 8). When rut values for these mileposts were excluded from the data set, the mean rut depth in the right wheel path was 0.148 inches (see Table

9), which is more in keeping with values measured for the left wheel path.

Next, the question as to why right wheel path rut depths were so much greater for the section between 1.663 and 3.048 miles was posed. DigitalHIWAY was employed to view images eastbound along S.R. 695. It was observed that for the section in question a longitudinal construction joint was located in the right wheel path, and that fatigue cracking existed along this joint. Before and after this section, the joint was generally located closer to the shoulder line or on the shoulder line itself. This same level of fatigue cracking was not observed in the left wheel path. Considering the above, it is possible that the increased rut depths for this section were caused by the existence of the longitudinal construction joint in the right wheel path. The fact that the density of the CIR base was lower in this section likely made the pavement more susceptible to rutting once a longitudinal crack did develop. Rut depths also correspond to grade (uphill or downhill). This will be discussed in greater detail in following sections of this report.

TABLE 7 S.R. 695 EB, Overall Descriptive Statistics for Rut Depths Determined from 2.4 Meter Partial-Width Transverse Profiles for Pavement over CIR Base (2008)

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	1158	.000	.472	.118	.0580
Right Wheel Path Rut Depth (inches)	1158	.000	1.142	.238	.1768

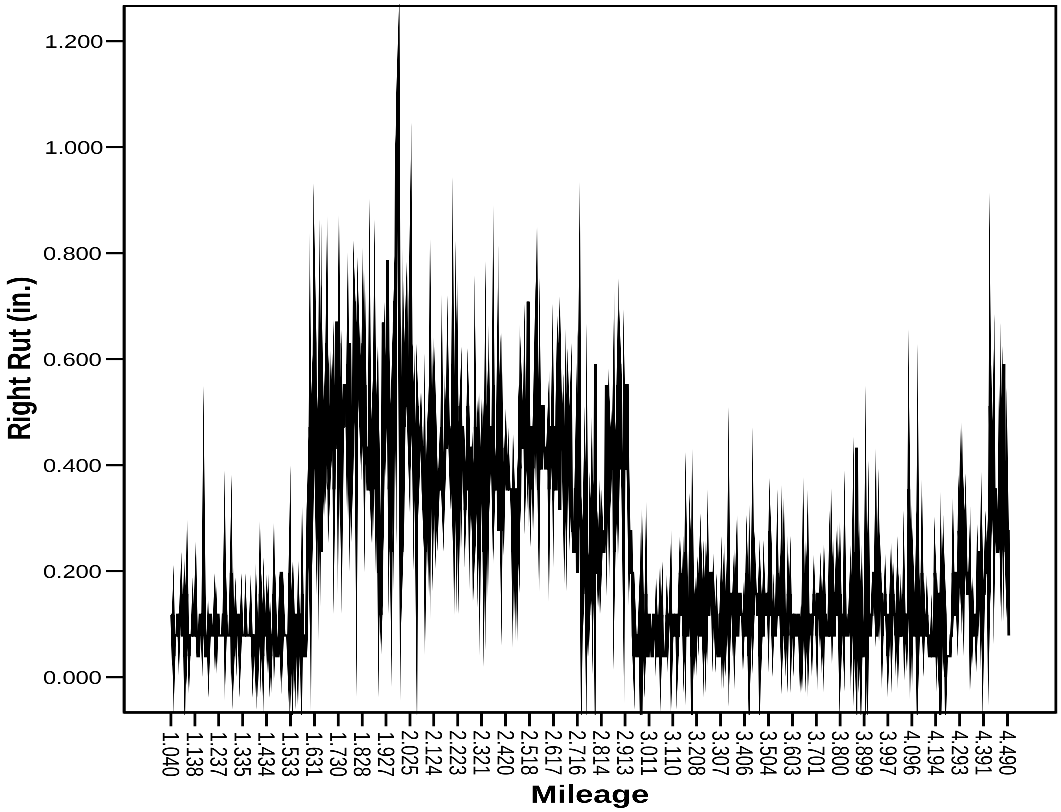


FIGURE 17 Right wheel path rut depths versus mileage for S.R. 695 EB pavement over CIR base determined from 2.4 meter transverse profiles.

TABLE 8 S.R. 695 EB, Descriptive Statistics for Mileposts Between 1.663 and 3.048 Miles, Longitudinal Joint Located in Right Wheel Path

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	445	.039	.433	.119	.056
Right Wheel Path Rut Depth (inches)	445	.118	1.142	.423	.131

TABLE 9 S.R. 695 EB, Descriptive Statistics for Mileposts Less than 1.663 and Greater than 3.048 Miles for Pavement over CIR Base

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	1064	.000	.472	.130	.060
Right Wheel Path Rut Depth (inches)	1064	.000	.591	.148	.086

Next, rut depth data, determined from 2.4 meter transverse profiles, were analyzed for 1999, 2000, 2002, 2004, 2006, and 2008. Mean rut depths for these years are presented in Table 10 for both the right and left wheel paths. Both wheel paths are presented for the entire eastbound section of S.R. 695 over CIR base as well as for just the section between 1.663 and 3.048 miles. For each column in Table 10, the rut depths steadily increased over time. This was more evident in the right wheel path between 1.663 and 3.048 miles, as the average rut depth increased from 0.166 inches in 1999 to 0.423 inches in 2008. Rut depths versus time are plotted for each of these cases in Figure 18.

TABLE 10 Rut Depths Measured on S.R. 695 EB over CIR Base Determined from 2.4 meter Transverse Profiles

Year	Right Rut Depth Mileage > 1.077 (inches)	Right Rut Depth Mileage > 1.663 < 3.048 (inches)	Left Rut Depth Mileage > 1.077 (inches)	Left Rut Depth Mileage > 1.663 < 3.048 (inches)
1999	0.105	0.166	0.059	0.080
2000	0.156	0.288	0.109	0.105
2002	0.177	0.275	0.103	0.107
2004	0.195	0.327	0.103	0.116
2006	0.223	0.383	0.117	0.129
2008	0.238	0.423	0.118	0.119

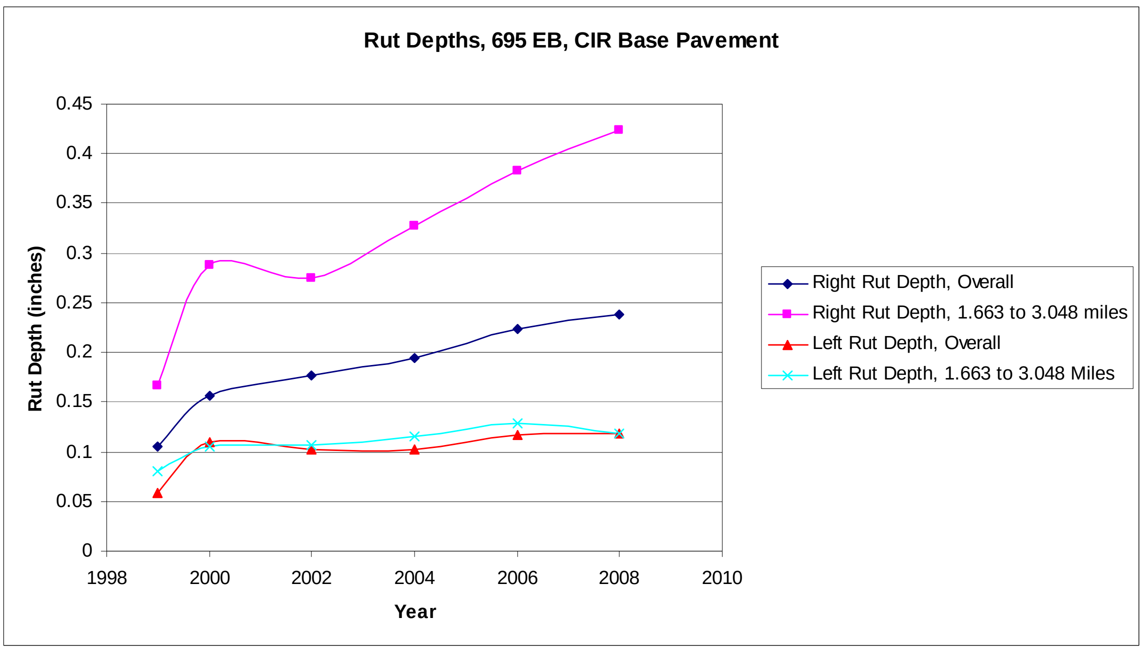


FIGURE 18 Rut depths on S.R. 695 EB over CIR base for right and left wheel paths split on the section between 1.663 to 3.048 miles. These were determined from 2.4 meter transverse profiles.

For comparison, 2004 rut depths were analyzed for the control pavement from 0 to approximately 1.0 miles. The year 2004 was selected for comparison because the age of the control section was the same age as that of the pavement with CIR base during this evaluation. The eastbound rut depths were 0.152 inches and 0.124 inches for the left and right wheel paths, respectively (see Table 11). These values compare similarly to those measured for pavement with CIR base, excluding the section discussed above in the right wheel path from 1.663 to 3.048 miles where the longitudinal joint was located in the wheel path.

TABLE 11 S.R.695 EB Control Pavement (Mileage < 1.0 miles), 2004 Descriptive Statistics for Rut Depths Determined from 2.4 meter Transverse Profiles

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	359	.039	.748	.152	.061
Right Wheel Path Rut Depth (inches)	359	.000	.433	.124	.074

Partial Transverse Profiles of S.R.695 WB (1999-2008)

Table 12 presents S.R. 695 WB rut depths for pavement over CIR base between 4.524 and 1.061 miles (reverse log direction). These are provided for the years 1999, 2000, 2002, 2004, 2006, and 2008. For each year analyzed, the right rut depth was greater than the left rut depth, except for 2000. This exception is likely due to the natural variation that exists between Photolog rut measurements from year-to-year.

The rut depths tended to increase over time. The left rut depth was 0.060 inches in 1999 and increased to 0.107 inches in 2008. The right rut depth increased steadily from 0.098 inches in 1999 to 0.182 inches in 2008.

The right wheel path rut depths were plotted versus mileage (see Figure 19), and similar to the eastbound direction, a section existed where the rut depths were more severe. This occurred between 4.115 and 2.989 miles. Once again, DigitalHIWAY was employed to look at this section and it was observed that for the more severe section, the longitudinal joint was located in the right wheel path. It is surmised that this led to the development of more severe rut depths.

In order to quantify this more severe condition, cases between 4.115 and 2.989 miles, where the longitudinal joint was located, were split from the rest of the data set. Rut depths between these mileage points are presented in Table 13, and those excluding this section are presented in Table 14. The left rut depth for this more severe section in Table 13 was 0.117 inches, which is slightly greater (14%) than that in Table 14 excluding this section (0.103 inches). The average 2008 right rut depth was considerably greater for this more severe section at 0.292 inches, which was more than twice as deep than that for pavement excluding this section (right rut depth = 0.129 inches).

TABLE 12 Rut Depths Measured on S.R. 695 WB Determined from Partial Width Transverse Profiles of 2.4 meters

Year	Left Rut Depth Chainage >= 1.744 Akm	Right Rut Depth Chainage >= 1.744 Akm
1999	0.060	0.098
2000	0.130	0.111
2002	0.110	0.152
2004	0.090	0.158
2006	0.109	0.186
2008	0.107	0.182

TABLE 13 Descriptive Statistics for SR 695 WB Section Between 4.115 and 2.989 Miles, Longitudinal Joint in Right Wheel Path, Measured in 2008 from 2.4 meter Transverse Profiles

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	361	.000	.591	.117	.0656
Right Rut Wheel Path Depth (inches)	361	.079	.669	.292	.1018

TABLE 14 Descriptive Statistics for SR 695 WB, Excluding Section Between 4.115 and 2.989 Miles, Longitudinal Joint not in Right Wheel Path, Measured in 2008 from 2.4 meter Transverse Profiles

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	762	.000	.591	.103	.0800
Right Wheel Path Rut Depth (inches)	762	.000	.551	.129	.0728

TABLE 15 S.R. 695 WB, Mileage < 1.061 Miles, No CIR Base, Determined from 2.4 meter Transverse Profiles

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth (inches)	356	.039	.709	.203	.0746
Right Wheel Path Rut Depth (inches)	356	.000	.551	.172	.0890

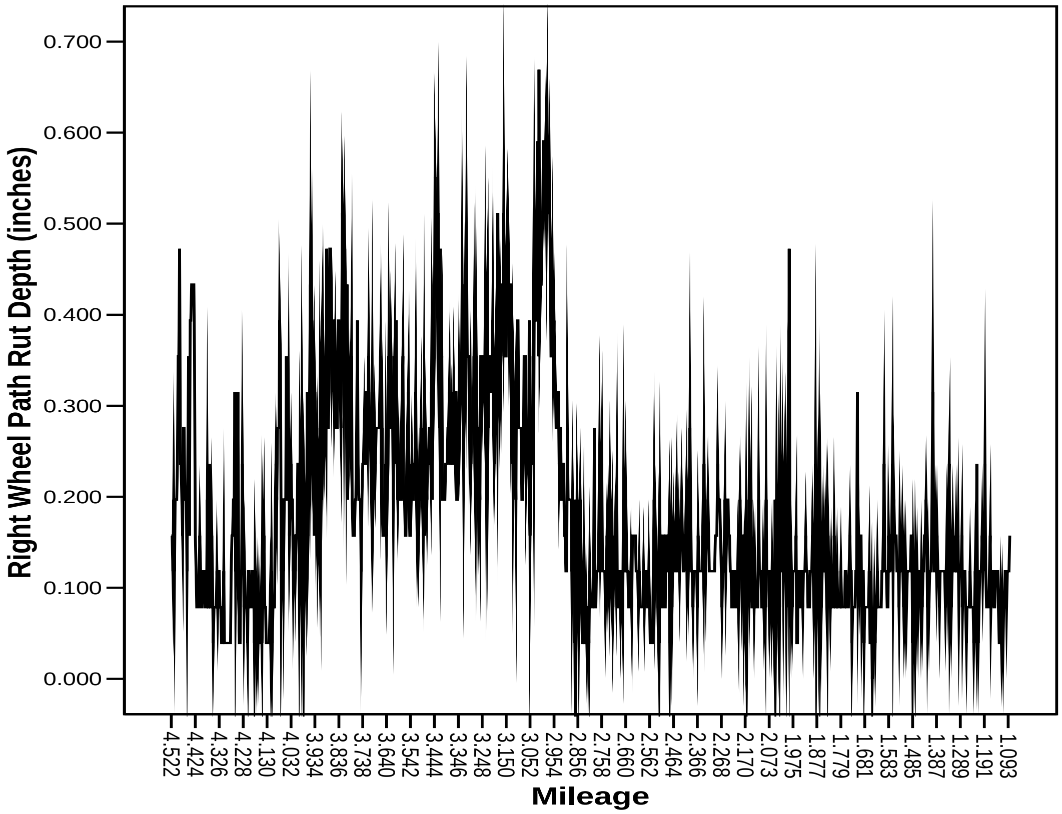


FIGURE 19 Right wheel path rut depths versus mileage for S.R. 695 WB determined from 2.4 meter transverse profiles.

Full Width Transverse Profiles of S.R. 695 EB (February 2009)

In order to obtain more accurate rut depth measurements and for comparison, it was decided to perform full width transverse profiles of S.R. 695 by extending the rut bars fully for a total width of 3.6 meters (approx. 12 ft). These measurements were taken in February 2009. Two escort vehicles accompanied the Photolog van in order to prevent other vehicles from passing the van while the bars were extended. For comparison, partial-width transverse profiles were measured with the bars extended so that the total width was 3.0 meters (approximately 10 ft) and 2.4 meters (approximately 8 ft).

Figure 20 presents a plot of distance measurements from sensors located on the front bumper assembly of the Photolog van (see Figure 16). Sensor numbers from left to right are shown on the x-axis. When full-width profiles were measured, 37 sensors were used. Rut depths were determined from this profile. This profile was measured at 1.712 miles of S.R. 695 EB. The right wheel path rut depth determined from this profile was 16 mm (0.630 inches) and the left wheel path rut depth was determined to be 4 mm (0.157 inches). If this same profile was measured with the rut bars extended so that the total width was 3.0 meters, sensors 1 through 3 and 35 through 37 would drop off, for a total of 31 sensors. If the rut bars were extended so the total width was 2.4 meters, sensors 1 through 6 and 31 through 37 would drop off, for a total of 25 sensors.

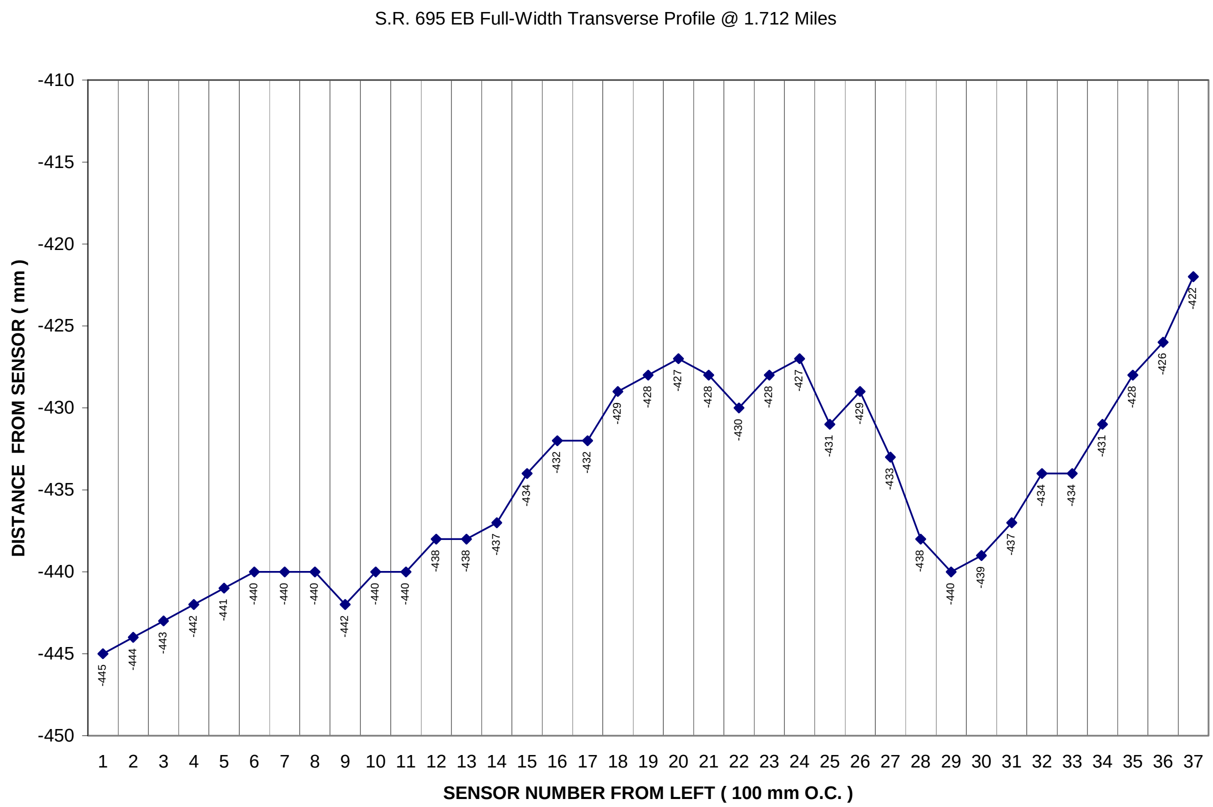


FIGURE 20 Full-Width Transverse Profile, S.R. 695 EB at 1.712 Miles.

Table 16 displays descriptive statistics for the left and right wheel path rut depths for S.R. 695 EB beginning at 1.080 miles and ending at 4.172 miles over CIR pavement. The variables represent rut depths calculated from partial-width transverse profiles of 2.4 meters and 3.0 meters, and from full-width transverse profiles of 3.6 meters, for the left and right wheel paths.

A significant difference between the magnitudes of the rut depths determined from the full-width versus partial-width profiles was observed. The 3.6 meter full-width mean rut depths were approximately two times those of the 2.4 meter partial-width mean rut depths, and they were approximately one-and-one-half (1.5) times those of the 3.0 meter partial-width profiles. This can be seen graphically in Figure 21 below, where 95% confidence intervals are plotted for each variable.

TABLE 16 Descriptive Statistics for Rut Depths (inches) Determined from Partial-Width Profiles of 2.4 Meters and 3.0 Meters, and from Full-Width Profiles of 3.6 Meters, S.R. 695 EB from 1.080 miles to 4.172 miles.

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depth, 2.4 M Profile	997	.039	.551	.127	.046
Right Wheel Path Rut Depth, 2.4 M Profile	997	.039	1.417	.250	.172
Left Wheel Path Rut Depth, 3.0 M Profile	997	.079	.984	.208	.086
Right Wheel Path Rut Depth, 3.0 M Profile	997	.039	1.181	.343	.229
Left Wheel Path Rut Depth, 3.6 M Profile	997	.079	.748	.250	.102
Right Wheel Path Rut Depth, 3.6 M Profile	997	.118	1.890	.480	.291

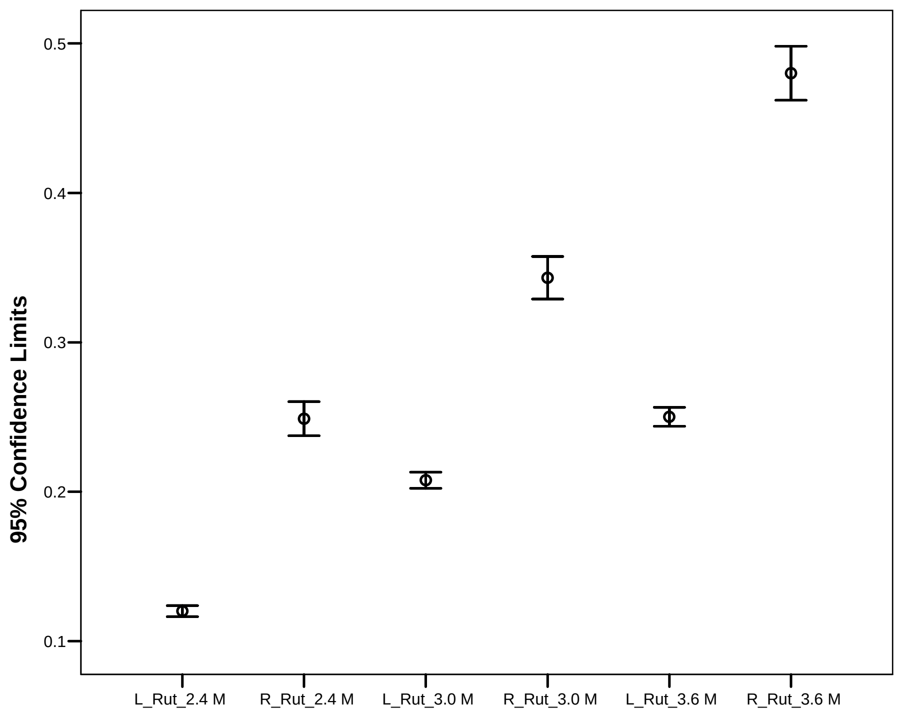


FIGURE 21 95% Confidence Levels for Rut Depths, S.R. 695 EB over CIR Base.

While a significant difference between the magnitudes of the rut depths was found, simple line charts demonstrate that rut depths determined from partial width profiles do provide relative indicators of rut severity. These are shown below in Figures 22 to 25. Notice that each of these three graphs follow the same pattern, but their magnitudes are different.

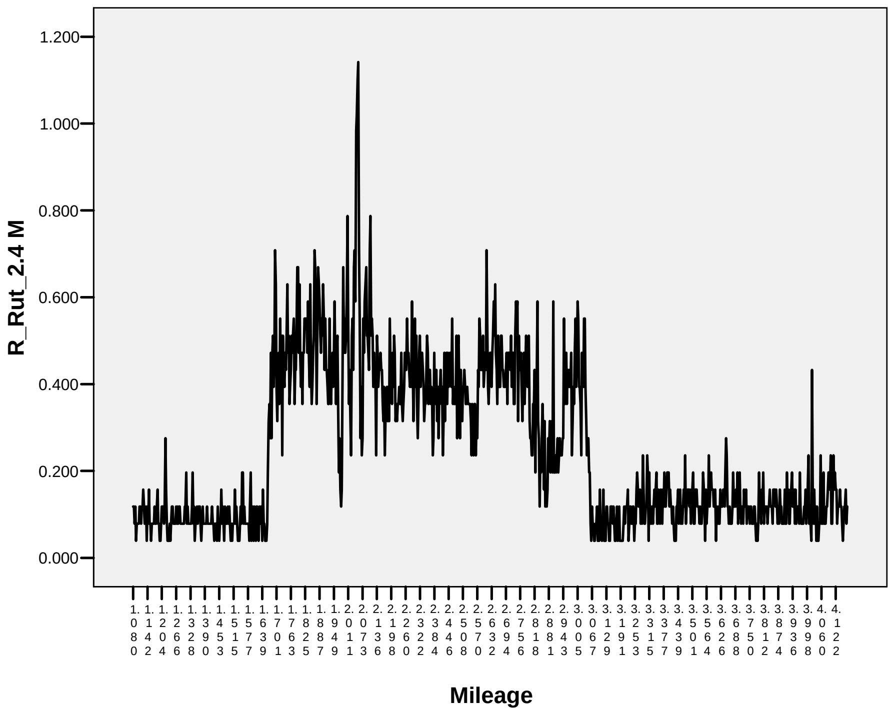


FIGURE 22 Simple Line Chart of Rut Depths Determined from Transverse Profiles of 2.4 Meters Versus Mileage for S.R. 695 EB over CIR Base.

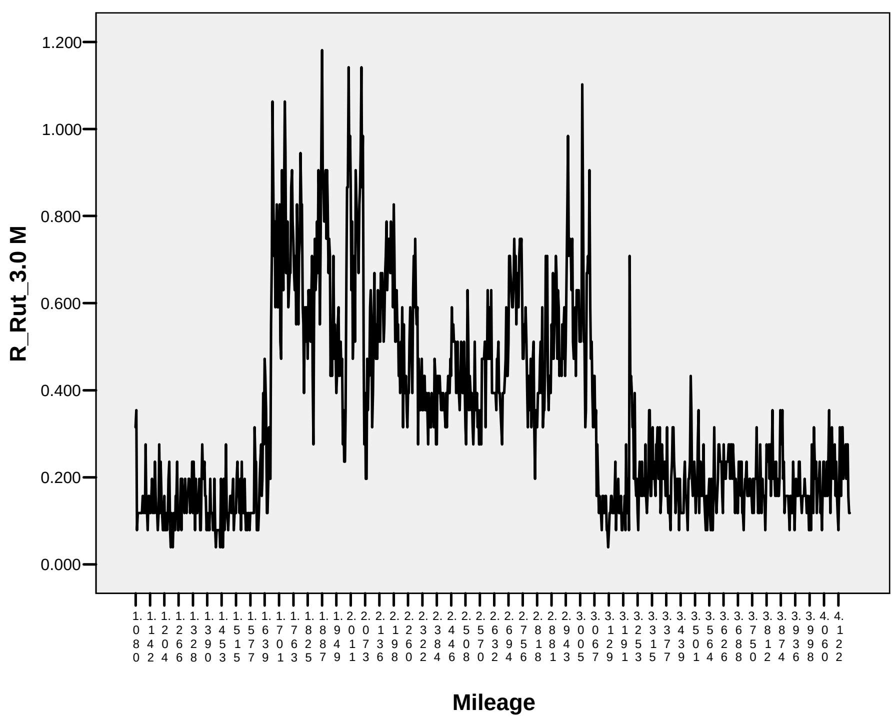


FIGURE 23 Simple Line Chart of Rut Depths Determined from Transverse Profiles of 3.0 Meters Versus Mileage for S.R. 695 EB over CIR Base.

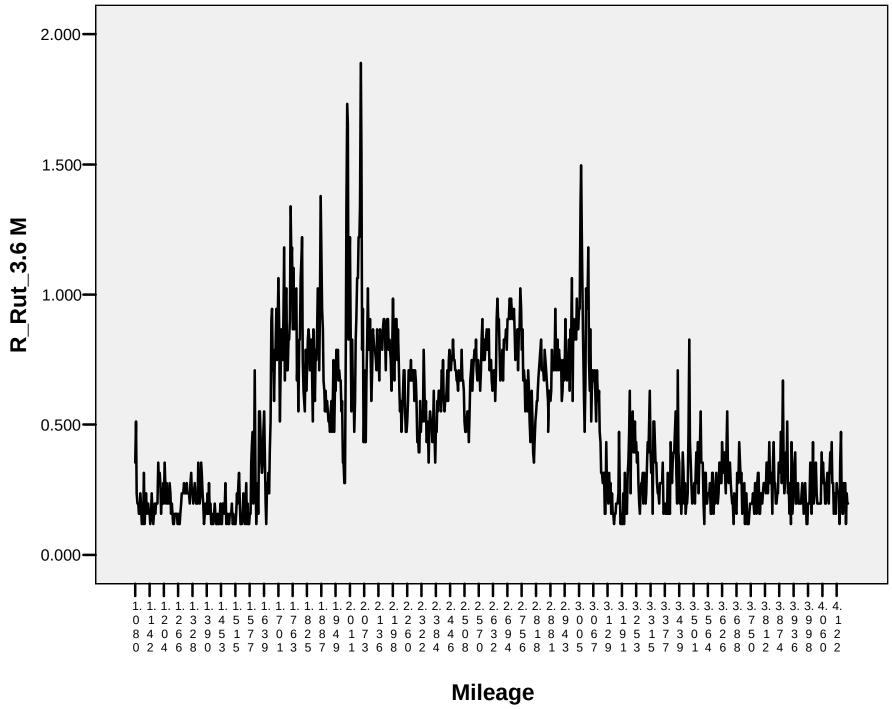


FIGURE 24 Simple Line Chart of Rut Depths Determined from Transverse Profiles of 3.6 Meters Versus Mileage for S.R. 695 EB over CIR Base.

