Thin Maintenance Surfaces

Phase Two Report with Guidelines for Winter Maintenance on Thin Maintenance Surfaces

Sponsored by
the Iowa Department of Transportation

Iowa Department of Transportation

and the Iowa Highway Research Board,
Project TR-435

Prepared by
the Department of Civil and Construction Engineering
IOWA STATE UNIVERSITY

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Thin Maintenance Surfaces: Phase Two (TR-435)

Abstract

In recent years there has been renewed interest in using preventive maintenance techniques to extend pavement life and to ensure low life cycle costs for our road infrastructure network. Thin maintenance surfaces can be an important part of a preventive maintenance program for asphalt cement concrete roads. The Iowa Highway Research Board have sponsored Phase Two of this research project to demonstrate the use of thin maintenance surfaces in Iowa and to develop guidelines for thin maintenance surface uses that are specific to Iowa.

This report documents the results of test section construction and monitoring started in Phase One and continued in Phase Two. The report provides a recommended seal coat design process based on the McLeod method and guidance on seal coat aggregates and binders. An update on the use of local aggregates for micro-surfacing in Iowa is included. Winter maintenance guidelines for thin maintenance surfaces are reported herein. Finally, Phase One's interim, qualitative thin maintenance surface guidelines are supplemented with Phase Two's revised, quantitative guidelines.

When thin maintenance surfaces are properly selected and applied, they can improve the pavement surface condition index and the skid resistance of pavements. For success to occur, several requirements must be met, including proper material selection, design, application rate, workmanship, and material compatibility, as well as favorable weather during application and curing. Specific guidance and recommendations for many types of thin maintenance surfaces and conditions are included in the report.

Thin Maintenance Surfaces

Phase Two Report
with Guidelines for Winter Maintenance on Thin Maintenance Surfaces

Principal Investigator (Phases One and Two)
Charles T. Jahren
Associate Professor of Civil and Construction Engineering
Iowa State University

Principal Investigator (Winter Maintenance Guidelines)
Wilfrid A. Nixon
Professor of Civil and Environmental Engineering
University of Iowa

Co-Principal Investigator (Phase One)
Kenneth L. Bergeson
Professor Emeritus of Civil and Construction Engineering
Iowa State University

Graduate Research Assistants
Ahmed Al-Hammadi
Serhan Celik
Jin Wook Chung
Gabriel Lau
Hernando Quintero
Jacob Thorius

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Prepared by
the Department of Civil and Construction Engineering
Iowa State University
394 Town Engineering Building
Ames, IA 50011

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Acknowledgments

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The research project advisory committee included the following members:

- Iowa DOT Office of Construction: Dave Jensen, P.E., and later Jeff Schmitt, P.E.
- Iowa DOT Office of Maintenance: John Selmer, P.E., and Francis Todey, P.E.
- Iowa DOT Office of Materials: John Heggen, P.E., and later Mike Heitzman, P.E.
- Carroll County: David Paulson, P.E.
- Kossuth County: Richard Scheik, P.E., L.S.
- City of Carroll: Randy Krauel, P.E.
- City of Newton: Neil Guess, P.E.
- Fort Dodge Asphalt: William Dunshee
- Koch Materials, Inc.: Bill Ballou (Dan Staebell, alternate)
- Sta-Bilt Construction Co.: Richard Burchett

CHAPTER 1. INTRODUCTION

In recent years there has been renewed interest in using preventive maintenance techniques to extend pavement life and to ensure low life cycle costs for our road infrastructure network. Thin maintenance surfaces (TMS) can be an important part of a preventive maintenance program for asphaltic concrete or bituminous roads.

The Iowa Department of Transportation (Iowa DOT) and Iowa Highway Research Board have sponsored a research project to demonstrate the use of thin maintenance surfaces in Iowa and to develop guidelines for thin maintenance surface uses that are specific to Iowa. This report documents the second phase of the research.

Summary of Phase One

Phase One of this research project included (1) a survey of local systems transportation officials to determine current practices in Iowa; (2) construction and monitoring of test sections; and (3) development of interim guidelines to help transportation officials to determine when and where to use thin maintenance surfaces.

Survey of Current Practices

The results of the survey of local systems transportation officials (Al-Hammadi 1998) indicated that seal coating using local aggregates is the most commonly used thin surface treatment. Counties and larger cities occasionally use slurry seal. Towns with populations under 5,000 had the greatest desire for additional information on thin maintenance surfaces.

Construction and Monitoring of Test Sections

Originally it was conceived that two sets of test sections would be constructed in July 1997 and monitored under the Phase One research project: one on US 151 east of Springville (northeast of Cedar Rapids) and the other on US 30 just west of the intersection with US 218 (west of Cedar Rapids). These test sections were constructed as part of two micro-surfacing maintenance projects. They included thin lift overlays, single chip seals (SCS), double chip seals (DCS), seal coat with fog seal (SC/fog), cape seal (seal coat with slurry seal top), slurry seal, micro-surfacing, and control sections. For various reasons, the contractor was delayed and did not place the test sections until September and October 1997. The test sections were of limited value because an inexperienced crew placed the seal coat and the cold weather that followed construction did not allow for adequate curing of many of the treatments.

After discussions with the research advisory committee, the decision was made to redirect the research effort. As a result, a third set of test sections was designed and constructed during the 1998 construction season, applying lessons learned from the previously completed test sections. These test sections were included as an extra work order for a micro-surfacing maintenance project on US 69 between Huxley and Ankeny, Iowa. Sta-Bilt was the contractor and Koch Materials, Inc., supplied the binder. The Minnesota Department of Transportation (Mn/DOT) offered to assist with the design and construction monitoring of the seal coat test sections.

Based on experience from the 1997 test sections, researchers and transportation officials noted that the finer surface of the 3/8-inch aggregate in a double seal coat was more desirable that of the 1/2-inch aggregate in a single seal coat. Therefore, all seal coat surface course aggregate was less than 3/8-inch for the 1998 test sections.

Test sections included various combinations of seal coat designs and materials: quartzite and limestone aggregate, cationic rapid set CRS-2P and high float rapid set HFRS-2P (polymer-modified) binder, and single and double chip seal. Other thin maintenance surface test sections included micro-surfacing and micro-surfacing with a chip seal interlayer. As a result of negotiations between the Iowa DOT and Koch Materials, Inc., two other test sections were added: an ultra-thin hot mix seal (Nova Chip) and a thin sand polymer hot mix overlay. A control section was also provided.

The seal coat designs resulted in a 25 percent to 33 percent savings in materials over the current practice, and initial performance has been favorable. Construction quality and curing conditions were much improved for the 1998 test sections; therefore, they promise to be a valuable source of performance data in the future.

Development of Interim Guidelines

The researchers developed an interim set of guidelines after reviewing the literature, examining the results of the survey of local systems transportation officials, reviewing test section performance, and holding discussions with the research advisory committee.

The interim guidelines (Jahren et al. 1999) provide a three-step process to guide the user as thin maintenance surfaces are selected. The first step is to assess the condition of the road network. The second is to identify treatments that are technically feasible by using a table and knowing the pavement condition and traffic load of the candidate road. The third step is to make a final selection between technically feasible alternatives by considering past practices, cost, durability, user preferences, neighbor preferences, and other factors that are difficult to quantify.

The interim guidelines were an improvement to the scattered information that previously existed. It was noted, however, that the guidelines could be improved by providing more rigorously defined decision points and guidance on when to use various types of aggregates and binders. The seal coat design process currently used by Mn/DOT is attractive because it reduces the amount of materials required when compared to current practice. When less aggregate is used, there is less fly rock, dust, and vehicle damage. It would be desirable to implement such a process on a statewide basis in Iowa. It would also be desirable to conduct additional technology transfer activities to make transportation officials more aware of the existing test sections and current interim guidelines. It was recommended that continued monitoring be provided for current test sections and additional test sections be constructed to provide additional comparisons between thin maintenance surface materials and mix design. Therefore, a second phase of research was proposed.

Phase Two Objectives

Phase Two of the research has six objectives:

1. Continue performance monitoring for previously placed test sections (see Chapter 2).
2. Construct and monitor additional test sections (see Chapter 2).
3. Evaluate design processes for seal coats and recommend one for implementation on a statewide basis (see Chapters 3 and 4).
4. Further investigate thin maintenance surface aggregates (see Chapter 5).
5. Investigate interactions between thin maintenance surfaces and winter maintenance activities (see Chapter 6).
6. Use the results of the performance monitoring, aggregate investigation, and additional test section construction to refine the guidelines for thin maintenance surfaces developed in Phase One. Provide additional guidance regarding the types and quality of material that should be used for various traffic loads, pavement conditions, and locations (e.g., urban vs. rural, turning and stopping traffic vs. steadily moving traffic). Also provide guidance regarding the amount and type of distress that can be addressed by thin maintenance surfaces. (Phase One interim guidelines are provided in Appendix A; the Phase Two revised guidelines are detailed in Chapter 7.)

Report Structure

Chapter 1 of this report contains the introductory material above. Chapter 2 reports the results of test section construction and monitoring (Objectives 1 and 2). Chapter 3 reports on seal coat materials—aggregates and binders, and Chapter 4 recommends a seal coat design process (Objective 3). Chapter 5 presents the results of the aggregate investigation for micro-surfacing (Objective 4). Chapter 6 provides winter maintenance guidelines conducted independently by Dr. Wilfrid A. Nixon and reported herein (Objective 5). Chapter 7 describes the thin maintenance surface guidelines developed under Objective 6. Chapter 8 provides conclusions and recommendations.

CHAPTER 2. TEST SECTION RESULTS

Three sets of test sections were constructed over the course of Phases One and Two of the research: US 151 and US 30 in 1997; US 69 in 1998; and US 218 in 1999.

For the sets of test sections built 1998 and after, each treatment was constructed in lengths of 1,600 to 10,560 feet. This allowed *surface condition index* (*SCI*) calculations to be based on the average of several samples, providing a good comparison of performance within the particular set of test sections. (The Army Corps of Engineers and other organizations use the terminology *pavement condition index* (*PCI*) when referring to deteriorations of pavements. For the purpose of this report we will refer to this as SCI because the state of Iowa uses PCI in a different way than what is meant in this report.) However, it is difficult to draw comparison of performance between sets of test sections because conditions varied too much from one set of test sections to another. Thus, each set of test sections (including the ones constructed before 1998) stand as an independent case study; conclusions and recommendations are not based on comparison of performance between sets of test sections.

No attempt was made to prioritize the relative importance of the SCI, skid resistance (SR), or roughness index (RI) measurements. The rating method for the SCI establishes the relative importance of the various types of surface distress that are present.

Maintenance crews were instructed to maintain the test sections according to their usual procedures, given the age and the type and severity of distress that the test sections were actually experiencing.

A demonstration project was also constructed in Carroll County, Iowa, on two county roads. Single and double limestone chip seals with designed application rates were used on a moderately trafficked road. Both CRS-2 and HFRS-2 emulsions were selected. A highly weathered road with light traffic had single and double pea gravel seal coats. Sections of limestone aggregate were also constructed.

Construction, data collection, and analysis of test sections are described in detail by Celik (1998), Lau (1999), and Quintero (2000). Their efforts are summarized below.

US 151 and US 30

In 1997, sets of TMS test sections were constructed on US 151, east of Springville, Iowa, and on US 30, north of Blairstown, Iowa, between IA 82 and US 218. The performance of the test sections was evaluated by comparing the SCI, individual distresses, and SR and RI values measured before construction to the same measurement criteria monitored over time after construction. The surface treatments applied in both sets of test sections included several types of seal coats with local aggregates, micro-surfacing, slurry seal, cape seal, and a thin lift hot mix overlay.

The section of US 151 used in this study was originally constructed in 1928 as a 20-foot wide, 7-inch-thick portland cement concrete (PCC) pavement. In 1953 it was widened to 24-feet and overlaid with 1.5 inches of asphalt cement concrete (ACC). In 1965, it was again overlaid with 1.5 inches of ACC, and in 1987 US 151 received a 2-inch ACC overlay.

The portion of US 30 that involved the test sections was originally constructed in 1949 as a 24-foot wide, 6.5-inch-thick PCC pavement. In 1965 it received a 3-inch ACC overlay, and in 1977 it received another 3-inch ACC overlay.

For each highway, starting from the west end, the test sections were generally placed in the following order and consisted of the following treatment types:

- control section 1
- micro-surfacing (Aggregate: Type 3 quartzite from Sioux Falls, South Dakota. Binder: a quick setting CSS-1H Polymer Modified Binder.)
- slurry seal (Aggregate: Type 3 limestone from Bowser/Springville Bed 7. Binder: CSS-1H.)
- cape seal (1/2-inch limestone seal coat on the bottom with a slurry seal top)
- single seal coat (SSC) (Aggregate: 1/2-inch limestone cover aggregate from Wendling South Cedar Rapids, Iowa, quarry. Binder: CRS-2P.)
- seal coat with fog seal (SC/fog) (Aggregate: 1/2-inch limestone cover aggregate from Wendling South Cedar Rapids, Iowa, quarry. Binder: CRS-2P.)
- double seal coat (DSC) (Aggregate: 1/2-inch bottom and 3/8-inch top limestone cover aggregate from Wendling South Cedar Rapids, Iowa, quarry. Binder: CRS-2P.)
- thin lift overlay (Overlay: 1.5-inch-thick Type A surface course. Aggregate: 1/2-inch with no special friction requirements. 50 blow Marshall Design.)
- control section 2

Each test section was approximately 1,500 feet long. On US 30, the micro-surfacing was placed after the thin lift overlay. See Figure 1 for the specific test section layout on US 151 (Jahren et al. 1999), and see Figure 2 for the specific test section layout on US 30 (Jahren et al. 1999).

The contracts for the construction of these test sections on US 151 and US 30 were included in micro-surfacing projects MP-151-6(701)45-76-57 and MP-30-6(700)229-76-06, respectively. These projects were awarded to Monarch Oil Company of Omaha, Nebraska. The test sections were planned for construction in July and August 1997, but due to the contractor's backlog of other projects, machine breakdowns, and difficult weather conditions, all the test sections except for the thin lift overlay were constructed in September and October 1997. The thin lift overlay was constructed on August 7 and 13, 1997, for US 151 and US 30, respectively, without major problems. However, several problems did occur during the construction of the seal coat, slurry seal, and micro-surfacing test sections.

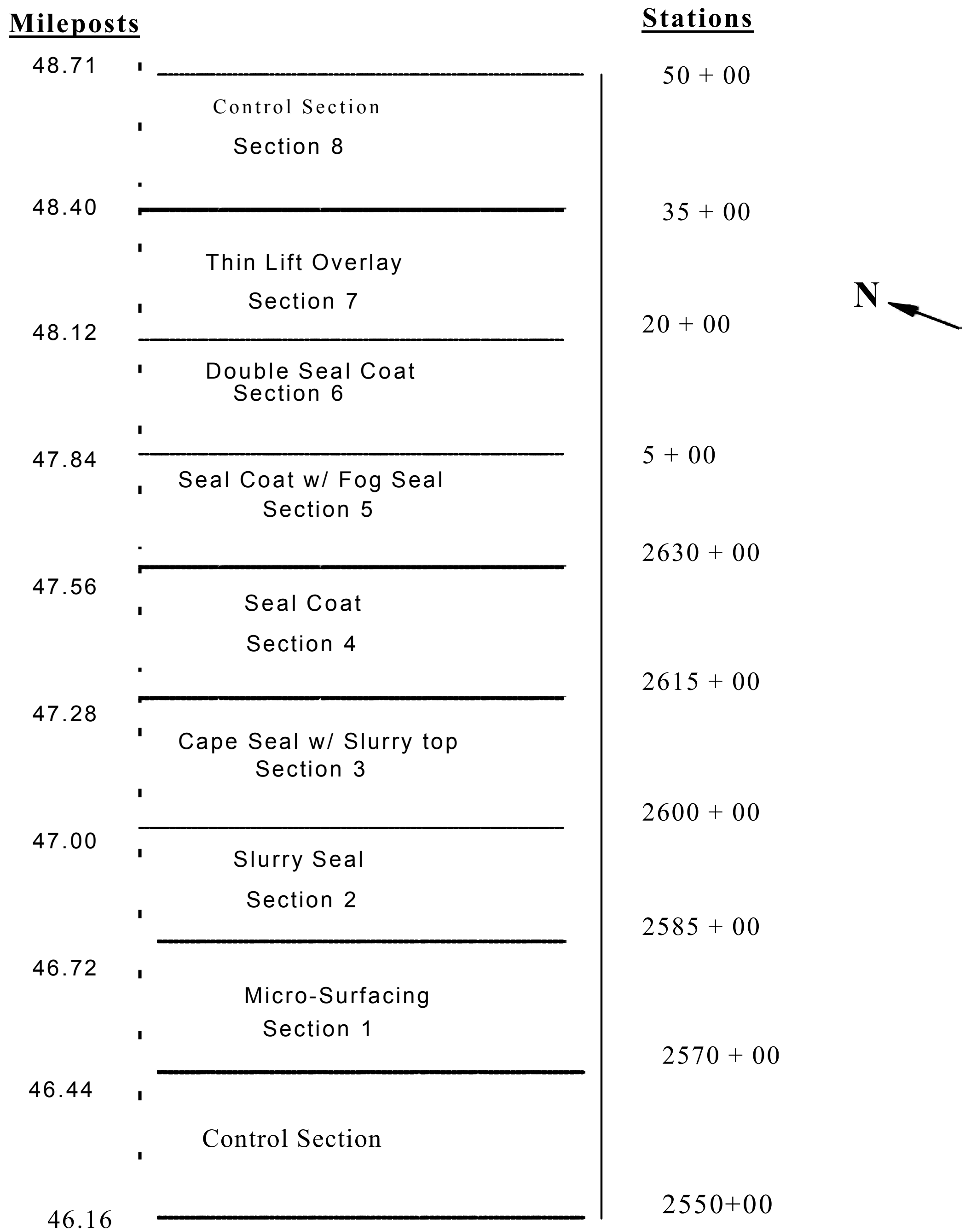


Figure 1. US 151 Test Section Layout

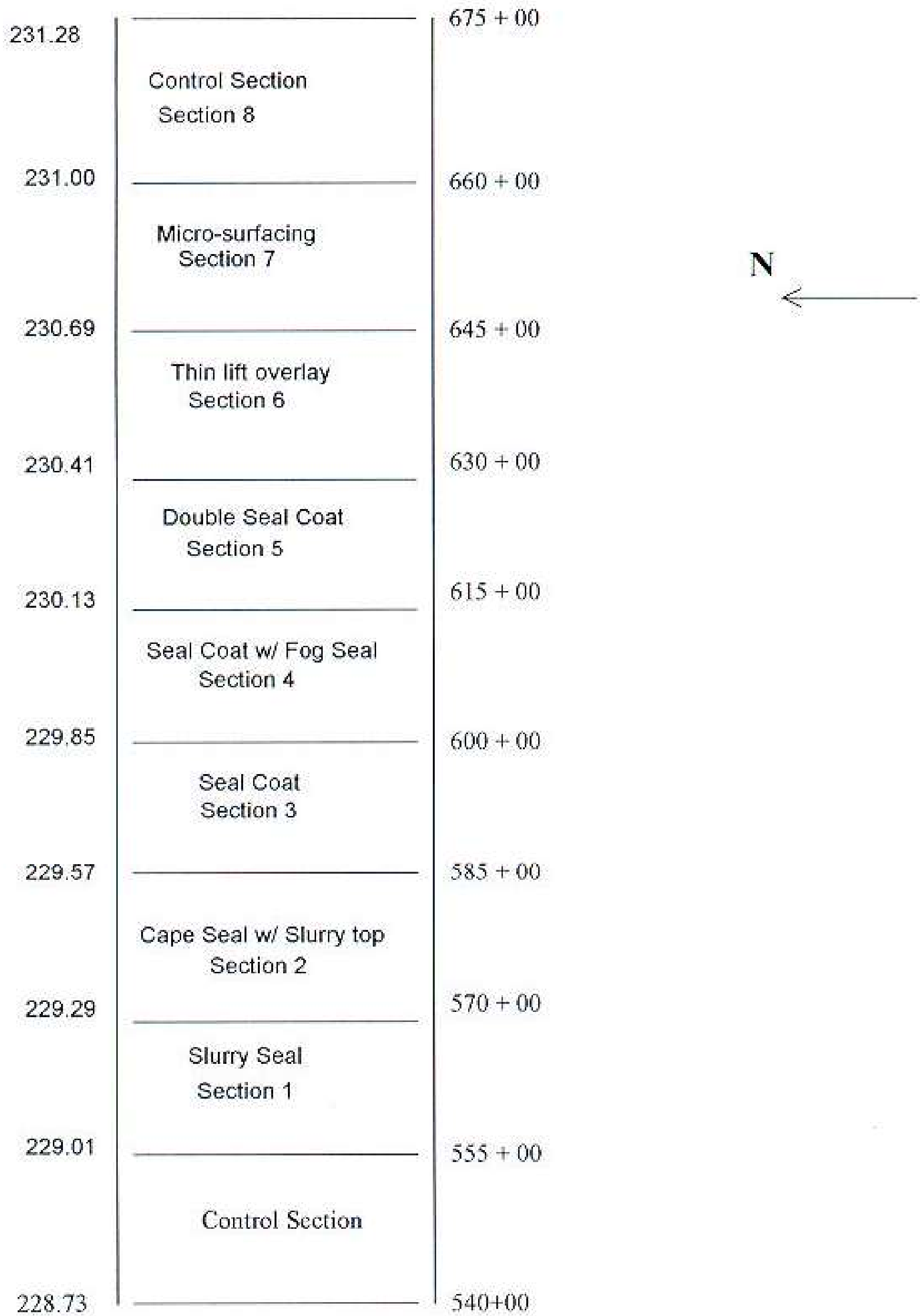


Figure 2. US 30 Test Section Layout

The application rates for the seal coats varied considerably from the intended rates. On US 151, calculation errors caused the rate at which the binder was applied for the seal coats to be low. As a result of the low amount of binder, the aggregate quickly raveled, leaving the reduced amount of binder exposed. For the double seal coat test section, the rate at which the binder was applied for the second layer was high, which likely caused the wheel tracks to flush. The sections using slurry seal on US 151 were initially placed on cold and rainy days, and in some cases insufficient time was given to allow the slurry to cure before traffic was returned to the road. This led to the slurry raveling in the wheel tracks and the consequent development of a ridge along the centerline where the slurry did not ravel. This centerline ridge was not addressed when the contractor reapplied the treatment and thus remained. The road surface on US 151 was also uneven within the micro-surfacing test sections. This caused the screed to scrape the road at high places and leave pools of material in the low places. These pools of material caused a flushed appearance in the low places, while the high places quickly raveled as a result of the thin application.

By June of 1998, excessive raveling had occurred on all the test sections on US 151 except for the double seal coat, thin lift overlay, and control sections. As a result of this raveling, micro-surfacing was applied to improve the road surface of those affected test sections. Further, the double seal coat test section was milled off in August 1998 due to excessive flushing and to improve skid resistance. Consequently, by the end of the trial period, the only test sections that were observed during the entire research period were the controls, thin lift overlay, and micro-surfacing.

The seal coats on US 30 had application rates for the aggregate that were high, thus causing considerable excess aggregate to be piled between the wheel tracks the morning after construction. This excess aggregate helped to loosen the aggregate that was bound to the road surface by wedging it out of place under traffic loads. The application rate of the slurry seal on US 30 was less than planned. This error occurred because slurry application started on the adjacent cape seal test section where the slurry application rate was supposed to be low. Consequently, when the machine placing the slurry entered the slurry seal test section, no adjustment was made to increase the application rate. Also, as a result of low temperatures at the time of construction, the westbound lanes of the cape seal and the slurry seal test sections were not surfaced with slurry seal as planned. The contractor then returned in May 1998 and only surfaced the westbound lane of the cape seal with slurry seal, because ensuing research had determined that slurry seal would unlikely be used on high-volume road (because of slurry seals' longer cure times that cause difficulty in opening the road back up to traffic).

For reasons similar to that of the slurry seal test sections, the micro-surfacing was only applied to the eastbound lane of US 30 in the fall of 1997. When the cold weather arrived, the material completely raveled away because it did not receive enough time to set up. When the contractor returned in May 1998, it finished placing the micro-surfacing test section on the westbound lane, and resurfaced the eastbound lane as a result of some raveling that had occurred.

On US 30, the slurry seal test sections failed completely due to raveling after they were constructed. The double seal coat test section failed in August 1998 as a result of flushing in the wheel tracks that decreased friction numbers (final range: 10 to 20). This range of friction numbers falls under the category of very hazardous driving conditions, so the Iowa DOT covered the double seal coat with slurry seal to restore friction. Because of the failure of these test sections, no further observations were conducted and no valid data were obtained.