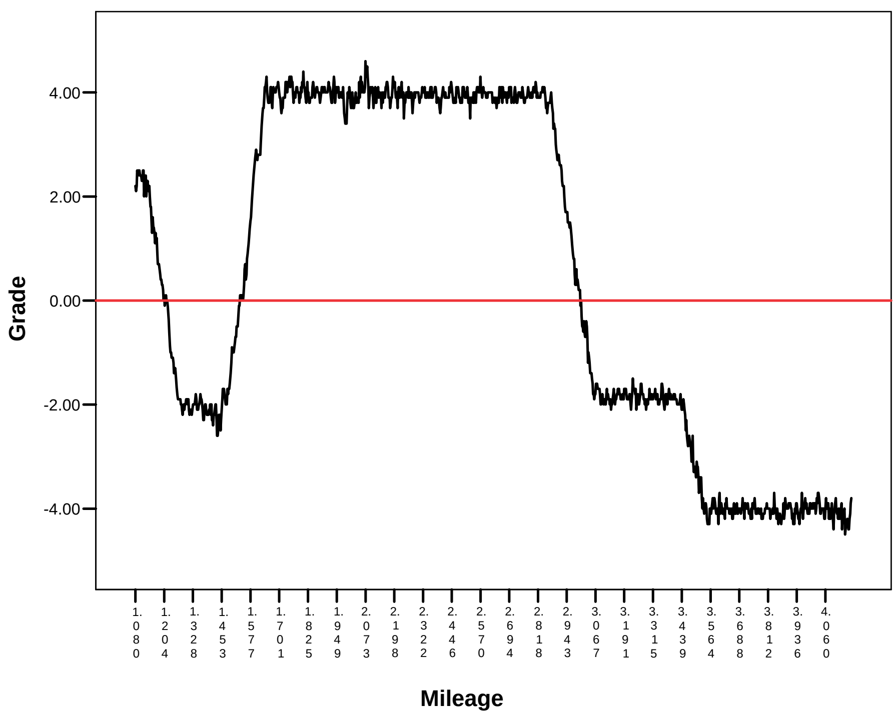


FIGURE 25 Simple Line Chart of Grade Versus Mileage for SR 695 EB over CIR Base.

Next, a simple line chart of grade versus mileage was plotted in Figure 25 above in order to see if grade related in any way to rut depths. Note: positive values represent grades uphill. In looking at this chart, it appears that uphill grades do correspond to mileages where more severe rut depths existed.

Accordingly, data were split between uphill and downhill grades, and descriptive statistics were determined. These are presented in Table 17 below. A significant difference was found between the uphill and downhill data sets. For the left wheel path, the average downhill rut depth was 0.213 inches, while the uphill rut depth was 0.291 inches. The difference in the right wheel path was more significant, as the downhill rut depth was 0.285 inches, while the uphill rut depth was 0.696.

TABLE 17 Descriptive Statistics for Rut Depths (inches) Determined from Full-Width Profiles of 3.6 Meters, S.R. 695 EB from 1.080 miles to 4.172 miles. Data Split Between Uphill and Downhill Grades.

Grade		N	Minimum	Maximum	Mean	Std. Deviation
Negative (downhill)	Left Wheel Path Rut Depths (in.)	524	.079	.748	.213	.097
	Right Wheel Path Rut Depths (in.)	524	.118	1.496	.285	.175
Positive (uphill)	Left Wheel Path Rut Depths (in.)	473	.079	.748	.291	.090
	Right Wheel Path Rut Depths (in.)	473	.118	1.890	.696	.236

In order to evaluate more severe grades, rut depths over pavement on 4% positive grades were compared to rut depths on 4% negative grades. Notice that for each row of Table 18 that the negative grades were slightly less than in Table 17 and that the positive grades were slightly greater than in Table 17. Therefore, as the grade steepened uphill, rut depths increased; and, as the grade steepened downhill, rut depths decreased.

TABLE 18 Descriptive Statistics for Rut Depths (inches) Determined from Full-Width Profiles of 3.6 Meters, S.R. 695 EB from 1.080 miles to 4.172 miles. Data Split Between Uphill Grades of 4% and Downhill Grades of -4%.

Grade		N	Minimum	Maximum	Mean	Std. Deviation
Negative (downhill -4%)	Left Wheel Path Rut Depths (in.)	210	.079	.433	.208	.073
	Right Wheel Path Rut Depths (in.)	210	.118	.669	.263	.091
Positive (uphill, 4%)	Left Wheel Path Rut Depths (in.)	387	.079	.748	.298	.090
	Right Wheel Path Rut Depths (in.)	387	.118	1.890	.725	.218

Next, data were split once again between areas where the right wheel paths were over longitudinal joints and areas where the right wheel paths were not over these joints, as observed in DigitalHIWAY. Areas where the right wheel paths were over longitudinal joints tended to coincide with positive grades, so the split descriptive statistics presented in Table 19 are similar to those in Table 17. Note that the right wheel path rut depths were more severe (0.744 inches) when the data were split on longitudinal joint location than on grade (0.696 inches).

TABLE 19 Descriptive Statistics for Rut Depths (inches) Determined from Full-Width Profiles of 3.6 Meters, S.R. 695 EB from 1.080 miles to 4.172 miles. Data Split Between areas where the right wheel path was over LG joints (1.663 miles to 3.048 miles) and where it was not over LG joints.

Longitudinal Jt.		N	Minimum	Maximum	Mean	Std. Deviation
Not Over Right Wheel Path	Left Wheel Path Rut Depths (in.)	551	.079	.748	.207	.088
	Right Wheel Path Rut Depths (in.)	551	.118	.866	.266	.127
Over Right Wheel Path	Left Wheel Path Rut Depths (in.)	446	.079	.748	.304	.091
	Right Wheel Path Rut Depths (in.)	446	.276	1.890	.744	.206

Full Width Transverse Profiles of S.R. 695 WB (February 2009)

Table 20 below presents mean rut depths in the westbound direction for S.R. 695 WB over pavement with CIR base, and Figure 23 presents 95% Confidence Intervals for each variable. Similar to the eastbound direction, the calculated rut depths were significantly different between those determined from full-width profiles (3.6 meters) and partial-width profiles (2.4 and 3.0 meters).

Line charts for rut depths determined from full-width transverse profiles of S.R. 695 WB versus mileage are presented for the left and right wheel paths in Figures 27 and 28, respectively. Finally, a line chart of grade versus mileage is presented in Figure 29. As was the case in the eastbound direction, uphill grades tended to correspond with more severe rut depths.

TABLE 20 Descriptive Statistics for Rut Depths (inches) Determined from Partial-Width Profiles of 2.4 Meters and 3.0 Meters, and from Full-Width Profiles of 3.6 Meters, S.R. 695 EB from 1.061 miles to 4.402 miles.

	N	Minimum	Maximum	Mean	Std. Deviation
L_Rut_2.4M	1074	.039	.315	.111	.037
R_Rut_2.4M	1074	.039	.669	.197	.132
L_Rut_3.0M	1074	.039	.551	.175	.058
R_Rut_3.0M	1074	.039	.787	.284	.173
L_Rut_3.6M	1074	.079	.551	.210	.076
R_Rut_3.6M	1074	.079	.866	.341	.203

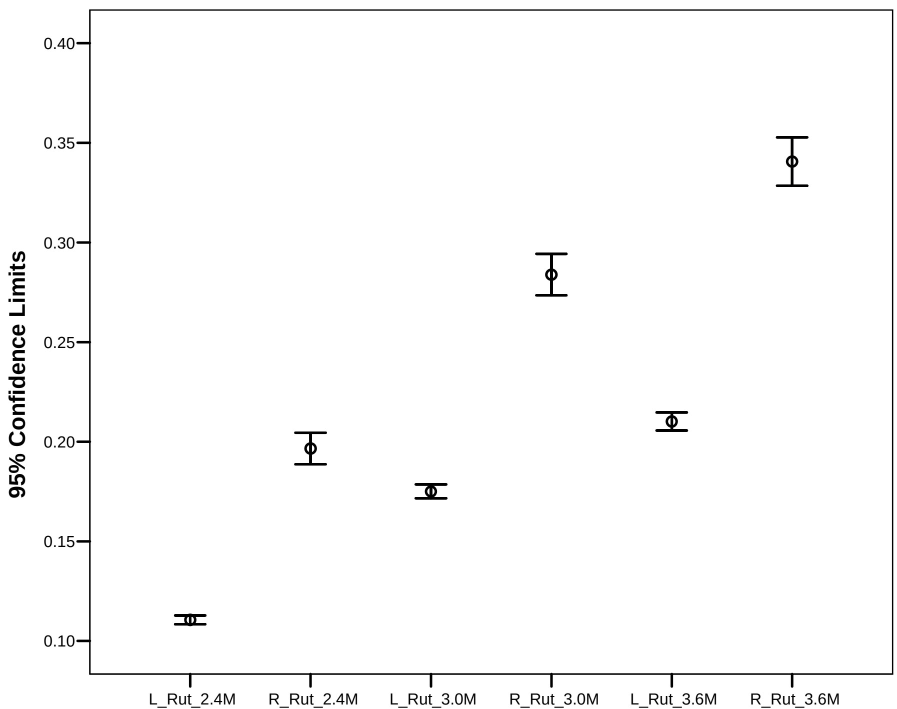


FIGURE 26 95% Confidence Levels for Rut Depths, S.R. 695 WB over CIR Base.

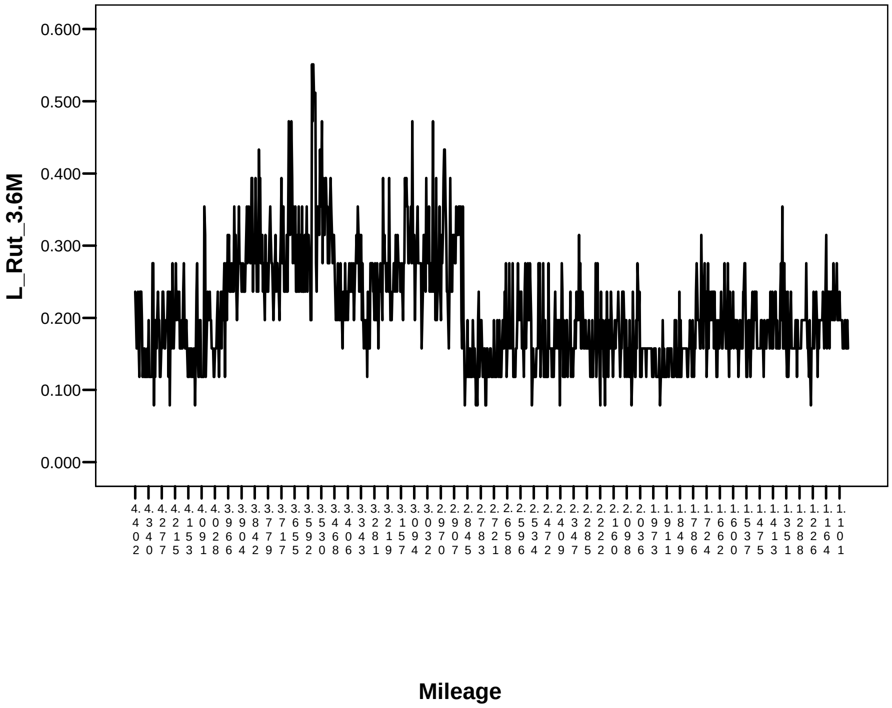


FIGURE 27 Simple Line Chart of Left Wheel Path Rut Depths Determined from Transverse Profiles of 3.6 Meters Versus Mileage for S.R. 695 WB over CIR Base

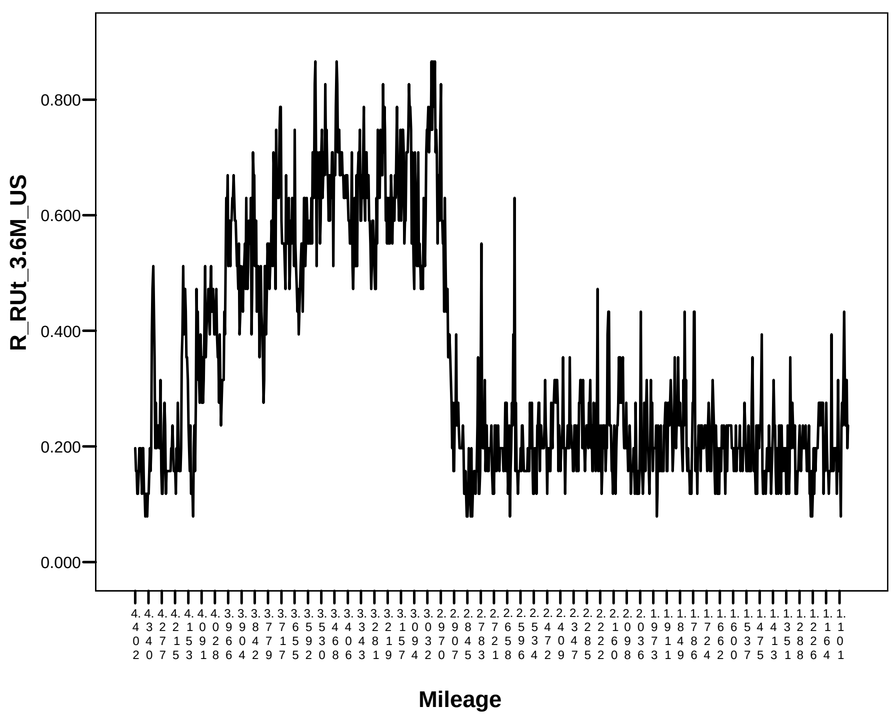


FIGURE 28 Simple line chart of right wheel path rut depths determined from transverse profiles of 3.6 meters versus mileage for S.R. 695 WB over CIR base.

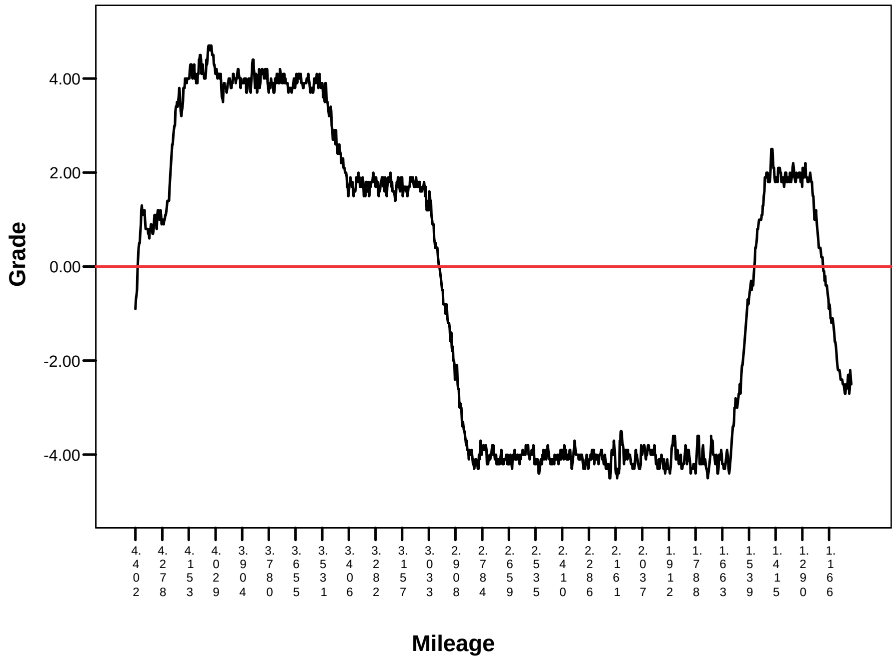


FIGURE 29 Simple Line Chart of Grade Versus Mileage for SR 695 WB over CIR Base.

The correspondence between rut depths and grade are quantified in Table 21 below with descriptive statistics. The average downhill rut depth in the left wheel path was 0.180 inches, while the average uphill depth was 0.238 inches. Similar to the eastbound direction, the difference was more profound in the right wheel path, where the downhill depth was 0.223 inches and the uphill depth was 0.451 inches.

TABLE 21 Descriptive Statistics for Rut Depths (inches) Determined from Full-Width Profiles of 3.6 Meters, S.R. 695 WB from 1.080 miles to 4.402 miles. Data Split Between Uphill and Downhill Grades

Grade		N	Minimum	Maximum	Mean	Std. Deviation
Negative (downhill)	Left Wheel Path Rut Depths (in.)	519	.079	.433	.180	.059
	Right Wheel Path Rut Depths (in.)	519	.079	.827	.223	.095
Positive (uphill)	Left Wheel Path Rut Depths (in.)	555	.079	.551	.238	.079
	Right Wheel Path Rut Depths (in.)	555	.079	.866	.451	.214

Just as was done in the eastbound direction, data were further split for more severe uphill and downhill grades of 4% and -4%, respectively (see Table 22). Similar to the eastbound direction, the more severe negative grades had slightly shallower rut depths, while the more severe positive grades had slightly deeper rut depths. The rut depths were affected by the more severe grades in the same way as they were in the eastbound direction.

TABLE 22 Descriptive Statistics for Rut Depths (inches) Determined from Full-Width Profiles of 3.6 Meters, S.R. 695 WB from 1.080 miles to 4.402 miles. Data Split Between Uphill 4% and Downhill 4% Grades

Grade		N	Minimum	Maximum	Mean	Std. Deviation
Negative (downhill -4%)	Left Wheel Path Rut Depths (in.)	396	.079	.315	.166	.047
	Right Wheel Path Rut Depths (in.)	396	.079	.630	.211	.072
Positive (uphill 4%)	Left Wheel Path Rut Depths (in.)	199	.079	.551	.260	.088
	Right Wheel Path Rut Depths (in.)	199	.079	.866	.494	.142

Next, data were split between areas where the right wheel path was over a longitudinal joint and not over a longitudinal joint (see Table 23 below). For areas where the right wheel path was over the longitudinal joint, the rutting was more severe, and the difference between the mean values was more significant than when the data were split on grade (positive versus negative). The right wheel path rut depth was 0.582 inches when it was over longitudinal joints, while it was 0.218 inches when it was not.

TABLE 23 Descriptive Statistics for Rut Depths (inches) Determined from Full-Width Profiles of 3.6 Meters, S.R. 695 WB from 1.080 miles to 4.402 miles. Data Split Between Areas where Right Wheel Path is Located Over LG Joints (2.989 to 4.115 miles) and not over LG Joints

Longitudinal Joint		N	Minimum	Maximum	Mean	Std. Deviation
Not over Right Wheel Path	Left Wheel Path Rut Depths (in.)	713	.079	.433	.180	.055
	Right Wheel Path Rut Depths (in.)	713	.079	.827	.218	.096
Over Right Wheel Path	Left Wheel Path Rut Depths (in.)	361	.118	.551	.270	.076
	Right Wheel Path Rut Depths (in.)	361	.236	.866	.582	.127

Full-Width Transverse Profiles of S.R. 695 over Conventional Base (February 2009)

Tables 24 and 25 below present descriptive statistics for rut depths measured on S.R. 695 for sections over conventional base. For S.R. 695 EB, the average left and right wheel path rut depths were 0.287 and 0.309 inches, respectively. For S.R. 695 WB, the average left and right wheel path rut depths were 0.363 and 0.281 inches, respectively.

TABLE 24 Descriptive Statistics for Rut Depths Measured on S.R. 695 EB over Conventional Base Pavement (Control), Calculated from Full-Width Transverse Profiles of 3.6 Meters

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depths	263	.118	.551	.287	.079
Right Wheel Path Rut Depths	263	.118	.748	.309	.118

TABLE 25 Descriptive Statistics for Rut Depths Measured on S.R. 695 WB over Conventional Base Pavement Calculated from Full-Width Transverse Profiles of 3.6 Meters

	N	Minimum	Maximum	Mean	Std. Deviation
Left Wheel Path Rut Depths	187	.118	.630	.363	.089
Right Wheel Path Rut Depths	187	.118	.591	.281	.100

Photolog Rideability (International Roughness Index)

Longitudinal pavement profiles were measured in each wheel path during the years 2000, 2002, 2004, 2006, and 2008. International Roughness Index (IRI) values were calculated from these profiles. Results for left and right wheel paths are presented in Table 26 for both the eastbound and westbound directions. As anticipated, IRI values generally increased over time, but not excessively (See Appendix H).

TABLE 26 IRI (inches/mile) Measured on S.R. 695 Pavement over CIR Base

Year	Left Mean IRI, EB, Pavement over CIR Base (in/mile)	Right Mean IRI, EB, Pavement over CIR Base (in/mile)	Left Mean IRI, WB, Pavement over CIR Base (in/mile)	Right Mean IRI, WB, Pavement over CIR Base (in/mile)
2000	78.6	84.9	74.1	81.1
2002	80.5	87.4	76.7	84.9
2004	85.5	96.3	80.5	88.7
2006	85.5	95.7	80.5	87.4
2008	88.1	103.9	80.5	88.7

Table 27 presents IRI values measured over pavement on S.R. 695 where traditional paving methods were used (control sections). Notice that the values are similar to those for the pavement over CIR base. The average of the 2008 IRI values for the pavement with CIR base (Table 26) was 90.3 inches/mile. Similarly, the average of the 2008 IRI values for the pavement without CIR base (Table 27) was 94.7 inches/mile. In order to synchronize the control pavement (no CIR) in time to the CIR base pavement, compare "Control" 2004 IRI values to "CIR Base" 2008 values. The average 2004 "Control" IRI was 92.2 in/mile after 10 years of service versus 90.3 inches/mile for the pavement with CIR base after 10 years of service.

TABLE 27 IRI (m/km) Measured on S.R. 695 Pavement not over CIR Base (Control)

Year	Left Mean IRI, EB, Control Pavement (in/mile)	Right Mean IRI, EB, Control Pavement (in/mile)	Left Mean IRI, WB, Control Pavement (in/mile)	Right Mean IRI, WB, Control Pavement (in/mile)
2000	93.9	88.2	82.2	73.7
2002	98.1	89.0	81.1	78.8
2004	103.5	95.1	88.4	81.7
2006	106.4	92.0	92.1	80.0
2008	103.6	98.1	86.7	90.5

WiseCrax Analysis

Photolog WiseCrax software was used to analyze cracking of the low-speed lane of S.R. 695. 2008 data were analyzed first. The control pavement (mileage less than 1.08 miles) was already crack sealed by 2008, while the CIR pavement was not crack sealed (see Figure 30). This may have biased the results, as sealed cracks are more readily identified, and appear more severe than if they are not crack sealed.

FIGURE 30 Photo showing that the control pavement was crack sealed, while the CIR pavement was not in year 2008.

Tables 28 and 29 present descriptive statistics for the Number of Cracks, Total Crack Length, and Average Crack Width per 5 meter segment of pavement (low-speed lane only) on S.R. 695. In the eastbound direction (Table 28) for the pavement over CIR base, there were a total of 875 cracks detected by WiseCrax. The average length of these cracks was 1.1 meters, and the average width was 7.7 mm. This amounts to 499 feet per lane-mile of pavement. Compare this to the control section in the eastbound direction, which had 2,812 cracks for a cumulative length of 1,665 meters. This amounts to 2,604 cracks per lane-mile, or 5,463 ft per lane-mile of pavement. Note again however that the control pavement was crack sealed, so it probably is not a fair comparison, because the software may identify sealed areas as more severe.

The pavement over CIR base in the westbound direction had more cracks (1,936) than that found in the eastbound direction (875), but still substantially less than the control pavement in the westbound direction (5,671). The average crack length for pavement over CIR base in the westbound direction was 1.41 meters, and the average width was 6.86 mm. This amounts to 1,037 feet of cracks per lane-mile of pavement. For the westbound control pavement, the average crack length was 12.67 meters and the average width was 9.93 mm. 13,942 feet of cracks per mile were found for the control pavement in the westbound direction, but once again, it was crack sealed.

TABLE 28 2008 Descriptive Statistics for S.R. 695 Eastbound, Number of Cracks per 5 Meter Segment, Split between Control Pavement and Pavement over CIR Base

Split		Minimum	Maximum	Sum	Mean	Standard Deviation
Mileage < 1.080 Miles - No CIR Base (control)	Number of Cracks	0	29	2812	7.81	3.80
	Total Crack Length (m)	.34	16.88	1665.34	4.68	2.41
	Average Crack Width (mm)	2.90	16.30	NA	11.82	1.62
Mileage >= 1.080 Miles - CIR Base	Number of Cracks	0	9	875	.77	1.28
	Total Crack Length (m)	.32	6.12	519.90	1.14	1.32
	Average Crack Width (mm)	3.50	15.90	NA	7.72	1.96

TABLE 29 2008 Descriptive Statistics for S.R. 695 Westbound, Number of Cracks per 5 Meter Segment, Split between Control Pavement and Pavement over CIR Base

Split		Minimum	Maximum	Sum	Mean	Standard Deviation
Mileage < 1.058 Miles - No CIR Base (control)	Number of Cracks	4	39	5671	15.97	6.23
	Total Crack Length (m)	3.42	29.29	4496.06	12.67	4.71
	Average Crack Width (mm)	6.30	14.80	NA	9.93	1.16
Mileage >= 1.058 Miles - CIR Base	Number of Cracks	0	21	1936	1.72	2.09
	Total Crack Length (m)	.32	12.84	1095.40	1.41	1.58
	Average Crack Width (mm)	3.80	14.40	NA	6.86	1.64

These differences can easily be seen graphically in Figures 31 and 32, where a simple line chart plots total crack length per 5 meter segment versus mileage. The pavement over CIR base begins at 1.08 miles in the eastbound direction. At that point, a severe drop in the total crack length per 5 meter segment can be seen. The average drops from 4.7 meters per 5 meter segment to 1.1 meter per 5 meter segment. In the westbound direction, it drops from 12.7 meters per 5 meter segment to 1.4 meters per 5 meter segment.

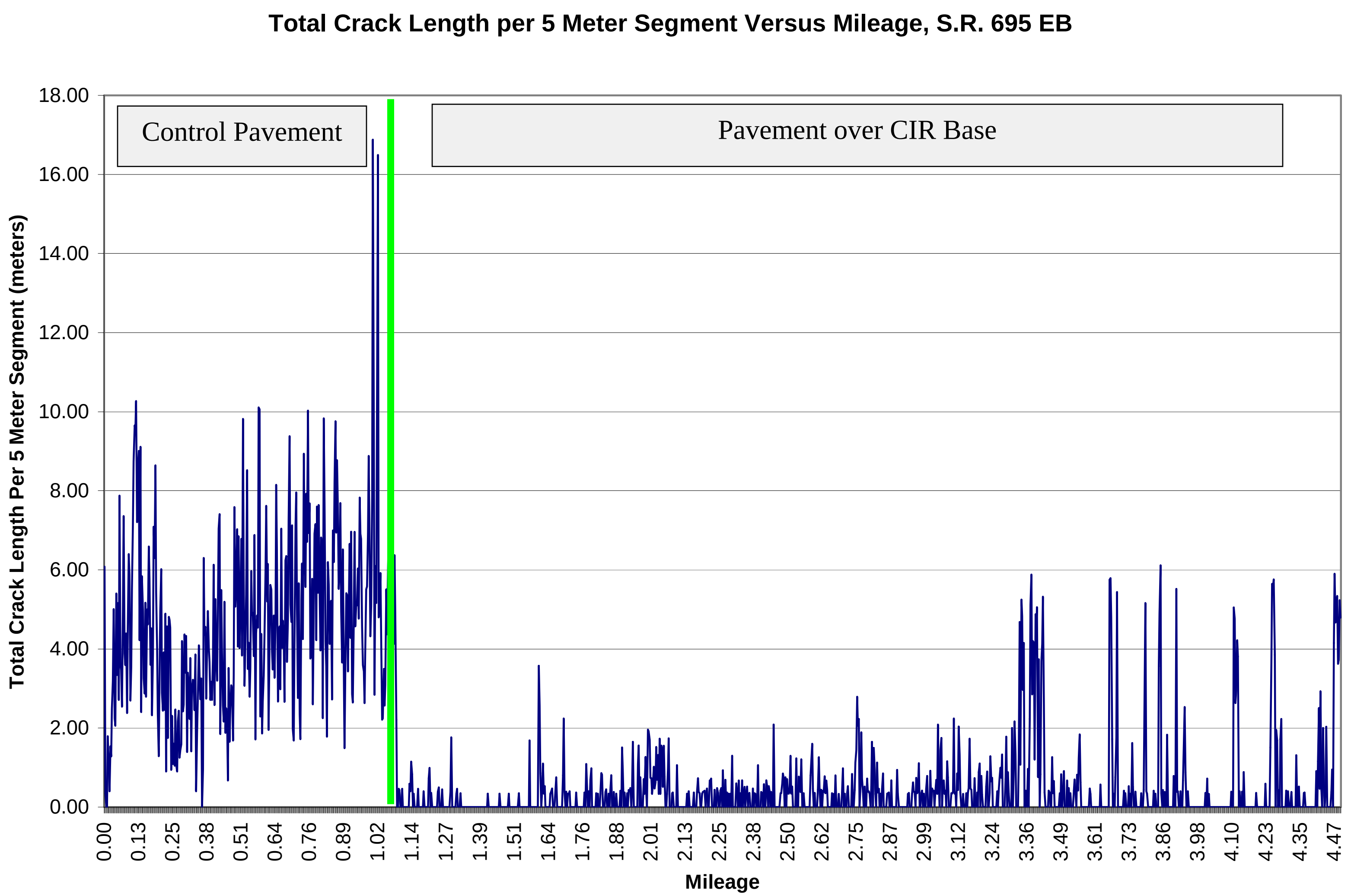


FIGURE 31 Total 2008 crack length per 5 meter segment versus mileage for the low speed lane of S.R. 695 eastbound.

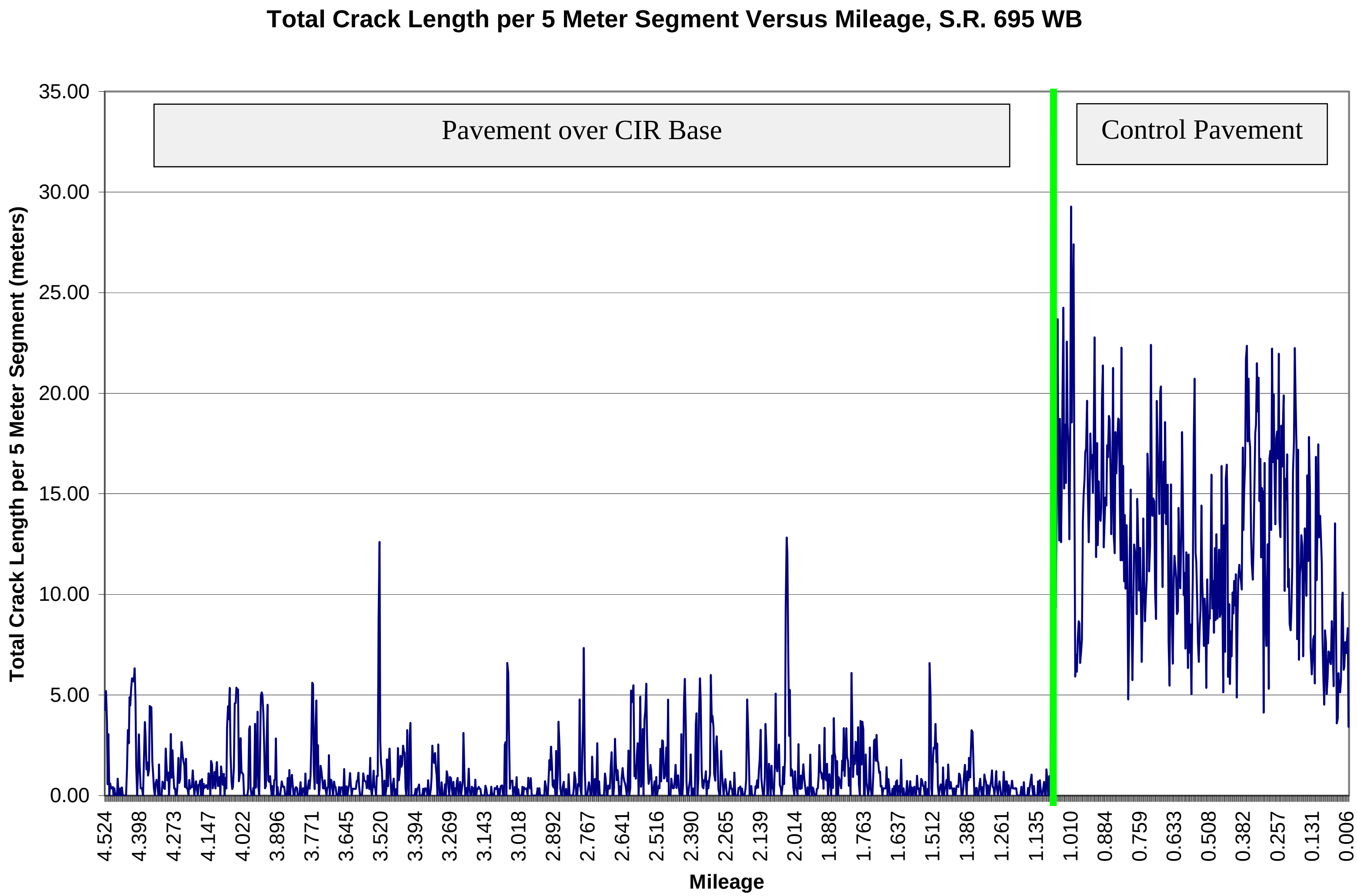


FIGURE 32 Total 2008 crack length per 5 meter segment versus mileage for the low speed lane of S.R. 695 westbound.