US 151 Surface Condition Index

Prior to construction of the test sections in 1997 on US 151, the road was in poor condition, with SCI values ranging from 27 to 43, indicating the need for reconstruction. The largest deduct value from the pre-construction SCI values came from severe cracking reflected from the PCC pavement below. Alligator cracking was reflecting from the joint of the 2-foot widening strip that was laid in 1953 (the road was originally built with 10-foot lanes). Also this section of road was experiencing moderate longitudinal and transverse (L&T) cracking and an overall rough riding surface with some raveling. The test section with the highest SCI value prior to construction was control 2, with a value of 37. The sections selected for thin lift overlay and micro-surfacing had pre-construction SCI values of 29 and 27. Control 1 did not have a SCI value recorded before construction, since its location was not established until after construction was completed. Its first SCI value was measured in October 1997 (immediately after construction), while the other three sections initial SCI values were measured in July 1997.

After construction of the US 151 test sections, the new SCI values ranged from a low of 29, for control 1, to a high of 93, for the thin lift overlay. The micro-surfacing had a SCI value of 84, while control 2 had a SCI value of 33. From the time immediately after construction to May 2000, the SCI for the thin lift overlay decreased 20 points, while the micro-surfacing SCI decreased by 26 points during that same time, ending the test period at 73 and 58 points, respectively. Also, at the end of the test period, both control sections had SCI values of 24. Table 1 lists the SCI values for the test sections as they were measured over time.

Table 1. US 151 SCI Values

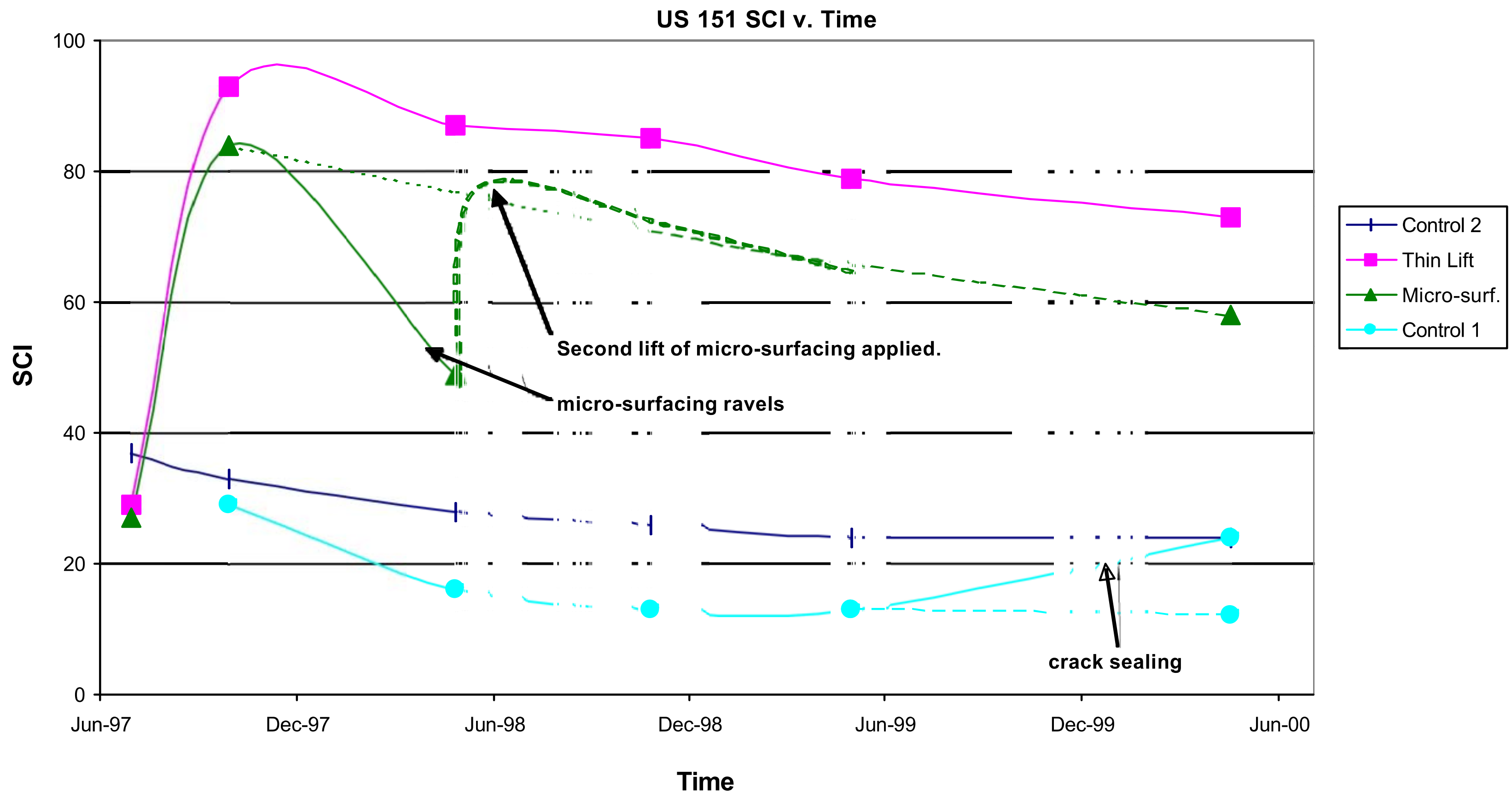
Survey	Control 2	Thin Lift	Micro-Surfacing	Control 1
07/1997	37	29	27	*
10/1997	33 (-4)	93 (+64)	84 (+57)	29
05/1998	28 (-9)	87 (+58)	49 (+22)	16 (-13)
11/1998	26 (-11)	85 (+56)	**	13 (-16)
05/1999	24 (-13)	79 (+50)	**	13 (-16)
05/2000	24 (-13)	73 (+44)	58*** (+31)	24**** (-5)

Note: Values in parentheses indicate change in SCI value from July 1997.
* Section location not established until after construction (added during construction).
** Not surveyed.
*** Second lift of micro-surfacing improved SCI.
**** Crack sealing improved SCI.

Figure 3 shows a graph of SCI values plotted against time. This graph helps show the trend of the SCI values over time and compares them with the other test sections. The control sections are shown at the bottom of the graph. They show steady deterioration throughout the study. The exception is that control 1 shows improvement during the last observation due to a crack-sealing program. A speculative dashed line shows the likely result if the cracks were not sealed. The thin lift overlay shows immediate post-construction improvement and then deteriorates at a rate similar to the control sections (note parallel line). The post-construction condition of the micro-surfacing had a lower SCI than the thin lift overlay. Rapid deterioration followed as the micro-surfacing raveled. A second lift of micro-surfacing was placed to correct the raveling, and the observation team did not make further observations because the test section was considered to have failed. However, one last observation was made in May 2000, when the researchers decided to treat the test section as two lifts of micro-surfacing. Note that if a line is plotted from the post construction micro-surfacing SCI to the final observation, the deterioration rate is similar to that of the thin lift overlay and the control sections.

While both TMS treatments helped improve the SCI value immediately after construction, the thin lift overlay improved the condition of the test section more and the improvement lasted longer. Two and a half years after construction, the main distresses affecting the thin lift overlay were low to moderate levels of L&T cracking, and low levels of joint reflective (JR) cracking and weathering/raveling. The main distresses that the micro-surfacing experienced two and a half years after construction were moderate levels of L&T cracking and low levels of alligator cracking, JR cracking, rutting, and weathering/raveling.

Meanwhile, control 1 and control 2 saw their respective SCI values decrease by 5 and 9 points after construction, with control 2 decreasing by 13 points from prior to construction. (Recall control 1 was not established until after the test sections were constructed.) Control 2's decrease in its SCI value was about the same each year, while control 1 decreased by 16 points the first year after construction and then increased the third year after construction, due to a crack sealing maintenance operation. The main distresses affecting control 1 were high levels of alligator cracking, moderate levels of block cracking and L&T cracking and low levels of JR cracking, patch/utility cutting, potholing, rutting, and weathering/raveling. Control 2 experienced high levels of alligator cracking, moderate levels of block cracking, L&T cracking, and rutting, and low levels of JR cracking, potholing, and weathering/raveling. Table 2 lists the deduct values for each distress that each test section experienced about three years after construction.



Note:
Dashed lines for micro-surfacing are speculative results for a second lift of micro-surfacing being applied during initial construction. Dashed lines for control 1 are speculative results if crack sealing had not been applied to control 1 during the testing period. The sections that were treated showed an initial increase in SCI value because all the cracks were filled and covered. However, over time the SCI values decreased because primarily longitudinal and transverse cracking began to reflect through the TMS.

Figure 3. US 151 SCI vs. Time

Table 2. Types of Distresses in US 151 Test Sections Three Years After Construction

Section	Alligator Cracking	Block Cracking	JR Cracking	L&T Cracking	Patch/Utility Cutting	Potholing	Rutting	Weathering/ Raveling
Control 2	(78.13) **78.28**	(13.10) **39.72**	(10.10) **10.11**	(47.09) **34.83**	(0.07) **0.0**	(0.0) **4.51**	(0.0) **50.88**	(0.0) **1.71**
Thin lift	(76.68) **0.0**	(14.94) **0.0**	(7.87) **7.87**	(45.15) **24.36**	(38.29) **0.0**	—	—	(0.0) **3.16**
Micro-surfacing	(80.71) **13.03**	(14.71) **0.0**	(19.22) **7.87**	(51.68) **40.54**	—	—	(0.0) **17.08**	(6.30) **7.08**
Control 1	(73.21) **73.99**	(25.78) **30.10**	(7.87) **7.87**	(37.13) **54.53**	(0.0) **1.24**	(2.00) **2.00**	(0.0) **17.08**	(0.0) **10.26**

Note: The deduct values for the types of distresses the US 151 test sections experienced about three years after construction are shown in bold. Values in parentheses are those experienced before construction.

US 151 Skid Resistance

Before construction, the SR values for both lanes ranged from 30.5 to 32.0. The SR values were then measured again after construction in October 1997 and July 2000. Table 3 lists the SR values that were measured over the test time period for each test section, and Figure 4 shows a graphical comparison of the SR values before and after construction. The October 1997 values ranged from 35.5 to 57.0. In July 2000, about three years after construction, all the test sections had an increase in their SR values from before construction and they ranged from 34.0 to 58.25.

Table 3. US 151 Skid Resistance Test Results

Section	Skid Resistance			Change in Friction*
	Before Construction	10/1997	07/2000	
Control 2	31.0	35.5	34.75	+3.75
Thin lift	32.0	50.0	42.0	+10.00
Micro-surfacing	30.5	57.0	58.25	+27.75
Control 1	32.0	35.75	34.0	+2.00

Note: Average of separate measurements for northbound and southbound lanes.

* Change in friction is between the July 2000 and before construction values.

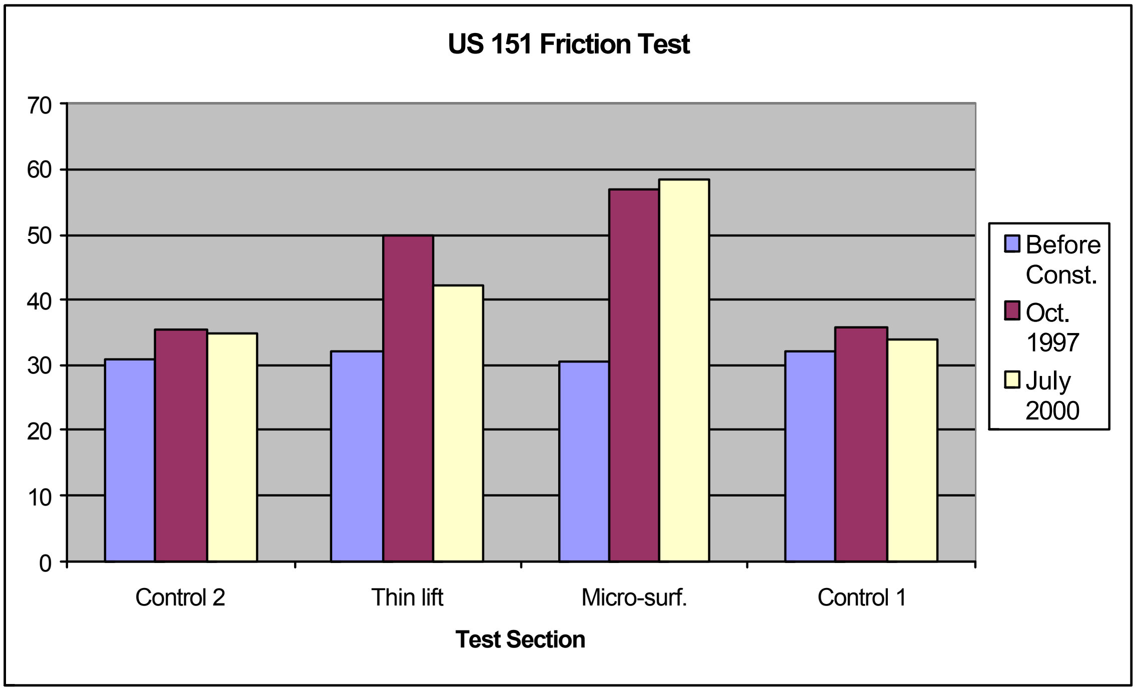


Figure 4. US 151 Friction Test Values

The thin lift overlay improved its after construction (October 1997) SR value from its before construction SR value by 18 points, while the micro-surfacing section improved its after construction SR value by 26.5 points. However, some of the improvement in the SR value for the micro-surfacing may be attributed to the fact that slurry strips were placed in the outer wheel tracks in 1999. The July 2000 SR value for the thin lift overlay lost about 8 points from its after construction SR value, while the micro-surfacing SR value remained about the same. Meanwhile, since before construction to July 2000, the control sections' SR values remained about the same.

US 151 Roughness Index

RI was used to assess roughness. Data were recorded by a South Dakota Type Profiler according to ASTM standards E950 and E1170. The results were averaged over the length of the test sections. The test equipment was adjusted to detect roughness with wavelengths up to 300 feet. According to Shahin (1994), wavelengths over 100 feet on highways have little effect on vehicle ride, whereas for airport runway wavelengths 400 feet might be significant. Shahin also states that the International Roughness Index (IRI) is sensitive to wavelengths between 4.2 feet and 75 feet. Therefore the RI used in the study cannot be compared to the IRI. However the RI used in the study can be used to compare roughness among test sections.

The RI values measured prior to construction in July of 1997 ranged from 2.406 to 2.979 for the four test sections. These values were again measured in July 2000, approximately three years after construction, and ranged from 1.555 to 3.110. Table 4 is a compilation of the RI values for each test section that was measured before and after construction, and Figure 5 is a graphical representation of the RI values obtained before and after construction. All but the test sections, except for control 1 had a decrease in their RI values in July 2000 from those measured in July 1997. The thin lift overlay had the greatest decrease in its RI value over the three-year period: its July 2000 value was 0.851 points less than the July 1997 value. The micro-surfacing decreased its RI value by 0.272 points. The July 2000 value for control 2 remained nearly the same as its 1997 value, only 0.097 points less, while control 1 increased its RI value in July 2000 by 0.131 points.

Table 4. US 151 Roughness Index

Section	RI (m/km)		Change in Roughness
	07/1997	07/2000	
Control 2	2.642	2.545	-0.097
Thin lift	2.406	1.555	-0.851
Micro-surfacing	2.507	2.235	-0.272
Control 1	2.979	3.110	+0.131

Note: Average of separate measurements for northbound and southbound lanes.

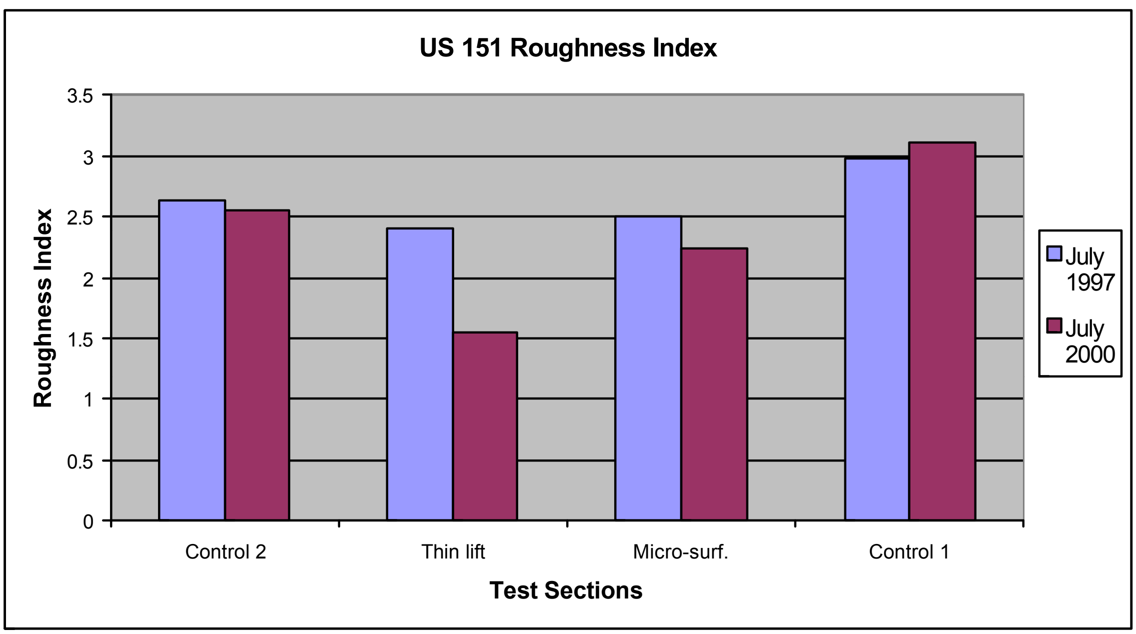


Figure 5. US 151 Roughness Index Values

US 30 Surface Condition Index

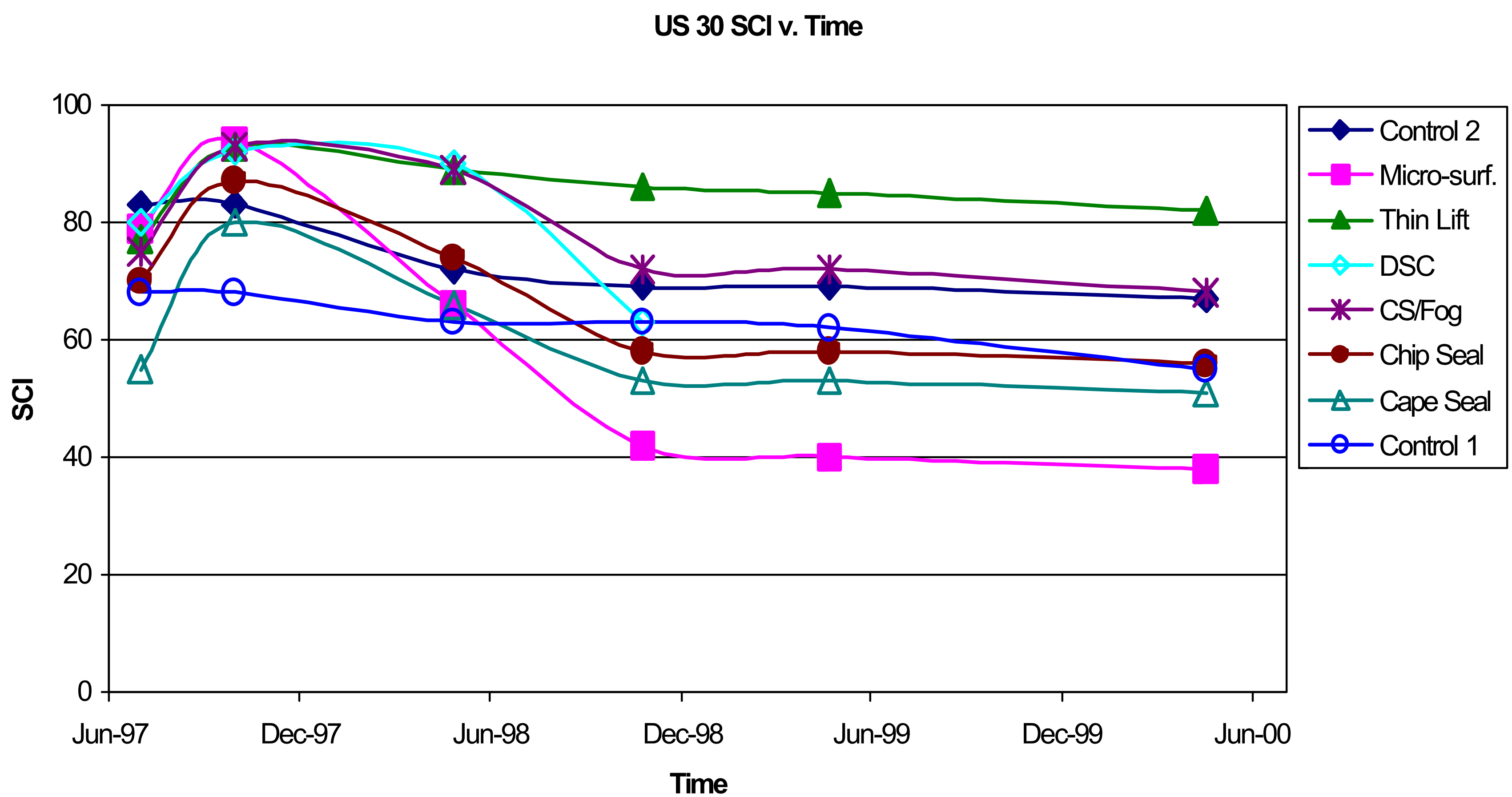
Prior to construction of the US 30 test sections in 1997, the road was in good condition, with SCI values ranging from 55 to 83. This road did exhibit some light to moderate cracking reflected from the PCC pavement below. The control 2 section had the highest measured pre-construction SCI value, with a value of 83. SCI values of 79, 77, 75, and 70 were recorded, respectively, for the micro-surfacing, thin lift overlay, seal coat with fog seal, and chip seal sections. The lowest recorded SCI value before construction was for the cape seal section, at 55, and the next lowest was the control 1 section, with a value of 68. All of the test sections were experiencing low to moderate levels of L&T cracking and low levels of edge cracking. Also the seal coat with fog seal, seal coat, and control 1 were experiencing low to moderate levels of patch/utility cutting distress, while the cape seal experienced moderate levels of patch/utility cutting distress. For US 30 the distress in this category consisted entirely of patches.

Initially after construction of the test sections on US 30, the new SCI values ranged from a low of 80, for the cape seal, to a high of 94, for the micro-surfacing. Table 5 summarizes the SCI values that were obtained for the test sections on US 30, and Figure 6 shows a graphical representation of the SCI values. The maintenance treatments indicate improvement from the pre-construction condition followed by deterioration in the first year (1998). The deterioration for the TMS was more rapid than that of the thin lift overlay. The micro-surfacing deterioration was especially rapid due to raveling. Subsequently, the deterioration rate was approximately the same for all treatments and the control sections.

Table 5. US 30 SCI Values

Survey	Control 2	Micro-Surfacing	Thin Lift	DSC	SC/Fog	Chip Seal	Cape Seal	Control 1
07/1997	83	79	77	80	75	70	55	68
10/1997	83 (0)	94 (+15)	93 (+16)	92 (+12)	93 (+18)	87 (+17)	80 (+25)	68 (0)
05/1998	72 (-11)	66 (-13)	89 (+12)	90 (+10)	89 (+14)	74 (+4)	66 (+11)	63 (-5)
11/1998	69 (-14)	42 (-37)	86 (+9)	*	72 (-3)	58 (-12)	53 (-2)	63 (-5)
05/1999	69 (-14)	40 (-39)	85 (+8)	*	72 (-3)	58 (-12)	53 (-2)	62 (-6)
05/2000	67 (-16)	38 (-41)	82 (+5)	*	68 (-7)	56 (-14)	51 (-4)	55 (-13)

Note: Values in parentheses indicate change in SCI value from July 1997.
* Covered with slurry seal due to severe bleeding.



The sections that were treated showed an initial increase in SCI value because all the cracks were filled and covered. However, over time the SCI values decreased primarily because longitudinal and transverse cracking began to reflect through the TMS. Rutting also played a significant role to the reduction in SCI value for the TMS, except for the thin lift overlay. Due to a late season placement of the micro-surfacing, weathering/raveling also contributed significantly to the decline of its SCI value.