Next, 2004 WiseCrax data were analyzed in order to eliminate the apparent bias caused by the application of crack sealant. The control pavement had not yet been crack sealed when it was Photologged in 2004. In looking at the descriptive statistics for these data and comparing them to those in 2008, it can be seen that there were considerably fewer cracks overall in 2004 than there were in 2008. It should be noted that data were reported on a 10 meter basis in 2004, whereas they were reported on a 5 meter basis in 2008. Therefore, the mean number of cracks reported in Tables 30 and 31 refer to the average number of cracks per 10 meter length. These same tabulated values in Tables 28 and 29 refer to average number of cracks per 5 meter length.

In the eastbound direction in 2004, there were 2,290 ft of cracks per lane-mile of control pavement. Compare this to 5,463 ft of cracks per lane-mile of control pavement in 2008. The 2008 pavement was crack sealed, whereas the 2004 pavement was not crack sealed, so many of the additional cracks detected in 2008 owe to the existence of the crack sealant. There were 370 ft of cracks per lane-mile of CIR pavement in 2004. Compare this to 499 ft of cracks per lane-mile in 2008, which seems a reasonable increase due to the additional 4 years of service. Both the 2004 and 2008 CIR pavements were free of any crack sealant.

In the westbound direction, there were 2,068 ft of cracks per lane-mile of control pavement. This is considerably less than that detected in 2008 (13,942 ft). The CIR pavement had just 280 ft of cracks per lane-mile in 2004, versus 1,037 ft per lane-mile in 2008.

In looking at these descriptive statistics and also at Tables 30 and 31, it can be seen that less cracking existed on the CIR pavement than the control pavement in 2004. This can also be seen graphically in Figures 33 and 34.

TABLE 30 2004 Descriptive Statistics for S.R. 695 Eastbound, Number of Cracks per 10 Meter Segment, Split between Control Pavement and Pavement over CIR Base

Split		Minimum	Maximum	Sum	Mean	Standard Deviation
Mileage < 1.080 Miles - No CIR Base (control)	Number of Cracks	0	29	1073	6.10	4.96
	Total Crack Length (m)	.36	21.86	753.79	4.49	4.23
	Average Crack Width (mm)	3.2	16.3	NA	7.37	1.77
Mileage >= 1.080 Miles - CIR Base	Number of Cracks	0	12	497	.86	1.67
	Total Crack Length (m)	.34	25.01	402.02	2.00	3.49
	Average Crack Width (mm)	2.3	17.2	NA	7.74	2.43

TABLE 31 2004 Descriptive Statistics for S.R. 695 Westbound, Number of Cracks per 10 Meter Segment, Split between Control Pavement and Pavement over CIR Base

Split		Minimum	Maximum	Sum	Mean	Standard Deviation
Mileage < 1.058 Miles - No CIR Base (control)	Number of Cracks	0	18	980	5.80	3.98
	Total Crack Length (m)	.35	14.39	668.25	4.10	3.12
	Average Crack Width (mm)	4.7	11.5	NA	7.00	1.16
Mileage >= 1.058 Miles - CIR Base	Number of Cracks	0	11	435	.82	1.66
	Total Crack Length (m)	.35	10.53	294.22	1.68	1.80
	Average Crack Width (mm)	3.4	15.4	NA	7.52	1.86

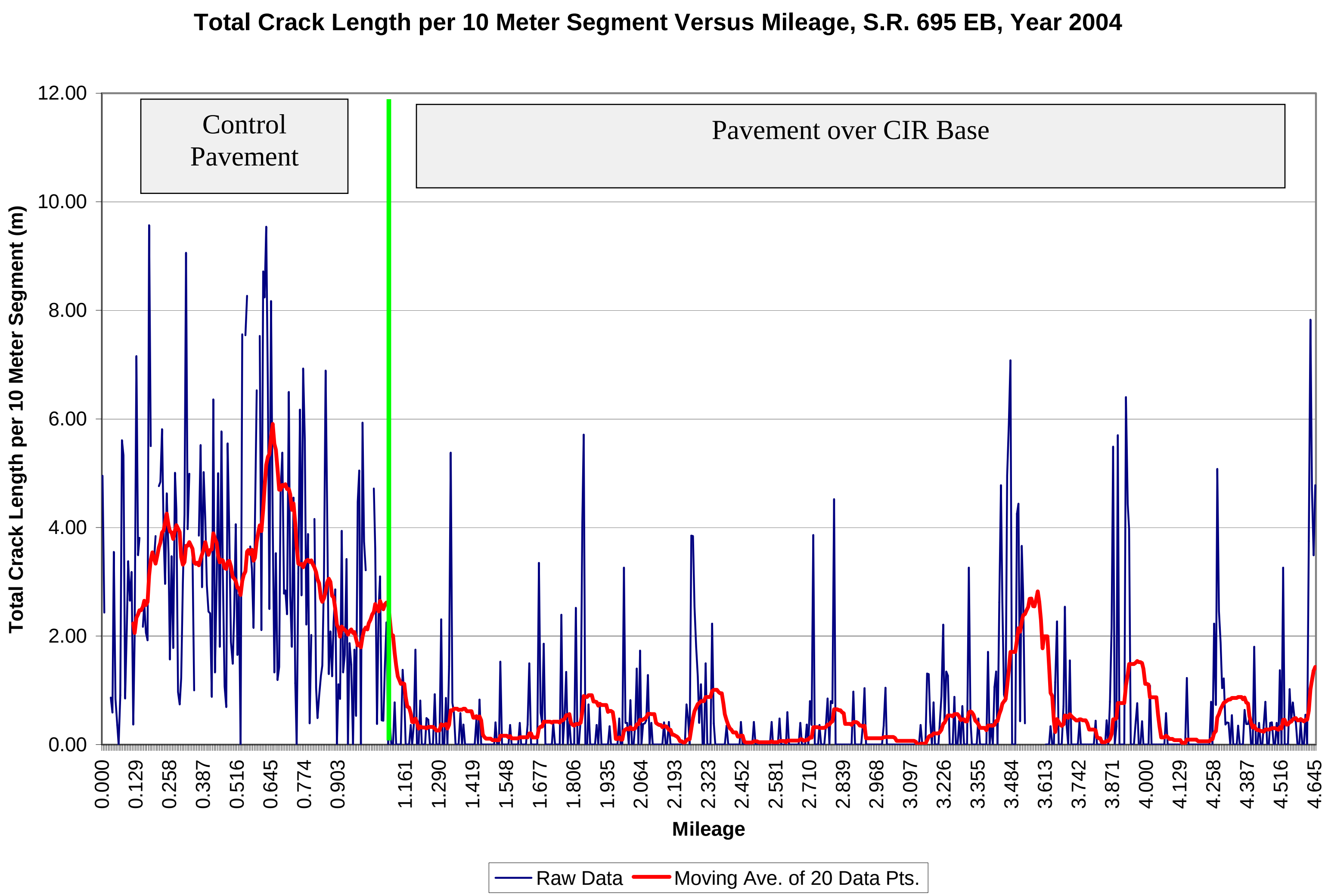


FIGURE 33 Total crack length per 10 meter segment for S.R. 695 EB, year 2004, no filters set (all data). Note: severe spikes, such as between 3.484 and 3.613 miles, often indicate invalid data.

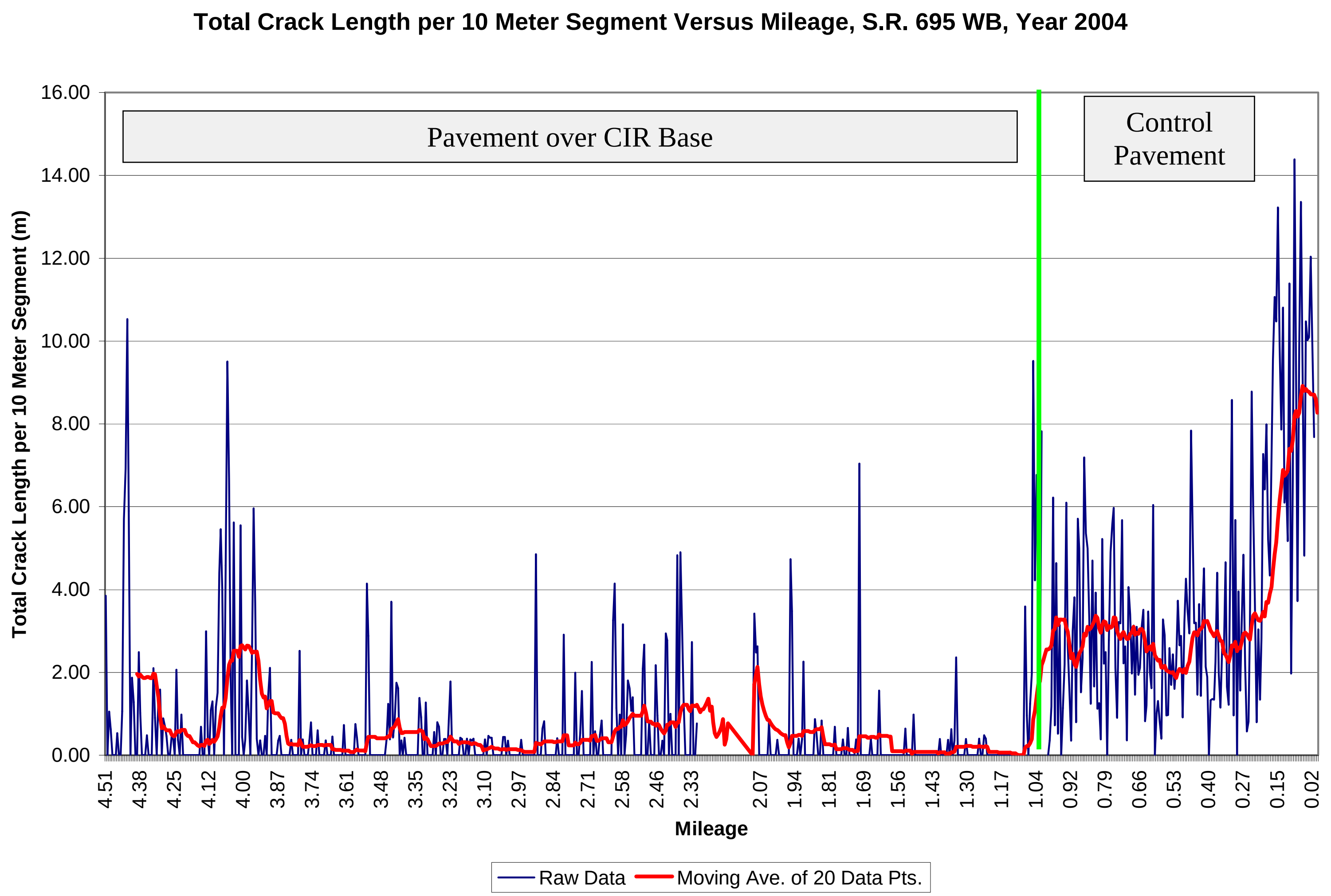


FIGURE 34 Total crack length per 10 meter segment for S.R. 695 WB, year 2004, no filters set (all data). Note: severe spikes, such as between 2.33 and 2.07 miles, often indicate invalid data.

Another problem with these comparisons between the pavement over CIR base and the control pavement is that the pavement over CIR base was 10-years old when these data were collected, whereas the control pavement was 14-years old. Accordingly, data for 2004 control pavement were merged with the 2008 data for CIR pavement. This synchronized the two pavements in time. The control pavement had not yet been crack sealed by 2004, so this bias was also removed. In the eastbound direction, this meant that year 2008 cases where the mileage was less than 1.08 miles were replaced with year 2004 data. This can be seen in Figure 35, as the part of the graph to the left of the green line represents 2004 data, while the part to the right of the green line represents 2008 data. The blue line plots the total crack length per 10 meter segment, while the red line plots its moving average over 20 cases. The purpose of the graph is to demonstrate the general trend of there being more cracks in the control pavement than the pavement in question.

In looking at Figure 35, it appears this was accomplished because it clearly shows that there was more cracking in the control pavement after 10 years than there was for the pavement in question (CIR base) after 10 years. In general terms, looking at the red line, the moving average tends to range between 1.00 and 1.50 for the control, while it ranges approximately between 0.50 and 1.00 for the CIR pavement. Therefore, it appears the CIR treatment was effective in mitigating reflective cracking, as evidenced by WiseCrax data after 10 years of service for each the control and the CIR pavement.

Descriptive statistics previously presented in Tables 28 through 31 can be used to quantify cracking after 10 years of service. After 10 years of service in the eastbound direction, there were 2,290 ft of cracks per lane-mile for the control (from 2004 data), while the CIR pavement had 499 ft per lane-mile after 10 years of service (from 2008 data). After 10 years of service in the westbound direction, there were 2,068 ft per lane-mile for the control (from 2004), and there were 1,037 ft per lane-mile for the CIR pavement (from 2008). These descriptive statistics bear out observations made in looking at the graphs in Figures 35 and 36, insofar as it supports that the CIR treatment was effective in mitigating cracking after 10 years of service.

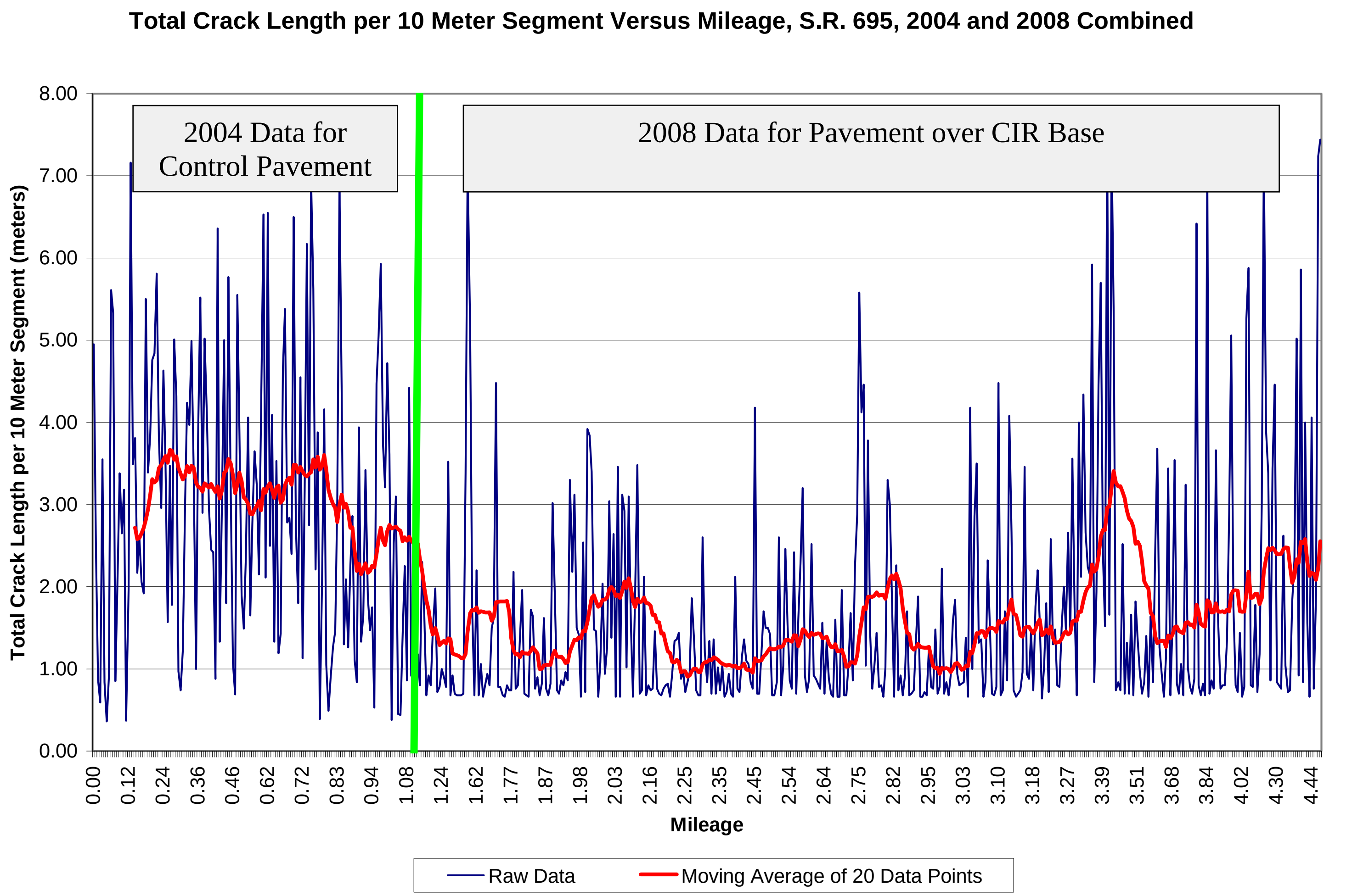


FIGURE 35 S.R. 695 EB Crack length data for 2008 pavement over CIR base (>1.08 miles) merged with 2004 control pavement data (<1.08 miles), crack lengths greater than 7.5 meters filtered-out. Note that the 2004 control data ranges approximately between 2.00 and 4.0 meters, while the CIR pavement ranges approximately between 1.0 and 3.0 meters.

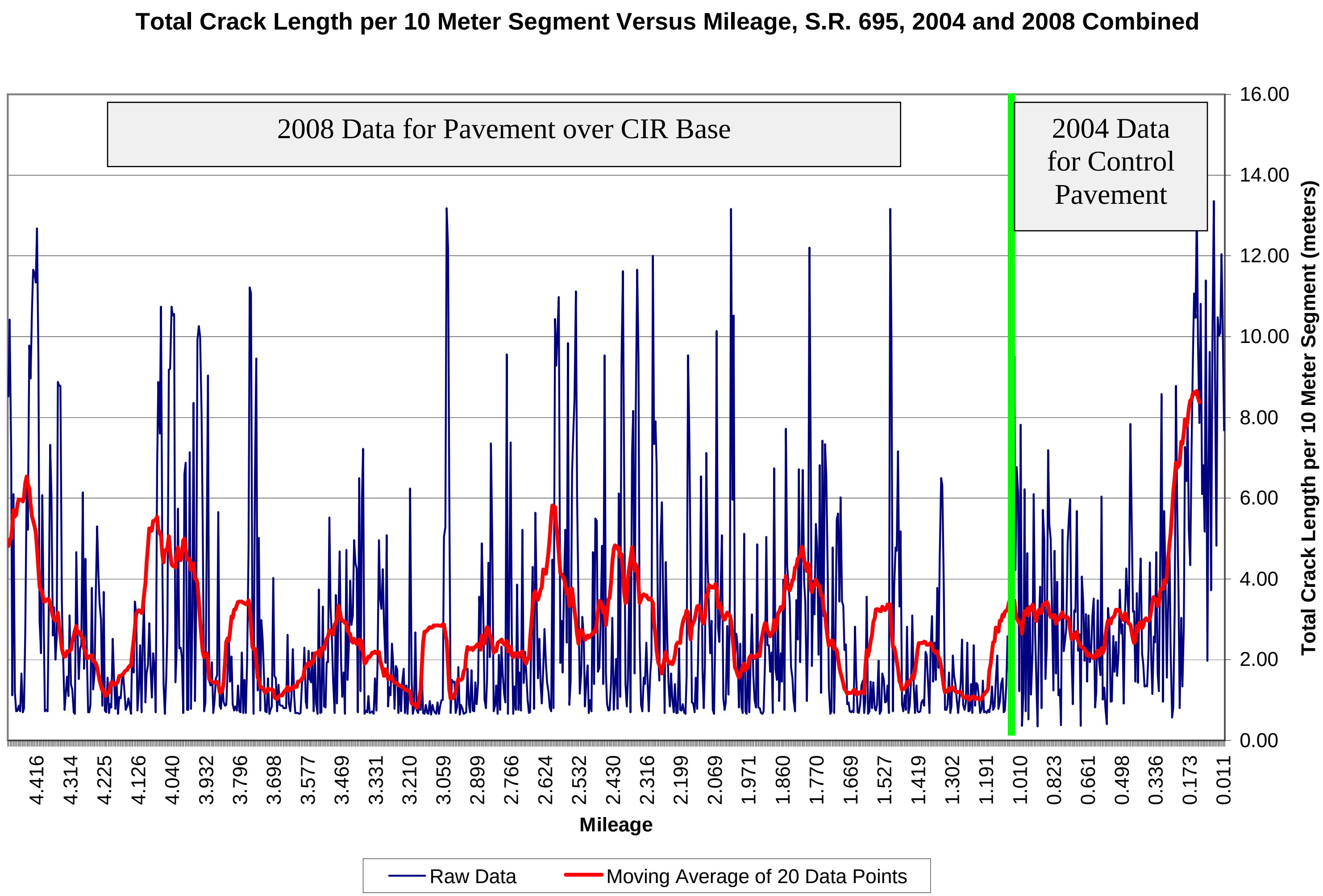


FIGURE 36 S.R. WB Crack length data for 2008 pavement over CIR base (>1.06 miles) merged with 2004 control pavement (<1.06 miles).

To conclude this section, Figure 37 shows an image file from WiseCrax taken from the Photolog van. Each of these images represent a 10 meter section of pavement in the lane traveled, which in this case was the low-speed lane. This particular image was taken in Section S2 at Station 1+00 (see the hash mark with the number 1 beside it). The image captures the entire width of the lane, from the dashed line between lanes to the shoulder line.

Figures 38 and 39 plot total crack lengths per 10 meter segment versus mileage for S.R. 695 eastbound (low-speed lane) for the years 2002 and 2001, respectively. These data were collected prior to any crack sealant material being applied to the pavement. Once again, it can be seen that the control pavement had more cracking than the CIR pavement, although the control pavement was 4 years older than the CIR pavement.

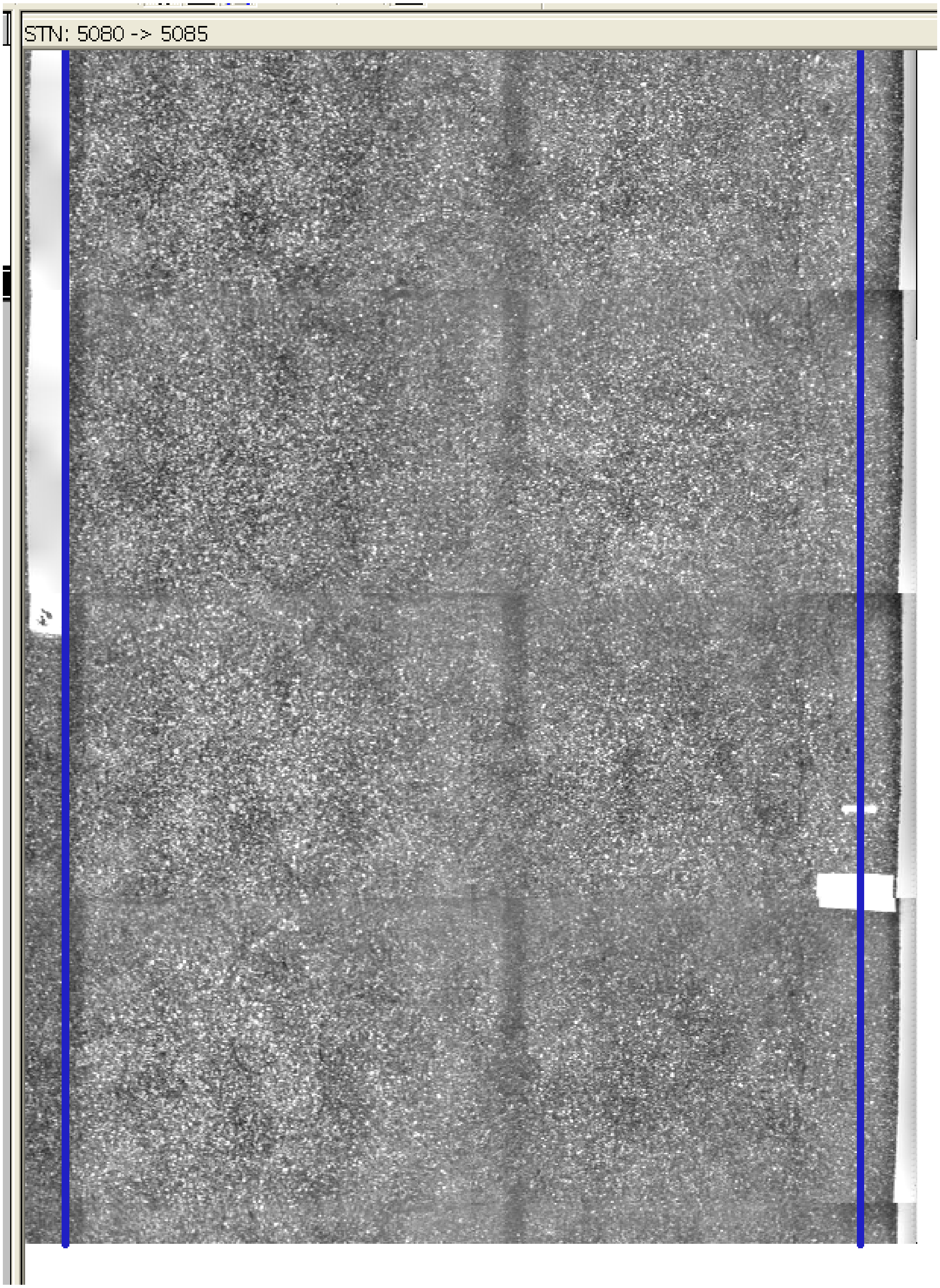


FIGURE 37 Wisecrax image file from chainage 5.080 to 5.085 kilometers, which corresponds with mileage 3.135 to 3.142 on S.R. 695 EB where Site S2 is located at Station 1+00.

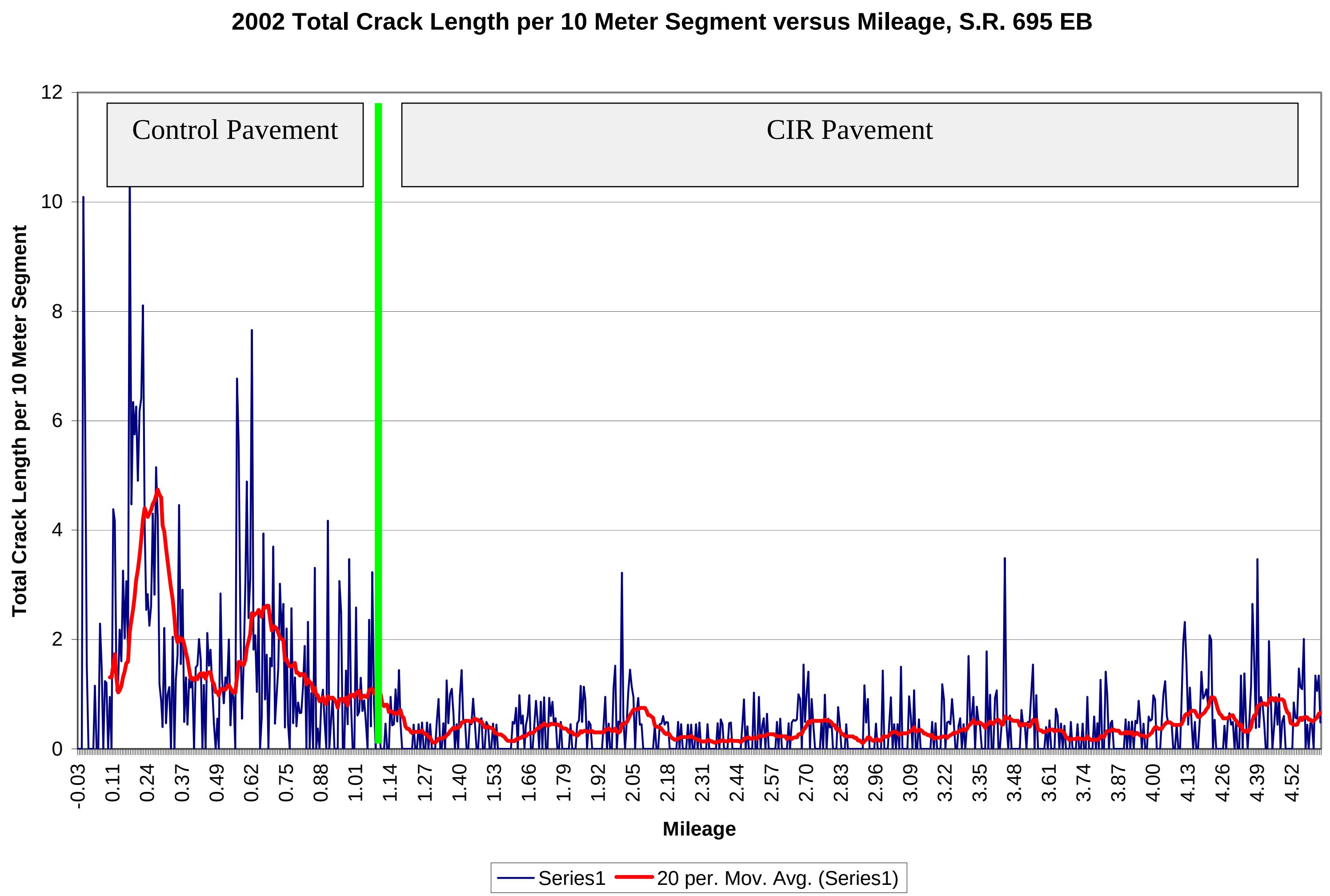


FIGURE 38 Total crack length per 10 meter segment versus mileage on S.R. 695 EB for 2002.