Figure 6. US 30 SCI vs. Time

Immediately after construction, the thin lift overlay had a SCI value of 93 along with the seal coat with fog seal, and the seal coat improved its SCI value to 87. Both the control sections had their SCI values remain the same as measured prior to construction. Two and a half years later the SCI values for the test sections ranged from a low of 38, for the micro-surfacing, to a high of 82 for the thin lift overlay. The seal coat with fog seal and control 2 had SCI values of 68 and 67, respectively, while the chip seal, control 1, and cape seal had SCI values of 56, 55, and 51, respectively. The thin lift overlay had a reduction in its SCI value of 11 points in two and a half years, but retained a higher SCI value than that recorded before construction. On the other hand, the micro-surfacing saw its SCI value decline by 56 points right after construction, and ended 41 points lower than recorded before construction. At the end of the test period, the thin lift overlay had improved the SCI value for that section of road by 5 points, and was the only test section method that had improved its overall SCI value.

The main distresses affecting the thin lift overlay included low levels of bleeding, edge cracking, L&T cracking, and weathering/raveling. The micro-surfacing performed the worst out of the test sections used on US 30, with the largest deduct values coming from moderate levels of weathering/raveling, followed by low to moderate levels of L&T cracking and rutting, and low levels of JR cracking. Low to moderate levels of L&T cracking and rutting were the main distresses affecting the cape seal and control 1 sections, while the rest of the test sections experienced low levels of numerous other types of distresses. Table 6 lists the deduct values for each distress experienced by the test sections about three years after construction.

Table 6. Types of Distresses in US 30 Test Sections Three Years After Construction

Section	Bleeding	Edge Cracking	JR Cracking	L&T Cracking	Patch/Utility Cutting	Polished Aggregate	Rutting	Weathering/ Raveling
Control 2	—	(5.62) **9.83**	(6.34) **7.87**	(8.82) **14.27**	(2.06) **9.04**	—	(0.0) **20.93**	(6.02) **8.00**
Micro-surfacing	—	(3.57) **0.0**	(0.0) **12.50**	(17.19) **28.34**	—	—	(0.0) **33.68**	(6.02) **45.42**
Thin lift	(0.0) **0.55**	(6.89) **6.48**	(4.17) **0.0**	(21.88) **13.08**	(9.57) **0.0**	—	(0.0) **0.0**	(6.02) **7.08**
SC/fog	(0.0) **18.17**	(13.63) **0.0**	(5.37) **5.37**	(16.39) **9.10**	(13.18) **0.0**	—	(0.0) **20.93**	(6.02) **6.02**
Chip seal	(0.0) **7.09**	(8.59) **6.48**	(5.37) **1.30**	(18.95) **11.37**	(20.01) **0.0**	—	(0.0) **32.52**	(6.02) **17.79**
Cape seal	(0.0) **7.05**	(16.98) **6.48**	(1.95) **0.0**	(23.95) **35.06**	(44.38) **5.67**	(0.0) **2.47**	(0.0) **32.52**	(6.02) **4.76**
Control 1	—	(11.05) **11.37**	(7.87) **20.70**	(22.64) **36.11**	(18.84) **18.84**	(0.0) **4.81**	—	(6.02) **6.02**

Note: The deduct values for the types of distresses the US 30 test sections experienced about three years after construction are shown in bold. Values in parentheses are those experienced before construction.

US 30 Skid Resistance

Before construction in 1997, the SR values for US 30 ranged from 41 to 43. The SR values were then measured again after construction in October 1997 and July 2000. The results from the SR tests performed before and after construction of the test sections on US 30 are listed in Table 7, and Figure 7 shows a graphical comparison of these values. The October 1997 SR values ranged from 49.25 to 64.0, and then in July 2000 they ranged from 37.5 to 61.0. The micro-surfacing improved its SR value the most over the test period: its July 2000 value was 18.50 points greater than that measured before construction. Increased skid resistance might have been a side benefit that resulted from the raveling of the micro-surfacing. The only other test section to improve its SR value was the thin lift overlay; its July 2000 value was 9.75 points greater than the one measured before construction. All the other test sections and control sections saw a decrease in their SR values after an initial increase in their respective SR values. The cape seal, CS/fog, and chip seal had decreases in their July 2000 SR values of 3.0, 3.5, and 5.25 points, respectively, from their before construction values. The July 2000 SR values decreased by 1.5 and 5.5 points from the values in October 1997 for control 2 and control 1, respectively.

Table 7. US 30 Skid Resistance Test Results

Section	Skid Resistance*			Change in Friction**
	Before Construction	10/1997	07/2000	
Control 2	—	49.25	47.75	-1.50
Micro-surfacing	42.5	51.0	61.0	+18.50
Thin lift	42.5	55.75	52.25	+9.75
SC/fog	41.0	52.5	37.5	-3.50
Chip seal	43.0	54.0	37.75	-5.25
Cape seal	41.0	64.0	38.0	-3.00
Control 1	—	51.0	45.5	-5.50

* Average of separate measurements for eastbound and westbound lanes.

** Change in friction is between July 2000 and the earliest measured SR value, either the before construction or October 1997 value, depending on the test section.

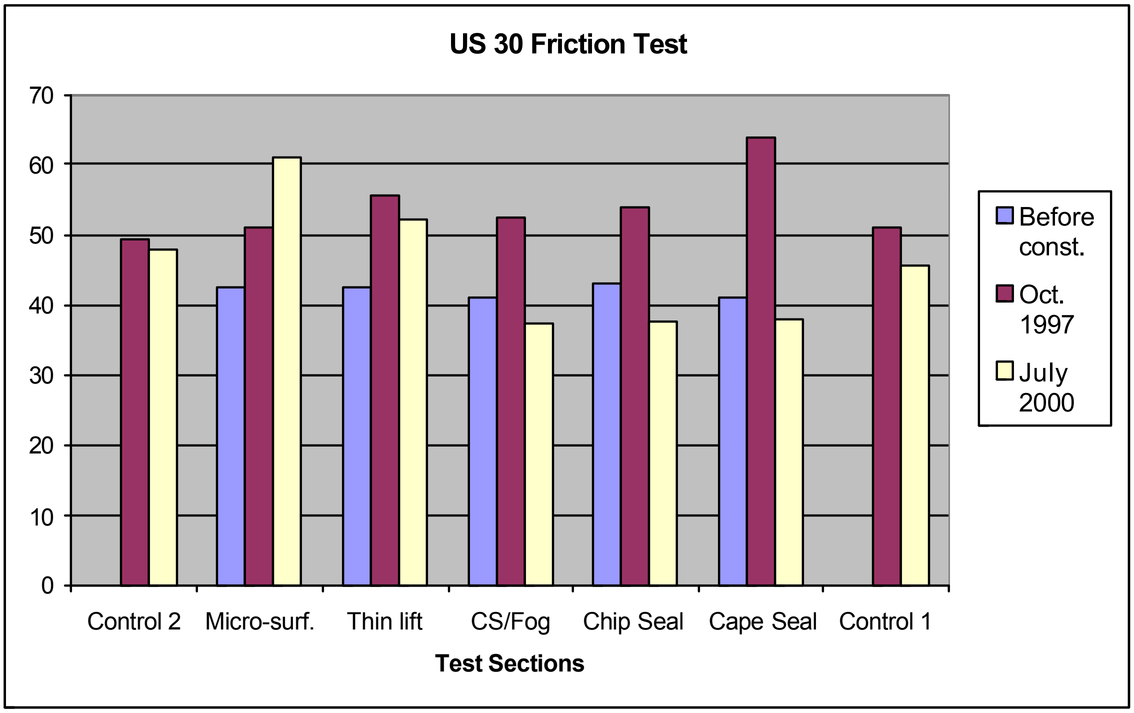


Figure 7. US 30 Friction Test Values

US 30 Roughness Index

RI values were measured in July of 1997, before construction of the test sections on US 30, and then about three years after construction in July of 2000. Table 8 comprises a list of these measurements, which were used to determine if the test sections changed the RI values for the section of interest on US 30, and Figure 8 shows a graphical representation of the RI values. The 1997 RI values ranged from 1.149 to 1.449. By July 2000, all the RI values had increased, except for the thin lift overlay, which decreased its RI value by 0.061 points from that measured before construction. The CS/fog section increased it RI value the least, only by 0.148, and then the control 1 and 2 sections increased their RI values by 0.204 and 0.207 points. Overall, the cape seal test section was the roughest section with an increase in its RI value of 0.484 points. The next roughest sections of road were the micro-surfacing and the chip seal, with an increase from their July 1997 to their July 2000 RI values of 0.475 and 0.459 points, respectively. The increase in the RI value for the micro-surfacing is most likely due to the raveling that occurred on the test section.

Table 8. US 30 Roughness Index

Section	Roughness Index* (m/km)		Change in Roughness
	07/1997	07/2000	
Control 2	1.410	1.617	+0.207
Micro-surfacing	1.241	1.716	+0.475
Thin lift	1.449	1.388	-0.061
SC/fog	1.149	1.297	+0.148
Chip seal	1.396	1.855	+0.459
Cape seal**	1.286	1.770	+0.484
Control 1	1.291	1.495	+0.204

*Average of separate measurements for eastbound and westbound lanes.
** Not surfaced with slurry seal in the westbound lane.

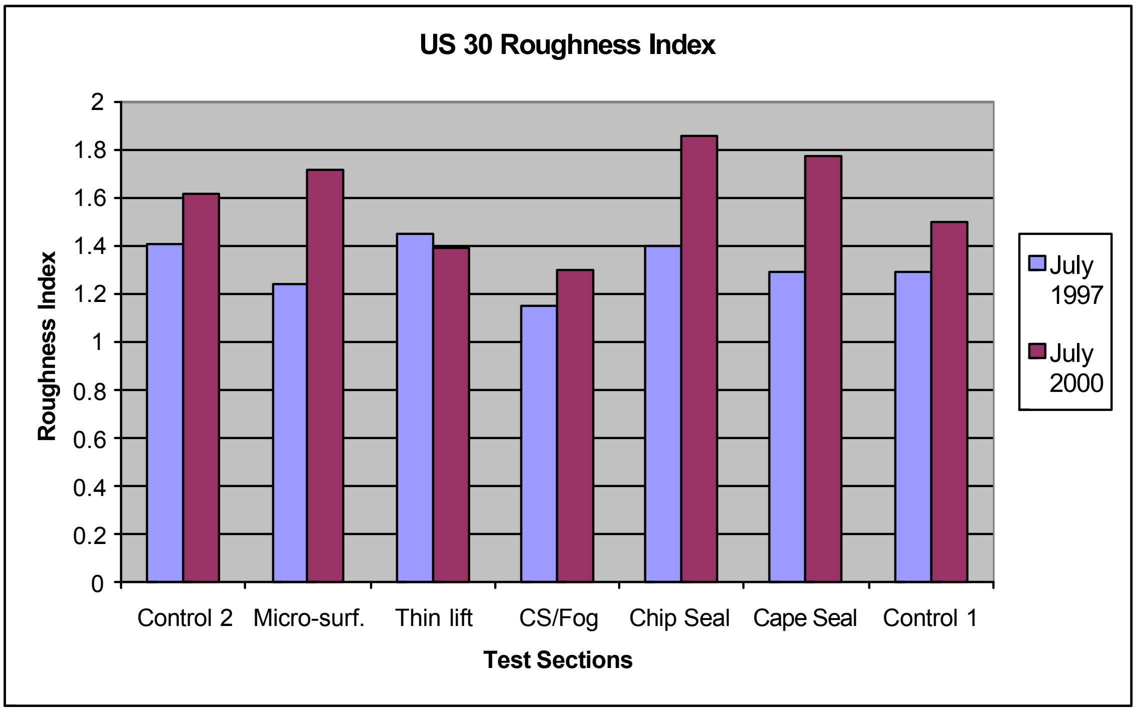


Figure 8. US 30 Roughness Index Values

US 69

In 1998, a set of TMS test sections was constructed on US 69, between Huxley and Ankeny, Iowa. The surface treatments used for this set of test sections included two types of thin lift overlays, Nova Chip and hot sand mix, micro-surfacing, and several types of seal coats using a variety of local and imported aggregates. The tests and observations conducted for this set of test sections are similar to those performed on US 151 and US 30.

The section of US 69 that included the test sections was originally constructed in 1948 as a PCC pavement. Then in 1956 and 1967 it was overlaid with hot mix asphalt (HMA), and in 1990 the road was milled and a 2-inch layer of HMA was laid. Starting from the south end of the test sections were placed in the following order and consisted of the following treatment types (see Figure 9 for the US 69 test section layout):

- micro-surfacing (Aggregate: Type 3 quartzite from L.G. Everest, Inc., Sioux Falls, South Dakota, with gradation on the coarse side of the allowable band. Binder: polymer-modified CSS-1H, specifically Ralumac, provided by Koch Materials, Inc.)
- control section
- double seal coat (DSC) #4 (southbound lane) and #8 (northbound lane) (Aggregate: 1/2-inch crushed limestone bottom course from Martin Marietta Ames Mine; 3/8-inch quartzite top course from L.G. Everest, Inc., Sioux Falls, South Dakota. DSC #4 aggregate is cleaner and more one-sized than that of DSC #8. Binder: CRS-2P.)
- single seal coat (SSC) #4 (southbound lane) and #8 (northbound lane) (Aggregate: 3/8-inch quartzite from L.G. Everest, Inc., Sioux Falls, South Dakota. SSC #4 aggregate is cleaner and more one-sized than that of SSC #8. Binder: CRS-2P.)
- single chip seal (SCS) w/ CRS-2P (Aggregate: 1/4-inch crushed limestone from Martin Marietta Ames Mine. Binder: CRS-2P.)
- double chip seal (DCS) w/ CRS-2P (Aggregate: 1/2-inch crushed limestone bottom course and 1/4-inch crushed limestone top course, both from Martin Marietta Ames Mine. Binder: HFRS-2P on the bottom course and CRS-2P on the top course.)
- double chip seal (DCS) w/ HFRS-2P (Aggregate: 1/2-inch crushed limestone bottom course and 1/4-inch crushed limestone top course, both from Martin Marietta Ames Mine. Binder: HFRS-2P on both courses.)
- single chip seal (SCS) w/ HFRS-2P (Aggregate: 1/4-inch crushed limestone from Martin Marietta Ames Mine. Binder: HFRS-2P.)
- thin lift overlays
  northbound lane: hot sand mix (Aggregate: 80% quartzite manufactured sand and 20% local mason sand. Binder: polymer modified.)
  southbound lane: Nova Chip (Aggregate: Gap graded blend of local limestone and imported quartzite of maximum size 1/2-inch.)

Both the hot sand mix and the Nova Chip test sections served as demonstrations for two products that had not been previously constructed in Iowa. The hot sand mix used a polymer-modified binder to help increase its stability in high temperatures and to reduce its cracking in low temperatures, two extremes that affect Iowa's roads. The hot sand mix was placed using traditional HMA paving methods, while the Nova Chip was placed with a special paver that first sprayed a heavy emulsion tack coat (branded Nova Bond) on the pavement surface and then placed a hot mix layer on top.

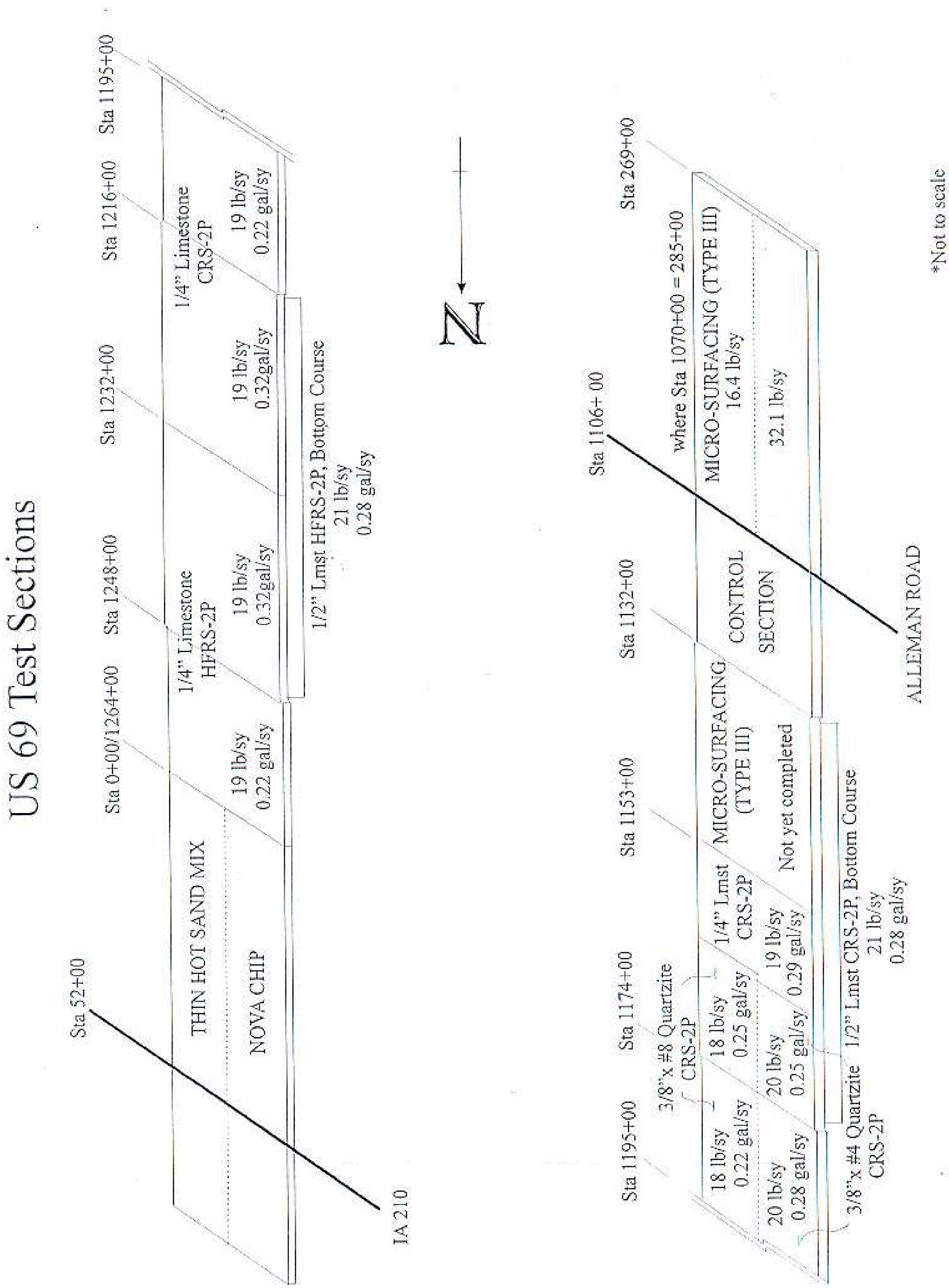


Figure 9. US 69 Test Section Layout

The test sections were constructed under the micro-surfacing maintenance contract MP-69-1(700)96-76-7, with an extra work order that was negotiated between the Iowa DOT and the contractor, Sta-Bilt Construction of Harlan, Iowa. The target application rates for the seal coat aggregate and binder were obtained using the Minnesota seal coat design procedure. Prior to construction, the chip spreader was calibrated to ensure target application rates. During construction, the target application rate was adjusted after visual inspection.

US 69 Surface Condition Index

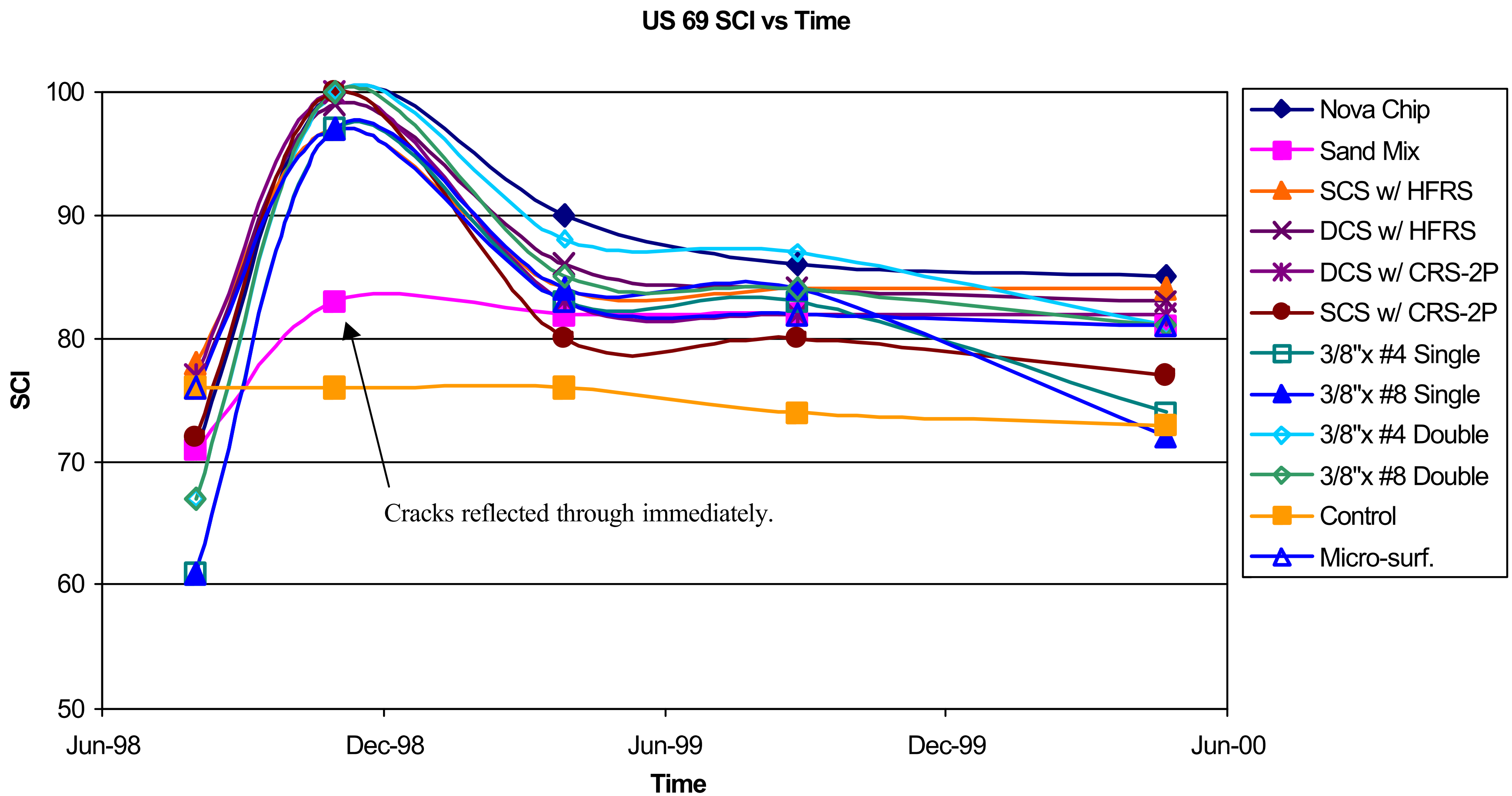
Prior to construction in August 1998, the SCI values ranged from 61 to 78, with all test and control sections showing similar types of distress and severity. Table 9 summarizes all SCI values that were obtained for the test sections placed on US 69, and Figure 10 shows a graphical representation of the values. Except for the hot sand mix, all of the treatments provided a post-construction SCI in excess of 95 and then deteriorated over the winter (mostly due to cracking, raveling, and bleeding) to SCIs between 80 and 90. Thereafter, deterioration rates matched the deterioration rate of the control section. There were some exceptions. Both quartzite seal coats and, to some extent, the single limestone chip seal with CRS-2P binder deteriorated at a faster rate during the winter of 1999/2000 because of snowplow damage. The hot sand mix did not attain a post-construction SCI that was as high as others, because immediately after construction hairline cracks reflected through from underlying cracks. The deterioration rate of the hot sand mix after the initial hairline crack matched the control sections.

Table 9. US 69 SCI Values

Survey	Nova Chip	Sand Mix	SCS w/ HFRS-2P	DCS w/ HFRS-2P	DCS w/ CRS-2P	SCS w/ CRS-2P
08/1998	71	71	78	76	77	72
11/1998	100 (+29)	83 (+12)	97 (+19)	99 (+23)	100 (+23)	100 (+28)
04/1999	90 (+19)	82 (+11)	84 (+6)	86 (+10)	83 (+6)	80 (+8)
09/1999	86 (+15)	82 (+11)	84 (+6)	84 (+8)	82 (+5)	80 (+8)
05/2000	85 (+14)	81 (+10)	84 (+6)	83 (+7)	82 (+5)	77 (+5)

Survey	SSC #4	SSC #8	DSC #4	DSC #8	Control	Micro-Surfacing
08/1998	61	61	67	67	76	76
11/1998	97 (+36)	97 (+36)	100 (+33)	100 (+33)	76 (0)	97 (+21)
04/1999	83 (+22)	84 (+23)	88 (+21)	85 (+18)	76 (0)	83 (+7)
09/1999	83 (+22)	84 (+23)	87 (+20)	84 (+17)	74 (-2)	82 (+6)
05/2000	74 (+13)	72 (+11)	81 (+14)	81 (+14)	73 (-3)	81 (+5)

Note: Values in parentheses indicate change in SCI value from August 1998.