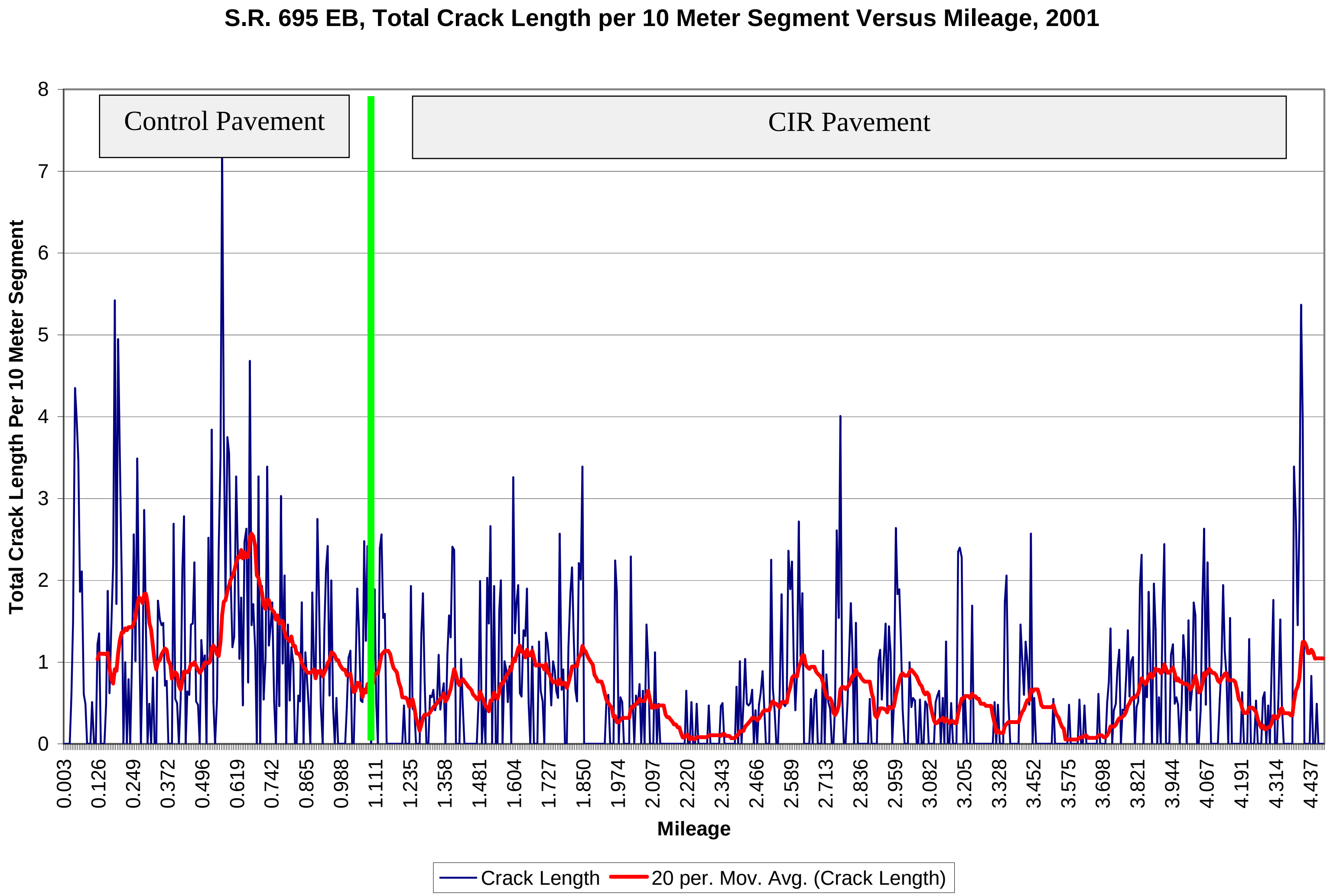


FIGURE 39 Total crack length per 10 meter segment versus mileage on S.R. 695 EB for 2001.

LIFE-CYCLE COST ANALYSIS

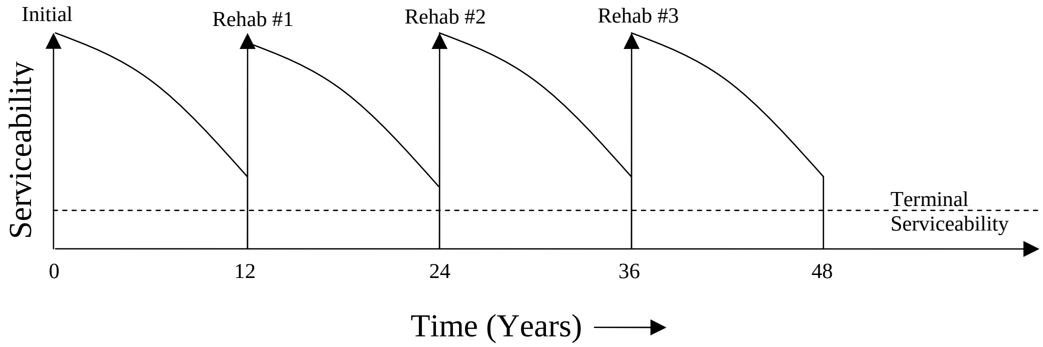
Two options were examined for a life-cycle cost analysis. The first option includes an initial CIR treatment of 3 inches of existing pavement, followed by a tack coat, and then a 3-inch Superpave overlay. The first rehabilitation was estimated for year 12, prior to reaching the pavement's terminal serviceability, before the pavement deteriorates to a point where greater repairs would be necessary. This was in keeping with what is actually proposed for S.R. 695, as PM personnel have indicated that they plan to overlay the existing pavement with 3 inches of Superpave next year (year 12). Rehabs #2 and #3 are estimated for years 24 and 36, which include milling and then overlaying in the same manner as for Rehab #1. A twelve year cycle for these rehabs was estimated because it appears the reflective cracking issue was addressed with the initial CIR treatment.

Compare this to Option #2, which does not include a CIR treatment. An 8 year cycle was assumed for this option because it was projected that reflective cracks would develop quickly without the CIR treatment, thereby decreasing the rehab's estimated life-cycle.

For this analysis, a currency interest rate of 1% was assumed for determining future costs, and a discount rate of 4% was used to determine present worth. Once the present worth was determined, it was annualized with a currency inflation rate of 1%.

The Option #1 AAC was calculated to be $10,515 per lane-mile per year, whereas the Option #2 AAC was $16,717 per lane-mile per year. By mitigating the reflective cracking problem with the CIR treatment, this model predicts a savings of $6,202 per lane-mile per year. Of course, many assumptions were made. The most significant assumption was that a 12 year rehabilitation cycle exists for Option #1, while an 8 year cycle exists for Option #2.

Option #1



1. Initial: CIR w/ 2" Superpave Overlay
   a. CIR 3 inches pavement for one lane-mile
      7,040 s.y. @ 3.15/s.y. = $22,176
   b. 2 inches HMA Superpave Overlay
      (7040 s.y.)(115/2000)(2 inches)($125/ton) = $101,200
   c. Tack Coat
      Assume $7.50/gal where 0.1 gallon covers a s.y.
      Total tack coat cost estimate =
      (7040 s.y.)($7.50/gal)(0.1 gal/s.y.) = $5,280

      Total of a, b, and c = $128,656/lane-mile

2. Rehab #1: 3 inch Superpave Overlay @ Year 12
   a. 3 inch Superpave Overlay
      (7040 s.y.)(115/2000)(3 inches)($125/ton) = $151,800
   b. Tack Coat
      $5,280

      Total of a and b = $157,080/lane-mile

      Future value at year 12 using 1% currency inflation rate compounded monthly:
      $F = P(1+i)^n$ = ($157,080)(1 + .01/12)^{144} = $177,098

3. Rehab #2: Mill 2 to 3 inches and Overlay 3 inches HMA at Year 24
   a. Mill 7,040 s.y. @ $5.75/s.y.
      (7040 s.y.)($5.75/s.y.) = $40,480
   b. 3 inch Superpave Overlay = $151,800
   c. Tack Coat = $5,280

      Total of a, b, and c = $192,280

Future value at year 24 using 1% currency inflation rate compounded monthly:
$F = P(1+i)^n = (\$192,280)(1+.01/12)^{288} = \$244,411$

4. Rehab #3: Mill 2 to 3 inches and Overlay 3 inches Superpave at Year 36
   a. Mill 7,040 s.y. @ $5.75/s.y.
      (7040 s.y.)($5.75/s.y.) = $40,480
   b. 3 inch Superpave Overlay = $151,800
   c. Tack Coat = $5,280

   Total of a, b, and c = $197,560

   Future value at year 36 using 1% currency inflation rate compounded monthly:
   $F = P(1+i)^n = (\$197,560)(1+.01/12)^{432} = \$283,126$

Present worth using a discount rate of 4% compounded monthly:
$PW = \$128,656 + \$177,098(1/(1+.04/12)^{144}) + \$244,411(1/(1+.04/12)^{288}) + \$283,126(1/(1+.04/12)^{432})$
$PW = \$128,656 + \$109,673 + \$93,733 + \$67,241$

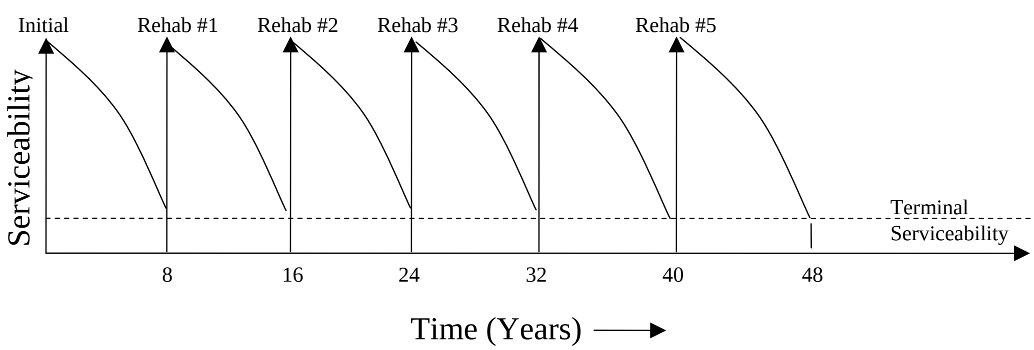
PW = $399,303 per Lane-Mile

Alternatively, expressed in terms of average annual cost (AAC) using a 1% currency inflation rate compounded yearly for 48 years:

$AAC = (PW)(i(1+i)^n)/((1+i)^n-1) =$
$(\$399,303)((.01)(1+.01)^{48})/((1+.01)^{48}-1)$

AAC = $ 10,515 per lane-mile per year

Option #2



1. Initial: Overlay the existing pavement with 3 inches of Superpave
   a. 3 inch Superpave Overlay
      (7040 s.y.)(115/2000)(3 inches)($125/ton) = $151,800
   b. Tack Coat
      $5,280

      Total of a and b = $157,080/lane-mile

2. Rehab #1: Overlay the existing pavement with 3 inches of Superpave @ 8 years

   Total = $157,080/lane-mile

   Future value at year 8 using 1% currency inflation rate compounded monthly:
   $F = \$157,080(1+.01/12)^{96} = \$170,157$

3. Rehab #2: Mill 3 inches and overlay with 3 inches of Superpave.
   a. Mill 7,040 s.y. @ $5.75/s.y.
      (7040 s.y.)($5.75/s.y.) = $ 40,480
   b. 3 inch Superpave Overlay = $151,800
   c. Tack Coat = $ 5,280

      Total of a, b, and c = $197,560

   Future value at year 16 using 1% currency inflation rate compounded monthly:
   $F = \$197,560(1+.01/12)^{192} = \$231,823$

4. Rehab #3: Mill 3 inches and overlay with 3 inches of Superpave.

   Total = $197,560

   Future value at year 24 using 1% currency inflation rate compounded monthly:
   $F = \$197,560(1+.01/12)^{288} = \$251,123$

5. Rehab #4: Mill 3 inches and overlay with 3 inches of Superpave.

   Total = $197,560

   Future value at year 32 using 1% currency inflation rate compounded monthly:

$F = \$197,560(1+.01/12)^{384} = \$272,029$

6. Rehab #5: Mill 3 inches and overlay with 3 inches of Superpave.

   Total = $197,560
   Future value at year 40 using 1% currency inflation rate compounded monthly:
   $F = \$197,560(1+.01/12)^{480} = \$294,676$

Present worth using a discount rate of 4% compounded monthly:

$PW = \$157,080 + \$170,157(1/(1+.04/12)^{96}) + \$231,823(1/((1+.04/12)^{192}) + \$251,123(1/(1+.04/12)^{288}) + \$272,029(1/(1+.04/12)^{384}) + \$294,676(1/(1+.04/12)^{480})$

PW = $157,080 + $123,625 + $122,369 + $96,307 + $75,795 + $59,653

PW = $634,829 per lane-mile

Alternatively, annualized with a 1% currency inflation rate compounded yearly for 48 years:
$AAC = (\boldsymbol{i}(1+\boldsymbol{i})^{n})/((1+i)^{n}-1) =$
$(\$634,829)((.01)(1+.01)^{48})/((1+.01)^{48}-1)$

AAC = $16,717 per lane-mile per year

IMPLEMENTATION

On February 9, 2009, the Principal Investigator (PI) for this study met with Connecticut State Representative Steven Mikutel at the Legislative Office Building in Hartford along with ConnDOT’s Deputy Commissioner of Transportation, Chief Engineer, Legislative Program Manager, and a Maintenance/Planning official. Representatives from Gorman Brothers, Inc. were also in attendance to promote the use of CIR. State Representative Mikutel is a Democrat legislator representing the 45th Assembly District, including the towns of Griswold, Lisbon, Plainfield and Voluntown. He is House vice chairman of the legislature’s Transportation Committee.

The PI presented research results included in this report. He indicated that overall, the CIR pavement treatment appears to be outperforming the previous treatment in 1987 with conventional paving methods and stated that the estimated pavement lifespan of 7 to 12 years will likely be exceeded. He said that this study as well as literature reviewed demonstrates that CIR treatment can mitigate reflective cracking. However, he did state that increased potential for rutting may be a side-effect, and recommended limiting CIR treatment to roads with ADTs of 8,000 or less, at least until the Department gains more experience with its use.

It was decided that the Department would select approximately four different construction projects, one from each District, to apply CIR treatment. The Department's Pavement Management section would determine which pavements were most suitable for this application.

DISCUSSION

The results of this study validate some of the recommendations made in a document titled, "Alternatives to the Conventional Overlay for Rehabilitation and Preservation of the State Highway Network" presented by a work group of ConnDOT personnel for presentation to ConnDOT executives on August 18, 2005 ***(10)***. Pages 12-14 of the abovementioned document describe the CIR treatment along with its pros and cons, costs, proper use, expected service life, and project-selection guidelines. A copy of these pages is provided in Appendix I.

The work group indicated that a benefit of a CIR treatment is that it can eliminate thermal crack patterns and their ensuing reflective cracking through surface layers. This was validated with the WiseCrax analysis presented in this report. It is indeed a viable alternative when reflective cracking is a concern.

CIR treatment was also tentatively shown to be cost effective, as presented through a life cycle cost analysis provided in this report, although a more detailed analysis with up-to-date cost data should be conducted.

They recommended limiting its use to pavements with traffic volumes <= 8,000 ADT. Considering the low densities, finer gradation, and rutting on sections of positive grades (uphill), this recommendation should be heeded. Traffic volumes on S.R. 695 were less than 5,000 ADT, and still sections of moderate rutting were observed. The literature reviewed and presented in this paper also supports limiting traffic volumes to such levels. Perhaps if higher densities can be achieved and demonstrated through future CIR treatments, and evidence of excessive rutting is not observed, ConnDOT can experiment with progressively higher ADTs. At this time, however, it is not advisable.

It is also recommended that every effort be made to locate longitudinal construction joints of the HMA overlay to between the lanes, along the line striping. It was observed that when these joints were located in the wheel paths, rut depths were more severe, especially for steep positive grades (>4% uphill). Joints were often located in the right wheel path of S.R. 695 whereas they were not observed in the left wheel path. Consequently, right wheel path rut depths were greater than left wheel path rut depths. This likely owes to water infiltrating through the joints and creating freeze thaw conditions, leading to a less stable pavement structure in the vicinity of the joint.

The Connecticut Transportation Institute (CTI) of the University of Connecticut School of Engineering recommends sealing pavement cracks in order to extend pavement life ***(11)***. They recently reprinted an article published by PennDOT ***(12)***. This article describes an economical pavement maintenance technique of sealing pavement cracks to prevent water from entering the base and subbase.

For a successful maintenance program of this type, pavement selection is critical ***(12)***. PennDOT suggests selecting newer pavements, where cracks are just beginning to form. Pavements that have more severe cracking are too far advanced. Next, they suggest using a "…flexible rubberized asphalt that bonds to the crack walls and moves with the pavement" to prevent water intrusion. They suggest that agencies perform a cost effectiveness analysis before choosing a product because modified and proprietary products are typically more expensive. Before application, they recommend thorough preparation of the surface, which is "…accomplished with compressed air (100 psi minimum) and a simple blowpipe." A hot-air lance should be used if the dirt in the cracks is wet. They also cited studies which have indicated that there is a 40 percent greater chance of success if the cracks are routed prior to sealing them in order to allow the sealant material to penetrate the voids.

CONCLUSIONS

- The CIR rehabilitation was a successful treatment that mitigated reflective cracking. This was demonstrated and quantified via a WiseCrax analysis of the pavement. The CIR rehabilitated pavement had 65% less cracking than the adjacent control pavement after 10 years of service for each.
- Overall, rut depths were 10% less severe for the CIR rehabilitated pavement than for the control pavement; however, where longitudinal joints were located in the wheel path, CIR treated pavement rut depths were 83% more severe than control pavement rut depths.
- There was a correspondence between roadway grade and rut depths. CIR pavement rut depths were 60% to 183% more severe on uphill grades ≥ 4% than downhill grades ≥ 4%. Although rut depths tended to be greater on steep uphill grades, these depths were not excessive, except for instances where longitudinal joints where located in the wheel path. This condition tended to exacerbate rutting to the extent that moderate levels (0.75 inches deep) existed in these sections.
- The density of CIR base remains a concern, as tests of cores yielded densities of between 80% and 90% of the maximum theoretical density. Note that lower density is inherent in CIR pavements, since preexisting air voids exist in the recycled material. These lower densities were not the result of poor construction methods, and the contractor communicated this to ConnDOT personnel prior to commencing work.
- Due to finer gradation and lower densities, rutting remains a concern for CIR pavements, regardless of whether or not lower densities are inherent in CIR materials.
- No significant differences in rideability between the CIR pavement and control pavement were found, as IRI values between these pavements were similar to one another.
- For selected applications, a life-cycle cost pavement analysis suggests 37% cost savings for CIR treated pavements vs. traditionally treated pavements over 48-year analysis period.

RECOMMENDATIONS

- Follow the project selection guidelines, developed in 2005 by ConnDOT's Pavement Preservation Workgroup (see Appendix I).
- Continue to limit the application of CIR treatment to pavement receiving fewer than 8,000 ADT.
- Following these guidelines, select additional lightly-traveled pavements with reflective cracking for CIR treatment of the base.
- During HMA paving over a CIR base, avoid locating longitudinal joints along wheel paths, because longitudinal cracks tend to develop along these joints, which exacerbate rutting, as water enters the cracks and undermines and weakens the CIR base.
- Include crack sealing as part of a pavement preservation program. The program should include selecting the proper sealants, preparation and application methods, and pavement selection.
- In future CIR projects, document the before and after conditions on these new projects to facilitate future monitoring and interpretation of the performance of this preservation treatment. Especially during HMA paving over a CIR base, avoid locating longitudinal joints along wheel paths to reduce future rutting.
- Through future CIR projects in Connecticut, develop experience with this preservation technique and gradually explore its application to pavement with higher ADT levels. Monitor subsequent rutting on these pavements to explore the viability of raising the ADT cutoff level in future revisions to the CIR project selection guidelines.

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APPENDIX A

5/3/95

Project No. 68-H006
Resurfacing and Safety
Improvements on S.R. 695
Town of Killingly

May 3, 1995

Mr. Thomas A. Harley
Principal Engineer
Bureau of Engineering and
Highway Operations

Colleen A. Kissane for
Joseph J. Obara
Manager of Design Services
Bureau of Engineering and
Highway Operations

As requested by your memorandum dated April 12, 1995, engineers from the Pavement Management section have investigated the pavement design requirements for the above captioned project.

This project involves resurfacing and safety improvements on S.R. 695 in Killingly from Ross Road easterly to U.S. Route 6 (MP 0.96 to MP 4.49).

The existing pavement is composed of 1.5 in. Bituminous Concrete Class 1 overlay (1987) on 1 in. Bituminous Concrete Class 138 surface treatment (1967) on the original 1958 flexible pavement structure. This original pavement structure is unknown since the plans indicate that a change was made during construction.

The pavement is distressed in the form of block cracking. This distress is a result of underlying block cracks reflecting up through the surface.

The 1986 photolog film shows signs of severe block/alligator cracking (See photolog print #1). Per our conversation with Mr. Richard Chick of District II Maintenance, the pavement below the 1987 overlay was very brittle (See photolog prints #2. #3, #4, #5, #6). Viewing these historical photolog images indicates that the initial cracking reflected through the 1987 overlay before 1990 (See photolog print #7). Sometime between 1990 and 1991, State Maintenance forces sealed all of the cracks to preserve the pavement structure.

In 1994, the block cracks have reached a moderate level of severity (see photolog print #8). Comparing photolog images indicates that the cracking has increased by approximately 30% since the sealing operation (see photolog print #9). The red lines indicate the new cracks that have developed.

Based on the above information, three possible rehabilitation alternatives were considered. They are:

1. Overlay the existing pavement with 2 in. Bituminous Concrete Class 1, on 1 in. Bituminous Concrete Class 2.

2. Cold-In-Place recycle 4 inches of the existing pavement immediately followed by Microsurfacing.

3. Cold-In-Place recycle 4 inches of the existing pavement immediately followed by a 2 in. Bituminous Concrete Class 1 overlay.

The expected lifespan and cost for each option are summarized in the following chart. All costs are in 1994 dollars.

Option	Lifespan (years)	Cost (millions)
1	4-7	1.43 *
2	5-9	0.94 *
3	7-12	1.41 *

* see attached sheet for breakdown of costs

We recommend Alternative #3 for the following reasons:

1. Based on the past overlay performance, the first alternative only provides a minimal service life before requiring additional maintenance.

2. Cold-In-Place recycling will rejuvenate the existing brittle pavement as a means of extending the service life.

3. The Average Daily Traffic (ADT) in 1993 was 2,515 vehciles, which makes this site an excellent candidate for Cold-In-Place recycling.

4. We anticipate that this combination of recycle/overlay will provide the longest service life for the cost.

We will provide a list of detailed recommendations once a final decision has been made as to which alternative will be utilized.

If there are any further questions regarding this project, please contact Mr. David Kilpatrick at extension 3257.

Attachments

David J. Kilpatrick:kw
cc: Earle Munroe
Joseph J. Obara
Joseph A. Misbach
Pavement Management

OPTION I: Bituminous Overlay

1. Overlay the existing pavement with the following:

   2 in. Bituminous Concrete Class 1 on
   1 in. Bituminous Concrete Class 2

2. Estimated total cost - $1,425,950

   Total square yards observed - 158,000

   A. Bituminous concrete
      Assume $50.00/ton for the bituminous concrete.

      Total bituminous concrete cost = $1,362,750

   B. Tack coat
      Assume $2.00/gallon where 0.1 gallon covers a square yard. Also, assume a layer of tack coat is applied on the existing pavement and between the Bituminous Concrete Class 1 & 2.

      Total tack coat cost = $63,200

3. The life of the overlay is estimated to be between 4-7 years.

OPTION II: Cold-In-Place Recycling and Micro Resurfacing

1. Cold-in-place recycle 4 in. of the existing pavement and then Micro resurface.

2. Estimated total cost - $940,100

   Total square yards observed - 158,000

   A. Cold-in-place recycling
      Assume $3.00/square yard.

      Total cold-in-place recycling cost = $474,000

   B. Micro resurfacing
      Assume $2.75/square yard.

      Total micro resurfacing cost = $434,500

C. Tack coat
Assume $2.00/gallon where 0.1 gallon covers a square yard.

Total tack coat cost = $31,600

3. The life of the cold-in-place recycling with micro resurfacing is estimated to be between 5-9 years.

OPTION III: Cold-In-Place Recycling and Bituminous Overlay

1. Cold-in-place recycle 4 in. of the existing pavement and then place 2 in. Bituminous Concrete Class 1.

2. Estimated total cost - $1,414,100

   Total square yards observed - 158,000

   A. Cold-in-place recycle
   Assume $3.00/square yard.

   Total cold-in-place recycling cost = $474,000

   B. Bituminous concrete
   Assume $50.00/ton for the bituminous concrete.

   Total bituminous concrete cost = $908,500

   C. Tack coat
   Assume $2.00/gallon where 0.1 gallon covers a square yard.

   Total tack coat cost = $31,600

3. The life of the cold-in-place recycling with bituminous overlay is estimated between 7-12 years.

APPENDIX B

PRELIMINARY DESIGN STATEMENT
STATE PROJECT NUMBER 68-184
FEDERAL AID PROJECT NO. NH - 54(101)
STATE ROUTE 695 RESURFACING AND SAFETY
IMPROVEMENTS IN THE TOWN OF KILLINGLY

LOCATION: The project is located on Route 695 (Governor John Davis Lodge Turnpike). It begins at the Ross Road underpass and ends at the Conn. / R.I. border at the intersection with U.S. Route 6. The total length of the project is 19,554 feet (5,960m) or 3.70 miles.

DESCRIPTION: The purpose of this project is to improve safety and extend the service life of this section of Route 695. The improvements include:

- cold-in-place recycling of the existing pavement prior to resurfacing the road,
- upgrading the guide rail and stabilizing rock outcroppings by clearing tree growth and removing unstable rocks.

Bridge Number 312 over Quaduck Brook will require parapet modifications.

DESIGN CONSIDERATIONS: Rock removal is subject to Waiver Committee concurrence per memo "Clear Zone - Treatment of Rock Removal" from Mr. Earle R. Munroe on January 19, 1995.

HIGHWAY CLASSIFICATION: S.R. 695 - Rural Principal Arterial (Freeway)

RECORDED USAGE DATA: **1994 ADT:** 2600 @ mile 0.96

	EB	WB
Average Speeds (MPH) @ mile 0.96	56.1	56.3
85th Percentile Speed: @ mile 0.96	60.5	61.3

These speeds are from 1979 - No current speeds are available

POSTED SPEED: 55 mph (88 kph)

DESIGN SPEED: Proposed: 70 mph (112 kph)

DESIGN STANDARDS: "Connecticut Department of Transportation's Guidelines for Highway Design January 1990", supplemented by "AASHTO 1994".

Criteria selected: 4R (Freeway)

DESIGN ELEMENTS:		Design Standards	Proposed for Project
Control of Access		Full	Full
Lane Width - Min.		12'	12'
Shoulder Width - Min.	LT	6'	5' (3' paved 2' Graded) *
	RT	10'	10'
Maximum Grade		4 %	4 %
Stopping Sight Distance - Min.		725' (-4 %)	490' *
Intersection Sight Distance - Min.		N\A	N\A
Maximum Superelevation allowed		0.06	0.06
Minimum Radius		2083'	2291'
Clear Zone (Minimum)		30'	15'

* existing conditions

EXCEPTIONS TO DESIGN STANDARDS: The following Design Element requires a Design Exception:

1. Clear Zone -

   To achieve the proposed 15′ minimum clear zone, rock will be removed to eliminate protrusions that extend out farther than the adjacent rock face. The removal of loose rock (scaling) will also be done at several other locations. The chart below shows the extent of rock removal. A total of 5000 C.Y. of rock will be removed. Trees and brush on the rock face will also be removed to prevent further widening of any cracks.

   To provide a 30′ clear zone per design standards, approximately 68,000 C.Y. of rock would have to be excavated at an estimated cost of $1,400,000.

   Traffic accident data for the last six years indicates that two out of ten total accidents involve rock that had fallen into the roadway. No cars have hit the rock ledge.

WESTBOUND ROADWAY				EASTBOUND ROADWAY			
STA.	Approx. Clear Zone	Proposed Rock Removal For Clear Zone	Proposed Rock Removal For Stability	STA.	Approx. Clear Zone	Proposed Rock Removal For Clear Zone	Proposed Rock Removal For Stability
AREA 1				**AREA 2**			
187+00 (970 lf) 196+40	Varies 15′-25′	2 Locations (30 lf)	2 Locations (280 lf)	187+00 (970 lf) 193+50	Approx. 20′	2 Locations (120 lf)	1 Location (120 lf)
AREA 3				**AREA 4**			
242+00 (1450 lf) 256+50	Varies 15′-30′	4 Locations (200 lf)	4 Locations (370 lf)	241+30 (1480 lf) 256+50	Varies 15′-30′ (25′-40′)	4 Locations (330 lf)	3 Locations (130 lf)
AREA 5				**AREA 6**			
307+70 (1380 lf) 321+50	Varies 15′-25′ (25′-35′)	6 Locations (710 lf)	NONE	308+90 (1110 lf) 320+00	Varies 20′-25′	NONE	NONE
Totals (3770 lf)		10 Locations (940 lf)	6 Locations (650 lf)	Totals (3240 lf)		6 Locations (450 lf)	4 Locations (250 lf)

Approx. Clear Zone - Indicates the existing clear zone ***EXCLUSIVE*** of rock outcrops and protrusions. The proposed (post construction) clear zone, without exclusions, after the rock removal is shown in parenthesis if different than the existing condition.

Proposed Rock Removal - Rock Removal is proposed for two safety purposes:

1. For Clear Zone - Remove outcrops, protrusions, snags and enhance the overall clear zone of the rock face.
2. For Stability - Remove loose rocks, overhangs created from differential rock weathering to lessen rockfall potential.

As stated in the Geometric Design Elements of Existing Highways (4R Design Criteria), the following Design Elements do not require Design Exceptions but will be included for information only since they do not meet current design standards:

Shoulder Width -

Allow the existing condition of a 5' left shoulder (3' paved and 2' graded) to remain within the original two lane section of the highway.

Currently, two sections of S.R. 695 have climbing lanes:

- Eastbound - Sta. 208+50 to Sta.288+50 (8,000') and
- Westbound - Sta. 277+00 to Sta. 340+00 (6,300')

This project proposes the elimination of these climbing lanes which will increase the shoulder widths within this section of roadway. The climbing lanes consist of a 12' lane and a 3' shoulder. This 15' will be used as a 10' right shoulder with the remaining 5' added to the left shoulder for a total left shoulder width of 10' (8' paved, 2' graded).

Design standards call for a 6' left shoulder in a 2 lane section. Removal of the climbing lane will not lower the highways Level of Service (service stays at LOS A) since the ADT is so low. } check As builts

To provide a wider left shoulder in the remaining portion of the highway (3700'), the median would require paving with the installation of P.C.B.C. The cost to pave the median, provide median barrier curb and install drainage is approximately $1,000,000 and would most likely require a Inland Wetland Permit Application.