The sections that were treated showed an initial increase in SCI value because all the cracks were filled and covered. However, over time the SCI values decreased because primarily longitudinal and transverse cracking began to reflect through the TMS. For both the #4 and #8 quartzite seal coats, raveling due to snowplow damage also contributed to the decline in SCI values.

Figure 10. US 69 SCI vs. Time

A detailed discussion follows about the about the distresses affecting the test sections. L&T cracking were the most notable distresses affecting the pre-construction values, while some other distresses affecting all SCI values were edge cracking, JR cracking, bleeding, and weathering/raveling. The 3/8-inch quartzite #4 and #8 SSC test sections had the lowest SCI values of 61 each; these test sections also had the highest L&T cracking and weathering/raveling deduct values, 27.9 and 17.06, respectively. The quartzite seal coat test sections suffered severe snowplow damage. The SCS w/ HFRS-2P had the highest SCI value of 78, while the control, micro-surfacing, DCS w/ HFRS-2P, and DCS w/ CRS-2P test sections also had similarly high SCI values (76, 76, 76, and 77, respectively).

Initially after construction of the test sections on US 69, the new SCI values ranged from a low of 76, for the control, to a high of 100, for the Nova Chip, DCS w/ CRS-2P, SCS w/ CRS-2P, and 3/8-inch #4 and #8 DSC. The DCS w/ HFRS-2P had a SCI value of 99 and the SCS w/ HFRS-2P, 3/8-inch #4 and #8 SSC, and micro-surfacing all had SCI values of 97, while the hot sand mix had a SCI value of 88. About one and a half years later the SCI values for the test sections ranged from a low of 73, for the control, to a high of 85 for the Nova Chip. The SCS w/ HFRS-2P, DCS w/ HFRS-2P, and DCS w/ CRS-2P had SCI values of 84, 83 and 82, respectively, while the hot sand mix, 3/8-inch #4 and #8 DSC, and micro-surfacing had SCI values of 81. The SCS w/ CRS-2P, 3/8-inch #4 and #8 SSC sections had SCI values of 77, 74, and 72, respectively.

By May 2000 (1.5 years after construction), the smallest reduction in SCI value after construction of the test sections was for the hot sand mix test section, a decrease of 2 points, but still a 10 point increase from before construction. During the one and a half years after construction, the Nova Chip, SCS and DCS w/ HFRS-2P, micro-surfacing, and 3/8-inch #4 and #8 DSC had their SCI values decrease by 15, 13, 16, 16, 19, and 19 points, respectively. However, those SCI values were an overall increase from before construction of 14, 6, 7, 5, 14, and 14 points, respectively. The DCS and SCS w/ CRS-2P and 3/8-inch #4 and #8 SSC test sections had a decrease of 18, 23, 23, and 25, respectively, but they saw a net increase of their SCI value measured prior to construction of 5, 5, 13, and 11 points. The only test section that did not improve its SCI value any time after construction was the control section, as expected. The control had a decrease in its SCI value of 3 points during the one and a half-years after construction.

In May 2000, the main distresses affecting all of the test sections were low to moderate L&T cracking, and low weathering/raveling, except for the 3/8-inch #4 and #8 SSC which experienced low to moderate weathering/raveling. The better performing test sections in terms of SCI values were the Nova Chip and the 3/8-inch #4 and #8 DSC, which improved their SCI values by 14 points. Next were the 3/8-inch #4 and #8 SSC sections, which improved their respective SCI values by 13 and 11 points, respectively. Table 10 lists the deduct values for each type of distress that was experienced by each test section about two years after their construction.

Table 10. Types of Distresses US 69 Test Sections Experienced Two Years After Construction

Section	Bleeding	Edge Cracking	JR Cracking	L&T Cracking	Rutting	Weathering/ Raveling
Nova Chip	(18.14) **0.0**	(9.57) **0.0**	(10.23) **0.0**	(17.89) **13.42**	—	(0.0) **1.67**
Hot sand mix	(5.30) **0.0**	(9.12) **0.0**	(9.60) **0.0**	(19.19) **16.80**	—	(5.99) **1.88**
SCS w/ HFRS-2P	(0.08) **0.15**	(4.70) **0.0**	(7.87) **0.0**	(15.38) **13.67**	—	(4.47) **4.55**
DCS w/ HFRS-2P	(5.65) **0.92**	(9.59) **0.0**	(7.87) **0.0**	(14.45) **13.69**	—	(6.57) **4.49**
DCS w/ CRS-2P	(3.24) **1.29**	(9.83) **0.0**	(7.87) **0.0**	(14.38) **14.56**	—	(4.80) **7.08**
SCS w/ CRS-2P	(2.24) **0.10**	(10.25) **0.0**	(7.87) **7.87**	(15.70) **18.91**	(6.45) **0.0**	(7.42) **7.08**
SSC #4	(9.29) **0.90**	(11.13) **0.0**	(1.49) **12.01**	(27.91) **10.06**	(5.58) **0.0**	(17.06) **23.50**
SSC #8	(9.29) **2.47**	(11.13) **0.0**	(1.49) **0.0**	(27.91) **16.31**	(5.58) **0.0**	(17.06) **25.08**
DSC #4	(17.15) **3.25**	(9.74) **0.0**	(7.87) **0.0**	(22.85) **12.48**	—	(11.12) **8.34**
DSC #8	(17.15) **5.10**	(9.74) **0.0**	(7.87) **0.0**	(22.85) **13.36**	—	(11.12) **8.34**
Control	(4.01) **4.01**	(9.13) **9.83**	(7.87) **7.87**	(16.22) **18.41**	—	(5.67) **5.29**
Micro-surfacing	(4.32) **0.29**	(4.73) **0.0**	(7.87) **0.0**	(14.05) **16.23**	**—**	(6.08) **4.58**

Note: Deduct values for the types of distresses the US 69 test sections experienced about two years after construction are shown in bold. Values in parentheses are those experienced before construction.

US 69 Skid Resistance

Prior to construction, the most recent SR values for these test sections were taken in the summer of 1995. These tests were conducted on two locations of US 69. The first location was in the southbound lane from the Polk/Story County line to the junction of US 69 with Alleman Road, the average SR value was 50. The second location was in the northbound lane from the junction of US 69 with Alleman Road to the junction of US 69 with IA 210, the average SR value was 55. The SR value used for the test sections in these locations will be the average of both locations, 52.5. Even though these values are older than preferred, they do give a general idea of what the SR value for the test sections was like prior to construction. To see how the SR test sections changed, SR values were measured both one and two years after construction and compared with each other. Where applicable, SR values are the average of separate tests taken for the northbound and southbound lanes. Such an average is not possible where different treatments were placed in different lanes. Table 11 is compilation of these SR values for each test section, and Figure 11 is a graphical representation of the values.

Table 11. US 69 Skid Resistance Test Results

Section	Skid Resistance*			Change from 07/1995	
	07/1995	10/1999	07/2000	10/1999	07/2000
Nova Chip	52.5	45.0**	49.0**	-7.5	-3.5
Hot sand mix	52.5	53.0***	57.0***	+0.5	+4.5
SCS w/ HFRS-2P	52.5	49.0	53.15	-3.5	+0.65
DCS w/ HFRS-2P	52.5	50.5	53.5	-2.0	+1.0
DCS w/ CRS-2P	52.5	51.5	53.35	-1.0	+0.85
SCS w/ CRS-2P	52.5	51.5	53.65	-1.0	+1.15
SSC #4	52.5	55.0**	57.0**	+2.5	+4.5
SSC #8	52.5	57.0***	55.3***	+4.5	+2.8
DSC #4	52.5	53.0**	54.7**	+0.5	+2.2
DSC #8	52.5	54.0***	55.0***	+1.5	+2.5
Control	52.5	47.5	46.8	-5.0	-5.7
Micro-surfacing	52.5	53.0	55.2	+0.5	+2.7

* Average of separate measurements for northbound and southbound lanes, where applicable.
** Placed in southbound lane only.
*** Placed in northbound lane only.

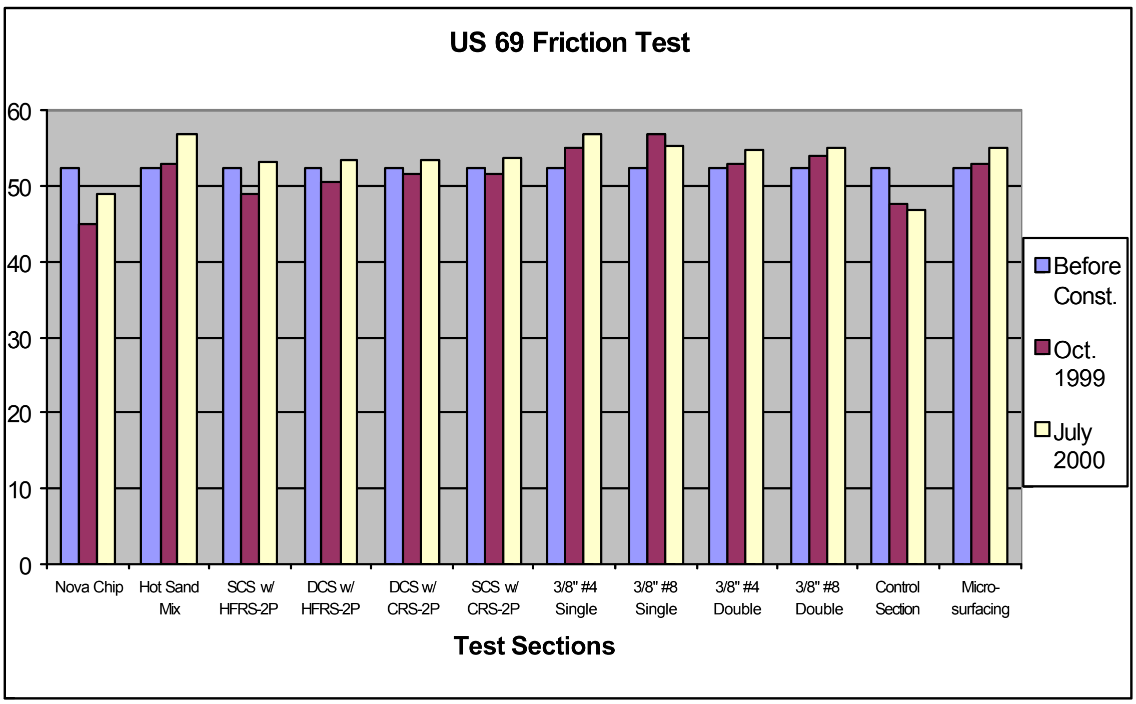


Figure 11. US 69 Friction Test Values

The October 1999 SR values ranged from 45 to 57; in July 2000, all the test sections had an increase in their SR values from October 1999, except for the 3/8-inch #8 SSC and the control section. In October 1999, the SR value improved the most, by 4.5 and 2.5 points, for the 3/8-inch #8 and #4 SSC sections, respectively, from July 1995. The Nova Chip, control, and SCS w/ HFRS-2P decreased their October 1999 SR values by 7.5, 5.0, and 3.5 points, respectively, from July 1995. In July 2000, the average SR value improved the most, by 4.5 points for the 3/8-inch #4 SSC and the hot sand mix. The 3/8-inch #8 SSC and DSC sections improved by 2.8 and 2.5 points, respectively, from July 1995. By July 2000, the only test sections to not show an improvement in their SR values were the control and the Nova Chip; respectively, they were 5.7 and 3.5 points less than the July 1995 SR values.

US 69 Roughness Index

The most recent RI values were measured in the summer of 1997 on the same two locations of road, from Polk/Story County line to junction of US 69 with Alleman Road and then with US 210, that were used for the old SR values. The first location had an average RI value of 1.60, while the second location had an average value of 1.74. The RI values were remeasured in July 2000 for each test section in the north and southbound lanes and are listed in Table 12, and Figure 12 is a graphical representation of these values.

The hot sand mix and the Nova Chip decreased their RI value by an average of 0.64 and 0.54 points, respectively. Meanwhile, the 3/8-inch #4 and #8 DSC sections decreased their RI values by an average of 0.42 and 0.22 points, respectively. The micro-surfacing had the greatest decrease, by an average of 0.155 points. During this same timeframe, the control and SCS w/ HFRS-2P RI values increased by an average of 0.04 points each, and the DCS w/ HFRS-2P and 3/8-inch #4 SSC sections increased their RI values by an average of 0.06 and 0.08 points, respectively.

Table 12. US 69 Roughness Index

Section	RI (m/km)				Average Change
	Summer 1997*	07/2000**			
		Southbound	Northbound	Average	
Nova Chip	1.74	—	1.20	1.20	-0.54
Hot sand mix	1.74	1.10	—	1.10	-0.64
SCS w/ HFRS	1.74	1.66	1.90	1.78	+0.04
DCS w/ HFRS	1.74	1.84	1.76	1.80	+0.06
DCS w/ CRS-2P	1.74	1.53	1.74	1.635	-0.105
SCS w /CRS-2P	1.74	1.59	1.74	1.665	-0.075
SSC #4	1.74	—	1.82	1.82	+0.08
SSC #8	1.74	1.65	—	1.65	-0.09
DSC #4	1.74	—	1.32	1.32	-0.42
DSC #8	1.74	1.52	—	1.52	-0.22
Control	1.74	1.90	1.65	1.775	+0.035
Micro-surfacing	1.60	1.44	1.45	1.445	-0.155

* Average historical data; tests were not taken in exact same location of test sections.
** Data given for southbound and northbound lanes where available; when not available, average includes only the value for the lane with data.

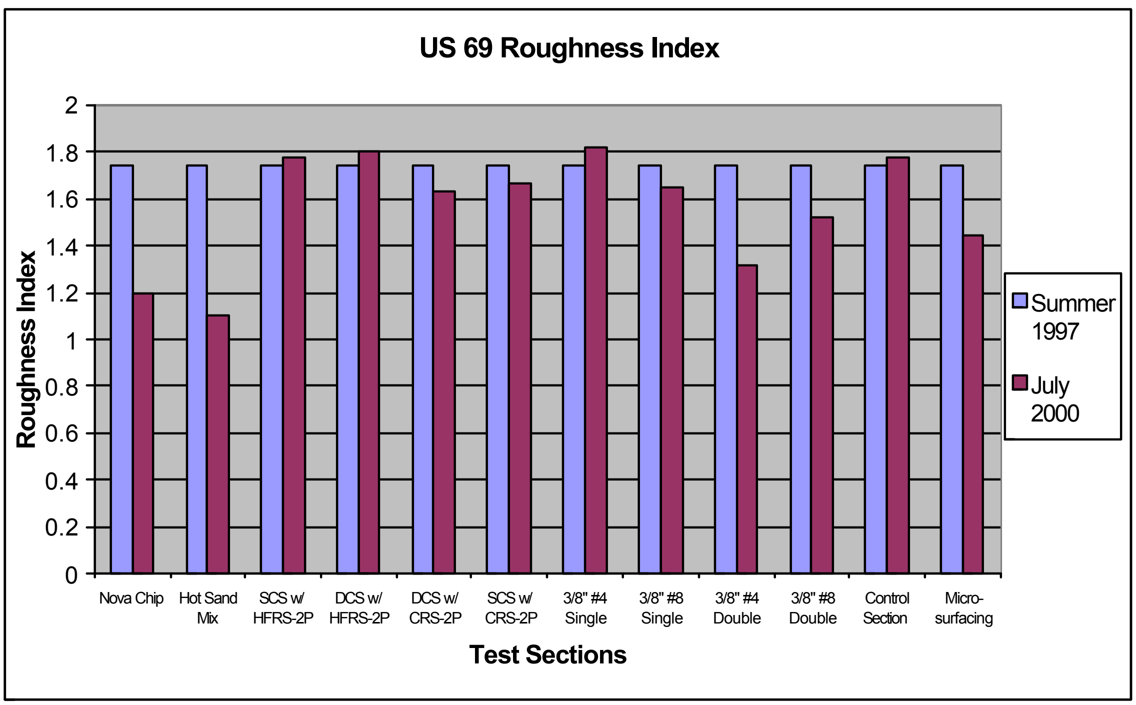


Figure 12. US 69 Roughness Index Values

The raveling and snowplow damage of the micro-surfacing may have been caused, at least in part, by the aggregate gradation (see Table 13 and Figure 13). This gradation is on the coarse side of the allowable range with a small amount passing the #200 sieve. The application rate from the micro-surfacing was low because the mix slid along the squeegee. Additional fine material might have helped stabilize the larger particles so they would go under. Also the lack of fine particles made the surface rough so the snowplow blades could engage the tops of the larger particles and scrape them off the road. More fine material might have protected these larger particles. During field visits to micro-surfacing sites the first author has noticed a similar appearance and damage for most roads micro-surfaced in that year. Roads micro-surfaced in previous years with mixes with finer gradations are smoother and have experienced less snowplow damage.

Table 13. Quartzite Aggregate Gradation for Micro-Surfacing on US 69

Sieve Size	Percent Aggregate Passing Sieve	
	Actual Values	Iowa Specifications
1/2”	100%	—
3/8”	99.8%	100%
1/4”	84.9%	—
#4	75.4%	70%–90%
#8	50.1%	45%–70%
#16	34.9%	28%–50%
#50	17.0%	12%–25%
#200	5.9%	7%–18%

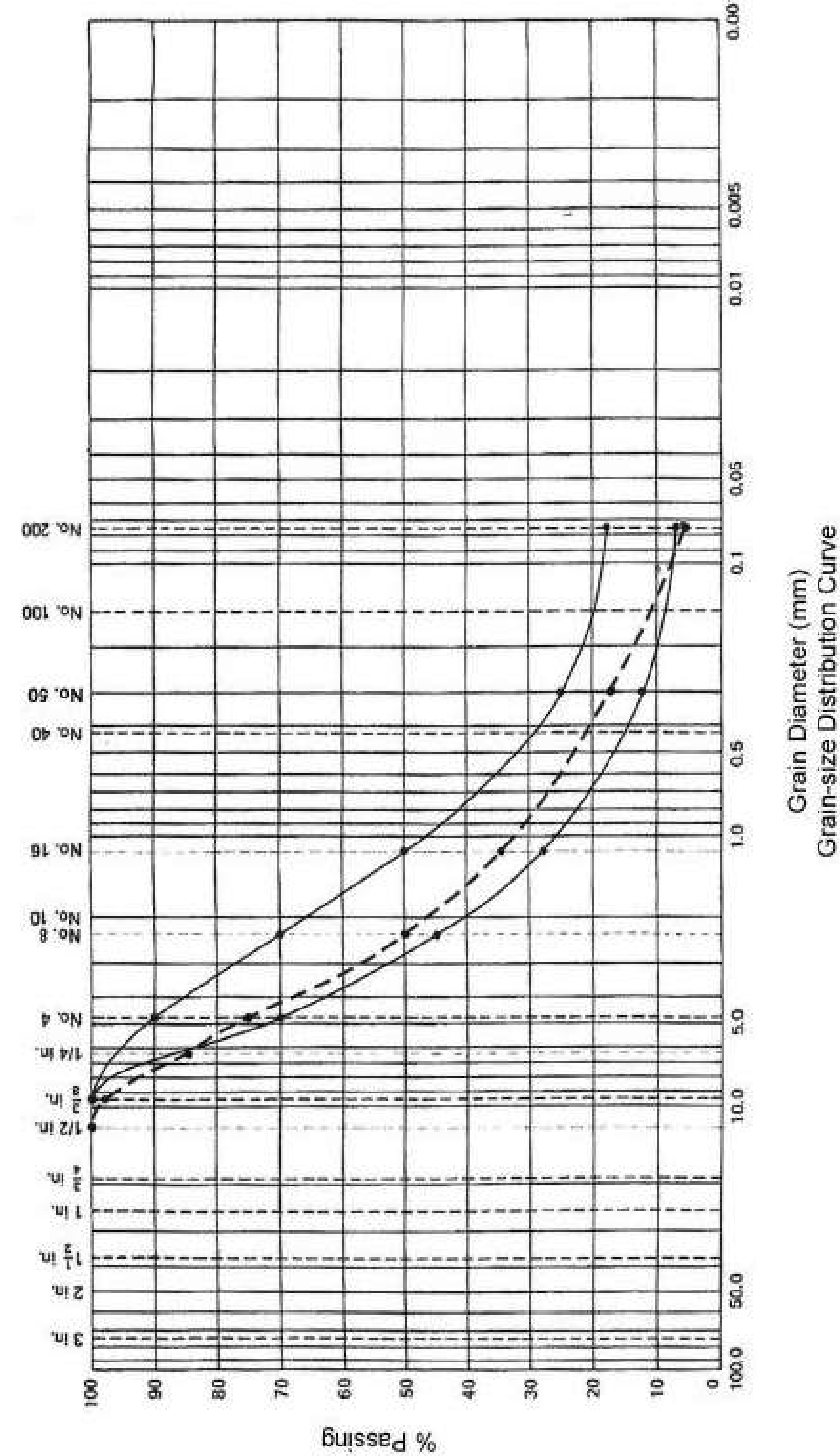


Figure 13. Aggregate Gradation Curve for Micro-Surfacing on US 69

US 218

In 1999, at the start of Phase Two, another set of test sections were constructed on US 218 between St. Ansgar and the Minnesota state line in Mitchell County, Iowa. The section of US 218 that included the test sections was originally constructed in 1933 as a 7-inch-thick PCC pavement. Then in 1962 it received a 2-inch ACC overlay, and in 1986, a 1.5-inch ACC overlay. Before constructing the test sections, the ruts were filled in with slurry seal. All the test sections used limestone aggregate from Falk Construction, St. Ansgar, Iowa.

Starting from the south end, the test sections were placed in the following order and consisted of the following treatment types:

- control section
- four sections using high float rapid set emulsion:
    1. Iowa DOT standard SSC (Aggregate: 1/2-inch cover aggregate. Binder: HFRS-2P.)
    2. designed SSC (Aggregate: 1/2-inch cover aggregate. Binder: HFRS-2P.)
    3. designed DSC (Aggregate: 1/2-inch bottom and 1/4-inch top cover aggregate. Binder: HFRS-2P.)
    4. designed SSC (Aggregate: 1/4-inch cover aggregate. Binder: HFRS-2P.)
- three sections using cationic rapid set emulsion:
    1. designed SSC (Aggregate: 1/2-inch cover aggregate. Binder: CRS-2P.)
    2. designed DSC (designed) (Aggregate: 1/2-inch bottom and 1/4-inch top cover aggregate. Binder: CRS-2P.)
    3. Mn/DOT-designed SSC (Aggregate: 1/4-inch cover aggregate. Binder: CRS-2P.)

Bituminous Materials and Supply of Tama, Iowa, provided both emulsions. All aggregate consisted of crushed limestone chips. See Figure 14 for US 218 test section layout. The contract for the construction of these test sections on US 218 was included in maintenance project MP-218-2(702)266-76-06. This contract was awarded to Manatts Inc., of Brooklyn, Iowa. The slurry seal portion of the project was then subcontracted to Fort Dodge Asphalt of Fort Dodge, Iowa. The slurry seal construction portion of the project took place in July 1999. This process of filling the ruts in the wheel paths with slurry seal consisted of using a three-foot wide slurry edge box. The slurry seal consisted of 3/16-inch crushed limestone from Martin Marietta Aggregates Fort Dodge Mine and CSS-1H asphalt emulsion provided by Bituminous Materials and Supply, of Tama, Iowa. In August 1999, the seal coat test sections were constructed. The spreader machine was calibrated and the test section application rates were designed using the seal coat design procedures found in the Minnesota Seal Coat Handbook (Janisch and Gaillard 1998). The application rates were adjusted based on the visual inspection of the test strips that were placed down on the test sections. The target application rates and the actual application rates are shown in Table 14.

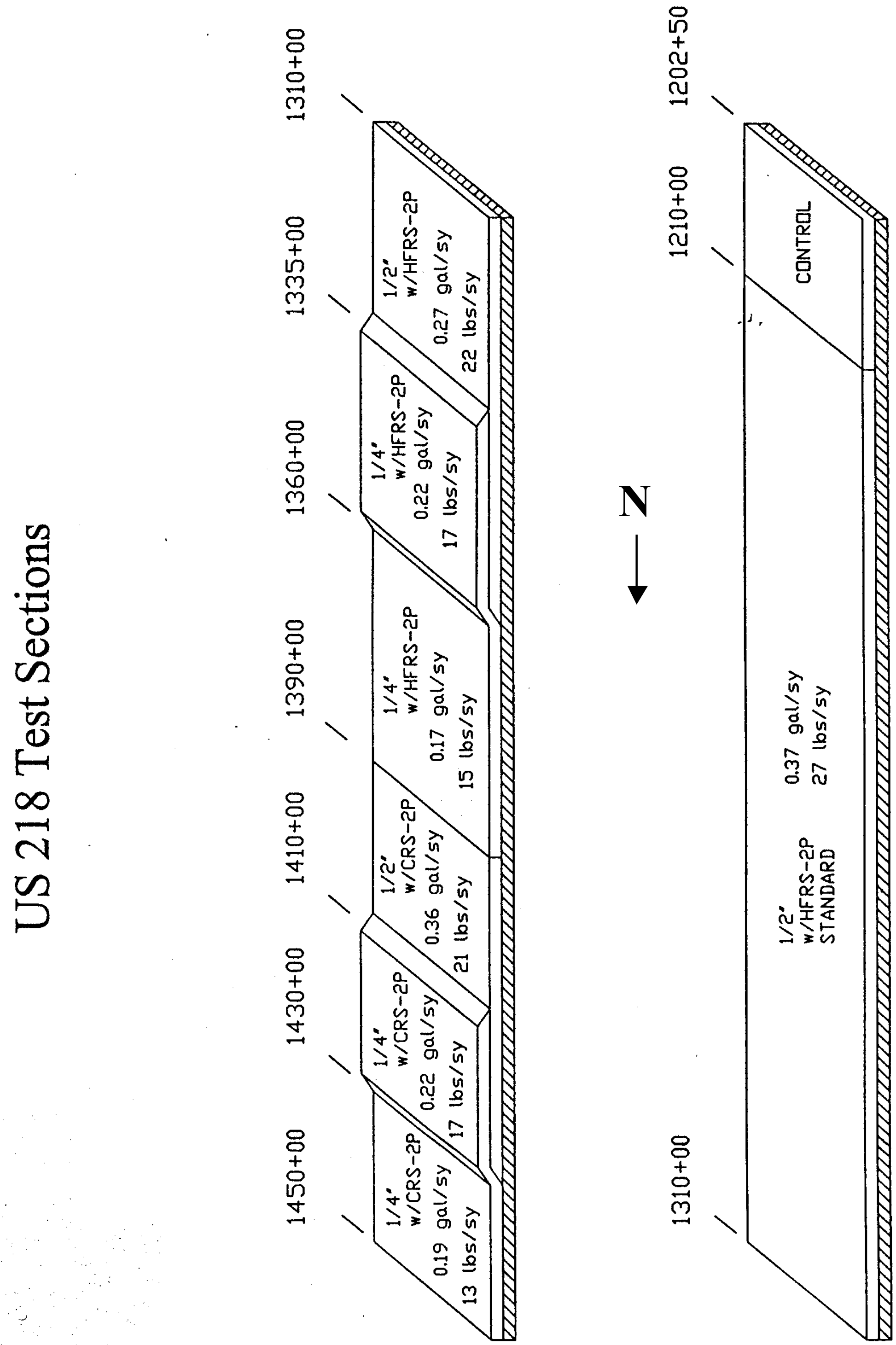


Figure 14. US 218 Test Section Layout and Application Rates

Table 14. US 218 Seal Coat Application Rates

Section	Target Application Rates Aggregate (lbs/yd^2) Binder (gal/yd^2)	Actual Application Rates Aggregate (lbs/yd^2) Binder (gal/yd^2)
With HFRS-2P binder:		
1/2” standard SSC	30.0 0.4	27.0 0.37
1/2” designed SSC	23.0 0.27	22.0 0.27
1/4” designed SSC	13.0 0.17	15.0 0.17
Designed DSC		
1/2” bottom	23.0 0.27	22.0 0.27
1/4” top	13.0 0.25	17.0 0.22
With CRS-2P binder:		
1/2” designed SSC	23.0 0.27	21.0 0.36
1/4” designed SSC	13.0 0.17	13.0 0.19
Designed DSC		
1/2” bottom	23.0 0.27	21.0 0.36
1/4” top	13.0 0.25	17.0 0.22

Before the construction of these test sections took place, the Iowa DOT Office of Materials Laboratory performed Iowa Test Method No. 630-B, the Modified Method of Test for Determining Compatibility of Rapid Setting Asphalt Emulsions and Aggregates. Samples of both emulsions were tested to check their compatibility with the proposed aggregate. The test procedure includes manually mixing the samples of aggregate and the emulsion for a short period of time and then letting that mixture sit. The amount that the emulsion coats the aggregate was then observed. The results showed that the HFRS-2P was more compatible with the crushed limestone chips than was the CRS-2P, because the HFRS-2P coated the limestone chips more than the CRS-2P did. This proved true in the field, because immediately after construction, the test sections that were constructed using the CRS-2P emulsion flushed in the wheel tracks; this was especially true for the 1/4-inch designed SSC.

The test sections were evaluated every six months by observing surface distresses and calculating the SCI according to Shahin (1994). The Iowa DOT performed friction tests and roughness index tests periodically and reported the results to the researchers.

US 218 Surface Condition Index

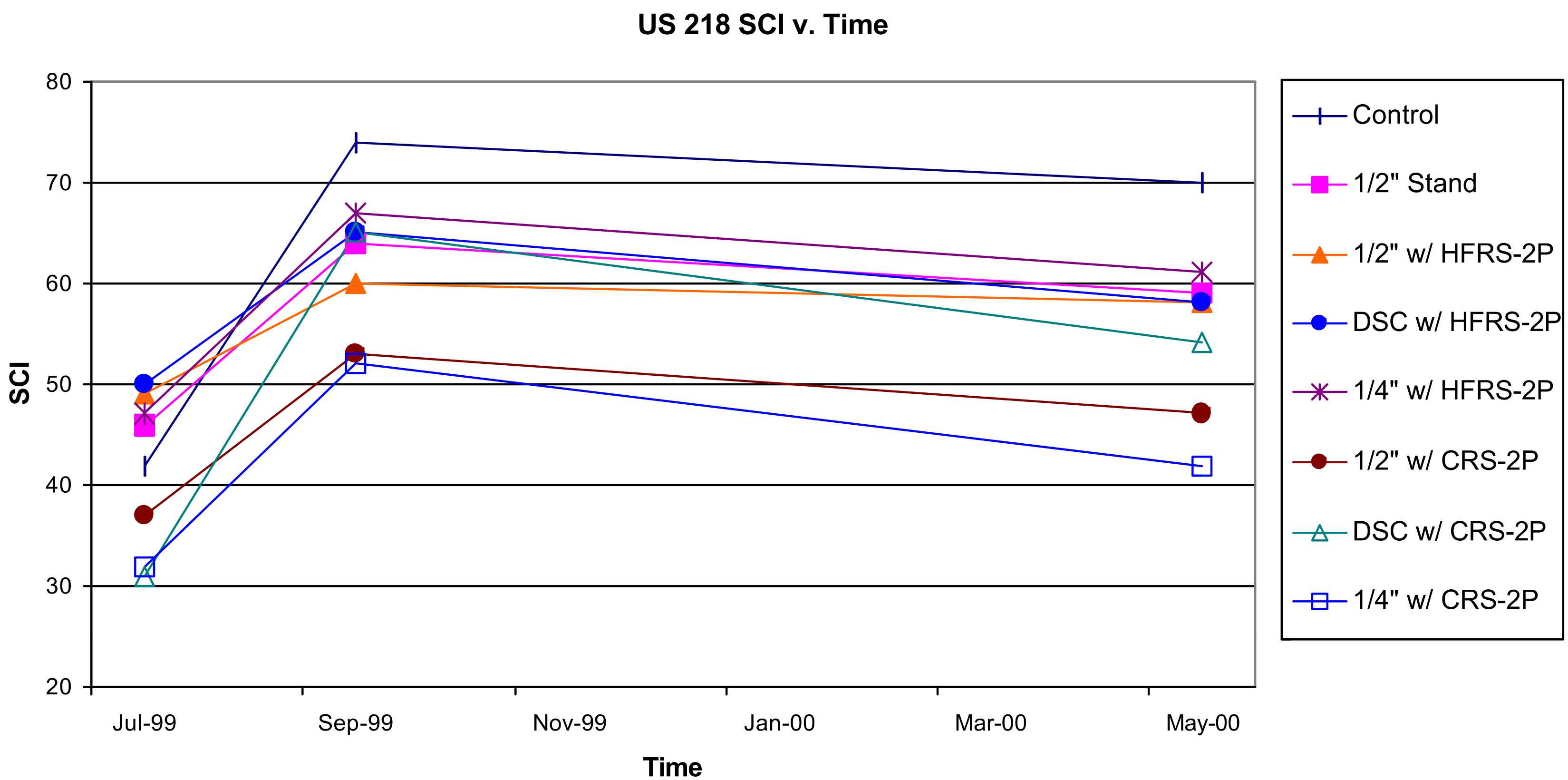
Prior to construction of these test sections in 1999, the road was in fair condition, with SCI values ranging from 31 to 50. US 218 did exhibit moderate to high levels of rutting in the wheel tracks. This section of road also experienced low to moderate levels of L&T cracking, low levels of JR cracking, and low levels weathering/raveling. The ruts in the wheel tracks were between 1/2-inch and 7/10-inch deep. Since this was the most significant distress affecting the pavement's SCI value, it was decided that the ruts had to be filled with slurry seal before construction of the seal coats.

The SCI of the test sections are summarized in Table 15 and Figure 15. In this case, all of the test sections including the control section had improved post-construction SCIs. The control section improved because it was not a control section in the usual sense, where the control section receives no treatments. This control section did have its ruts filled with slurry seal, because the researchers felt the ruts should be filled for safety reasons. However, after the ruts were filled, no further treatments were applied.

Table 15. US 218 SCI Values

Survey	Control	HFRS-2P				CRS-2P		
		1/2" Standard SSC	1/2" Designed SSC	Designed DSC	1/4" Designed SSC	1/2" Designed SSC	Designed DSC	1/4" Designed SSC
07/1999	42	46	49	50	47	37	31	32
09/1999	74 (+32)	64 (+18)	60 (+11)	65 (+15)	67 (+20)	53 (+16)	65 (+34)	52 (+20)
05/2000	70 (+28)	59 (+13)	58 (+9)	58 (+8)	61 (+14)	47 (+10)	54 (+23)	42 (+10)

Note: Values in parentheses indicate change in SCI value from July 1999.



The sections that were treated showed an initial increase in SCI value because all the cracks were filled and covered. However, over time the SCI values decreased because primarily longitudinal and transverse cracking began to reflect through the seal coat. For these treatments, rutting reappeared, although not as severe as the preconstruction ruts, and contributed to the decline of the SCI values. Bleeding also added to the decline in SCI values for the seal coats that used CRS-2P binder.

Figure 15. US 218 SCI vs. Time

Note that there was a wide range (nearly 30 points) of initial SCIs due to wide variations in rut depth. Between September 1999 and May 2000, most of the deterioration rates are similar to that of the control section. However, all of the seal coat treatments had lower SCIs than the control section possibly due to an increase in bleeding and rutting of the test sections compared to what the control section experienced. The seal coats using HFRS-2P performed better than those using CRS-2P, as discussed later.

The DSC designed w/ CRS-2P and the 1/4-inch SSC designed w/ CRS-2P had the lowest initial SCI values of 31 and 32, respectively; they each had a deduct value of 80 for the rutting. The highest initial SCI value of 50 was for the DSC designed w/ HFRS-2P. SCI values of 49, 47, 46, 42, and 37 were recorded, respectively, for the 1/2-inch designed w/ HFRS-2P, 1/4-inch designed w/ HFRS-2P, 1/2-inch standard w/ HFRS-2P, control, and 1/2-inch designed w/ CRS-2P.

After construction, SCI values were measured in September 1999 and ranged from 52 to 74. The control section had the highest SCI value, while the 1/4-inch SSC designed w/ HFRS-2P had a SCI value of 67. Both the DSC sections had SCI values of 65, and the 1/2-inch standard SSC and the 1/2-inch designed SSC w/ HFRS-2P had SCI values of 64 and 60, respectively. The 1/4-inch and 1/2-inch designed SSC CRS-2P sections had the lowest SCI values of 52 and 53, respectively. The main distress affecting the test sections was still rutting, but these deduct values had decreased substantially compared to the deduct values before construction.

The SCI values were measured again in May 2000 to determine how the test sections were holding up. The SCI values ranged from a low of 42 and 47 for the 1/4-inch and 1/2-inch designed SSC CRS-2P sections, respectively, to a high of 70 and 61 for the control and 1/4-inch SSC designed w/ HFRS-2P, respectively.

For this set of test sections, it is helpful to compare decreases in SCI from the post-construction condition to one year later. The test section that experienced the smallest post-construction decrease in SCI one year after construction was the 1/2-inch SSC designed w/ HFRS-2P; its SCI value decreased 2 points. The control and 1/2-inch standard SSC w/ HFRS-2P test sections had decreases of 4 and 5 points respectively. These three test sections had an overall increase from before construction of 9, 28, and 13 points, respectively. The 1/4-inch SSC designed w/ HFRS-2P and the 1/2-inch SSC designed w/ CRS-2P experienced a decrease of 6 points during the one-year time period after construction, with both have a net increase from before construction of 14 and 10 points each. About one-year after construction, the DSC w/ HFRS-2P, 1/4-inch SSC designed w/ CRS-2P, and DSC w/ CRS-2P had a decrease in their respective SCI values of 10, 11, and 12 points. Those three test sections had an overall increase in their SCI value from before construction of 8, 10, and 23. The low SCI values for the seal coats using CRS-2P are a result of the bleeding that occurred shortly after they were constructed, which was caused by the binder and aggregate not being compatible with each other. Table 16 lists the US 218 deduct values for each distress experienced about one year after construction.

Table 16. Distress Experienced on US 218 Test Sections One Year After Construction

Section	Bleeding	Edge Cracking	JR Cracking	L&T Cracking	Polished Aggregate	Rutting	Weathering/ Raveling
Control	—	—	(20.69) **7.87**	(36.71) **16.03**	(3.7) **0.0**	(41.05) **23.18**	—
With HFRS-2P binder:							
1/2” standard SSC	(0.51) **8.94**	—	(17.09) **7.87**	(24.98) **17.59**	(2.79) **2.47**	(41.05) **32.21**	—
1/2” designed SSC	(2.51) **8.94**	—	(6.88) **7.87**	(22.64) **14.21**	(0.68) **2.47**	(41.05) **36.09**	(9.20) **0.0**
Designed DSC	(0.0) **9.82**	—	(6.88) **7.87**	(22.81) **13.44**	(0.68) **4.81**	(41.05) **32.21**	(9.20) **6.57**
1/4” designed SSC	(0.0) **8.94**	—	(6.88) **17.23**	(26.68) **14.22**	(0.68) **2.47**	(41.05) **28.56**	(8.82) **0.0**
With CRS-2P binder:							
1/2” designed SSC	(0.0) **26.23**	(0.0) **3.79**	(6.88) **7.87**	(24.76) **12.07**	(0.88) **2.87**	(55.90) **42.25**	(9.20) **9.69**
Designed DSC	(0.0) **22.77**	—	(6.88) **20.69**	(25.37) **8.72**	(0.88) **3.70**	(79.78) **28.56**	(8.82) **0.0**
1/4” designed SSC	(0.0) **25.14**	—	(6.88) **20.69**	(24.78) **13.02**	(0.68) **3.00**	(79.78) **38.89**	(8.82) **3.05**

Note: Deduct values for the types of distresses the US 218 test sections experienced about one year after construction are shown in bold. Values in parentheses are those experienced before construction.

US 218 Skid Resistance

The most recent SR values for US 218, before construction, were measured in the summer of 1998. Tests were conducted between St. Ansgar, Iowa, to the Iowa/Minnesota state line, with the average SR value for that entire stretch of US 218 being 51. Since there was no specific SR value for the test sections, this average pre-construction SR value of 51 was used to compare the improvements the test sections might have on the SR value after construction. The post-construction measurement of the SR values for the test sections was done in July 2000, with average scores ranging from 30.0 to 49.50. Table 17 summarizes the SR values that were obtained for the test sections, and Figure 16 shows a graphical representation of those values.

All the test and control sections had average SR values that were lower than before construction; however, all the test sections, except for the DSC designed w/ CRS-2P, had SR values that were equal to or higher than the SR values for the control section. The control section had an average SR value of 37.75. The highest average SR value was for the 1/4-inch SSC designed w/ HFRS-2P, 49.50. The 1/2-inch standard SSC w/ HFRS-2P had the next highest average SR value, 46.25. The next two best performing test sections were the 1/4-inch SSC designed w/ CRS-2P, which had an average SR value of 45.75, and then the DSC designed w/ HFRS-2P, which had an average value of 44.75. The lowest average SR value was for the DSC designed w/ CRS-2P, which had an average value of 30.0. The low SR values can be most likely attributed to the bleeding and flushing problems that occurred right after construction with the test sections that were constructed with the CRS-2P emulsion.

Table 17. US 218 Skid Resistance Test Results

Section	SR				Change
	07/1998*	07/2000			
		Northbound	Southbound	Average	
Control	51	36.0	39.5	37.75	-13.25
With HFRS-2P binder:					
1/2" standard SSC	51	46.5	46.0	46.25	-4.75
1/2" designed SSC	51	33.5	42.0	37.75	-13.25
Designed DSC	51	47.5	42.0	44.75	-6.25
1/4" designed SSC	51	50.5	48.5	49.50	-1.50
With CRS-2P binder:					
1/2" designed SSC	51	41.5	37.0	39.25	-11.75
Designed DSC	51	29.5	30.5	30.00	-21.00
1/4" designed SSC	51	43.0	48.5	45.75	-5.25

* Average historical friction values; tests were not taken in exact location of test sections.

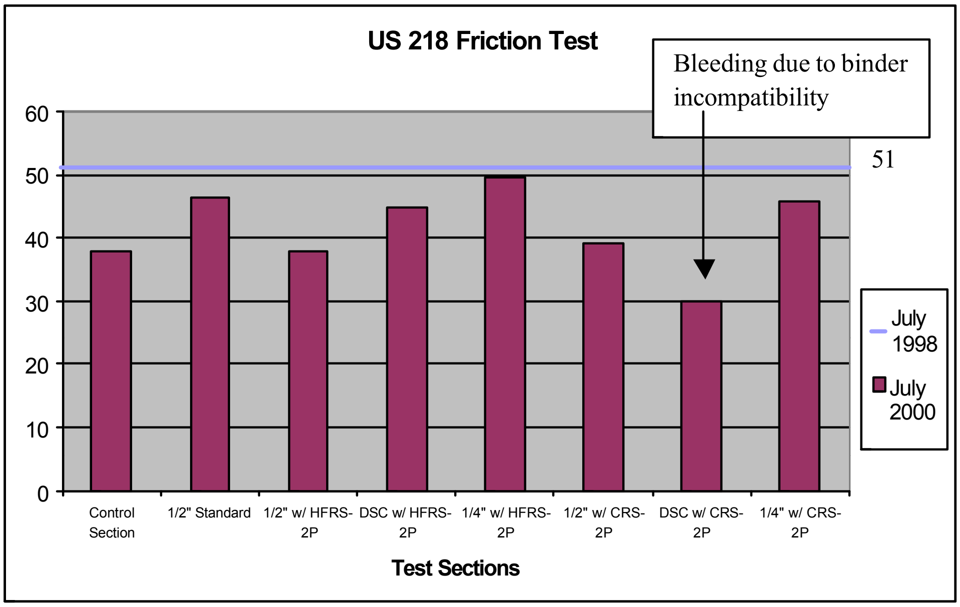


Figure 16. US 218 Friction Test Values

US 218 Roughness Index

The RI values before construction were also measured in the summer of 1998. The roughness testing was done on the same strip of US 218 as the friction testing, from St. Ansgar, Iowa, to the Iowa/Minnesota state line. The average roughness value for that entire stretch of road was found to be 2.13. Because no specific RI values were measured for each test section, this average RI value of 2.13 was used to compare the RI values for each test section about one-year after construction to determine how the roughness condition of the road changed. A summary of the measured RI values is listed in Table 18, with a graphical representation of these values appearing in Figure 17.

Table 18. US 218 Roughness Index

Section	RI (m/km)				Change
	07/1998*	07/2000			
		Northbound	Southbound	Average	
Control	2.13	2.64	1.70	2.170	+0.040
With HFRS-2P:					
1/2" standard SSC	2.13	1.77	1.45	1.610	-0.520
1/2" designed SSC	2.13	1.97	1.72	1.845	-0.285
Designed DSC	2.13	1.85	1.63	1.740	-0.390
1/4" designed SSC	2.13	1.72	1.69	1.705	-0.425
With CRS-2P binder:					
1/2" designed SSC	2.13	1.92	1.83	1.875	-0.255
Designed DSC	2.13	1.96	1.87	1.915	-0.215
1/4" designed SSC	2.13	1.95	2.36	2.155	+0.025

* Average historical roughness; the tests were not taken in exactly the same location as the test sections.

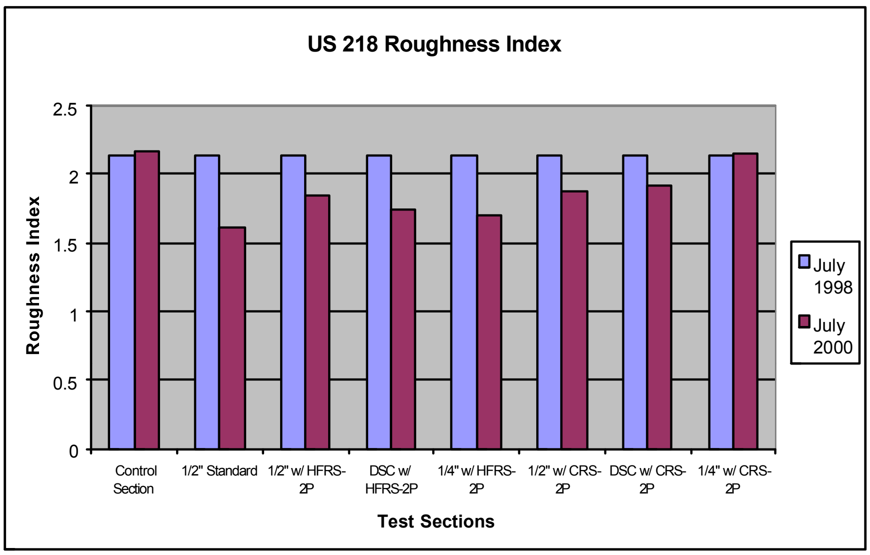


Figure 17. US 218 Roughness Index Values

In July 2000 the post-construction RI values were measured and had a range of 1.54 to 2.64; the average RI values between both lanes of test sections ranged from 1.610 to 2.170. All the test sections exhibited lower average RI values than those measured in 1998, except for the control and the 1/4-inch SSC designed w/ CRS-2P sections. The 1/2-inch standard SSC w/ HFRS-2P had the lowest average RI value, 1.610. The 1/4-inch SSC designed w/ HFRS-2P and the DSC w/ HFRS-2P had average RI values of 1.705 and 1.740, respectively. Overall, the RI in the southbound lane decreased, on average, the most from the July 1998 RI value. However, the 1/4-inch SSC w/ CRS-2P increased its RI value in the southbound lane by 0.23, which is why that section, along with the control, has an average RI value that is greater than the value from July 1998. The control only saw an increase in its RI value for the section on the northbound lane; it increased by 0.51 while the southbound decreased by 0.43.

Conclusions from Test Sections

These case studies showed that thin maintenance surface treatments are more effective/perform better when they are applied to a pavement that is in good condition compared to when they are applied to a pavement that is already in poor condition. However, in an environment of competing dollars, there is potential for the public or the less informed observer to think that the money used to place the TMS on a good road is wasted when there are other roads that are in dire need of repair. In order to combat this impression that may come about as a result of doing preventive maintenance, the public needs to be educated as to the long-term cost savings and benefits of timely maintenance and as to why treatments are focused on good roads. TMS treatments are usually not as effective on roads that are in poor condition because the base and the existing surface, to which they are applied, cannot hold the freshly applied TMS. Usually within one year of application on a road that is in poor condition, the TMS treatment showed significant signs of bleeding and flushing, actually worsening the overall condition of the road after construction compared to that state prior to construction.

When TMS are to be used as a preventive maintenance treatment, they need to be applied under weather conditions that are ideal for the placement of the particular TMS treatment planned. If the TMS is not applied under ideal weather conditions, there is a greater chance for surface failure, resulting in loss of realized savings in the road maintenance program. In determining which TMS treatment to use, one needs to also analyze the type and severity of existing and potential distresses on the road by determining the SCI value for that road. This is necessary because two roads may have identical SCI values, but the factors for determining SCI value may come from different types of distresses. So the TMS treatment that is selected needs to be sure to address the types of distresses that the particular road is experiencing.