2. Stopping Sight Distance -

   Allow the existing minimum SSD of 486' to remain. Three crest vertical curves exist within the project limits. Bringing the SSD up to design standards would require the reconstruction of 4700' of roadway with a cost of $1,250,000, not including earth excavation and drainage improvements.

RIGHTS OF WAY: No Rights of Way involvement is anticipated.

UTILITIES: No utilities involvement is anticipated.

MAINTENANCE AND PROTECTION OF TRAFFIC: Traffic will be maintained on the existing roadway. Traffic will be reduced to utilizing one lane where required in the areas of rock excavation and will be stopped for a minimum period of time if blasting of the rock outcrops is required.

PAVEMENT COMPOSITION:

2 inches - Bituminous concrete class 1 overlay, on
4 inches - Cold-in-Place recycling of the existing pavement

ENVIRONMENTAL CONSIDERATIONS: FEMA Flood Insurance Mapping shows that the roadway is within a designated flood hazard zone (Zone A). ConnDOT has performed a hydraulic analysis which shows that the roadway is well above the base flood elevation in the area of Quaduck Brook. Therefore, ConnDOT has concluded that the proposed improvements do not require Flood Management Certification (FPM Permit). DEP has concurred with this determination in their January 2, 1997 memorandum.

ENDANGERED SPECIES: It has been determined that there are no threatened or endangered species in the project area.

HISTORIC INVOLVEMENT: There will be no effect on historic, architectural, or archaeological resources. (SHPO memorandum dated September 19, 1996)

ESTIMATED COST:

	Project Initiation (10/27/94)	Current Estimate *(1/29/97)
PE	$ 200,000	$ 200,000
ROW	--------	---------
Utilities	--------	---------
Incidentals to Construction	375,000	415,000
Construction Contract Items and Contingencies	2,675,000	2,955,000
Total Cost	$3,250,000	$3,570,000

*estimated by Highway Design

ACCIDENT DATA ON S.R. 695 FROM 01-01-90 TO 12-31-95:

Fixed Object	-	9	- guide rail (6) - boulder (2) - utility pole (1)
Overturn	-	1	- snow
Total	-	10	(No accidents reported since May 1994)

ACCIDENT DATA ON ROUTE 6 FROM 01-01-90 TO 12-31-95:

Fixed Object	-	4	- guide rail (4)
Rear End	-	2	
Side Swipe	-	2	
Overturn	-	1	- snow
Total	-	9	(No accidents reported since Jan 1994.)

PROJECT SCHEDULE:

FDP:	4-16-97
DCD:	5-28-97
ADV:	6-25-97

TIP/STIP STATUS OF ROW AND CONSTRUCTION PHASES: There is no ROW phase for this project and construction is included in the TIP/STIP for Federal Fiscal Year 1997.

Approved by:________________________ Date:__________________
Bradley J. Smith
Manager of State Design

PROJECT NO. 68-184

SUBJECT: RESURFACING, SAFETY, AND BRIDGE IMPROVEMENTS
SEMI-FINAL ESTIMATE

CONTRACT ITEMS SUMMARY

ROADWAY COST		**2,280,000**
STRUCTURE COST		**10,000**
SUBTOTAL		**2,290,000**
CLEARING & GRUBBING	(4 %)	**91,600**
M & P OF TRAFFIC	(3 %)	**68,700**
MOBILIZATION	(4 %)	**91,600**
TRAFFIC ITEMS	(8 %)	**183,200**
CONSTRUCTION STAKING	(1.5 %)	**34,350**
CONTRACT ITEMS		**2,759,450**
SAY		**2,760,000**

COST ESTIMATE SUMMARY

CONTRACT ITEMS			**2,760,000**
CONTINGENCIES	(7 %)		**193,200**
INCIDENTALS TO CONSTRUCTIO	(15 %)*		**414,000**
UTILITIES	(------- %)		**-**
PRELIMINARY ENGINEERING	(7.2 %)	**$**	**200,000**
RIGHTS OF WAY			
RAILROAD FORCE ACCOUNT			**-**
TOTAL ESTIMATED COST		**$**	**3,567,200**

SAY: **$3,570,000**

YEAR OF ESTIMATE: 97

* Estimate is based on Contract Items

ESTIMATED BY: RCT
UNIT: 1305
DATE: Jan 1997

F:\LEBLANC\68-184\ESTIMATE.XLS

PRELIMINARY COST ESTIMATE

TOWN(S): Killingly

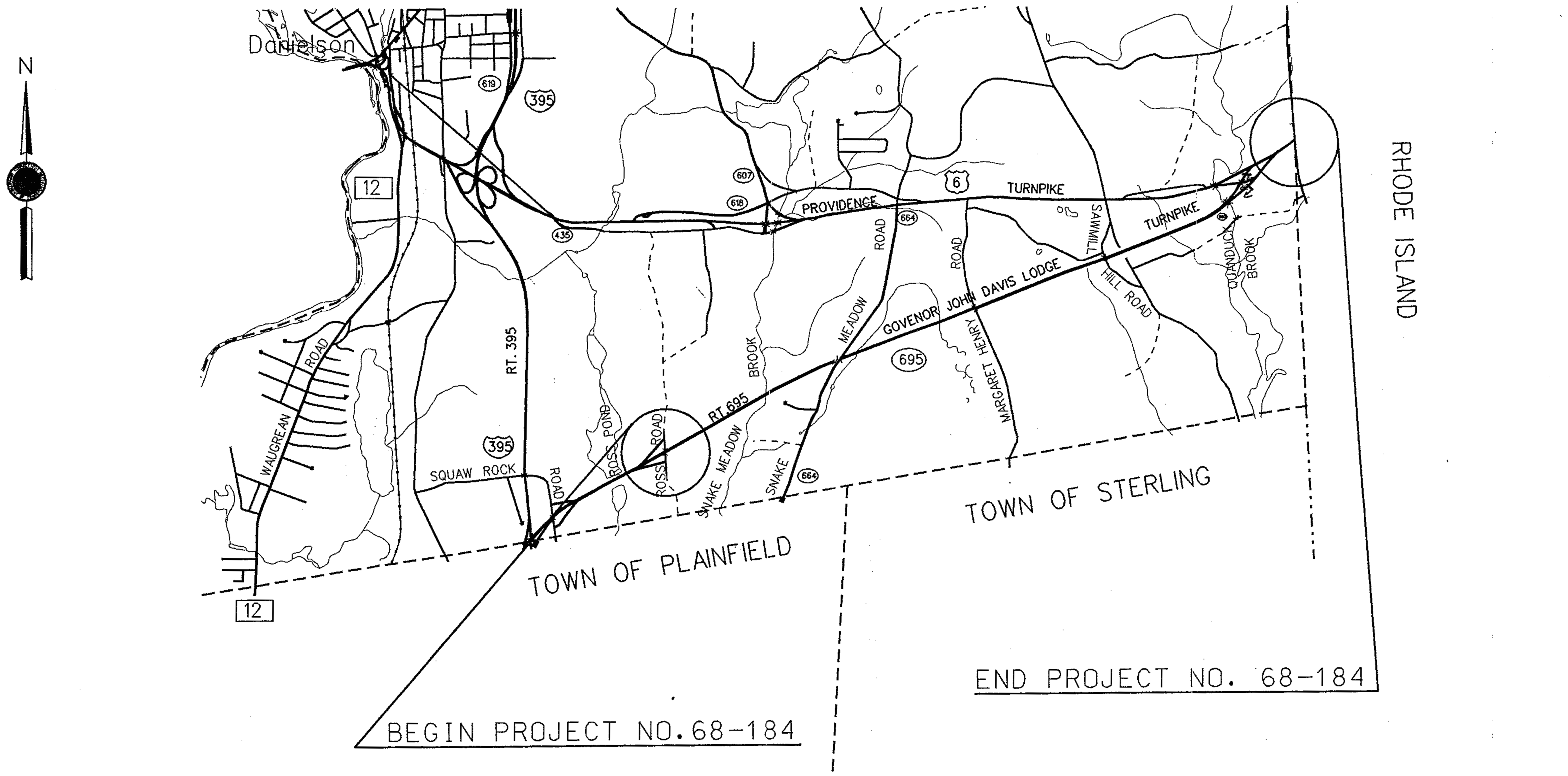
PROJECT: 68-184

Resurfacing & Safety Improvements on S.R. 695

Earthwork		Unit	Quantity	Price	Cost
	Rock Excavation	C.Y.	5120	$18.00	$92,160
	Processed Agg. (for railing)	TON	1290	$16.50	$21,285
	Grade Median (Form. Subgrade)	S.Y.	50100	$0.50	$25,050
	Trench Excavation 0'-4'	C.Y.	100	$6.00	$600
	Erosion Control Matting	S.Y.	12000	$2.00	$24,000
	Geotextile	S.Y.	100	$2.50	$250
	Turf Establishment	S.Y.	40000	$0.45	$18,000
	Topsoil (4") in median & abut bclc	S.Y.	40000	$1.50	$60,000
	2" Crushed Stone	Ton	675	$21.50	$14,513
Pavement					
	Cold-In-Place Recycling	S.Y.	184800	$3.00	$554,400
	Material for Tack Coat	Gal.	9300	$2.00	$18,600
	Subbase	C.Y.	200	$22.00	$4,400
	Bit. Conc. Class 1	Ton	21300	$34.00	$724,200
Drainage					
	Replace Type "C" C.B. Top	Ea.	65	$275.00	$17,875
	Reset Type "C" C.B. Top	Ea.	65	$410.00	$26,650
	Replace Type "C-L" C.B. Top	Ea.	50	$300.00	$15,000
	Reset Type "C-L" C.B. Top	Ea.	50	$773.00	$38,650
	Type "C-L" C.B.	Ea.	3	$1,100.00	$3,300
	Paved Ditch	S.Y.	270	$20.00	$5,400
	Convert "C" to "C-L"	Ea.	23	$500.00	$11,500
	Convert "C" to "CMCS"	Ea.	3	$1,100.00	$3,300
	12" R.C.P.	L.F.	200	$32.00	$6,400
	24" M.C.E.	Ea.	1	$300.00	$300
	4" Underdrain	L.F.	400	$12.00	$4,800
	6" Underdrain	L.F.	850	$14.00	$11,900
	Clean existing catch basins	Ea.	140	$75.00	$10,500
Curbing					
	BCLC	L.F.	21800	$2.00	$43,600
	Temporary P.C.B.C.	L.F.	1000	$15.00	$15,000
	Relocate Temporary P.C.B.C.	L.F.	4000	$6.00	$24,000
	Concrete Barrier (42" "F" Shape)	L.F.	680	$32.00	$21,760
Guide Railing					
	Remove Cable Guide Railing	L.F.	1200	$1.25	$1,500
	Convert 3-Cable Guide Railing	L.F.	12650	$2.50	$31,625
	Remove M.B.R.	L.F.	22500	$1.75	$39,375
	M.B.R. (MD-B 350)	L.F.	18000	$15.00	$270,000
	Reset Type MD-I	L.F.	125	$3.50	$438
	M.B.R. (R-B 350)	L.F.	5300	$15.00	$79,500
Guide Rail Anchors					
	Type I	Ea.	27	$640.00	$17,280
	RB Type I	Ea.	5	$450.00	$2,250
	RB Ledge	Ea.	1	$425.00	$425
	RB Bridge	Ea.	2	$500.00	$1,000
	RB Buried	Ea.	2	$600.00	$1,200
Other Items					
	Block Wall	L.F.	25	$40.00	$1,000
	C.A.T.	Ea.	2	$5,000.00	$10,000
Roadway Total					$2,272,985
				Say:	$2,280,000
Structures					$10,000

F:\LEBLANC\68-154\COST.XLS

APPENDIX C



LOCATION PLAN

PROJECT NO.68-184

STATE ROUTE ROUTE 695

KILLINGLY

CONNECTICUT

METRIC SCALE 300m 0 300m 600m 900m

SCALE 1000 0 1000 2000 3000 FEET

APPENDIX D

7/27/98

KL 7-31
Nick

GORMAN BROS.
INCORPORATED

Telephone 518-462-5401
518-462-5402
Dispatcher 518-472-9342
Fax 518-462-1296

ASPHALT DISTRIBUTING CONTRACTORS

Church Street, Port of Albany
Albany, NY 12202

July 27, 1998

Thomas Moore
Cardi Corp. Inc.
400 Lincoln Ave.
Warwick, R.I. 02886

Dear Tom,

This is in response to Keith Lane's July 10, 1998 memo to Mike Wilson regarding the cold in-place recycling on the Rt. 695 project.

1.) Attached is a copy of our lab test results on the HFMS-2 emulsion we are supplying to the job. A copy of this also was sent with our emulsion load last Friday.

2.) Our lab mix design is conducted under ideal conditions using RAP generated by crushing 12 cores representing nearly 200,000 s.y. of pavement. While the lab tests indicated 1.5% was the best application rate, we started at 2.0% to assure that mixing, paving and compaction were acceptable. We feel it's better to start at a safe percentage of emulsion and cut back if the recycled mix paves and compacts well. I'm sure that Keith and Nick Corona don't have much more confidence in the Marshall Mix Design that I do.

3.) Regarding moisture content, we always do our mix design at 4.0% total fluids (emulsion plus water). In the field we try to do the same – ie increase water if emulsion content decreases. However, the in-place pavement usually contains 0.5% to 2.0% moisture. We estimate 1% and add enough water plus the emulsion to get at least 4% total fluids. Obviously with the in-place moisture constantly changing it is difficult to know exactly what the content is at all times.

4.) I do not know the % air void in the recycled mix since we have not done a maximum specific gravity determination on this RAP or RAP plus emulsion. My best guess would be that the mix contains 8 to 15% air voids. While this sounds high compared to hot mix, remember that our aggregate (the RAP) contains about 5 to 8% air voids.

RECEIVED
JUL 30 1998

GORMAN BROS.
INCORPORATED

Telephone 518-462-5401
518-462-5402
Dispatcher 518-472-9342
Fax 518-462-1296

ASPHALT DISTRIBUTING CONTRACTORS

Church Street, Port of Albany
Albany, NY 12202

5.) Regarding moisture content in the recycled mat prior to placing the HMA overlay, it is generally recommended that the free moisture be less than 1.5%. Free moisture is defined as total moisture minus the in-place moisture prior to recycling. We will conduct a few tests to determine the in-place moisture content of the Rt. 695 pavement and send the results to you.

If there are any further questions, please call.

Sincerely,

Ed

Edward J. Kearney, P.E.
Chief Engineer

cc Keith R. Lane, P.E.

Lot No. 12
NYS Item No. 702-3301
Grade HFMS-2
Gallons @ 60°F 49,370

Primary Source: Name: MOHAWK ASPHALT EMULSIONS
Address: 6 FREEMANS BRIDGE ROAD
SCOTIA, N.Y. Zip 12302
Telephone No. Area Code 518 - 372-9988

ANIONIC ASPHALT EMULSIONS

Type	RAPID SETTING		MEDIUM SETTING		SLOW SETTING	
Required Test	Demulsibility N/50 (%)		Stone Coating Test	Satis.	Cement Mixing Test (%)	
	Residue by Distillation (%)		Residue by Distillation (%)	68.97	Residue by Distillation (%)	
	Oil Distl., Vol. Tot. Emul. (%)		Oil Distl., Vol. Tot. Emul. (%)	2.10	Pen. Distl., Res. @ 77°F	
	Pen. Distl. Res. @ 77°F		Pen. Distl. Res. @ 77°F	192		
			Float Test, 140°F (Sec.) (High Float Types Only)	1200+		

CATIONIC ASPHALT EMULSIONS

Type	RAPID SETTING		MEDIUM SETTING		SLOW SETTING	
Required Test	Classification Test		Stone Coating Test		Cement Mixing Test (%)	
	Particle Charge Test		Particle Charge Test		Particle Charge Test or Ph	
	Residue by Distillation (%)		Residue by Distillation (%)		Residue by Distillation (%)	
	Oil Distl., Vol. Tot. Emul. (%)		Oil Distl., Vol. Tot. Emul. (%)		Pen. Distl., Res. @ 77°F	
	Pen. Distl., Res. @ 77°F		Pen. Distl. Res. @ 77°F			
	Sp. gr. 1.014					

INSTRUCTIONS: Use this form to certify Emulsions to the Materials Bureau. To certify Asphalt Cement use Form BR 301b

This Form supplied by N.Y.S.D.o.T. - Materials Bureau
1220 Washington Ave., Albany, NY 12232

I hereby certify that the above material conforms to N.Y.S.D.o.T. Specifications.
Authorized Signature [signature]
Title PRODUCTION MANAGER Date 7/20/98

APPENDIX E

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION

memorandum

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

subject Project No. 68-184
S.R. 695, Killingly
Core Results

date February 24, 2000

to Mr. Joseph A. Misbach
Transportation Maintenance Manager
Bureau of Engineering and
Highway Operations

from Keith R. Lane, P.E.
Director of Research and Materials
Bureau of Engineering and
Highway Operations

In response to your memorandum of October 20, 1999, personnel from this office obtained core samples on the subject project on October 28, 1999. The results of our findings are attached.

The grading of the recycled material is considerably finer than the original core samples taken before the recycling process. The recycled material meets our current class 2 criteria, while the original samples were class 1.

The density of the material has always been and is still of concern to us, ranging from a low of 83.1% to a high of 89.9%. With the combination of a fine gradation, high liquid content and relatively low density, we would recommend that this road be monitored in the future for evidence of rutting.

Any questions about this information may be directed to Mr. Nicholas R. Corona at this office. His telephone number is (860) 258-0326.

Attachment
Nicholas R. Corona/gp/S\gp\corona\M_misbach-contr.Sealing-Jnts

cc: Louis R. Malerba - Michael D. Turano
Keith R. Lane
Colleen A. Kissane - David J. Kilpatrick
Nicholas R. Corona
DMT Files

TABLE 1

Project: 68-184
Loctaion: Rte. 695 Killingly
Date Sampled: 10/28/99

CORE #	DEPTH * (in.)	% VOIDS AASHTO T-166	LOCATION
1	1 1/2	10.44	E.B. Low Speed Lane – 75' from start of recycle area - 5' rt of centerline
2	2 3/8	14.04	E.B. Low Speed Lane – 1 mile east of core #1 6' - rt of centerline
3	1 3/4	10.08	E.B. Low Speed Lane – 1 mile east of core #2 - 300' east of Saw Mill Hill Road 10' rt of centerline
4	2	11.74	E.B. Low Speed Lane – 1.2 miles east of core #3 - 7' rt of centerline
5	2	12.57	W.B. Shoulder – 0.5 miles west of east end of expressway - 13' rt of centerline
6	1 7/8	11.45	W.B. Low Speed Lane – 1 mile west of core #5 - 5' rt of centerline
7	1 7/8	11.21	W.B. Low Speed Lane – 100' west of Margaret Henry Rd. overpass - 8' rt of centerline
8	1 7/8	13.09	W.B. Low Speed Lane – 1 mile east of Exit 90 - 5' rt of centerline
9	2 3/4	16.90	E.B. High Speed Lane – 1 mile east of Exit 90 - 6' rt of centerline
10	2 1/8	16.50	E.B. High Speed Lane – 1.4 miles from core #9 - 7' left of centerline
11	2 3/4	12.91	E.B. High Speed Lane – 1.1 miles from core #10 - 8' left of centerline
12	3 1/4	13.21	W.B. High Speed Lane at Saw Mill Hill Rd. overpass - 9' left of centerline
13	2 1/4	16.61	W.B. High Speed Lane – 1.2 miles from core #12 - 6' left of centerline
14	3	13.77	W.B. High Speed Lane – 1.2 miles from core #13 - 9' left of centerline

* Recycled Portion Only

Abson Recovery

Core #	Viscosity @ 140°F	Penetration at 77°F
1, 2,3	29,560	23
5, 6, 7	32,020	22

GRADATION ANALYSIS PERCENT PASSING/SIEVE SIZE									
CORE	3/4"	1/2"	3/8"	#4	#8	#30	#50	#200	AC
4 & 9	100	98	95	75	58	34	23	5.8	6.67
8 & 12	100	99	93	74	56	34	24	7.7	7.28

APPENDIX F

CONTROL SITE

MANUAL DISTRESS SURVEY

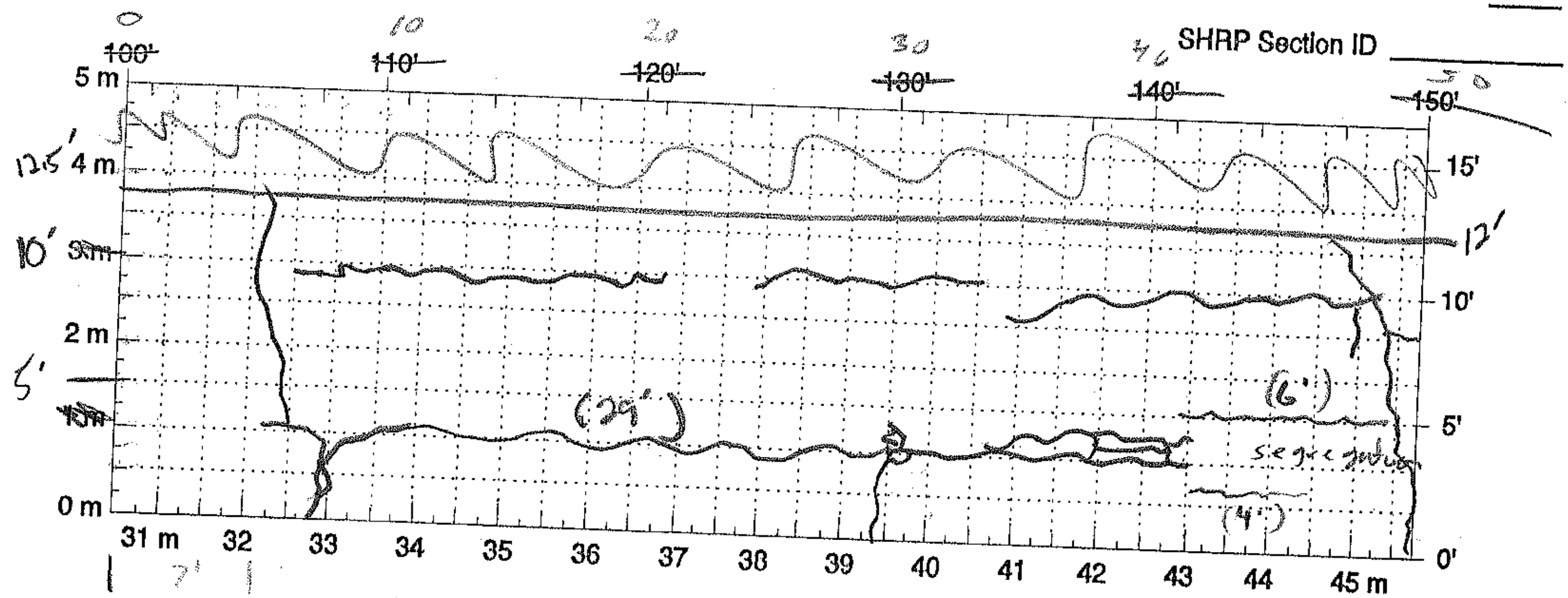
Control Site

control C/O

State Assigned ID ____________

State Code ______

SHRP Section ID ____________



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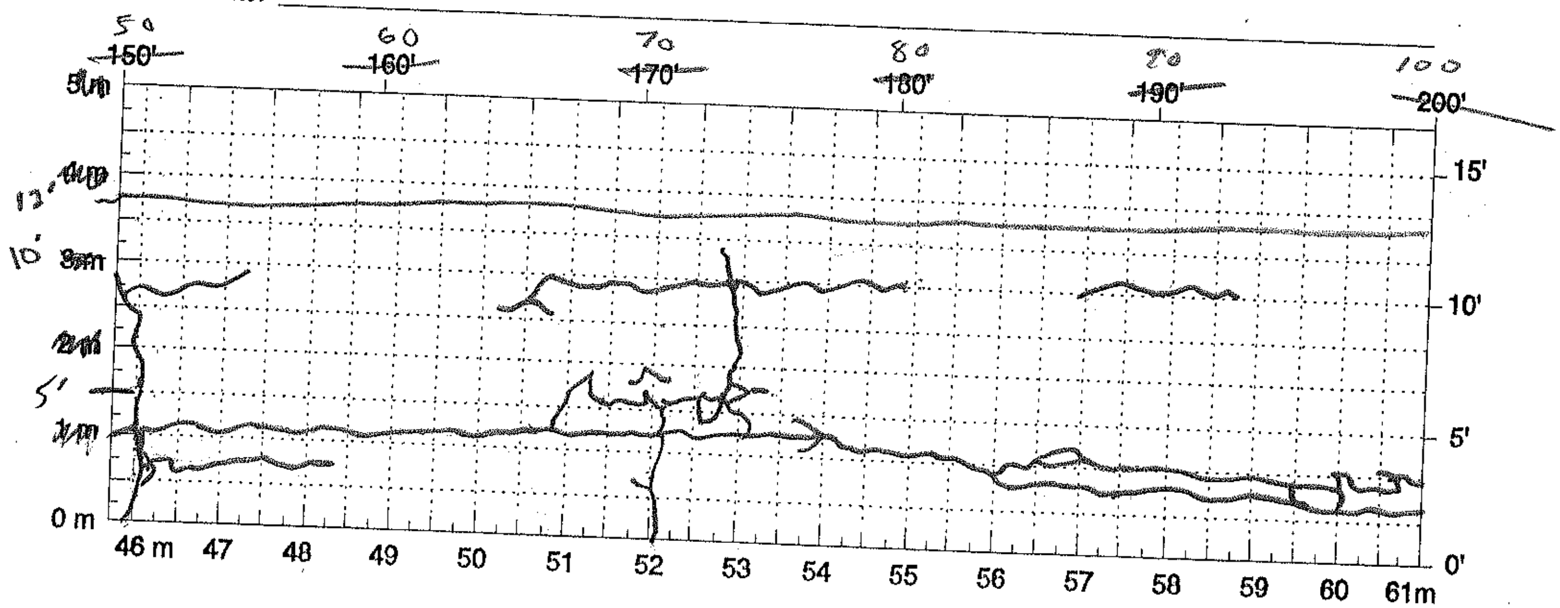


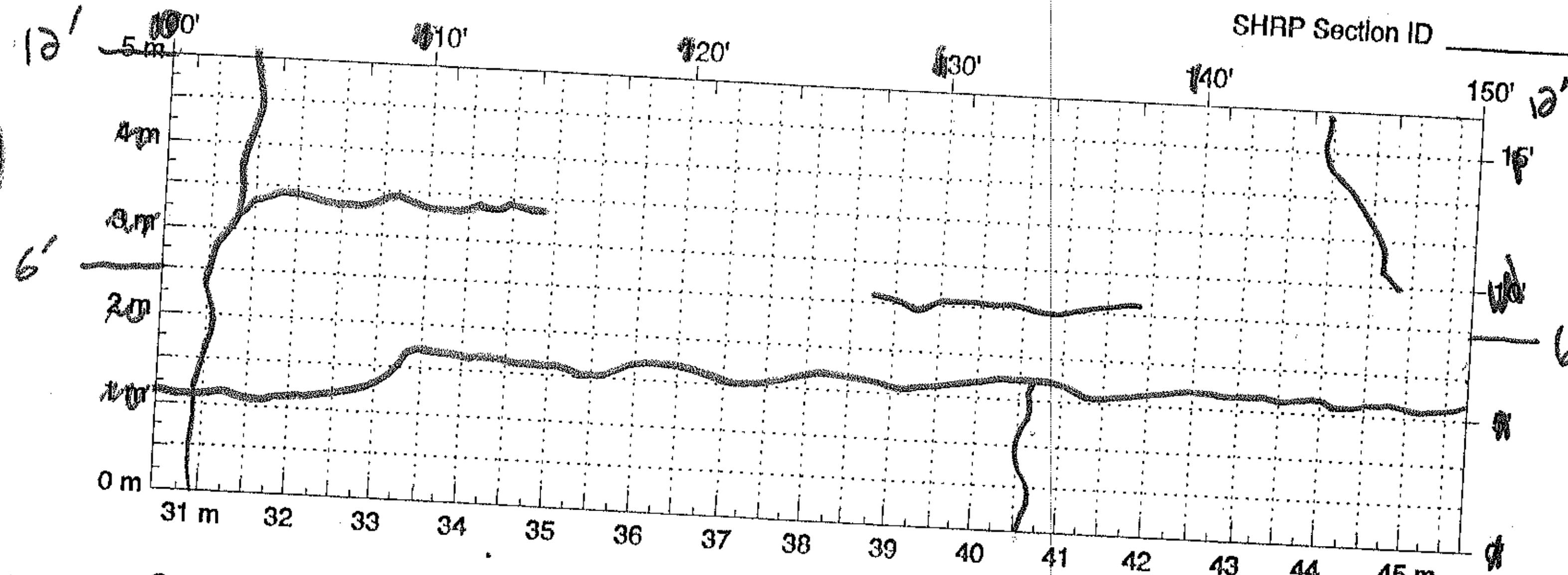
Comments: ____________

State Assigned ID ______

State Code ______

SHRP Section ID ______

Section 1



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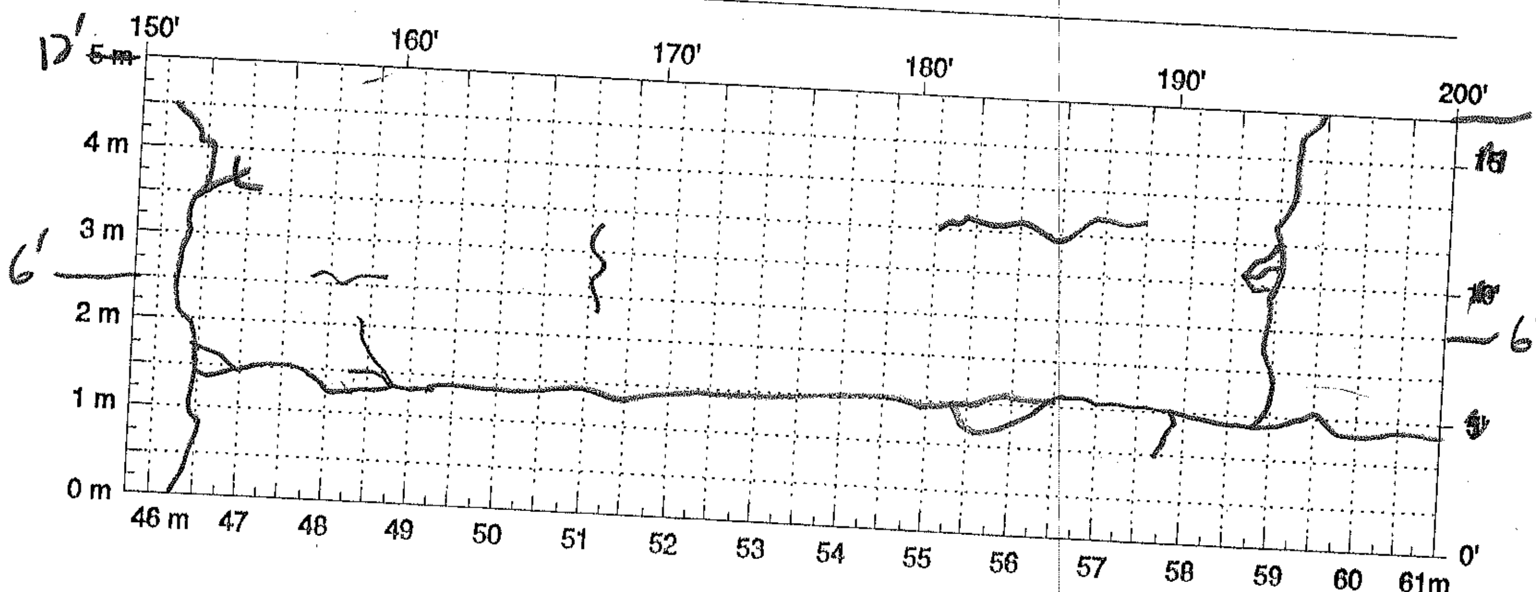