From these four case studies, it can be concluded that the thin lift overlay sections exhibited the best performance with respect to SCI and RI values. The SR value for the thin lift overlay was usually improved, but some TMS treatments yielded more improvement than did the thin lift overlay. These three values showed the most overall improvement for each test section after construction compared to the values that were recorded before construction. The thin lift overlays addressed the following pavement distresses well when checked one, two, and three years after construction: rutting, raveling, and L&T cracking; while the micro-surfacing and the slurry seals appeared to perform the best on pavements that were experiencing low cracking. Based on information from these case studies, the micro-surfacing treatment can be used to improve SR values of a road; however, this type of treatment had poor performance with respect to improving SCI values because of raveling that occurred. The test sections that used micro-surfacing experienced high raveling, rutting, and L&T cracking. The chip seals that were used in these case studies performed better than the other treatments when they were applied to pavements that were experiencing large amounts of cracking prior to construction.

After construction, all of the seal coats exhibited fair to good performance with respect to their SCI and RI values and high SR values were obtained on all test sections, except for those on US 218. These case studies showed that when the single and double seal coats were applied to the road surface, they both experienced about the same results for their SCI, SR, and RI values that were being used to measure and evaluate their performance as a maintenance surface. The most significant distresses that the seal coats experienced affecting their SCI values were rutting, JR cracking, L&T cracking, and weathering/raveling. The highest increase in SCI value, after construction, for sections of road that were seal coated appeared on the US 69 test sections that were constructed with imported quartzite. However, the single seal coats with quartzite had higher raveling values than did the other seal coat test sections. The test sections of seal coats that were constructed with local limestone exhibited good SCI values without the high raveling values.

The one advantage that the double seal coats appeared to have over the single seal coats in these case studies is that they were found to be less noisy, a result of a tighter bonded and smoother surface. When using a seal coat as a TMS, a design method needs to be implemented to make sure the seal coat is adjusted for specific road and traffic conditions that it will experience. This was especially evident on US 218 where a standard 1/2-inch SSC and a designed 1/2-inch SSC performed similarly in the values that were being measured to determine their performance. They both had about the same SCI value, but the designed seal coat had a lower RI value while the standard seal coat had a higher SR value. However, the advantage of using the designed seal coat over the standard seal coat was that the designed seal coat reflected fewer L&T cracking and used less binder and aggregate per square yard compared to the standard seal coat. Imperative in the use of seal coats is the need for good compatibility between binder and aggregates used in the seal coat. If this does not occur, low friction values may be obtained as a result of the binder not binding well to the aggregate.

CHAPTER 3. SEAL COAT MATERIAL CONSIDERATIONS

Even though there are several different combinations of aggregate and binder that could be used in thin maintenance surfaces, only a few of those combinations are used successfully in TMS. This chapter discusses the different types of aggregates and binders that have been successfully used in TMS.

Aggregates

Aggregate Types

According to Iowa DOT Materials Instructional Memorandum T-203, there are six main functional types of aggregate classifications in accordance with their frictional characteristics for bituminous construction:

- **Type 1.** Type 1 aggregates are generally a heterogeneous combination of minerals with coarse-grained microstructure of very hard particles (generally, a Mohs hardness range of 7 to 9) bonded together by a slightly softer matrix. These aggregates are typified by those developed for and used by the grinding wheel industry such as calcinated bauxite (synthetic) and emery (natural). They are not available from Iowa sources. Due to their high cost, these aggregates would be specified only for use in extremely critical situations, such as quartzite, granite, and slags.
- **Type 2.** Natural aggregates in this class are crushed quartzite and granites. The mineral grains in these materials generally have a Mohs hardness range of 5 to 7. Synthetic aggregates in this class include some air-cooled steel furnace slags and others with similar characteristics.
- **Type 3.** Natural aggregates in this class are crushed trap rocks, and/or crushed gravels. The crushed gravels shall not contain more than 60 percent total carbonate (limestone). Synthetic aggregates in this class are the expanded shales with a Los Angeles abrasion loss less than 35 percent.
- **Type 4.** Aggregates crushed from dolomitic or limestone ledges in which 80 percent of the grains are 20 microns or larger. The mineral grains in the approved ledges for this classification generally have a Mohs hardness range of 3 to 4. For natural gravels, the Type 5 carbonate (see below) particles, as a fraction of the total material, shall not exceed the non-carbonate particles by more than 20 percent.
- **Type 4D.** A subgroup of the Type 4 category comprised of those aggregates near, but exceeding, the 20-micron minimal grain size. Type 4D aggregates are not acceptable for use in any asphalt cement concrete surface courses requiring the use of Type 4 or better material.
- **Type 5.** Aggregates crushed from dolomitic or limestone ledges in which 20 percent or more of the grains are 30 microns or smaller.

The aggregate used for a TMS is typically either of Type 2, 3, 4, or 4D friction classification in accordance with the above classes of aggregate. Type 4 and 4D aggregates generally require the use of more binder because they are more absorptive. The advantages and disadvantages of using quartzite, limestone, or pea gravel as the aggregate for a TMS are listed in Table 19.

Table 19. Advantages and Disadvantages of Aggregate Types

Type	Advantages	Disadvantages
Quartzite	• Particles have sharp edges that provide excellent skid resistance. • High durability. • Pink coloration can be contrasted with other aggregates to delineate portions of the road.	• Binder must be properly formulated to mitigate stripping. • Sharp edges catch snowplow blades, causing wear on blade or plucking improperly bound aggregate from road. • High transportation costs in portions of Iowa that are not close to sources.
Limestone	• Limestones with low clay content are easily and permanently bound by most binders. Higher clay content limestones can be retained by carefully selecting binder. • Sharp edges promote good skid resistances until worn; however, some limestone has excellent durability. • Some limestone has microstructure that promotes good skid resistance by providing rough crystalline faces after worn. • Locally available in many places in Iowa, thus lower transportation costs. • Individual stones may shear apart during snowplowing operations preventing them from being plucked from the road.	• High clay content limestone requires careful binder selection. • Soft limestones will not be durable, loosing sharp edges under traffic. • Individual stones may shear during snowplowing, reducing macro-texture (may not occur in wheel ruts).
Pea gravel	• Round stones provide smoother road surface that may be friendlier to pedestrians, bike riders, skate boarders, and in-line skaters. • Round stones may not catch snowplow blades. • Inexpensive local sources reduce cost.	• Round particles provide less macro-texture for skid resistance. • Round particles provide less stability; this may make them more susceptible to snowplow damage if the blade engages them. • Particles are not from a homogeneous source, so their chemical behavior with binders is less predictable.

Since the construction and monitoring of the test sections in this study, an 11 mile stretch of Pocahontas County Road N28 (between Laurens and Fonda) has been successfully micro-surfaced with the use of limestone from the Martin-Marietta pit in Fort Dodge, Iowa (Lehmann 2003).

Aggregate Sources

Type 4 crushed aggregates are generally found throughout the state of Iowa (see exceptions in next paragraph), while Type 2 and 3 crushed aggregates are not available in Iowa. However, some Type 2 crushed synthetic aggregate can be obtained in Muscatine

County (slag from a steel plant). Most Type 2 and 3 crushed aggregates must be imported from the neighboring states of Minnesota and South Dakota. The raw material Type 3 and 4 crushed gravels can also be found in many portions of Iowa; however, they are rarely produced.

Figures 18 and 19 are maps of the state of Iowa that show the approved locations for aggregate and crushed stone, respectively, to be used in TMS (maps from Iowa DOT). Most of the approved aggregate and crushed stone is located in eastern Iowa, particularly in the northern half, within two or three counties of the Mississippi River. Every county in Iowa either has an approved aggregate source or there is one located within a neighboring county. However, this does not hold true for those counties in the northwest portion of the state. In this part of the state there are very few approved sources, but most of the aggregate can be easily shipped from the Sioux City, South Dakota area.

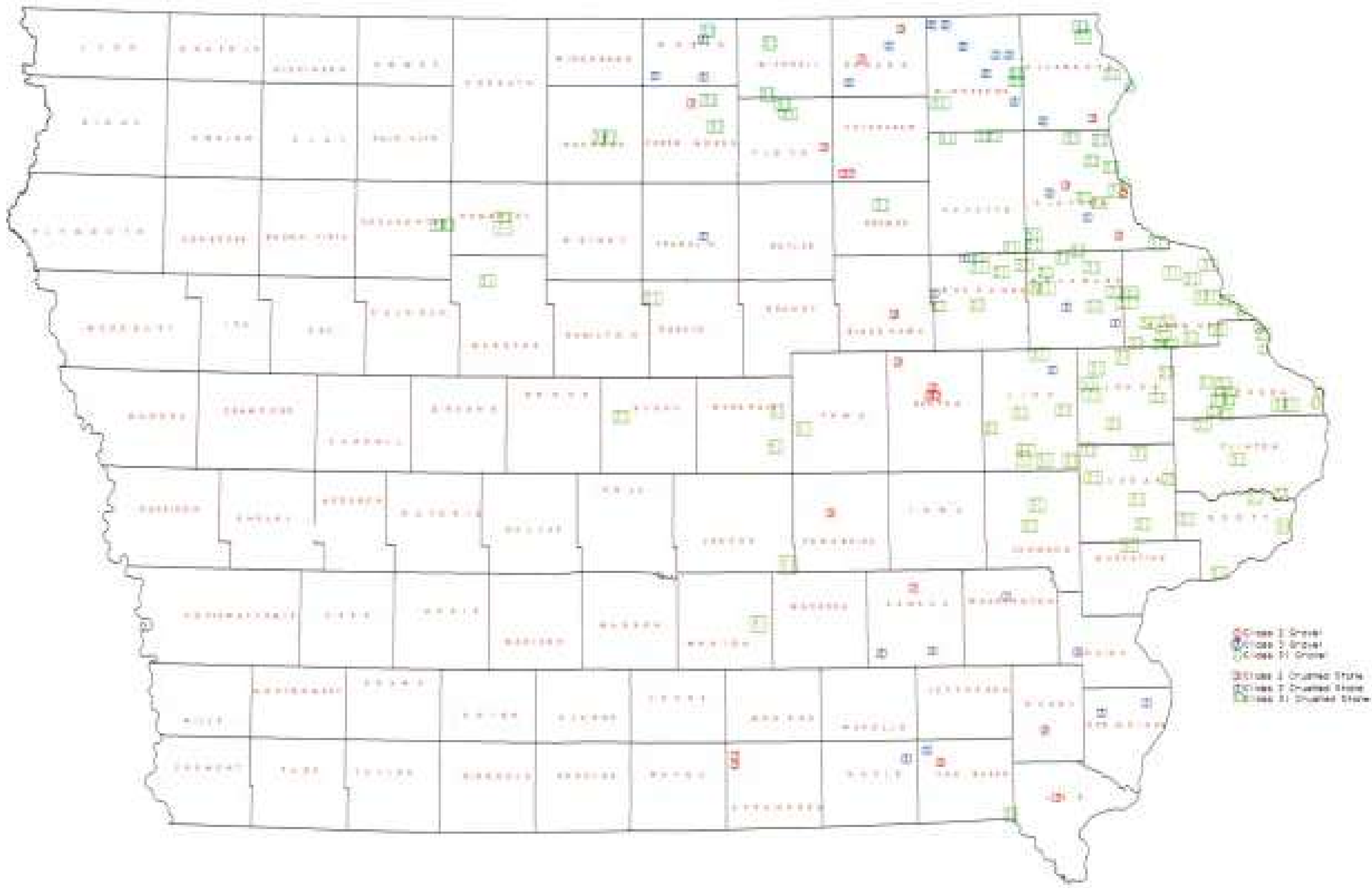
Aggregate Wear Resistance

Of the aggregates typically used for a TMS, Type 2 aggregates are the hardest and can withstand the wear of snowplow blades and traffic better than Type 3, 4, or 4D aggregates. However, as experienced by our test sections, Type 2 aggregates may be dislodged easier because a snowplow blade may catch their sharp corners and edges easier and pluck them from the binder if they are not well bound. The aggregates that are crushed down to a smaller size from the initial parent aggregate tend to be angular so particles interlock with each other better. Therefore, they are better able to withstand the starting, stopping, and turning forces from vehicles on the roadway.

Aggregate Skid Resistance

The skid resistance on a road surface comes from the macro- and micro-texture of the aggregate that is spread on the road. Macro-texture is the large-scale texture on the road surface caused by the size and shape of the aggregate in the asphalt binder. Micro-texture is the small-scale texture of the individual aggregate chips caused by the hard mineral grains distributed through the softer mineral material of the aggregate chips.

The following points (following the maps) were made by Abdul-Malak, Meyer, and Fowler in “Research Program for Predicting the Frictional Characteristics of Seal-Coat Pavement Surfaces” (1989) and “Major Factors Explaining Performance Variability of Seal Coat Pavement Rehabilitation Overlays” (1993). More information on aggregate skid resistance can be found in those papers.



APPROVED AGGREGATE SOURCES T-203

Figure 18. Aggregate Locations in Iowa Approved for Use in Thin Maintenance Surfaces

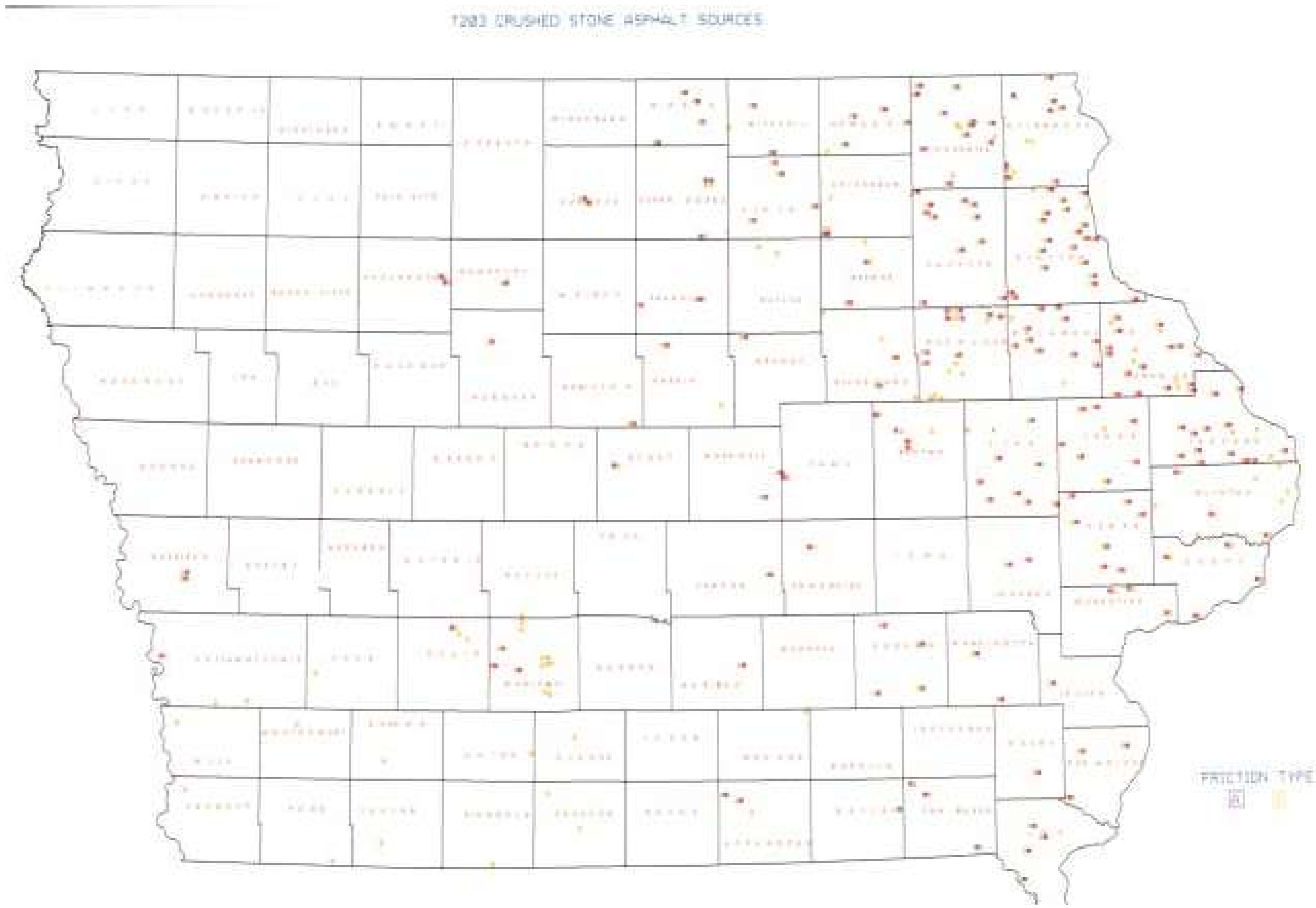


Figure 19. Crushed Stone Locations in Iowa Approved for Use in Thin Maintenance Surfaces

- Aggregates that are composed of a combination of hard and soft minerals seem to have a higher skid resistance than aggregates composed of minerals of relatively the same hardness. The idea behind this is that the soft mineral grains wear away first, exposing the hard grains, which provides the increased skid resistance. These particles and the matrix holding them together are then worn down exposing fresh unpolished particles, thus allowing the process to repeat itself.
- Aggregates that contain larger and more angular mineral grains or crystals in the individual aggregate chips are expected to have a higher skid resistance.
- The more uniform distribution of these coarser and harder mineral grains throughout the softer minerals, the higher the expected skid resistance.
- The variations of frictional resistance along roadway surfaces are deemed to be from short- and long-term seasonal changes: Long dry periods tend to allow for the aggregate to be polished more. Long wet periods tend to allow for the aggregate to be rejuvenated by exposing fresh, angular crystals.
- Freeze-thaw cycles seem to create a rejuvenating effect on the micro-texture of the aggregate chips, which is caused by the softer particles coming off the surface leaving the harder particles exposed. This leads to an increase in skid resistance.
- Soft materials that wear easily may have high skid resistance before they wear below the level of the binder for reasons stated above.

Aggregate Shapes

The shape of aggregate used in TMS is considered to be either flat, cubical, or round.

Flat Aggregate

- The flatter the aggregate, the more susceptible it will be to change in orientation from the impacts of traffic. Aggregate chips that are flat and in the wheel paths are caused, from the tire loads, to lie on their flattest side. This causes a thinner TMS and a chance for increased bleeding if the binder is too thick in the wheel paths or loss of aggregate in the non–wheel paths if the binder is applied to thin. This is more of a problem for seal coats; however, a hot mix is also weakened by the use of flat aggregates. Figure 20 shows how the flat aggregate chips are re-oriented in the wheel paths of traffic (from Janisch and Gaillard 1998).
- For parking lots and roadways with very low volumes of traffic, such as residential streets, the use of flat aggregate chips may not create a problem. This is because there may not be enough traffic or the traffic may not be confined to specific wheel paths, as experienced on a road with higher traffic counts, to cause the aggregate chips to be re-orientated.

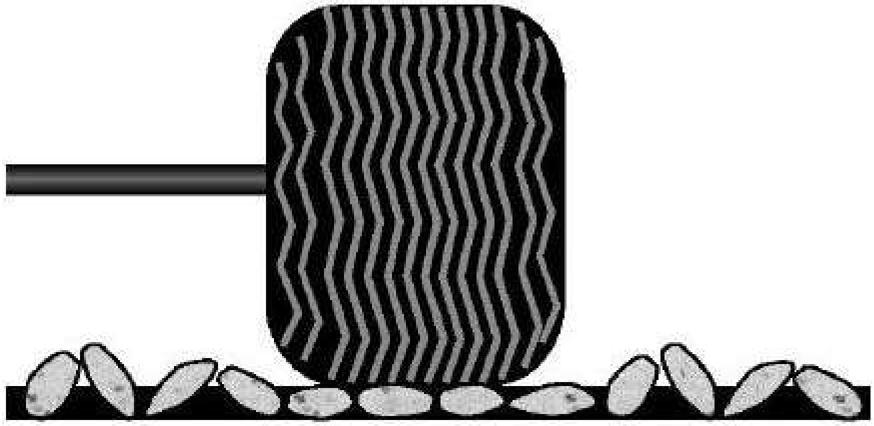


Figure 20. Flat Aggregate Chips Being Re-oriented Under Traffic in the Wheel Path

Cubical Aggregate

- As a result of cubical chips all being relatively uniform in shape, traffic will not reorient the chips. Since the chips are uniform in shape, whichever way they are orientated the TMS will have relatively the same thickness, even in the wheel tracks. Figure 21 shows how cubical aggregate chips withstand traffic forces better than flat chips (from Janisch and Gaillard 1998).
- Because of their angular edges, cubical aggregate chips interlock together, creating a surface that is more resistant to the pounding of traffic and snowplow blades.
- They are also better able to withstand the starting, stopping, and turning actions of vehicles as they travel the roadway.

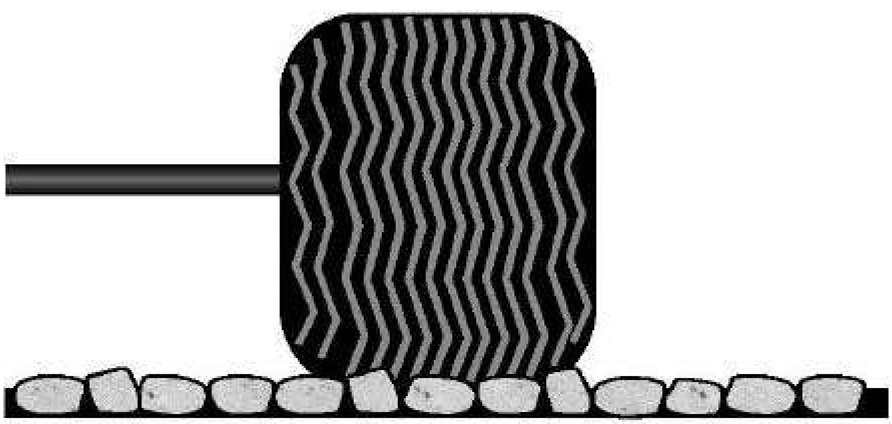


Figure 21. Cubical Aggregate Experiences Little Effect in Orientation from Traffic

Round Aggregate

- The rounder the aggregate, the more susceptible it will be to rolling and displacement under stopping and turning actions of traffic. Because of this, round aggregates should not be used on high volume roadways where many turning, starting, and stopping forces may be experienced.
- Round aggregates are susceptible to being dislodged under snowplow blades because they do no interlock with each other as well. However, because of the aggregate's roundness, the blade does not catch them as easily as aggregate with jagged edges. The authors found anecdotal evidence that round aggregate withstood snowplowing well in residential areas.
- The use of graded round aggregate allows for the chips to lock more readily with each other. The smaller chips are able to fill the voids between the larger chips, thus locking them together.
- Round aggregates tend to create a smoother surface, especially when used in a double seal fashion. This provides a surface that is more comfortable to walk on, especially in parking lots and places where people may fall down and skin their knees.

Table 20 lists the advantages and disadvantages of using different shaped aggregates in a TMS.

Table 20. Advantages and Disadvantages of Aggregate Shapes

Shape	Advantages	Disadvantages
Cubical	• Greater stability in wheel tracks and areas where traffic is turning. • Allows higher shot rate for binder, ensuring that aggregate particles are better attached.	• Requires more expensive production techniques, which raise cost and limit availability, and in some cases increases volume of waste products in quarry.
Flat	• May be more easily produced and therefore lower in cost in some areas.	• Flat particles reorient under traffic to lowest possible elevation, possibly submerging in binder and causing tracking and bleeding. • Reduces design shot rate, which may cause some particles to be less firmly bound.
Round	• Create a smoother surface that is more comfortable to walk on. • Do not catch the blade of a snowplow because of roundness.	• More susceptible to rolling and displacement under starting, stopping, and turning actions of traffic. • Do not interlock with each other as well, unless a graded aggregate is used, thus dislodged easier by snowplow blades.

Aggregate Gradation

Aggregates used in TMS are either of roughly one size or of multiple sizes—this is, a graded aggregate.

One-Size Aggregate

- Aggregate is considered one-size if nearly all the aggregate is retained on two consecutive sieves. When one size aggregate is used, an individual aggregate chip does not stick up higher than others around it, so no individual chip can be dislodged easier from a snowplow blade. Figure 22 shows a cross section of one-size aggregate on a roadway surface (from Janisch and Gaillard 1998). Note that all particles are fairly close to the same size and no one particle is easier to dislodge than the others. This is an idealized artist's rendition; actual aggregate pieces will probably have sharp corners and edges. Actual cross sections would likely show more angular particles, some of which stand taller than those shown in the drawing.
- Because the aggregate chips are of one-size, there is a greater contact area between the tires and the road surface.
- As a result of the channels between the aggregate chips on the surface, there is increased drainage of water, which also increases the effective frictional value of the wet road surface by reducing the tendency to hydroplane.

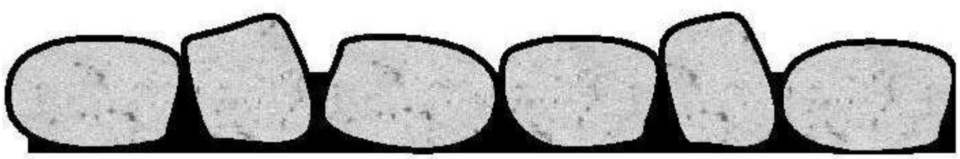


Figure 22. Cross Section of One-Size Aggregate in a Thin Maintenance Surface

The US 69 test sections included one test section with one-sized quartzite aggregate, which suffered considerable snowplow damage, possibly due to angular particles that caught the snowplow blade.

Graded Aggregate

- Generally it is thought that the more graded an aggregate, the less desirable it is, because there is less room for the binder to fit between the chips. The use of graded aggregate reduces the tolerance regarding the amount of binder used. Thus, usually bleeding occurs because of too much binder being used, or there is loss of aggregate because not enough binder was used. Figure 23 shows a cross section of graded aggregate on a roadway surface (from Janisch and Gaillard 1998).
- Some aggregate chips protrude farther above than many others, making them easier to dislodge under traffic and snowplow blades.
- Portions of the aggregate chips may become completely imbedded in the binder, resulting in an increased opportunity for bleeding to occur on the road surface.

- There is a greater chance for more dust to be present with the use of graded unwashed aggregate, causing the binder to not stick to the aggregate as well and loss of aggregate to occur.
- Graded aggregates tend to produce a tighter bound surface. This tighter bound surface leads to a quieter ride for the vehicle occupants traveling over the road surface.
- The use of graded aggregate creates less contact surface area between the tire and road because of different sized aggregate particles protruding up farther than others.

Figure 23. Cross Section of Graded Aggregate in a Thin Maintenance Surface

Despite the difficulties mentioned with graded aggregate, several test sections constructed under this project with graded aggregate exhibited excellent performance. The advantages and disadvantages of using either one-size or graded aggregate for a TMS are listed in Table 21.

Dusty Aggregate

The amount of dust in the aggregates used for a TMS should be kept to a minimal amount. CRS emulsions should not be used with dusty aggregates, while high float emulsions work with small dust amounts and cutback emulsions work better with dusty aggregate. More on the types of emulsions and dusty aggregate will be discussed later. The presence of dust on the cover aggregate prevents good adhesion between the aggregate and the applied binder, resulting in a loss of aggregate chips when the roadway is subjected to traffic. For CRS emulsions, a rule of thumb is that if you pick up a hand full of aggregate and throw it down and notice dust on your hand, it is too dusty.

Table 21. Advantages and Disadvantages to Using Either One-Size or Graded Aggregate

Gradation	Advantages	Disadvantages
One-size	• More void space (compared to graded aggregate) for more binder to be shot and allows more tolerance with regard to binder application rate. • Allows spreading of aggregate in one layer so each particle is bound to the road surface, not other aggregate particles. • Mitigates tracking by keeping tires away from binder. • Theoretically prevents snowplow blades from catching single aggregate particles that stand above others and plucking them out. • Requires lower application rate by weight per square area compared to graded aggregate.	• Road may seem rough or noisy to occupants (though no worse than any other road with an open texture). • May add to cost of aggregate if fine material becomes a waste product, which cannot be used in another product. • May not be produced in certain geographic areas, thus requiring long distance transportation and more expense. • Some aggregate may be plucked or sheared off by snowplow blade because sharp corners may stand above other aggregate pieces.
Graded	• Provides a smoother, tighter road surface. • Uses fine material from quarry that may otherwise become a waste product. • High availability locally in Iowa, thus low transportation costs.	• Reduces macro-texture, thus increasing risk of hydroplaning. • Lack of void space allows less binder to be shot and less tolerance in binder shot rate, compared to one-size aggregate. • Subject to tracking because some aggregate may be submerged in binder. • Subject to aggregate loss because some aggregate is not bound directly to the road surface. • Requires higher application rate in weight per square area when compared to one-size aggregate.

Aggregate Size

The size of aggregate used in a TMS falls in one of two categories: small aggregate (<=3/8”) or large aggregate (>3/8”).

Small Aggregate

- The design shot rate is smaller when smaller aggregate is used. This is because it takes less binder to bind the aggregate particles to the roadway. As a result of a smaller shot rate, there is a lower cost.
- Because design shot rate is smaller, there is less binder available to seal the cracks and there is less room for error in the binder application rate. If too much binder is used, flushing occurs, and if too little binder is used, the aggregate particles will not stay bound to the roadway.

- A smoother tighter road surfaced is created with the use of smaller aggregate, but this leads to less macro-texture on the road surface.
- A smaller weight per square area of aggregate to be spread is required.
- If the aggregate is picked up by the tires, often called fly rock, less damage is done to vehicles with the use of a small aggregate.

Large Aggregate

- The design shot rate is larger with the use of large aggregate. Thus, there is more binder available to seal the cracks. Also, there is more room for error in the binder application rate. However, the higher design shot rate leads to higher costs.
- Larger aggregate is less likely to wear to the point where the tires and binder would be in contact with each other.
- There is more macro-texture, increased skid resistance, with large aggregate. However, this greater macro-texture leads to more road noise.
- A larger weight of aggregate per square area to be spread is required.
- If the large aggregate becomes dislodged and picked up by the tires, there is a greater chance of damage to the vehicles.

The advantages and disadvantages of aggregate sizes used in TMS are listed in Table 22.

Table 22. Advantages and Disadvantages of Aggregate Sizes

Size	Advantages	Disadvantages
Small (<=3/8")	• Provides a smoother, tighter road surface. • Requires a smaller weight per square area of aggregate to be spread. • Fly rock does less damage to vehicles. • Design shot rate is smaller, thus lower cost.	• There is less room for error in the binder application rate (the distance from the top of the aggregate to the top of the binder is smaller). • Design shot rate is smaller, thus less binder available to seal cracks. • The top of the aggregate may wear down more quickly allowing tire contact with the binder. • Less macro -texture.
Large (>3/8")	• Less sensitive to errors in binder application rate. • Design shot rate is larger; more binder available to seal cracks • Aggregate is less likely to wear sufficiently to allow tires to contact the binder. • More macro-texture.	• Like other open surfaces with high macro-texture, more road noise for vehicle occupants. • Larger weight of aggregate per square area to be spread. • Fly rock is heavier and more likely to damage vehicle. • Design shot rate is higher, thus higher cost.

Binders

For TMS applications, asphalt is used as a binder because of two key properties: it is waterproof, and it adheres relatively well to the aggregate. Since asphalt is too stiff at room temperature to apply to the road surface, it is usually applied as either a cutback asphalt or an asphalt emulsion.

Cutback Asphalt

Cutback asphalt is asphalt that is thinned with solvents such as kerosene or naphtha (gasoline), which is called cutter. The following factors should be considered in the use of cutback asphalt:

- The type of solvent used controls the curing time of the cutback.
- Rapid curing cutbacks use naphtha, while medium curing cutbacks use kerosene.
- The higher the content of cutter in the cutback asphalt, the less viscous and more fluid the cutback asphalt will be.
- Cutback asphalts are useful when the penetration of a hard pavement surface is needed and when the seal coating process must be extended late into the construction season.
- Cutback asphalts also have a much higher percentage of residual asphalt compared to emulsions, which leads to more asphalt being left on the road surface for the same amount of binder applied.
- Cutback asphalts stay active longer, which means that they are able to penetrate and coat the dust that may be on the aggregate.
- A disadvantage is that the solvents used to thin the asphalt evaporate, give off hydrocarbons into the atmosphere, and pose environmental risks and safety problems when they are used.

Asphalt Emulsion

Asphalt emulsions are fine asphalt particles that are brought into contact with a chemical solution (emulsifier) to provide stabilization, and then are dispersed in water. This makes them less harmful to the environment and safer to work with, which is the primary reason why they are used more often than cutback asphalts. The following factors should be considered in the use of asphalt emulsion:

- Asphalt emulsions are divided into three major types: cationic, anionic, and non-ionic. Only the first two types are used in construction and have a positive (cationic) and negative (anionic) charge.
- Emulsions are then further classified based on how fast they “break,” revert back to their asphalt state. Classifications include rapid, medium, or slow setting emulsions.

- The principal investigator has noted anecdotally that in New Zealand emulsions have been formulated that work late in the season, thus extending the construction season, which contradicts the usual practice in the United States.

High Float Emulsion

High float emulsions are made with a special family of emulsifying agents that leaves a gel structure behind in the asphalt residue. The following factors should be considered in the use of high float emulsion:

- High float emulsions were developed for low volume roads in areas where a graded cover aggregate is to be used.
- High float emulsions are also quite effective when used with somewhat dusty aggregates because they provide a thicker asphalt film on the aggregate and the aggregate can penetrate much more uniformly. This is because high float emulsions are slightly anionic (sets slower than most cationic emulsions) and there is a small amount of solvents in them that act as a cutter in penetrating the dust. A thicker asphalt film coats the aggregate; therefore, high float emulsions do not flow and drain as readily as conventional emulsions.
- Rapid setting high float emulsions set slower than rapid setting cationic emulsions; this slower setting time allows for the liquid to have more time to penetrate the layers of dust that may be present on the aggregate.
- Reportedly, bleeding at high temperatures and brittleness at low temperatures is less likely to occur with high float emulsions because after the emulsion is allowed to cure, the residue that is left behind has a higher viscosity from a gel like structure that is left behind. Results of our test section performance did not always corroborate this claim.

Cationic Emulsion

Since aggregates are negatively charged, cationic emulsions are more often used than anionic emulsions. Cationic emulsion droplets have a positive charge; thus they are attracted to the negatively charged aggregate, since opposite electrical charges attract each other. When the asphalt particles and the aggregate particles are attracted to each other, this event is called breaking. According to the *Minnesota Seal Coat Handbook,* "Breaking refers to the event when the asphalt and water separate from each other. This occurs as the emulsifier leaves the surface of the asphalt particles due to its attraction to the surface of the aggregate. Since asphalt is heavier than water, the asphalt particles will settle to the bottom of the solution" (Janisch and Gaillard 1998). Figures 24 and 25 show depictions of this breaking process (from Janisch and Gaillard 1998).

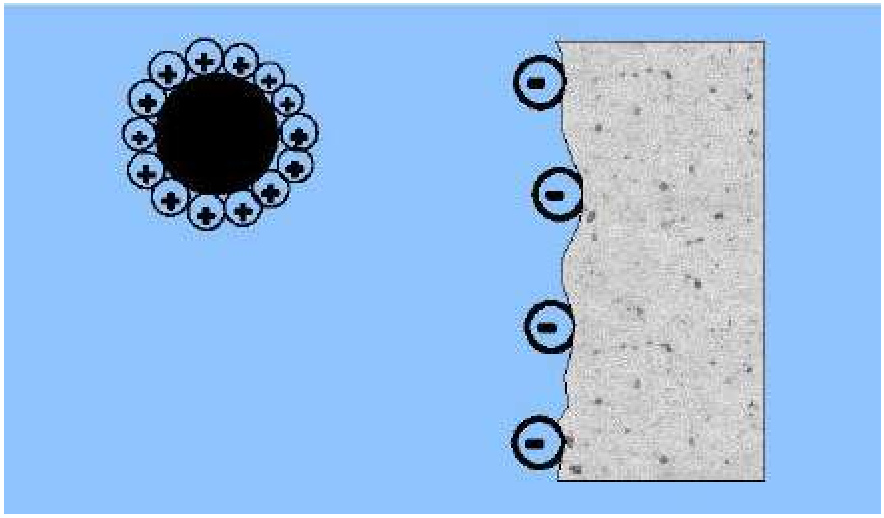


Figure 24. Cationic Emulsion Before It Begins to Break

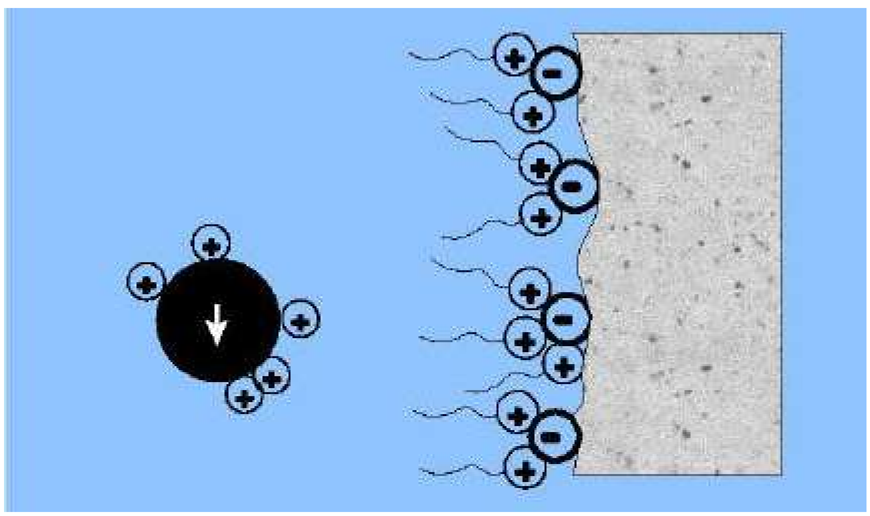


Figure 25. Cationic Emulsion Beginning to Break

Note:

- Given the correct aggregate, cationic emulsions have performed reliably in the field and they set up more quickly than anionic emulsions.
- Cationic rapid set (CRS) emulsions adhere to the aggregates much faster, thus allowing for the road to be opened to traffic sooner. However, when a CRS emulsion is used, the cover aggregate must be placed much faster so as to ensure the emulsion breaks after it has had time to coat the aggregate.
- CRS emulsions work well with clean and dust-free aggregate. However, if dusty aggregate is to be used, then pre-coating the aggregate prior to its use is required; this will be discussed more later.

Polymer-Modified Binder

Properties of asphalt emulsions can be enhanced with the addition of polymers to the emulsion, creating a polymer-modified emulsion. Note:

- When polymers are added to an emulsion, there is an increase in early stiffness of the binder, which leads to a better early aggregate chip retention.
- When compared with non-polymer-modified binders, the flexibility of the treated surface is increased in cold weather and over time as a result of the emulsion being modified with the addition of polymers.
- Bleeding and flushing of surfaces treated with polymer-modified emulsions is reduced in warm weather because polymers enhance binder stiffness at high temperatures.
- When polymer-modified emulsions are used, there is an increase in cost, typically about 30 percent.

Depending on the roadway and the circumstances for the road, the benefits of the polymer-modified emulsion may warrant its use. Some roads that may warrant their use are high volume roads and areas where more turning, starting, and stopping occurs, such as roads in municipalities. For each of the previously mentioned types of asphalt binders, Table 23 lists their advantages and disadvantages for use in TMS.

Aggregate and Binder Interactions

Dusty Aggregate Problems

Dusty aggregate does not react well with some binders that are used in TMS. This is because the dust particles, which have a large negative charge on them, prevent good adhesion between the aggregate and the asphalt binder because the binder binds to the dust instead of the aggregate. If dust-free aggregate is not available, the following must be done:

- The material needs to be washed with clean potable water and then the cleaned aggregate needs to be restockpiled and allowed to dry.
- A cutback or high float emulsion should be used for the binder as stated above.
- The aggregate should be pre-coated with a thin film of asphalt emulsion or hot asphalt cement. Precoating aggregate increases aggregate retention.
- If a precoating process is to be performed, the dust content should be limited to no more than three percent (Kandhal and Motter 1991).
- Even though some asphalt has been applied to the aggregate during the pre-coating process, the amount of asphalt binder to be applied to the roadway should be the same as that for non-precoated aggregate. The aggregate chips should be considered as "black rock" with the precoat asphalt assumed to provide little actual binding properties.

- For precoated aggregates more than 90 percent of the visible area should be covered (Kandhal and Motter 1991).
- The cost of precoated aggregate is higher than untreated aggregate, but there is less aggregate loss and better bonding between the aggregate and the asphalt binder. The cost of pre-coated aggregate is around $19/ton, delivered (Parker 2002).

Stripping Problems of Aggregate

Some high-quality aggregates do not bind well with any type of binder; quartzite is one example of these aggregates. To reduce the stripping of the aggregate from the road surface and the binder, two things can be done; either the material can be dried or an anti-stripping agent can be added to the material.

The following points were made by Selim and Tham in "Improving Chip Retention and Reducing Moisture Susceptibility of Seal Coats" (1993) and Selim in "Enhancing the Bond of Emulsion-Based Seal Coats with Antistripping Agents" (1989). More information on stripping problems of aggregate can be found in those papers.

Dried Aggregate

- The susceptibility of stripping aggregate from the roadway is reduced when dried aggregate is used compared to the use of aggregate with its natural field moisture content.
- The amount of stripping of dried aggregate is reduced by about 25 percent by using dried aggregate compared to aggregate at its field moisture content.

Anti-Stripping Agents

- The use of anti-stripping agents with the aggregate enhances chip retention the most when compared to using aggregate with its field moisture content and aggregate that has been dried.
- With the use of Redicote-82-S as the anti-stripping agent, the amount of aggregate loss is about 30 percent less than the amount of aggregate loss when using aggregate with its field moisture content.
- Anti-stripping agents should be added to the emulsion instead of applied to the aggregate itself for the following reasons: This yields a higher friction value. It is much easier and cheaper to add the agent to the emulsion than to try and coat the aggregate.
- The use of an anti-stripping agent and dried aggregate further increases the amount of initial aggregate that is retained.
- The skid resistance of the road surface seems to be improved with the use of an anti-stripping agent. The addition of the agent reduces the amount the aggregate is allowed to rotate.

Table 23. Advantages and Disadvantages of Binder Types

Type	Advantages	Disadvantages
Cutback asphalt	• Best at binding dusty aggregate. • Some possible penetration into dry road surfaces increases bond. • Will retain aggregate that is not spread immediately after shooting binder.	• Subject to bleeding and tracking. • Some products are flammable. • Emits hydrocarbons during curing process. • Curing can take considerable time. • Aggregates must be dry.
Cationic rapid setting (CRS)	• Binds clean aggregates with low clay content securely to road surface. • Cures quickly. • Works with damp aggregate. • Commonly available and familiar to industry participants.	• Ineffective for dusty aggregates or aggregates with high clay content. • Aggregate must be spread immediately behind distributor truck.
High float	• Binds aggregate with more dust and clay when compared to CRS. • Cures quickly, but not as quickly as CRS. • Works with damp aggregate. • May coat aggregate more thickly, yet reduce movement that causes bleeding due to "gel" structure of cured emulsion.	• Does not cure as quickly as CRS (but more quickly than cutback). • Industry participants may not be as familiar with this product as CRS, depending on geography and local experience. • Some hydrocarbons released during curing due to the use of cutter (kerosene) in this product (much less than standard cutback).
Polymer-modified (added either to CRS or high float)	• Greater flexibility during cold weather, mitigates cracking. • Greater stiffness in warm weather, mitigates bleeding and retains aggregate in areas of turning, accelerating, and decelerating traffic. • Higher early strength during curing leads to better chip retention.	• Higher cost compared to non-polymer-modified binder.

CHAPTER 4. RECOMMENDED SEAL COAT DESIGN METHOD

Comparison and Selection of Seal Coat Design Methods

Researchers selected a seal coat design method for use in Iowa. Comparisons between design methods are available in the literature. Therefore, the selection was made based on literature review. The current practices of Iowa's neighboring state of Minnesota also had considerable influence on the decision.

Comparison between Simplistic and Sophisticated Methods

The Pennsylvania Department of Transportation (Penn DOT) compared seven seal coat design procedures: (A) ASTM, (B) Asphalt Road Materials, (C) Bituminous Materials, (D) Asphalt Institute, (E) McLeod, (F) Penn DOT, and (G) Chevron (Roque et al. 1989). According to Penn DOT's comparison, those seven procedures can be divided into two groups: simplistic procedures (ASTM, Asphalt Road Materials, and Bituminous Materials) and sophisticated procedures (Asphalt Institute, McLeod, Penn DOT, and Chevron).

Table 24 shows which design parameters are considered by each method (Roque et. al. 1989). According to Roque et al. (1989), the three simplistic procedures underestimate the application rates for both the emulsion and aggregate. The remaining procedures (except Asphalt Institute) predict nearly identical application rates for the aggregate, agreeing well regarding emulsion application rates (see Table 25 and Figure 26, from Roque et. al. 1989).

Comparison between McLeod and Texas DOT Methods

Shuler (1990) compared the McLeod and the Texas DOT methods for seal coat design. The only differences he noted were when synthetic aggregate was used. Neither of these products are commonly used in Iowa, so from Iowa's point of view, there is little difference between the two methods.

Selection of Recommended Method

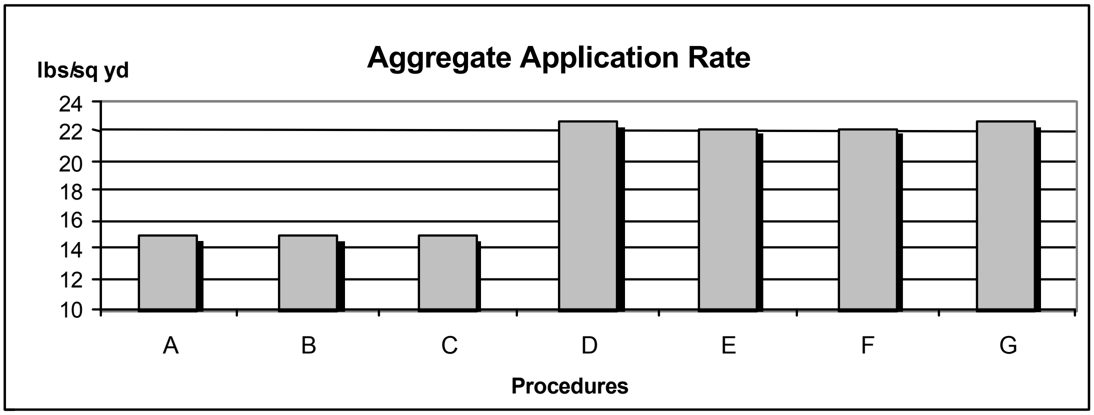
It is recommended that Iowa adopt the McLeod method for seal coat design, as modified in the *Minnesota Seal Coat Handbook* (Janisch and Gaillard 1998). The McLeod method has a long history of satisfactory performance and gives design application rates that are little different from other methods. Minnesota has made a considerable investment in documentation and training materials to implements its use. Iowa would be wise to share in the benefits of this investment. Most of the remaining portion of this chapter is a summary of the McLeod design procedure as described in the *Minnesota Seal Coat Handbook*; more detailed information can be found there.

Table 24. Design Parameters Considered by Various Seal Coat Design Procedures

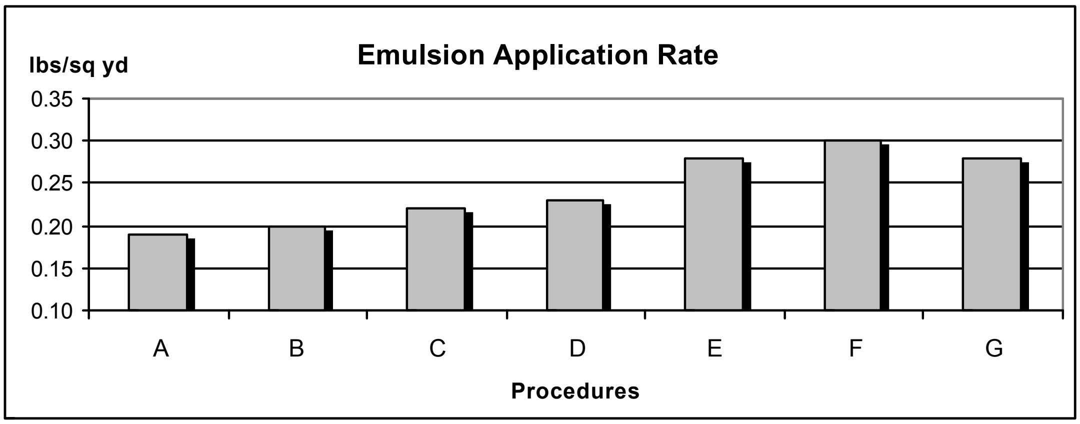
Design Parameters		(A) ASTM	(B) Asphalt Road Materials	(C) Bituminous Materials	(D) Asphalt Institute	(E) McLeod	(F) Penn DOT	(G) Chevron
1.	Aggregate type (crushed slag, gravel, sand, etc.)	■	■	■	■	■	■	■
2.	Aggregate condition (wet or dry, dirty, or clean)							■
3.	Aggregate compatibility (with existing pavement, emulsion)							■
4.	Emulsified asphalt type (based on set time, application temperature)				■	■	■	■
5.	Emulsion compatibility (with existing pavement, aggregate)							■
6.	Existing pavement condition		■	■	■	■	■	■
7.	Traffic volume/annual average daily traffic (AADT)		■	■		■	■	
8.	Application type (single or double)	■				■	■	
9.	Application temperature of asphalt		■			■	■	
10.	Field conditions of site (rain, sunny, etc.)	■	■	■	■	■	■	■
11.	Climate of site (wet, dry, humid, etc.)							
12.	Flakiness index					■		
13.	One-sized vs. graded aggregate							

Table 25. Application Rates for Each Seal Coat Method

Design Method (ID)	Application Rate	
	Aggregate (lbs/yd^2)	Binder (gal/yd^2)
ASTM (A)	15	0.19
Asphalt Road Materials (B)	15	0.20
Bituminous Materials (C)	15	0.22
Asphalt Institute (D)	22.5	0.23
McLeod (E)	22	0.28
Penn DOT (F)	22	0.30
Chevron (G)	22.5	0.28



(a)



(b)

Figure 26. Comparison of (a) Aggregate and (b) Binder Application Rates for Each Seal Coat Method

Recommended Seal Coat Design Method

In the McLeod seal coat design method, the aggregate application rate depends on the aggregate gradation, shape, and specific gravity. The binder application rate depends on the aggregate gradation, absorption, shape, traffic volume, existing pavement condition, and the residual asphalt content of the binder.