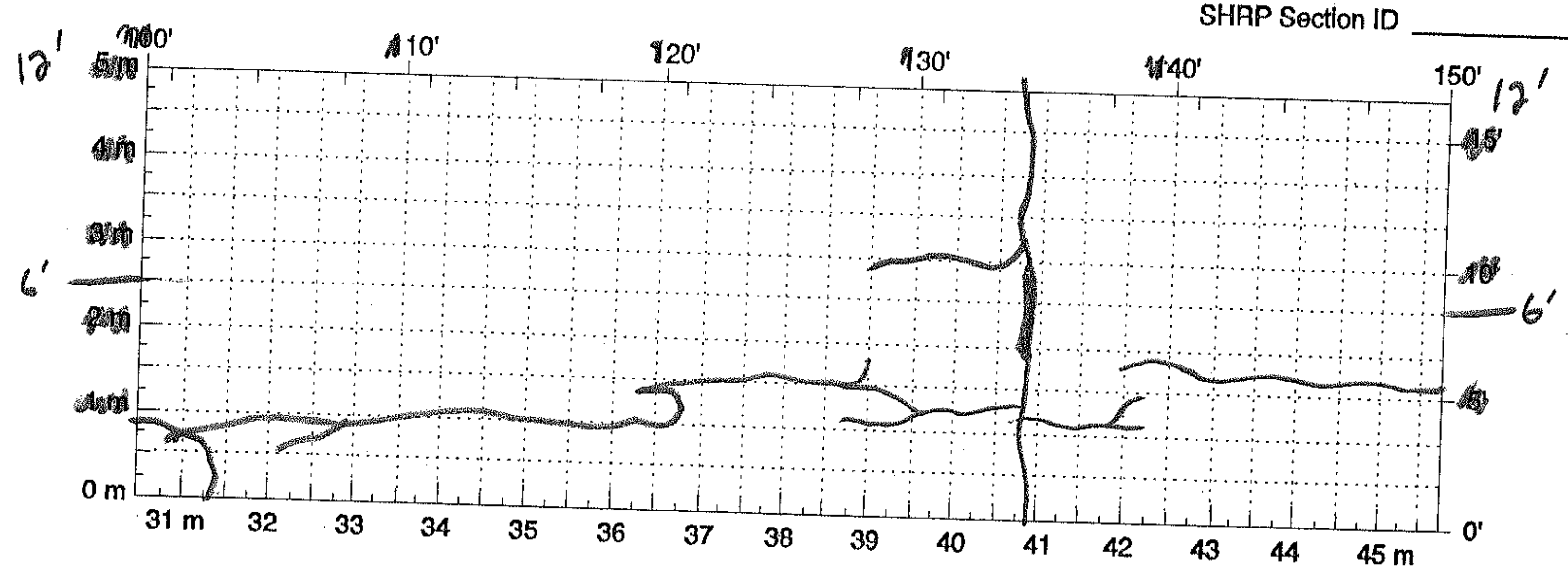
Comments: ______

State Assigned ID ____________

State Code ______

SHRP Section ID ____________

Section ②



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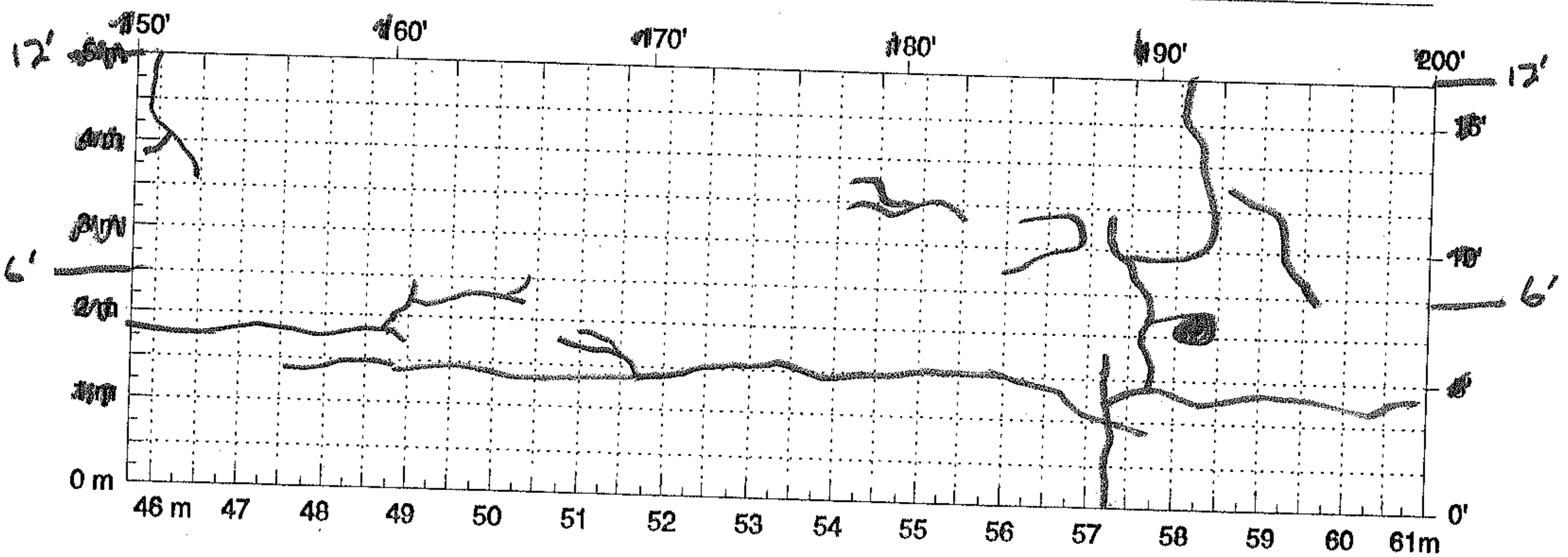


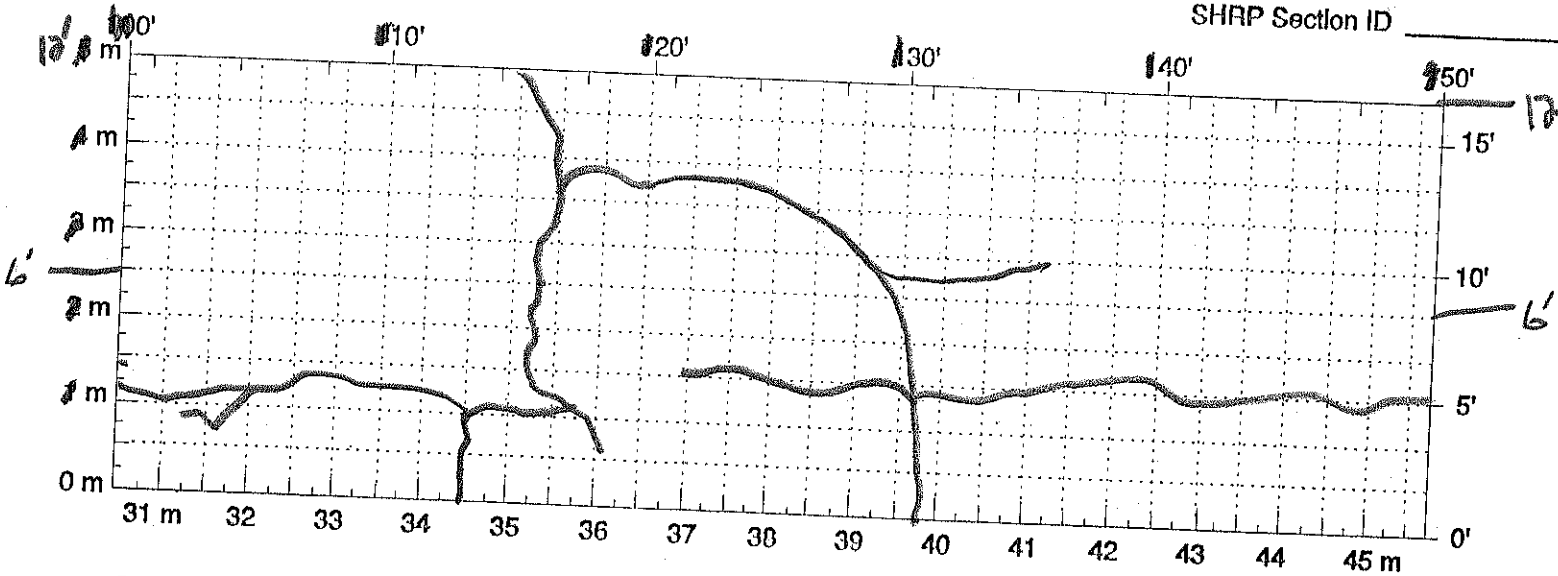
Comments: ____________

Section 3

State Assigned ID ______

State Code ______

SHRP Section ID ______



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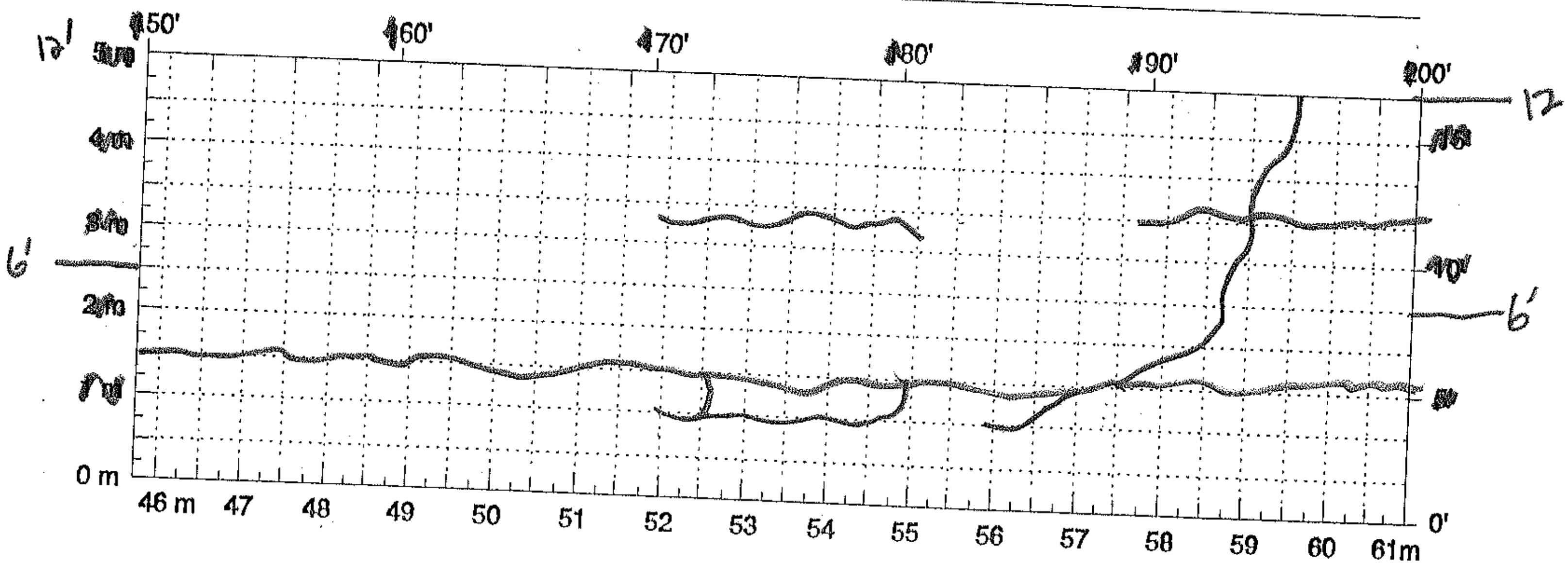


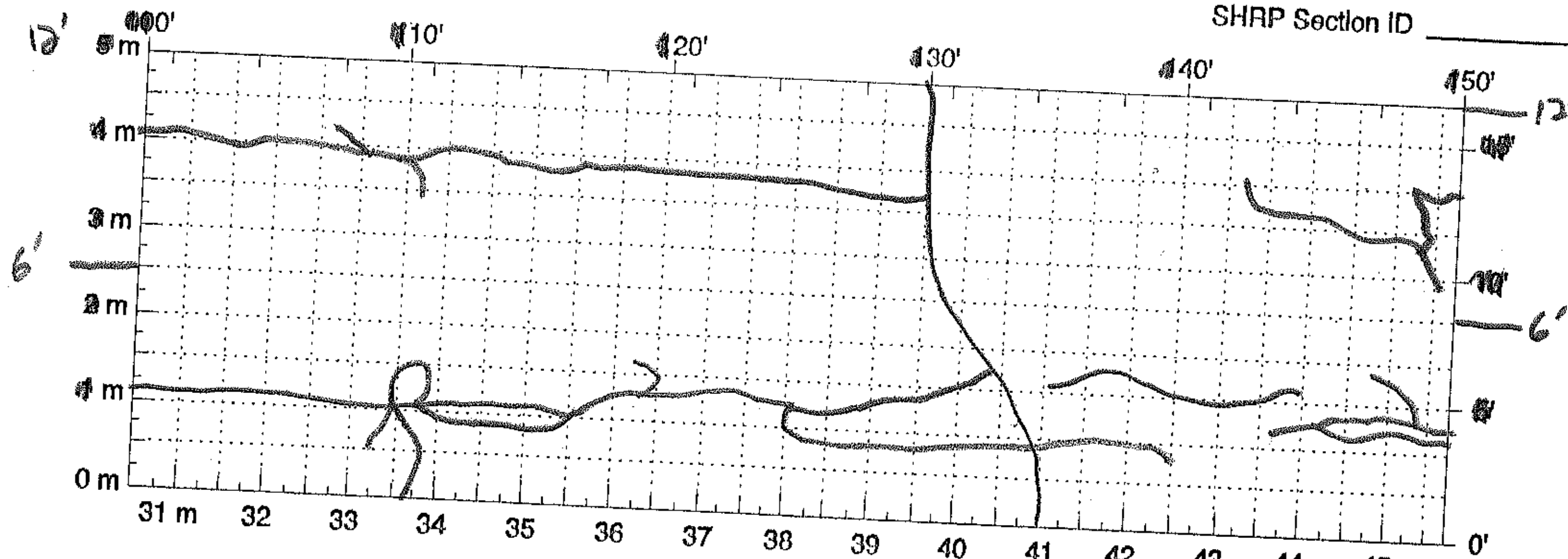
Comments: ______

State Assigned ID ____________

State Code ______

SHRP Section ID ____________

Section 4



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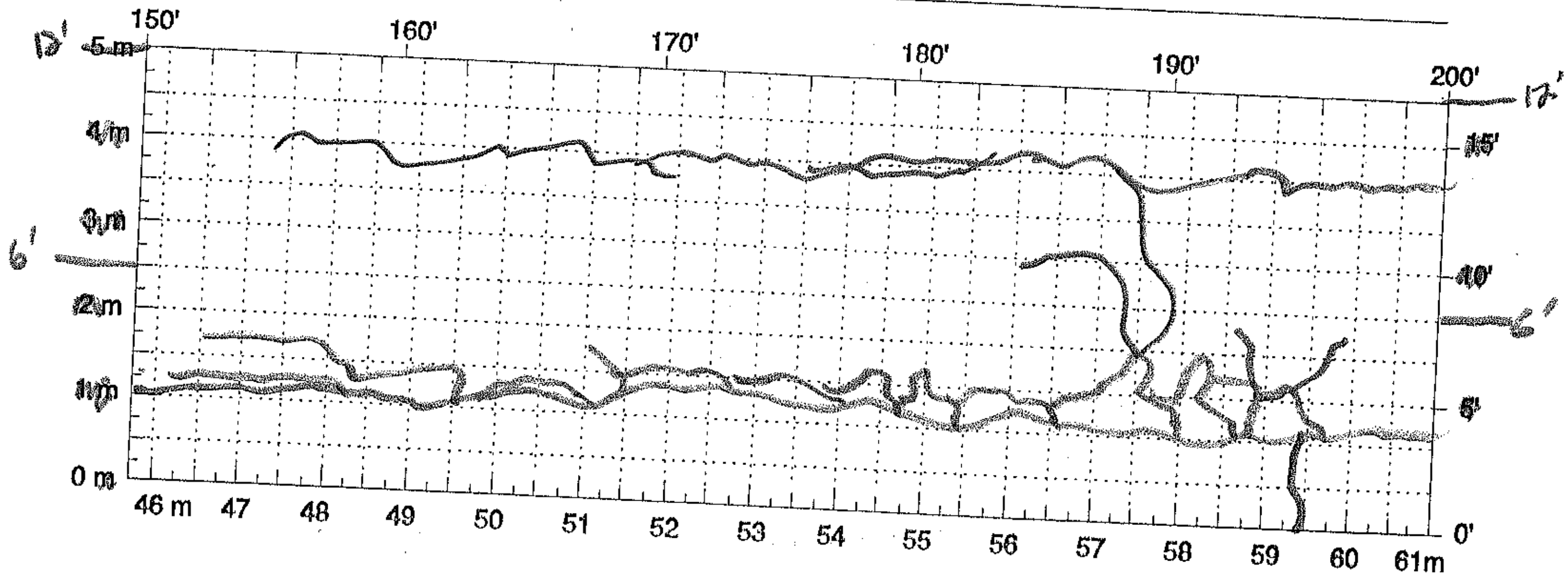


Comments: ____________

APPENDIX F

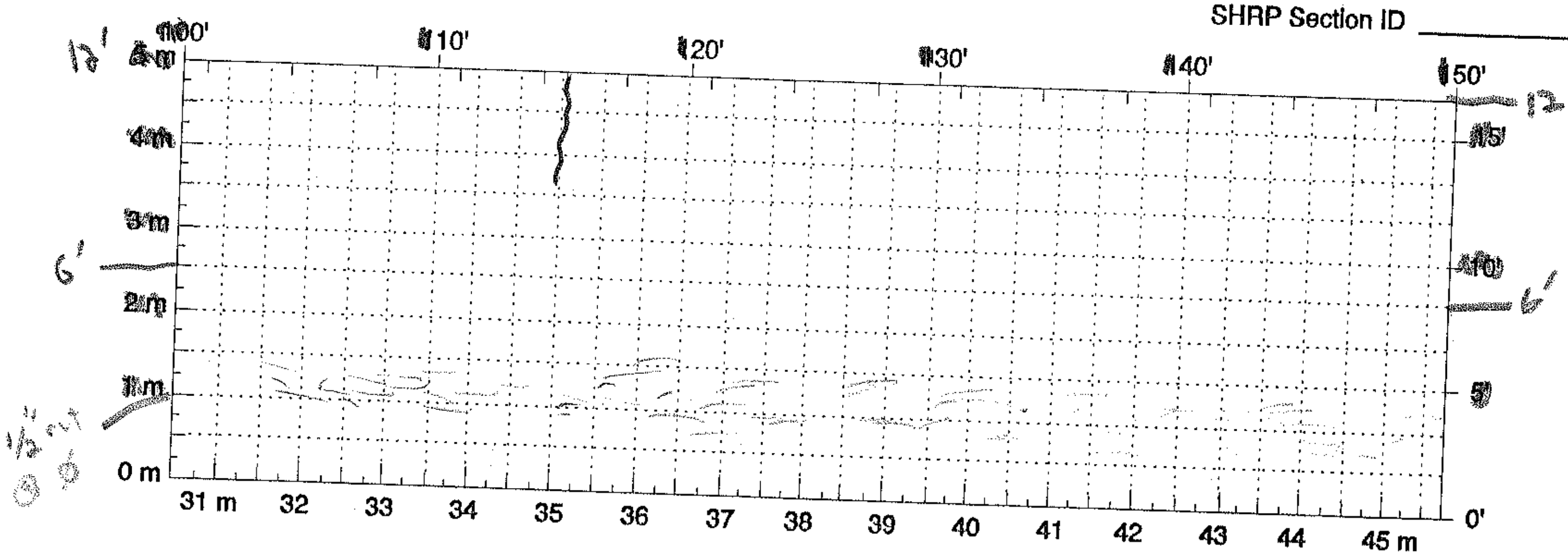
SITE S1 MANUAL DISTRESS SURVEY

Site 1

State Assigned ID ______

State Code ______

SHRP Section ID ______



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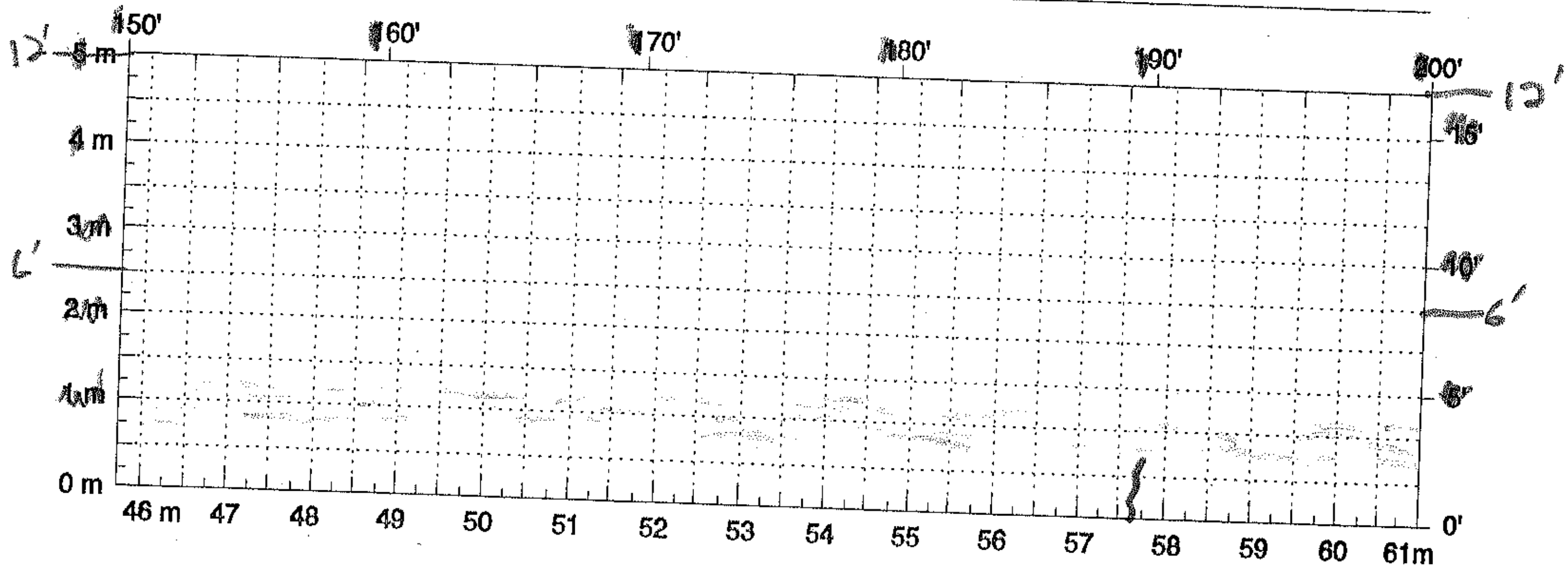


Comments: ______

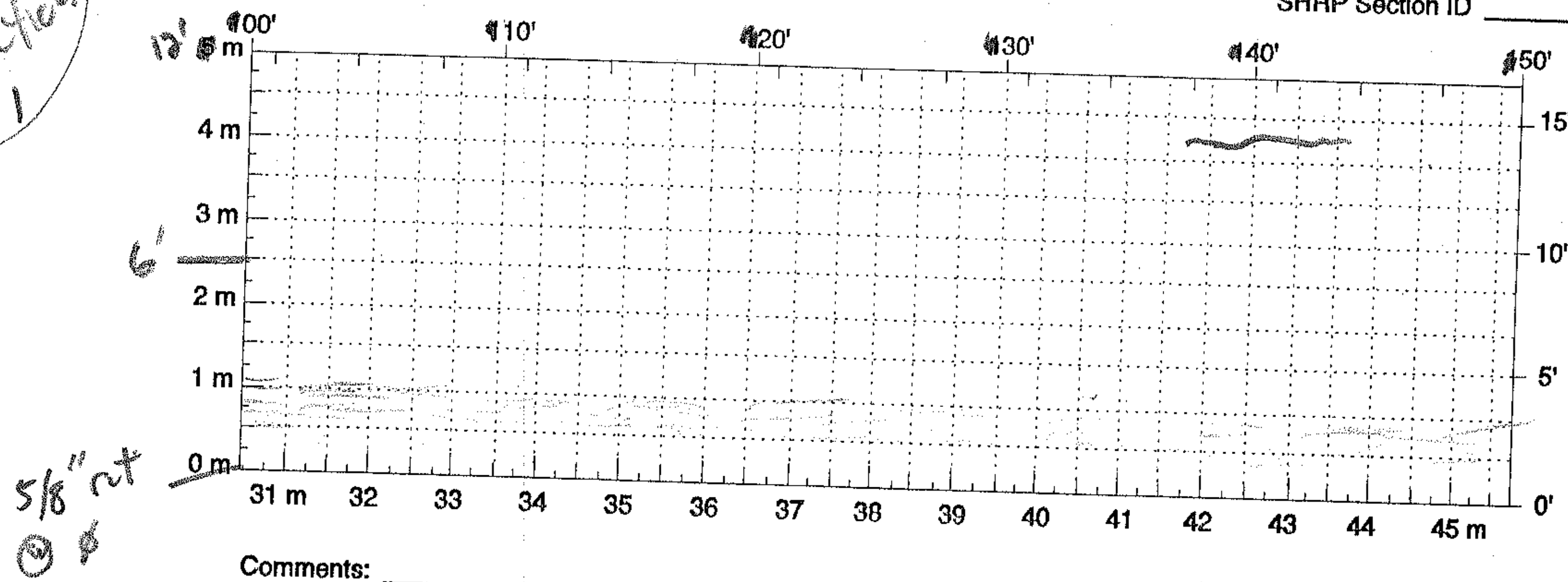
Site 1

State Assigned ID ____________

State Code ____

SHRP Section ID ____________

Section 1



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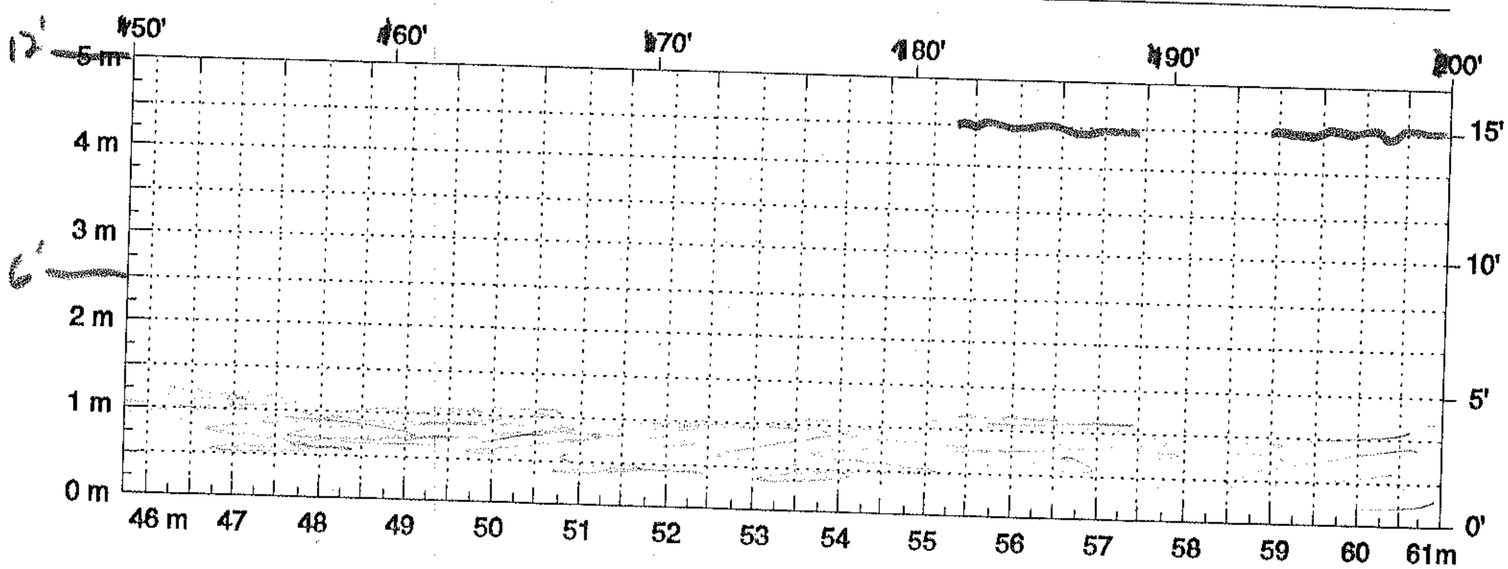


Comments: ____________

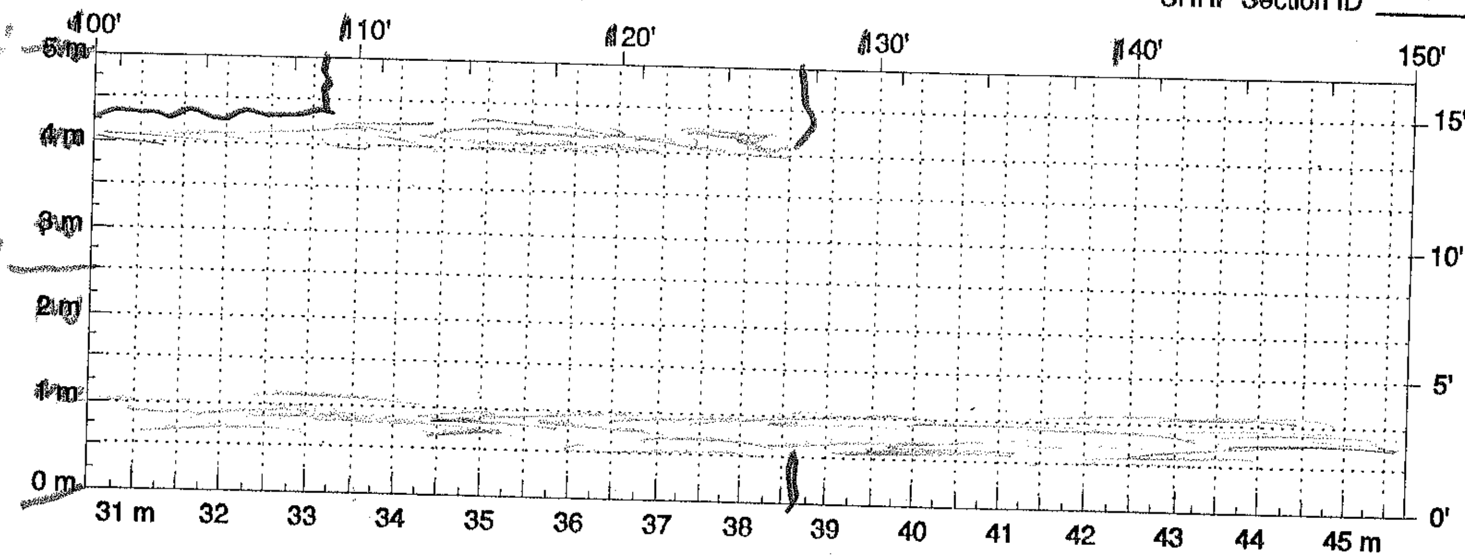
Site 1

Section 2

State Assigned ID ____________

State Code ____

SHRP Section ID ____________



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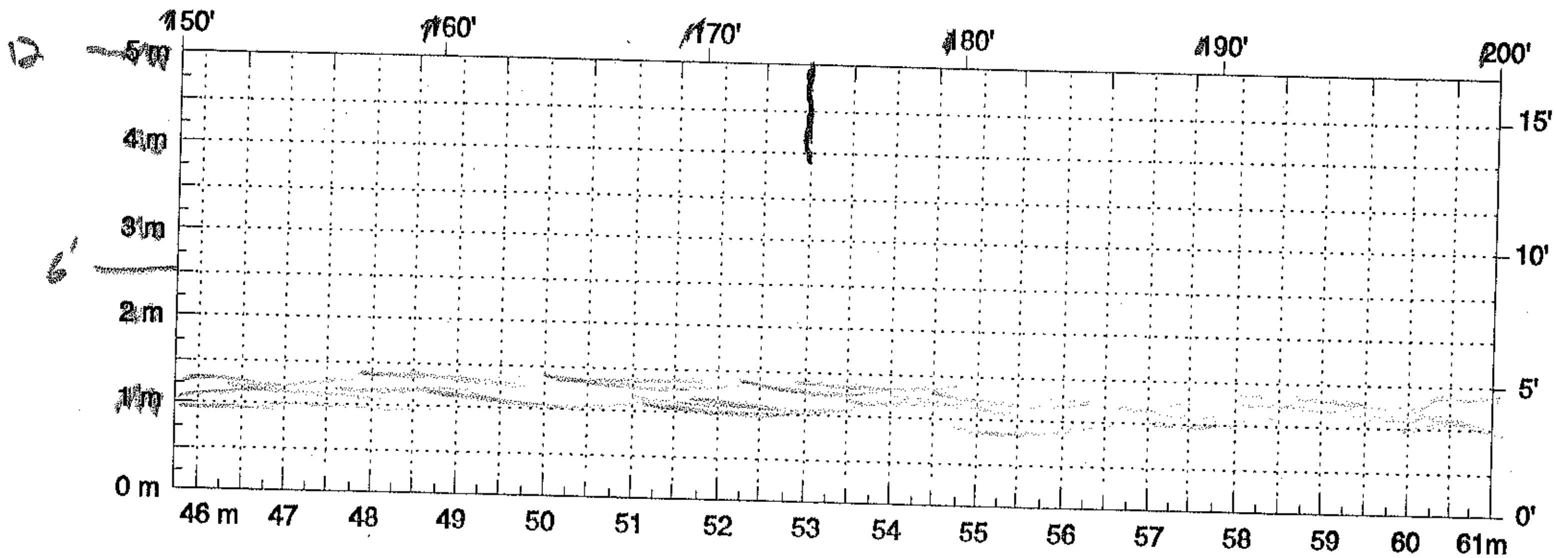


Comments: ____________________________

Site 1

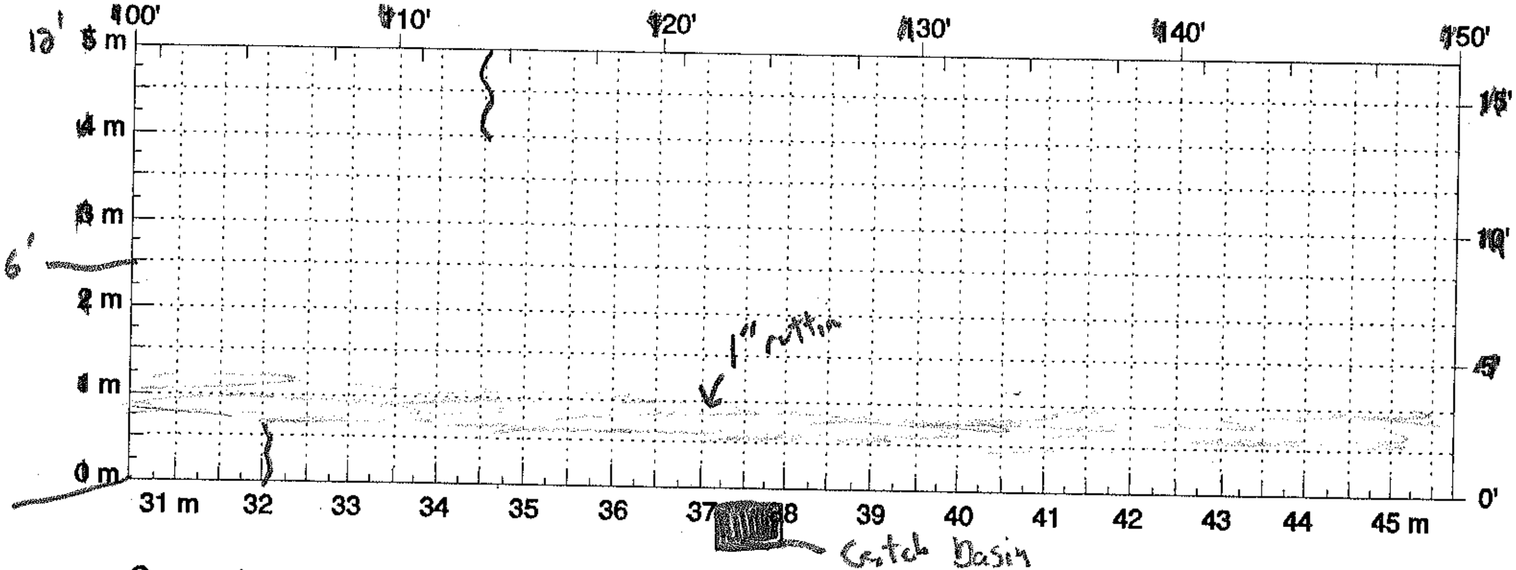
State Assigned ID ____________

State Code ____

SHRP Section ID ____________

Section 3

3/4" rut @ ϕ



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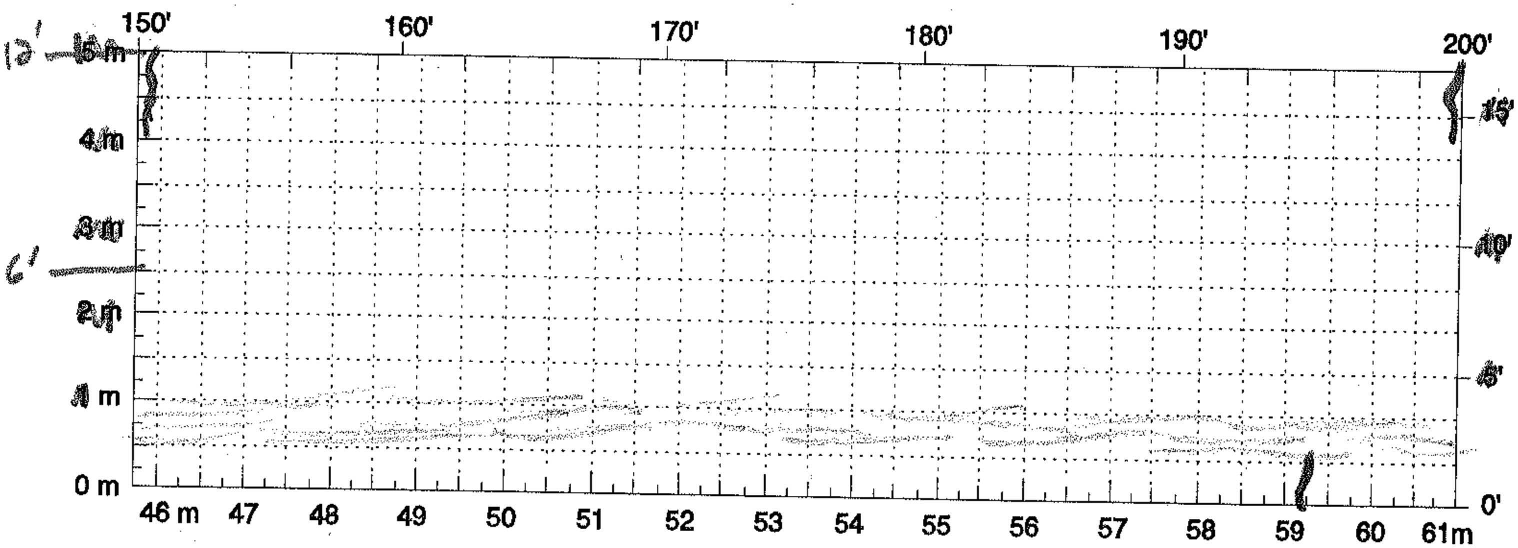


Comments: ____________________

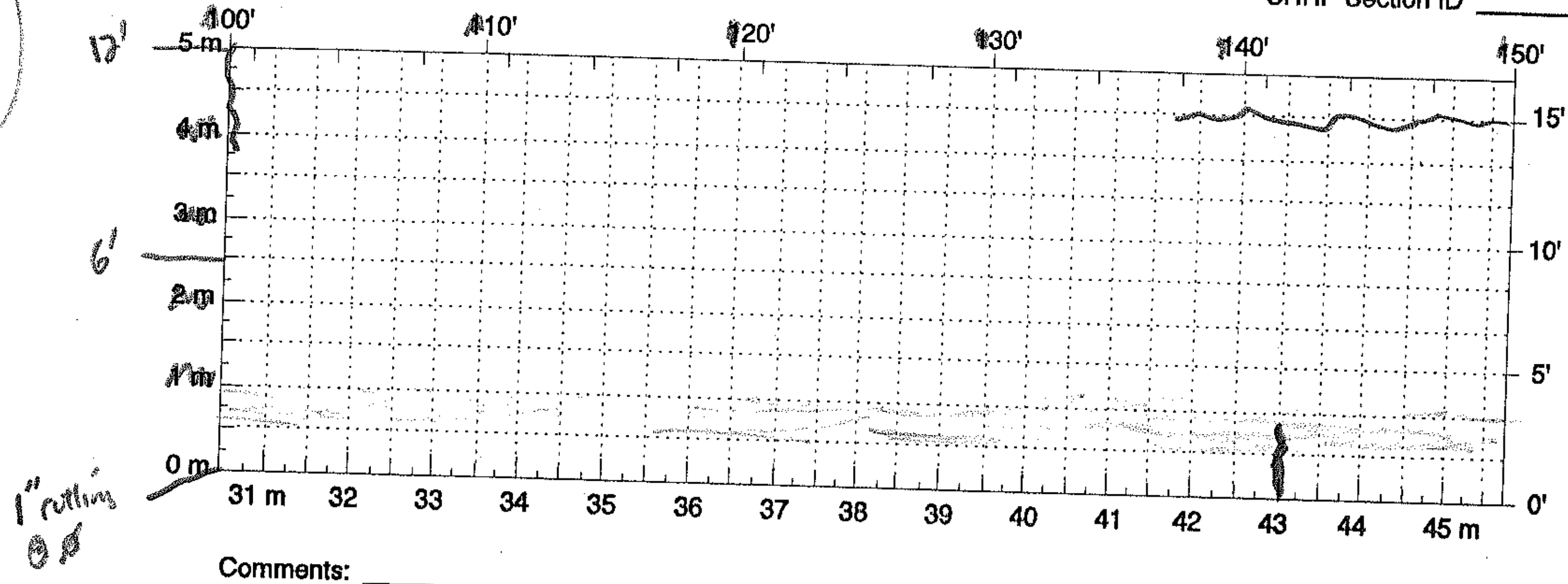
Site 1

Section 4

State Assigned ID ____________

State Code ______

SHRP Section ID ____________



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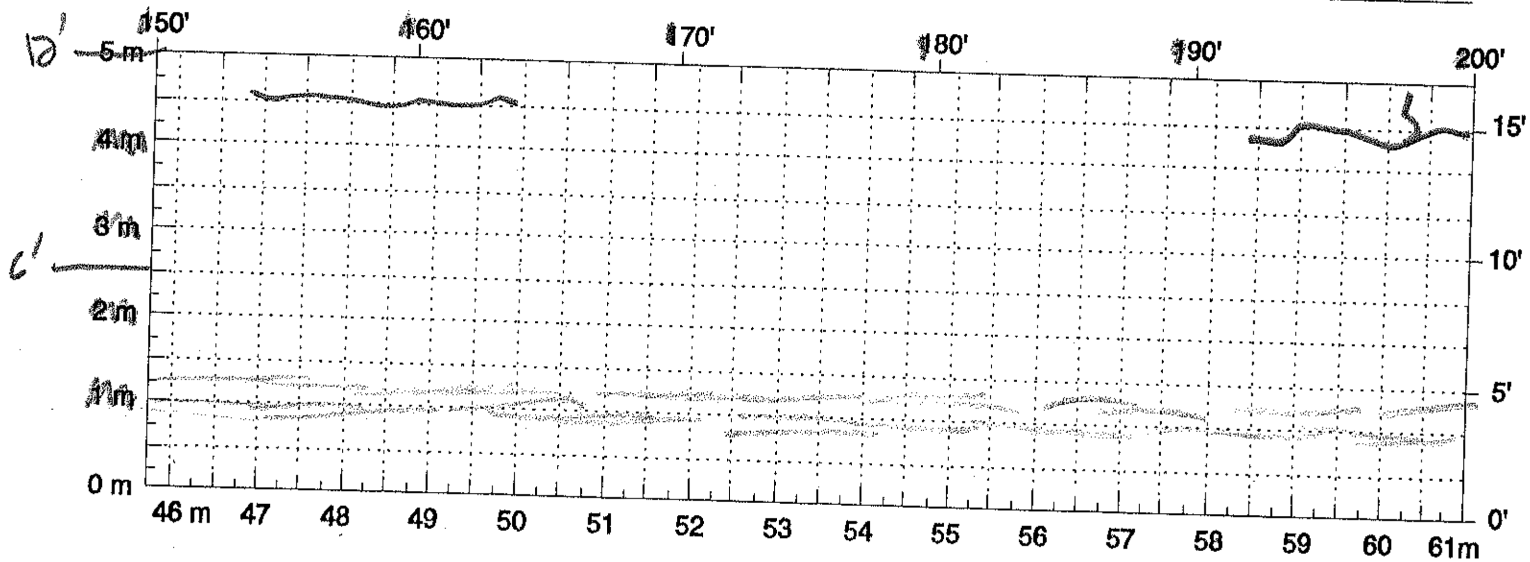


Comments: ____________

APPENDIX F

SITE S2 MANUAL DISTRESS SURVEY

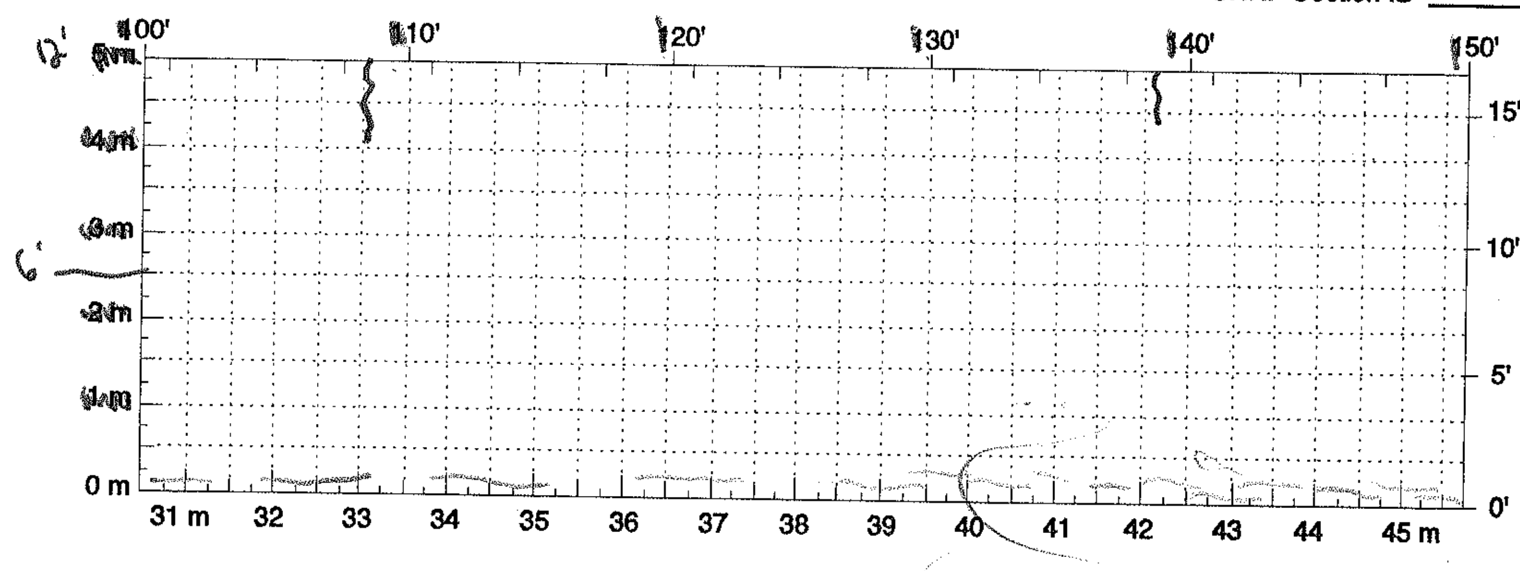
Site 3

5210

State Assigned ID ____________

State Code ______

SHRP Section ID ____________



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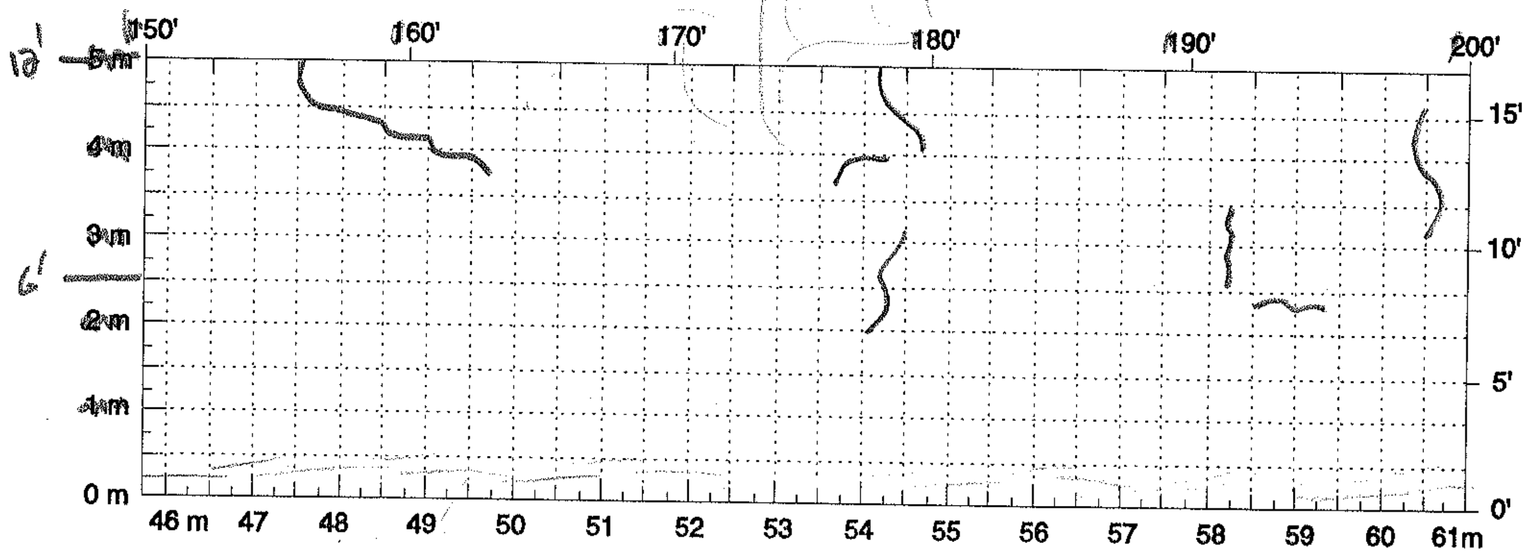


Comments: ____

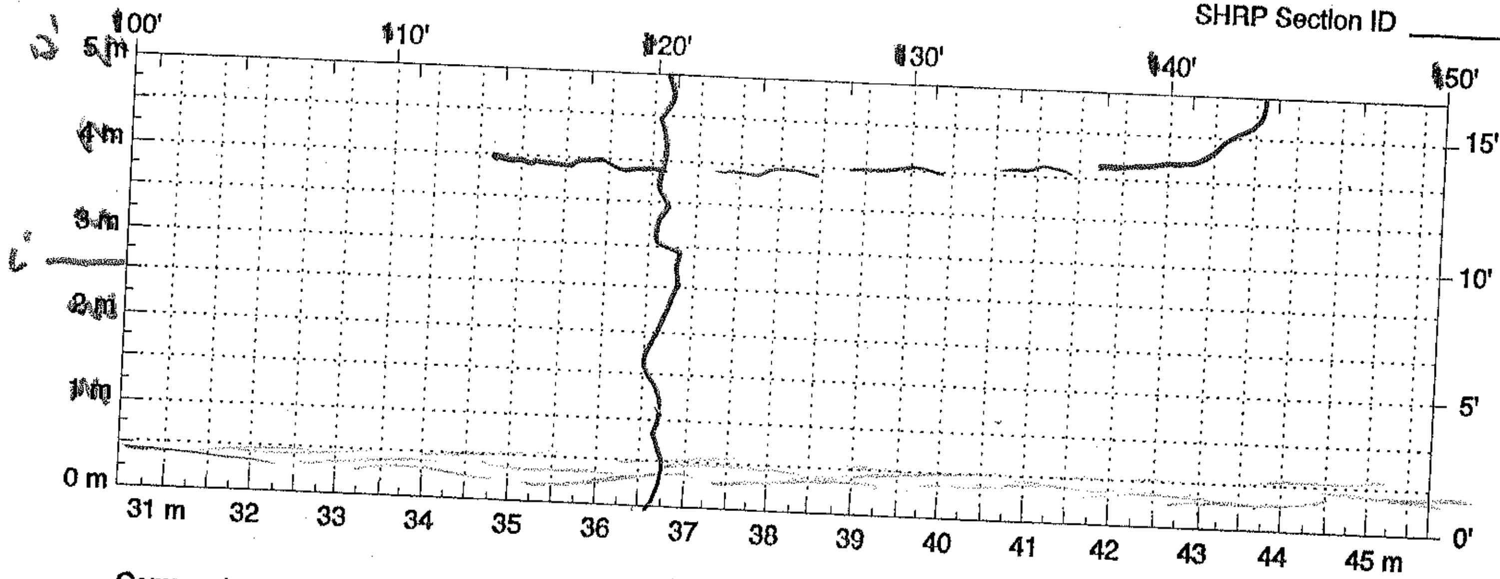
Site 3

Section 1

State Assigned ID ____________

State Code ____

SHRP Section ID ____________



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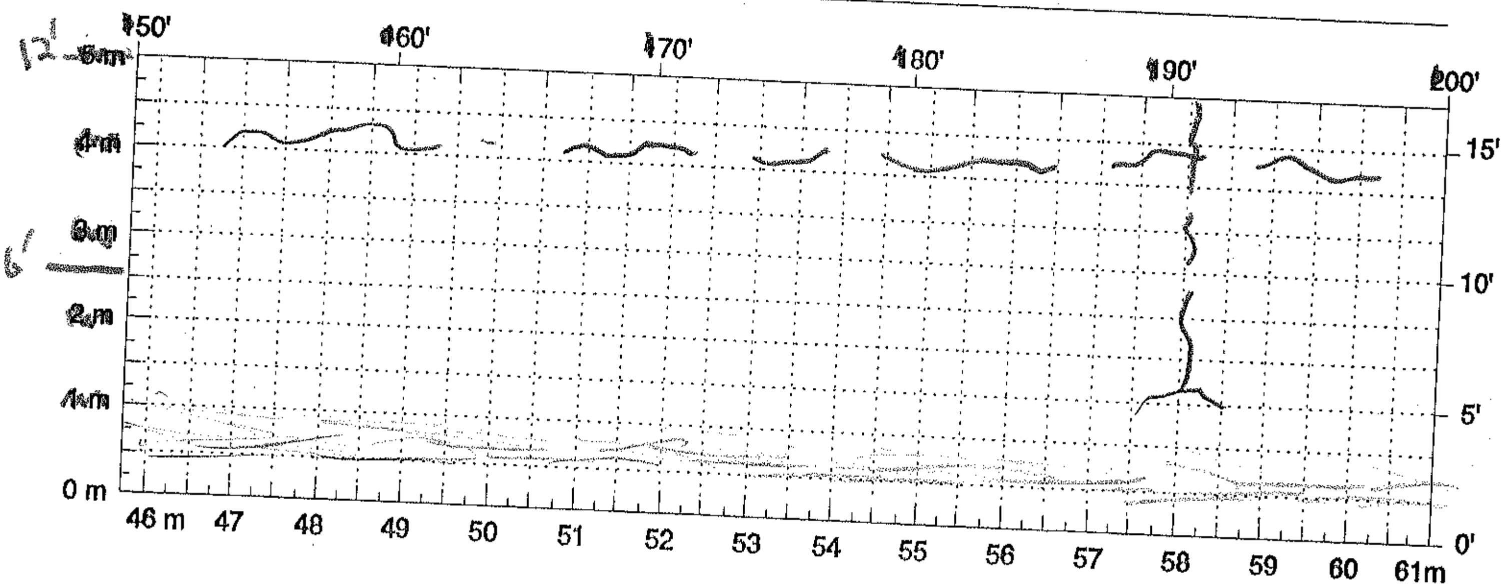

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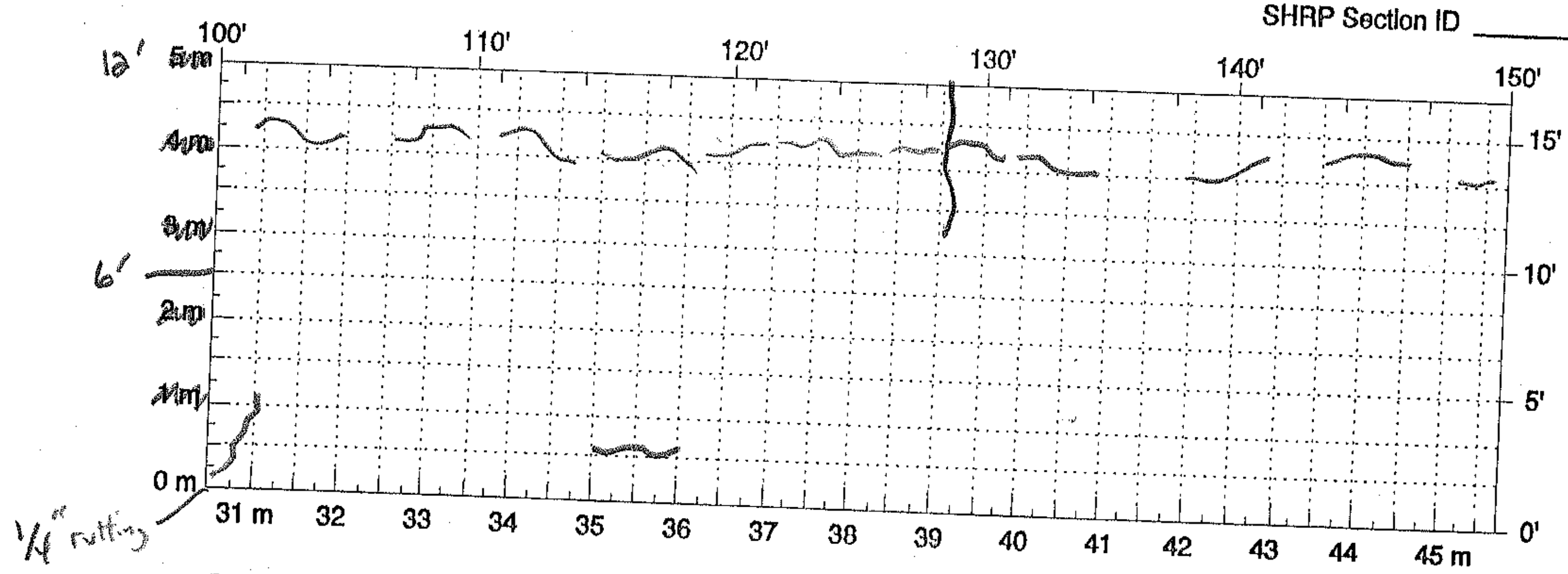
Section 2

Site 2

State Assigned ID ____________

State Code ____

SHRP Section ID ____________



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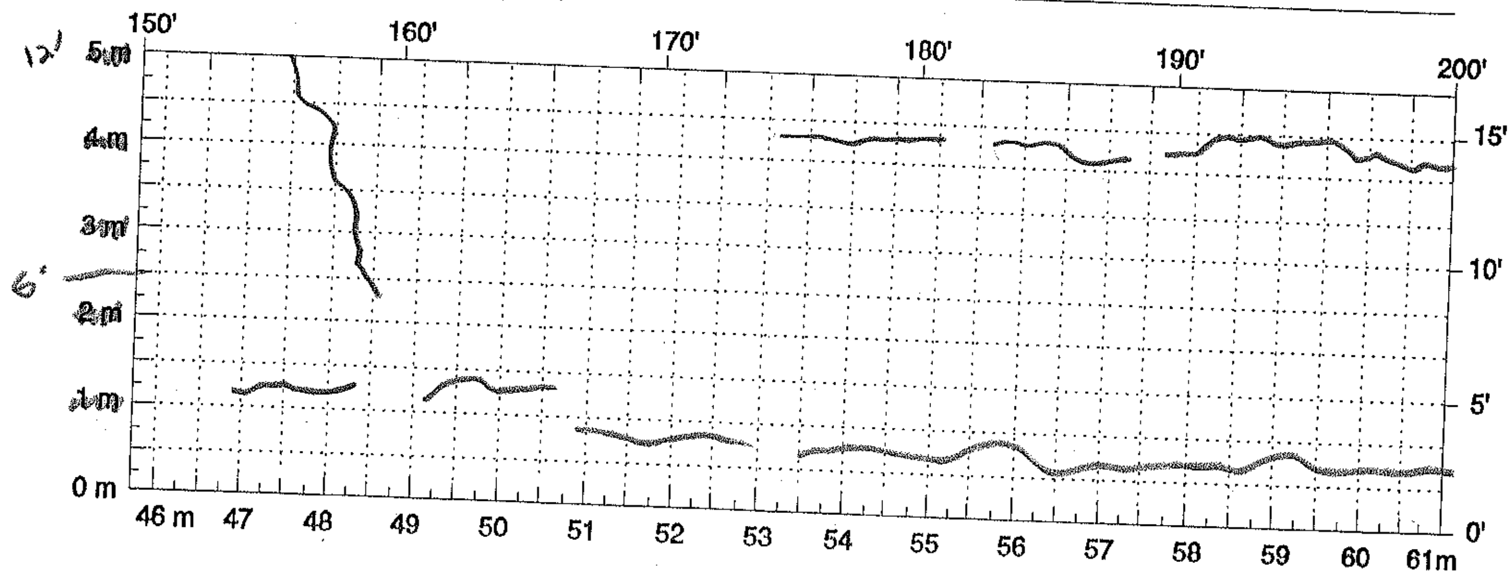

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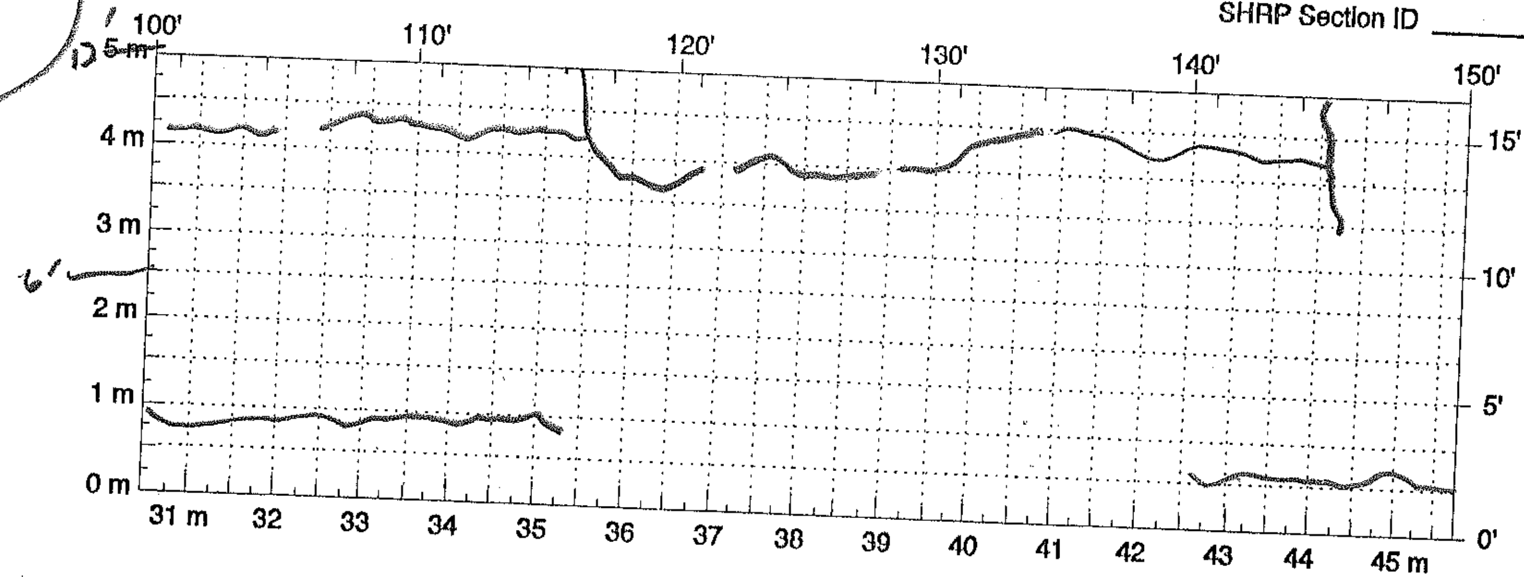
Site 3

Section 3

State Assigned ID ____________

State Code ____

SHRP Section ID ____________



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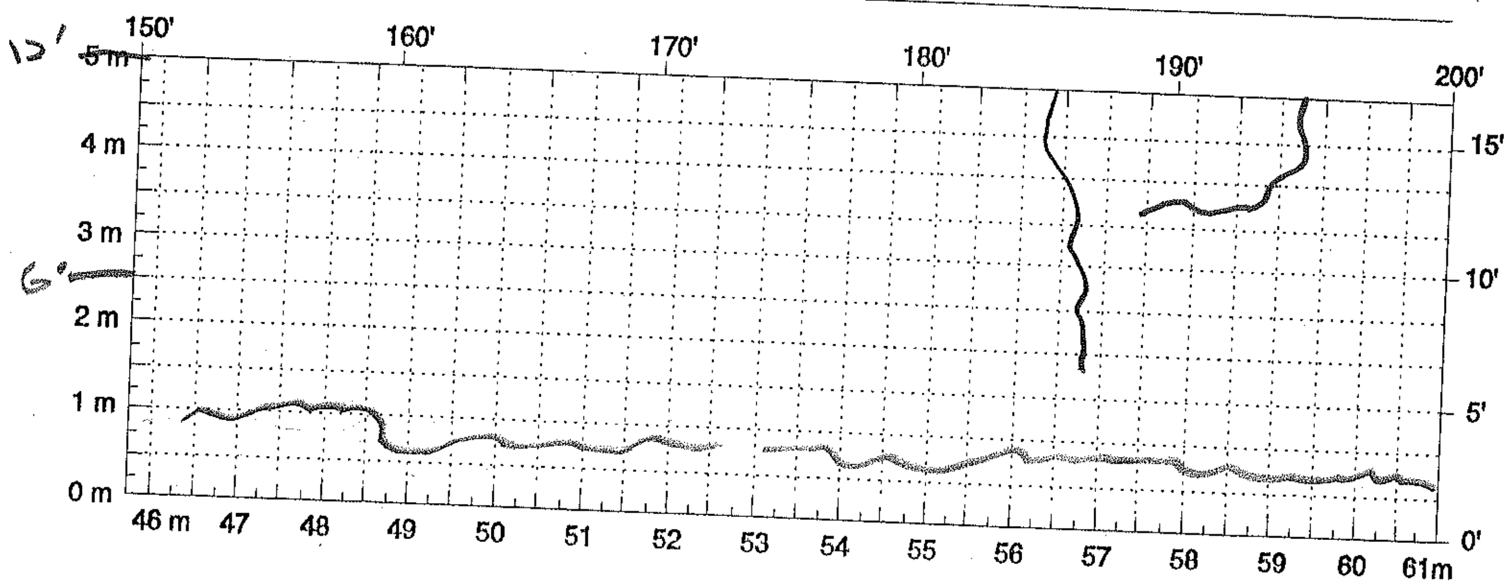


Comments: ____________

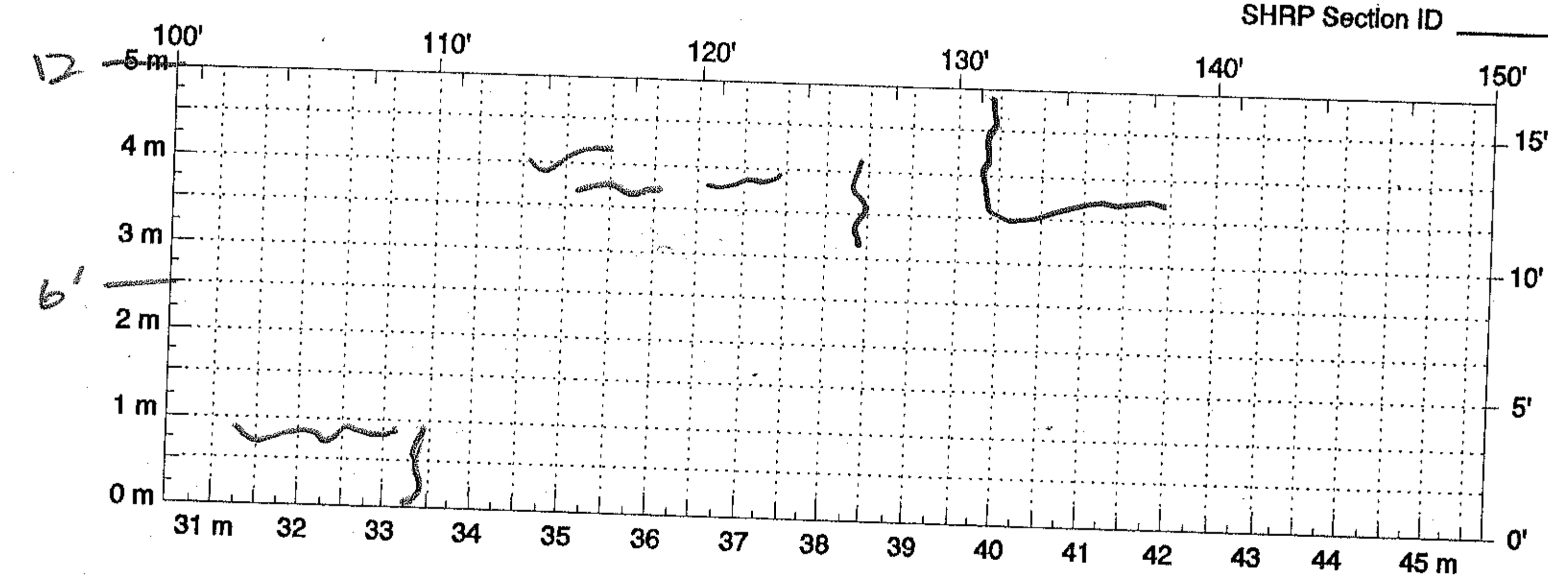
Section 4

Site 3

State Assigned ID ______

State Code ______

SHRP Section ID ______



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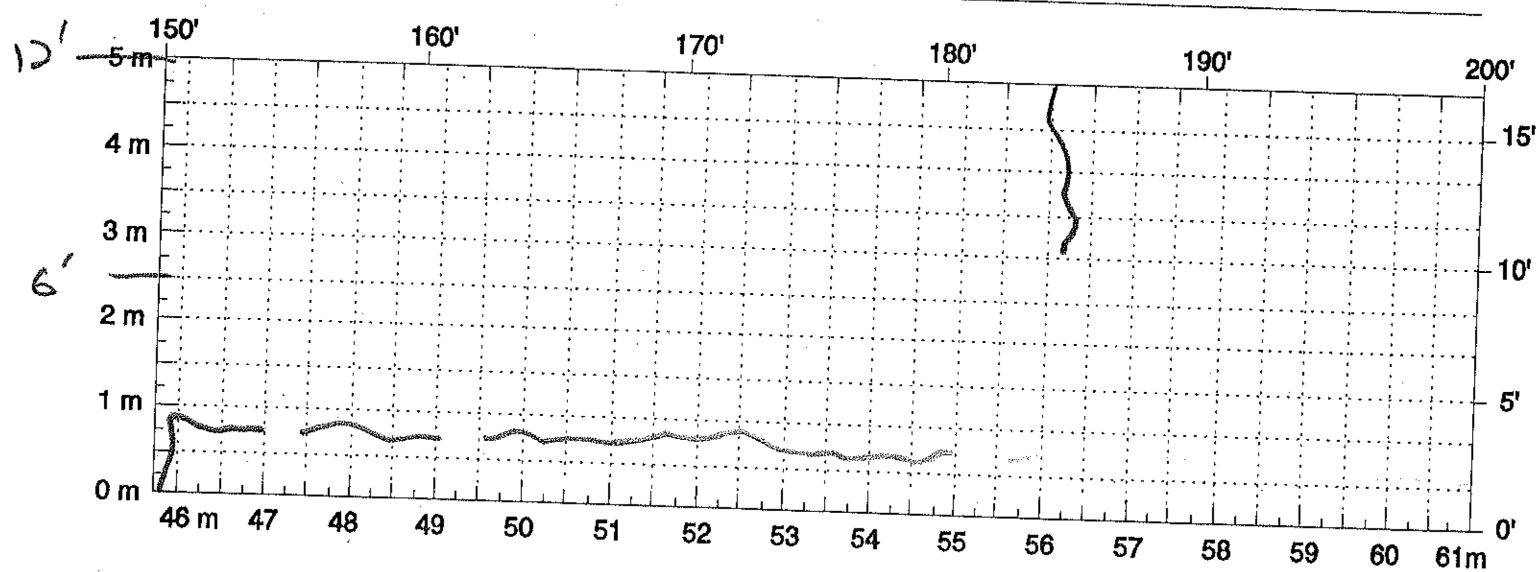

Comments: ______

APPENDIX G

TABLE G-1 Site S1, Sample 1A Grain Size Distribution, Asphalt Content = 5.21%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class 1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	63.6	6.57	3-8	3-8
0.150	#100	126.0	13.0		
0.300	#50	215.0	22.2	6-26	8-26
0.600	#30	321.0	33.2	10-32	16-36
1.18	#16	419.9	43.4		
2.36	#8	526.5	54.4	28-50	40-64
4.75	#4	656.6	67.8	40-65	55-80
9.5	3/8”	857.8	88.6	60-82	90-100
12.5	½”	921.7	95.2	70-100	100
19.0	¾”	968.0	100	90-100	
25.0	1”				
37.5	1-1/2”				
50.0	2”				

TABLE G-2 Site S1, Sample 2A Grain Size Distribution, Asphalt Content = 6.36%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class 1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	97.2	7.25	3-8	3-8
0.150	#100	186.9	13.9		
0.300	#50	312.0	23.3	6-26	8-26
0.600	#30	456.5	34.1	10-32	16-36
1.18	#16	595.9	44.5		
2.36	#8	748.5	55.9	28-50	40-64
4.75	#4	926.1	69.1	40-65	55-80
9.5	3/8”	1173.5	87.6	60-82	90-100
12.5	½”	1251.9	93.4	70-100	100
19.0	¾”	1324.4	98.9	90-100	
25.0	1”	1339.8	100		
37.5	1 ½”				
50.0	2”				

TABLE G-3 Site S1, Sample 3B Grain Size Distribution, Asphalt Content = 7.46%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class 1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	107.6	5.91	3-8	3-8
0.150	#100	229.2	12.6		
0.300	#50	407.2	22.4	6-26	8-26
0.600	#30	589.0	32.3	10-32	16-36
1.18	#16	760.1	41.7		
2.36	#8	962.9	52.9	28-50	40-64
4.75	#4	1214.4	66.7	40-65	55-80
9.5	3/8”	1540.3	84.5	60-82	90-100
12.5	½”	1720.0	94.4	70-100	100
19.0	3/4'”	1821.8	100	90-100	
25.0	1”				
37.5	1-1/2”				
50.0	2”				

TABLE G-4, Site S1, Sample 4A Grain Size Distribution, Asphalt Content = 6.72%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class 1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	144.8	7.80	3-8	3-8
0.150	#100	265.0	14.3		
0.300	#50	427.9	23.0	6-26	8-26
0.600	#30	618.0	33.3	10-32	16-36
1.18	#16	833.7	44.9		
2.36	#8	1094.1	58.9	28-50	40-64
4.75	#4	1409.1	75.9	40-65	55-80
9.5	3/8”	1768.0	95.2	60-82	90-100
12.5	½”	1833.6	98.8	70-100	100
19.0	3/4'”	1856.5	100	90-100	
25.0	1”				
37.5	1-1/2”				
50.0	2”				

TABLE G-5, Site S2, Sample 1A Grain Size Distribution, Asphalt Content = 6.41%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class 1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	152.5	7.26	3-8	3-8
0.150	#100	293.8	14.0		
0.300	#50	492.7	23.5	6-26	8-26
0.600	#30	714.4	34.0	10-32	16-36
1.18	#16	937.8	44.6		
2.36	#8	1209.1	57.6	28-50	40-64
4.75	#4	1537.9	73.2	40-65	55-80
9.5	3/8”	1920.8	91.4	60-82	90-100
12.5	½”	2033.9	96.8	70-100	100
19.0	3/4’”	2100.9	100	90-100	
25.0	1”				
37.5	1-1/2”				
50.0	2”				

TABLE G-6, Site S2, Sample 2B Grain Size Distribution, Asphalt Content = 6.50%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class 1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	87.6	7.56	3-8	3-8
0.150	#100	167.2	14.4		
0.300	#50	269.7	23.3	6-26	8-26
0.600	#30	384.1	33.1	10-32	16-36
1.18	#16	511.3	44.1		
2.36	#8	667.8	57.6	28-50	40-64
4.75	#4	846.8	73.0	40-65	55-80
9.5	3/8”	1057.6	91.2	60-82	90-100
12.5	½”	1132.8	97.7	70-100	100
19.0	3/4’”	1159.3	100	90-100	
25.0	1”				
37.5	1-1/2”				
50.0	2”				

TABLE G-7, Site S2, Sample 3A Grain Size Distribution, Asphalt Content = 6.40%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class 1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	68.6	7.13	3-8	3-8
0.150	#100	136.6	14.2		
0.300	#50	232.6	24.2	6-26	8-26
0.600	#30	337.2	35.0	10-32	16-36
1.18	#16	437.4	45.4		
2.36	#8	553.3	57.5	28-50	40-64
4.75	#4	699.8	72.7	40-65	55-80
9.5	3/8”	865.0	89.9	60-82	90-100
12.5	½”	937.1	97.4	70-100	100
19.0	3/4’”	962.6	100	90-100	
25.0	1”				
37.5	1-1/2”				
50.0	2”				

Table G-8, Site S2, Sample 4A Grain Size Distribution, Asphalt Content = 6.85%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class 1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	109.1	7.32	3-8	3-8
0.150	#100	215.7	14.5		
0.300	#50	371.9	25.0	6-26	8-26
0.600	#30	535.0	35.9	10-32	16-36
1.18	#16	692.8	46.5		
2.36	#8	882.2	59.2	28-50	40-64
4.75	#4	1139.9	76.5	40-65	55-80
9.5	3/8”	1389.9	93.3	60-82	90-100
12.5	½”	1479.4	99.3	70-100	100
19.0	3/4’”	1489.9	100	90-100	
25.0	1”				
37.5	1-1/2”				
50.0	2”				

Table G-9, Control Site, Sample 1B and 2B Grain Size Distribution, Asphalt Content = 8.35%

Sieve Size				Specification	
		Mass Passing	% Ind. Passing	Class1	Class 2
(mm)	(in)	(g)	(%)	(%)	(%)
0.075	#200	91.3	6.24	3-8	3-8
0.150	#100	178.9	12.2		
0.300	#50	370.3	25.3	6-26	8-26
0.600	#30	641.3	43.8	10-32	16-36
1.18	#16	900.1	61.5		
2.36	#8	1148.4	78.5	28-50	40-64
4.75	#4	1315.3	89.9	40-65	55-80
9.5	3/8”	1460.0	99.8	60-82	90-100
12.5	½”	1463.4	100	70-100	100
19.0	3/4'”			90-100	
25.0	1”				
37.5	1-1/2”				
50.0	2”				

APPENDIX H

2008 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		left IRI	right IRI
N	Valid	1159	1159
	Missing	0	0
Mean		1.3931	1.6418
Median		1.2400	1.4400
Mode		1.11	1.08(a)
Std. Deviation		.68576	.89721
Minimum		.33	.39
Maximum		6.34	10.06

a Multiple modes exist. The smallest value is shown

2007 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		l_iri	r_iri
N	Valid	559	559
	Missing	0	0
Mean		1.3735	1.5905
Median		1.2900	1.4500
Mode		1.00(a)	1.38
Std. Deviation		.51864	.73679
Minimum		.43	.50
Maximum		3.70	7.32

a Multiple modes exist. The smallest value is shown

2006 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		l_iri	r_iri
N	Valid	556	556
	Missing	0	0
Mean		1.3500	1.505
Median		1.2500	1.365
Mode		1.14(a)	1.2
Std. Deviation		.53956	.6598
Minimum		.45	.5
Maximum		4.14	5.4

a Multiple modes exist. The smallest value is shown

2004 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		l_iri	r_iri
N	Valid	581	581
	Missing	0	0
Mean		1.3485	1.5193
Median		1.2100	1.3800
Mode		.89(a)	1.24
Std. Deviation		.59862	.65456
Minimum		.49	.47
Maximum		7.21	7.77

a Multiple modes exist. The smallest value is shown

2003 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		l_iri	r_iri
N	Valid	554	553
	Missing	0	1
Mean		1.2857	1.458
Median		1.1950	1.360
Mode		1.02(a)	1.4
Std. Deviation		.48574	.5923
Minimum		.42	.5
Maximum		3.34	5.4

a Multiple modes exist. The smallest value is shown

2002 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		l_iri	r_iri
N	Valid	575	575
	Missing	0	0
Mean		1.266	1.3778
Median		1.170	1.2900
Mode		.8(a)	1.08
Std. Deviation		.4801	.53698
Minimum		.4	.44
Maximum		3.3	4.56

a Multiple modes exist. The smallest value is shown

2001 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		l_iri	r_iri
N	Valid	556	556
	Missing	0	0
Mean		1.2518	1.3647
Median		1.1700	1.2700
Mode		1.00	.76(a)
Std. Deviation		.48873	.56217
Minimum		.36	.36
Maximum		4.04	4.68

a Multiple modes exist. The smallest value is shown

2000 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		l_iri	r_iri
N	Valid	555	555
	Missing	0	0
Mean		1.2353	1.3430
Median		1.1400	1.2500
Mode		1.17	.95
Std. Deviation		.46308	.53958
Minimum		.39	.37
Maximum		3.14	4.01

1999 Statistics Measured on S.R. 695 EB Pavement over CIR Base, Mileage >= 1.077 miles

		l_iri	r_iri
N	Valid	585	585
	Missing	0	0
Mean		1.271	1.3976
Median		1.200	1.3200
Mode		1.2	.93
Std. Deviation		.4770	.52571
Minimum		.4	.53
Maximum		3.6	4.36

2008 Statistics Measured on S.R. 695 WB Pavement over CIR Base, Mileage >= 1.061 miles

		Left IRI	Right IRI
N	Valid	1123	1123
	Missing	0	0
Mean		1.2749	1.3983
Median		1.1200	1.2900
Mode		.85	1.33
Std. Deviation		.63878	.61442
Minimum		.29	.30
Maximum		4.91	6.03

2006 Statistics Measured on S.R. 695 WB Pavement over CIR Base, Mileage >= 1.061 miles

		l_iri	r_iri
N	Valid	560	560
	Missing	1	1
Mean		1.2714	1.3846
Median		1.1600	1.3050
Mode		.90	1.25
Std. Deviation		.52770	.48387
Minimum		.42	.45
Maximum		3.57	3.41

2004 Statistics Measured on S.R. 695 WB Pavement over CIR Base, Mileage >= 1.061 miles

		l_iri	r_iri
N	Valid	561	561
	Missing	0	0
Mean		1.2658	1.4000
Median		1.1500	1.3000
Mode		.88(a)	1.02
Std. Deviation		.54051	.48741
Minimum		.40	.52
Maximum		3.91	3.38

a Multiple modes exist. The smallest value is shown

2002 Statistics Measured on S.R. 695 WB Pavement over CIR Base, Mileage >= 1.061 miles

		l_iri	r_iri
N	Valid	561	561
	Missing	0	0
Mean		1.2108	1.3391
Median		1.1000	1.2700
Mode		.88	1.11
Std. Deviation		.50958	.46329
Minimum		.35	.46
Maximum		3.67	3.22

2000 Statistics Measured on S.R. 695 WB Pavement over CIR Base, Mileage >= 1.061 miles

		l_iri	r_iri
N	Valid	559	559
	Missing	1	1
Mean		1.1650	1.2840
Median		1.0400	1.2100
Mode		.84(a)	1.14
Std. Deviation		.48136	.46628
Minimum		.38	.41
Maximum		4.75	3.13

a Multiple modes exist. The smallest value is shown

2000 Statistics Measured on S.R. 695 WB Pavement (Control), Mileage < 1.061 miles

		l_iri	r_iri
N	Valid	178	178
	Missing	0	0
Mean		1.2970	1.1637
Median		1.1550	1.0700
Mode		1.08	.85(a)
Std. Deviation		.52012	.48616
Minimum		.52	.52
Maximum		3.34	3.64

a Multiple modes exist. The smallest value is shown

2002 Statistics Measured on S.R. 695 WB Pavement (Control), Mileage < 1.061 miles

		l_iri	r_iri
N	Valid	178	178
	Missing	0	0
Mean		1.280	1.2443
Median		1.155	1.1550
Mode		.9(a)	1.14
Std. Deviation		.4587	.45134
Minimum		.5	.54
Maximum		2.9	3.69

a Multiple modes exist. The smallest value is shown

2004 Statistics Measured on S.R. 695 WB Pavement (Control), Mileage < 1.061 miles

		l_iri	r_iri
N	Valid	178	178
	Missing	0	0
Mean		1.3954	1.2892
Median		1.2700	1.2150
Mode		.84(a)	1.01(a)
Std. Deviation		.49477	.46148
Minimum		.59	.44
Maximum		3.34	3.44

a Multiple modes exist. The smallest value is shown

2006 Statistics Measured on S.R. 695 WB Pavement (Control), Mileage < 1.061 miles

		l_iri	r_iri
N	Valid	178	178
	Missing	0	0
Mean		1.4541	1.263
Median		1.3600	1.190
Mode		1.12(a)	1.4
Std. Deviation		.43167	.4915
Minimum		.55	.5
Maximum		2.75	3.6

a Multiple modes exist. The smallest value is shown

2008 Statistics Measured on S.R. 695 WB Pavement (Control), Mileage < 1.061 miles

		left IRI	right IRI
N	Valid	357	357
	Missing	0	0
Mean		1.3690	1.4280
Median		1.2900	1.3500
Mode		1.02(a)	.90(a)
Std. Deviation		.53788	.51281
Minimum		.45	.45
Maximum		3.53	4.59

a Multiple modes exist. The smallest value is shown

APPENDIX I

Alternatives to the Conventional Overlay for Rehabilitation and Preservation of the State Highway Network

For presentation to Connecticut Department of Transportation Executives

Completed August 18, 2005

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Treatment: Cold-In-Place Recycling	
Description	• In-place pulverization of existing hot-mix-asphalt (HMA.) pavement at a typical thickness range between 2½ and 4½ inches, with the addition of binder (emulsified) and possibly virgin aggregate, all in a single pass, followed by compaction and subsequent overlay with either a surface-treatment or HMA, depending on traffic volume and structural need.
Benefits	• Elimination of thermal crack patterns (and the ensuing reflection through the surface layer.) • Homogenization of a potholed/patched/cracked surface. • Viable alternative when reflective cracking is a concern (an overlay is not effective at controlling distress from reflecting to the surface.) • Provides a uniform bound base for a durable pavement structure. • Can be preceded by milling to achieve crack elimination on very thick pavement structures, without the need to adjust other roadway appurtenances. • Depending on an evaluation of in-situ materials, additional virgin aggregate may be incorporated in order to provide additional thickness and structure. • Cost-effectiveness when compared to an equivalent treatment (milling and overlaying at the same depth), because of the elimination of the need for heating, hauling, and disposing of materials. • Environmentally friendly (energy consumption and emissions.) • 100% recycling of treated layer (no waste products). • Isolated, localized failed areas can be repaired after cold-in-place recycling and before the surface course is placed.
Concerns	• In the initial curing period the mix is substantially softer than HMA, which limits application to lower-volume roadways (Average Daily Traffic of 5,000 vehicles per lane; this suggests that a value in the range of 3,000 to 4,000 vehicles per lane (i.e. 6,000-8,000 two-way ADT for a two-lane highway) should be used as a conservative traffic "maximum" for project selection.) • Difficult-to-impossible to obtain full-depth cores during initial curing period. • Compaction at high end of the thickness range (> 4-1/2" recycling depth) may be difficult; requires use of heavy rollers. • Underlying base/subgrade failures are not addressed • Recycling at low thickness (< 2-1/2") raises uniformity concerns • Sufficient thickness of existing pavement is needed in order to achieve a quality product; a thickness of bound layers of > 4" is advisable • Cold-in-place recycling is made more complex by the abundance

	of utility manholes and outlets; may present constructability challenges in such areas (urban streets with a high number of these artifacts, for instance). • Compaction testing is usually accomplished by establishing the maximum possible compaction in the initial day of construction. • Addition of rejuvenator may complicate matters more than it helps; good results using polymer-modified emulsion and treatment of recycled material as "black aggregate" may be simpler, more effective, and avoid "soft asphalt" problems. • Direct-emulsion-injection systems of cold-in-place recycling should only be used on low-volume roadways. • Susceptible to failure if exceptional heavy rain during the first day or two of construction. • Requires some degree of (existing) pavement sampling and site-specific mix design. • Temperature-sensitive process would limit the recycling season approximately to about mid-May to late September. • Permeability can typically be on the high side, making it essential that the surface treatment or overlay of this material provide a waterproof cover over the recycled cold mix.
Cost	There no up-to-date cost data for cold-in-place recycling at this time. Connecticut's previous cold-in-place recycling project (dating from 1997) cost was…
Proper use	• Utilize on thick (>4") HMA pavements that exhibit environmental distress (raveling, thermal and shrinkage cracking.) • Limit to roads with ADT of 8,000 or less; alternatively, a limit based on number of trucks/day could be considered. • Specify a surface layer over the cold-in-place layer (suggest HMA at 3" for ADT 2,000-5,000, 4"+ (actual thickness based on traffic needs) for ADT > 5,000, thinner surface layer for ADT <=2,000.) • Sample in-place material prior to project and perform a mix design (to validate emulsion selection and determine whether addition of virgin aggregate is necessary.) • Repair localized areas of base failure prior to recycling. • Provide for repair, prior to overlay, of any isolated/localized areas of base failure left after recycling. • Best results obtained with full-lane recycling trains with re-crusher and pugmill. • Specify at least one heavy roller (25 tons or more, rubber-tired), a vibratory roller, and a finish roller, and control compaction through measurement of maximum achievable density.
Expected service life	• Depends on structural needs and pavement preservation. 20 years or longer could easily be obtained; in this sense it is similar

	to reconstruction. • Expected life of the surface layer depends on surface layer type (e.g. HMA overlay, rubber or conventional chip seal, microsurfacing, etc), routine maintenance, and preservation strategy. Refer to the appropriate section for each layer type. • Structural contribution comparable to HMA base.
Guidelines for selection	• Pavement condition: Pavements with more than 4" of bound-layer thickness; extensive transverse and longitudinal cracking, block/alligator cracking, and patching; no pervasive base/subgrade deficiencies. • Traffic volumes <=8,000 ADT. • Provide for HMA surface preservation just as in full-depth HMA pavements.
Ability to Implement	• ConnDOT specifications are in place for this treatment. Majority of effort would be in project selection and mix design.
References	http://www.pavementpreservation.org/library/getfile.php?journal_id=305