Basic Principles

The McLeod method is based on two basic principles (see Figure 27):

1. The application rate of a given cover aggregate should be determined so that the resulting seal coat will only be one-stone thick.
2. The voids in the aggregate layer need to be 70 percent filled with asphalt cement for good performance on pavements with moderate levels of traffic.

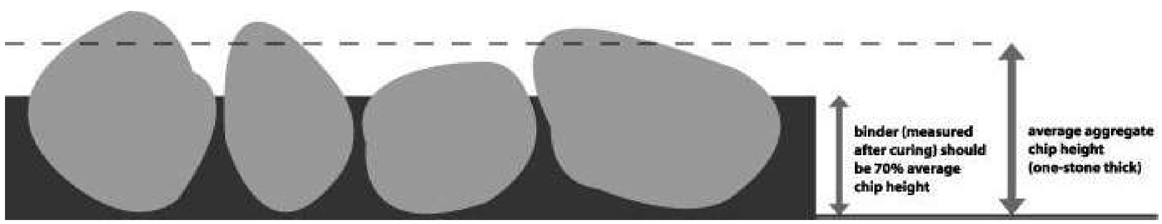


Figure 27. Recommended One-Stone Thickness and Proper Embedment

Design Procedures

Step 1. Determine Median Particle Size of the Aggregate

The median particle size (M) is the theoretical sieve size through which 50 percent of the material passes. Figure 28 shows an example gradation chart for median particle size (M) (Janisch and Gaillard 1998).

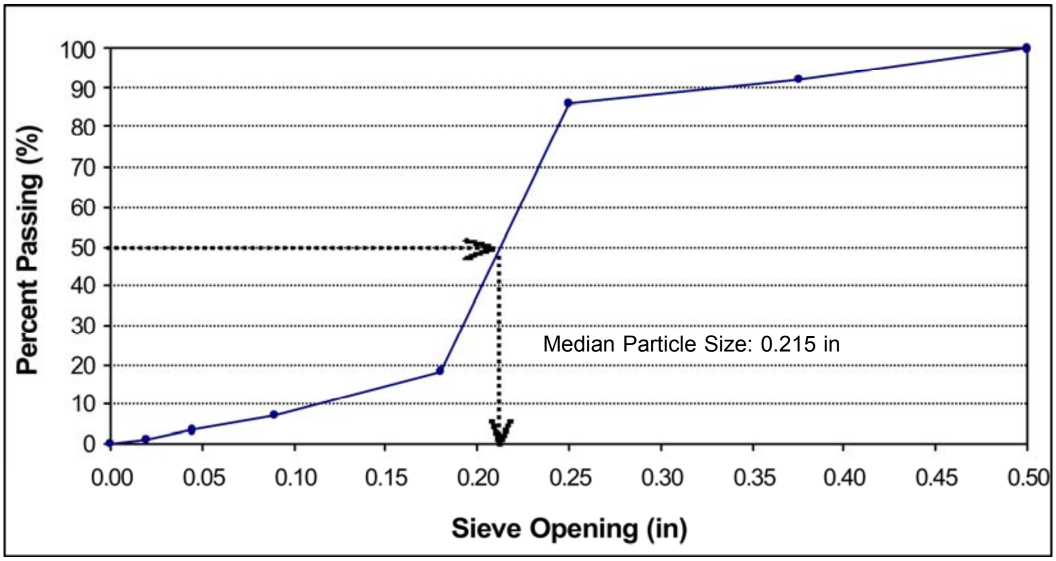


Figure 28. Gradation Chart for Design Example Showing Median Particle Size

Step 2. Measure Flakiness Index of the Aggregate

The flakiness index (*FI*) is a measure of the percent of flat particles in terms of weight. It is determined by testing a small sample of aggregate particles for their ability to fit through a slotted plate. There are five different sized slots in the plate. Table 26 lists the size of the slot and which materials pass through the slots, and Figure 29 shows an illustration of how the slots look. If the chips can fit through the slotted plate they are considered to be flat. If not, they are considered to be cubical. The weight of material passing all of the slots is then divided by the total weight of the samples to give the percent of flat particles, by weight, or flakiness index. The lower the *FI*, the more cubical the material is.

Table 26. Size of Aggregate and Slot to Use

Slot	Size of Material		Slot Width (inches)
	Passing	Retained on	
Slot 1	1” sieve	3/4” sieve	0.525
Slot 2	3/4” sieve	1/2” sieve	0.375
Slot 3	1/2” sieve	3/8” sieve	0.263
Slot 4	3/8” sieve	1/4” sieve	0.184
Slot 5	1/4” sieve	No. 4 sieve	0.131

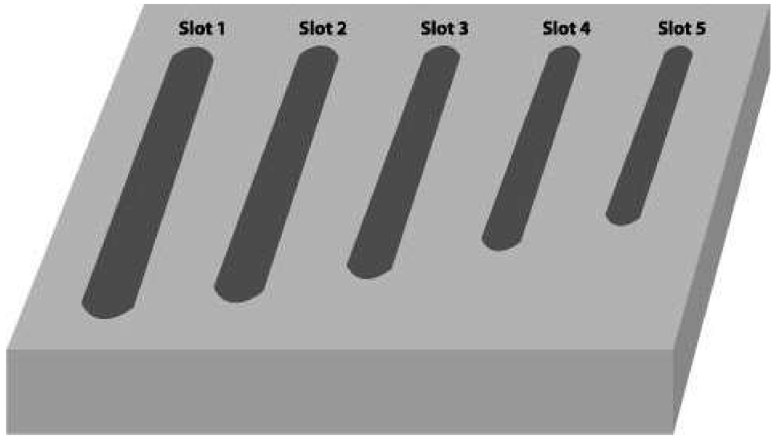


Figure 29. Flakiness Index Slotted Testing Plate

Step 4. Calculate Average Least Dimension of the Seal Coat

The average least dimension (ALD or H) is calculated using the median particle size (M) and flakiness index (FI). It represents the expected seal coat thickness in the wheel paths where traffic forces the flat chips to lie on their flattest side.

$$H = M / (1.139285 + 0.011506 * FI)$$

where H = average least dimension (inches or mm), M = median particle size (inches or mm), and FI = flakiness index.

Step 5. Determine Unit Weight or Bulk Specific Gravity of the Aggregate

Different aggregates have different specific gravities or unit weights. The bulk specific gravity (G) must be taken into account in the design procedure because it will take more pounds of a heavy aggregate than a light aggregate to cover a square yard of pavement. Table 27 shows the typical bulk specific gravity of common seal coat aggregates (Janisch and Gaillard 1998).

Table 27. Typical Bulk Specific Gravity of Common Seal Coat Aggregates

Aggregate Type	Bulk Specific Gravity (G)		
	Maximum	Average	Minimum
Limestone	2.67	2.61	2.40
Pea rock	2.66	2.62	2.55
Quartzite	2.63	2.62	2.59
Granite	2.75	2.68	2.60
Trap rock	2.98	2.97	2.95

Step 6. Calculate Loose Unit Weight of the Aggregate

The loose unit weight (*W*) is determined according to standard test method ASTM C29 and is needed to calculate the air voids expected between the chips after initial rolling. Loose unit weight depends on the gradation, shape, and specific gravity of the aggregate. Well-graded aggregate and aggregate with high fines content will have the highest loose unit weight because the particles pack together tightly leaving little room for air. This air space between the aggregate particles is the only space available to place the binder.

$$W = \frac{\text{Weight of Aggregate}}{\text{Volume of Cylinder}}$$

where W = loose unit weight (lbs/ft^3 or kg/m^3).

Step 7. Calculate Voids in the Loose Aggregate

The voids in the loose aggregate (*V*) approximate the voids present when the aggregate is placed by the spreader onto the pavement. Generally, this is nearly 50 percent for one-size aggregate, less for graded aggregate. After initial rolling, the voids are assumed to be reduced to 30 percent and will reach a low of about 20 percent after sufficient traffic has oriented the stone on their flattest side. However, if there is very little traffic, the voids will remain nearly 30 percent and the seal coat will require more binder to ensure good chip retention.

For U.S conventional units: $V = 1 - \frac{W}{62.4 * G}$

For S.I. metric units: $V = 1 - \frac{W}{1000 * G}$

where V = voids in the aggregate (percent expressed as a decimal), W = loose unit weight (lb/ft^3 or kg/m^3), and G = bulk specific gravity of the aggregate.

Step 8. Determine Traffic Whip-Off Factor

The McLeod procedure recognizes that some of the cover aggregate will get thrown to the side of the roadway by passing vehicles as the fresh seal coat is curing. The amount of aggregate that will do this is related to the speed and number of vehicles on the new seal coat. To account for this, a traffic whip-off factor (*E*) is included in the aggregate design equation. A reasonable value to assume is 5 percent for low volume, residential type traffic, and 10 percent for higher speed roadways, such as county roads. The traffic whip-off or wastage factor is given in Table 28 (Asphalt Institute 1979).

Table 28. Traffic Whip-Off Factor Table

Percent Waste Allowed	Traffic Wastage Factor (*E*)
1%	1.01
2%	1.02
3%	1.03
4%	1.04
5%	1.05
6%	1.06
7%	1.07
8%	1.08
9%	1.09
10%	1.10
11%	1.11
12%	1.12
13%	1.13
14%	1.14
15%	1.15

Step 9. Calculate Cover Aggregate Application Rate

The cover aggregate application rate (*C*) should include a correction for traffic whip-off:

$$C = 46.8 * (1 - 0.4 * V) * H * G * E$$

where *C* = cover aggregate application rate (lb/yd^2 or kg/m^2), *V* = voids in loose aggregate, *H* = average least dimension (inches or mm), *G* = bulk specific gravity of the aggregate, and *E* = traffic whip-off factor.

Step 10. Determine Aggregate Absorption Factor

Most aggregates absorb some of the binder that is applied. The design procedure must be able to correct for this condition to ensure that enough binder will remain on the pavement surface. McLeod suggests an absorption correction factor (*A*) of 0.02 gal/yd^2 (0.09 L/m^2) if the aggregate absorption is around 2 percent. In this seal coat design process, there are two options for the aggregate absorption correction factor (*A*): 0.02 gal/yd^2 (0.09 L/m^2) and 0.03 gal/yd2 (0.136 L/m^2). Table 29 can be used as a guideline for the typical aggregate absorption factors (Janisch and Gaillard 1998).

Table 29. Typical Aggregate Absorption Factors of Common Seal Coat Aggregates

Aggregate Type	Aggregate Absorption Factor (*A*)		
	Maximum (%)	Average (%)	Minimum (%)
Limestone	5.44	2.80	1.75
Pea rock	2.32	1.69	1.14
Quartzite	0.72	0.67	0.61
Granite	0.92	0.59	0.40
Trap rock	0.59	0.43	0.31

Step 11. Determine Traffic Volume Factor

The traffic volume, the number of vehicles per day, on the pavement surface must be taken into consideration to determine the amount of binder needed. Generally speaking, the higher the traffic volume, the lower the binder application rate. This is because there is a greater chance chips be lying on their flat sides with higher traffic volumes. Consequently, less asphalt binder is needed to achieve the desired 70 percent embedment. If this in not taken into account, the wheel paths will likely bleed. The McLeod design procedure uses Table 30 to estimate the required embedment, based on the number of vehicles per day on the roadway. *T* is the traffic volume factor.

Table 30. Traffic Volume Factor Table

Number of Vehicles Per Day	Traffic Volume Factor (*T*)
Under 100	0.85
100 to 500	0.75
500 to 1,000	0.70
1,000 to 2,000	0.65
Over 2,000	0.60

Step 12. Determine Pavement Surface Condition Factor

The condition of the existing pavement plays a major role in determining the amount of binder required to obtain proper embedment. A new smooth pavement with low air voids will not absorb much of the binder applied to it. Conversely, a dry, porous, and pocked pavement surface can absorb a tremendous amount of the binder. Failure to recognize when to increase and decrease the binder application rate to account for the pavement condition can lead to excessive chip loss or bleeding. The surface condition factors (*S*) used in the McLeod procedure are listed in Table 31.

Table 31. Pavement Surface Condition Factor Table

Existing Pavement Surface	Surface Condition Factor (*S*)	
	S.I. Metric Units (liters/m^2)	U.S Customary Units (gal/yd^2)
Black, flushed asphalt	-0.04 to -0.27	-0.01 to -0.06
Smooth, non-porous	0.00	0.00
Slightly porous, oxidized	0.14	0.03
Slightly pocked, porous, oxidized	0.27	0.06
Badly pocked, porous, oxidized	0.40	0.09

Step 13. Calculate Binder Application Rate

The binder application rate (*B*) should include a correction for aggregate absorption (*A*), traffic volume (*T*), surface condition factor (*S*), and residual asphalt content of binder (*R*). In calculating the binder application rate, it must be remembered that it is not practical to assume that all roadways to be sealed in a given project will need the same amount of asphalt binder. A single project may include new pavements, old pavements, porous pavements, flushed pavements, etc.

$$B = \frac{((2.244 * H * T * V) + S + A)}{R}$$

where B = binder application rate (gal/yd^2 or liters/m^2), H = average least dimension (inches or mm), T = traffic volume factor, V = voids in loose aggregate (percent expressed as a decimal), S = surface condition factor (gal/yd^2 or liters/m^2), A = aggregate absorption factor (gal/yd^2 or liters/m^2), and R = residual asphalt content of binder (percent expressed as a decimal).

Spreadsheet

Table 32 is a spreadsheet for seal coat design that works according to the McLeod seal coat design procedure, as has been recommended above. The table has a function to convert the design quantities of U.S. conventional units into those of the S.I. metric unit system. The spreadsheet also has two options for the aggregate absorption factor (A): 0.02 gal/yd^2 (0.09 liters/m^2) and 0.03 gal/yd^2 (0.136 liters/m^2). A working version of this spreadsheet will accompany electronic versions of this report.

Sensitivity Analysis

A sensitivity analysis (Case 1) was performed to find how the cover aggregate application rate (C) and binder application rate (B) change with the change of each design parameter. Each design parameter is increased by 10 percent.

The result of this analysis is that the cover aggregate application rate (C) is most sensitive to changes in the median particle size (M), bulk specific gravity (G) of an aggregate, and traffic whip-off (E) than to any other design parameters. The binder application rate (B) is most sensitive to median particle size (M), bulk specific gravity (G), loose unit weight (W), traffic volume factor (T), and residual asphalt content of the binder (R). Table 33 provides a summary of Case 1; more detail can be found in Table 34.

A second sensitivity analysis (Case 2) was done to confirm the results from the first analysis. Like the first analysis, all parameters were increased by 10 percent. The results of the second analysis show that changes in cover aggregate application rate (C) and in binder application rate (B) are in the same direction as those from the first sensitivity analysis. However, the magnitudes of the changes in Case 2 are different from those in Case 1. A summary of Case 2 can be found in Table 35; more detail is available in Table 36.

Table 32. Seal Coat Design Spreadsheet (Recommended McLeod Method)

SEAL COAT DESIGN			Unit Selection: S.I. Metric Unit / U.S. Conventional Unit (selected)
	unit		**inputs****
Median Particle Size (**M**)	in	0.210	0.210
Flakiness Index (**FI**)	%	29.00	29.00
Average Least Dimension (H)	in	**0.143**	
Bulk Specific Gravity (**G**)		2.62	Quartzite / Average
Loose Unit Weight (**W**)	lbs / cu. ft	100.00	100.00
Void (V)	% as a decimal	**0.388**	
Aggregate Wastage Factor (**E**)		1.05	5
Cover Aggregate Application Rate (C)	lbs / sq. yd	**15.504**	
Traffic Factor (**T**)		0.70	500 to 1,000
Surface Condition Factor (**S**)	gal / sq. yd	0.06	Slightly pocked, porous & oxidized
Aggregate Absorption	%	2.80	Limestone / Average
Residual Asphalt Content of Binder (**R**)	% as a decimal	0.67	0.67
Binder Application Rate (B) for	gal / sq. yd		**Aggregate Absorption Factor (A)**
Wheel paths (Flat)		**0.316**	0.03
Non-Wheel paths (Not Flat)		**0.327**	0.03
Starting Point in the field (Avg.)		**0.321**	
** Numerical inputs must have U.S. Conventional Unit.			Aggregate Absorption Factor: Factor 1 (0.02) / Factor 2 (0.03) (selected)

The effects of design parameters on cover aggregate application rate (*C*) and binder application rate (*B*) can be summarized as follows:

- An increase in median particle size (*M*) increases average least dimension (*H*) and binder application rate (*B*) for non-wheel paths. An increase in average least dimension (*H*) causes both the cover aggregate application rate (*C*) and binder application rate (*B*) for wheel paths to increase.
- A change in flakiness index (*FI*) does not affect the binder application rate (*B*) for non-wheel paths. *FI* does affect both the cover aggregate application rate (*C*) and binder application rate (*B*) for wheel paths.
- A change in bulk specific gravity (*G*) causes changes in the percent of voids in the aggregate (*V*) (mathematical change, not physical change, that is), cover aggregate application rate (*C*), and binder application rate (*B*).
- An increase in loose unit weight (*W*) increases cover aggregate application rate (*C*), but decreases binder application rate (*B*) for both wheel and non-wheel paths.
- A change in the traffic whip-off factor (*E*) changes only the aggregate application rate (*C*).
- Binder application rate (*B*) is only influenced by the traffic volume factor (*T*), surface condition factor (*S*), residual asphalt content of the binder (*R*), and, when applicable, aggregate absorption factor (*A*).

Table 33. Summary of Case 1

Parameter	***C***	***B* (wheel paths)**	***B* (non-wheel paths)**
Median particle size (*M*)	10.00%	5.86%	6.76%
Flakiness index (*FI*)	-2.22%	-1.30%	0.00%
Bulk specific gravity (*G*)	2.79%	3.96%	4.57%
Loose unit weight (*W*)	2.99%	-10.34%	-11.93%
Traffic whip-off factor (*E*)	0.48%	0.00%	0.00%
Traffic volume factor (*T*)	0.00%	4.18%	4.83%
Surface condition factor (*S*)	0.00%	20.72%	16.22%
Aggregate absorption factor (*A*)	0.00%	20.72%	16.22%
Residual asphalt content (*R*)	0.00%	-9.09%	-9.09%

Table 34. Sensitivity Analysis for Case 1

SEAL COAT DESIGN

Unit Selection: S.I. Metic Unit / U.S. Conventional Unit

Aggregate Absorption Factor: 0.136 L/sq. m (0.03 gal/sq. yd. / 0.09 L/sq. m (0.02 gal/sq. yc

	unit		M	FI	G **	W	E	T **	S **	AA **	R
Median Particle Size (*M*)	in	0.220	**0.242**	0.220	0.220	0.220	0.220	0.220	0.220	0.220	0.220
Flakiness Index (*FI*)	%	29.00	29.00	**31.90**	29.00	29.00	29.00	29.00	29.00	29.00	29.00
Average Least Dimmension (H)	in	**0.149**	**0.164**	**0.146**	**0.149**	**0.149**	**0.149**	**0.149**	**0.149**	**0.149**	**0.149**
Bulk Specific Gravity (*G*)		2.51	2.51	2.51	**2.61**	2.51	2.51	2.51	2.51	2.51	2.51
Loose Unit Weight (*W*)	lbs / cu. ft	100.00	100.00	100.00	100.00	**110.00**	100.00	100.00	100.00	100.00	100.00
Void (V)	% as decimal	**0.362**	**0.362**	**0.362**	**0.386**	**0.298**	**0.362**	**0.362**	**0.362**	**0.362**	**0.362**
Aggregate Wastage Factor (*E*)		1.05	1.05	1.05	1.05	1.05	**1.055**	1.05	1.05	1.05	1.05
Cover Aggregate Application Rate (C)	lbs / sq. yd	**15.758**	**17.334**	**15.409**	**16.198**	**16.229**	**15.833**	**15.758**	**15.758**	**15.758**	**15.758**
Traffic Factor (*T*)		0.700	0.700	0.700	0.700	0.700	0.700	**0.750**	0.700	0.700	0.700
Surface Condition Factor (*S*)	gal / sq. yd	0.060	0.060	0.060	0.060	0.060	0.060	0.060	**0.090**	0.060	0.060
Aggregate Absorption* (*AA*)	%	0.610	0.610	0.610	0.610	0.610	0.610	0.610	0.610	**1.750**	0.610
Residual Asphalt Content of Binder (*R*)	% as decimal	0.670	0.670	0.670	0.670	0.670	0.670	0.670	0.670	0.670	**0.737**
Binder Application Rate (B) for	gal / sq. yd										
Wheel paths (Flat)		**0.216**	**0.229**	**0.213**	**0.225**	**0.194**	**0.216**	**0.225**	**0.261**	**0.261**	**0.196**
Non-Wheel paths (Not Flat)		**0.276**	**0.295**	**0.276**	**0.289**	**0.243**	**0.276**	**0.289**	**0.321**	**0.321**	**0.251**
Stating Point in the field (Avg.)		**0.246**	**0.262**	**0.245**	**0.257**	**0.218**	**0.246**	**0.257**	**0.291**	**0.291**	**0.224**

* When aggregate absorption (*AA*) is less than 1.5%, aggregate absorption factor (**A**) is 0, which means no adjustment, and when aggregate absorption is greaer than or equal to 1.5%, the factor (**A**) is applied.

** Instead of 10% increament, Tables are used.

Table 35. Summary of Case 2

Parameter	***C***	***B* (wheel paths)**	***B* (non-wheel paths)**
Median particle size (*M*)	10.00%	6.99%	7.61%
Flakiness index (*FI*)	-1.65%	-1.16%	0.00%
Bulk specific gravity (*G*)	10.27%	9.00%	9.80%
Loose unit weight (*W*)	2.56%	-7.43%	-8.08%
Traffic whip-off factor (*E*)	0.48%	0.00%	0.00%
Traffic volume factor (*T*)	0.00%	5.38%	5.85%
Surface condition factor (*S*)	0.00%	15.04%	11.95%
Aggregate absorption factor (*A*)	0.00%	0.00%	0.00%
Residual asphalt content (*R*)	0.00%	-9.09%	-9.09%

Table 36. Sensitivity Analysis for Case 2

SEAL COAT DESIGN

Unit Selection: ○ S.I. Metic Unit ◉ U.S. Conventional Unit

Aggregate Absorption Factor: ◉ 0.136 L/sq. m (0.03 gal/sq. yd.) ○ 0.09 L/sq. m (0.02 gal/sq. yd

	unit		M	FI	G **	W	E	T **	S **	AA **	R
Median Particle Size (*M*)	in	0.270	**0.297**	0.270	0.270	0.270	0.270	0.270	0.270	0.270	0.270
Flakiness Index (*FI*)	%	20.00	20.00	**22.00**	20.00	20.00	20.00	20.00	20.00	20.00	20.00
Average Least Dimmension (H)	in	**0.197**	**0.217**	**0.194**	**0.197**	**0.197**	**0.197**	**0.197**	**0.197**	**0.197**	**0.197**
Bulk Specific Gravity (*G*)		2.61	2.61	2.61	**2.97**	2.61	2.61	2.61	2.61	2.61	2.61
Loose Unit Weight (*W*)	lbs / cu. ft	83.90	83.90	83.90	83.90	**92.29**	83.90	83.90	83.90	83.90	83.90
Void (V)	% as decimal	**0.485**	**0.485**	**0.485**	**0.547**	**0.433**	**0.485**	**0.485**	**0.485**	**0.485**	**0.485**
Aggregate Wastage Factor (*E*)		1.05	1.05	1.05	1.05	1.05	**1.055**	1.05	1.05	1.05	1.05
Cover Aggregate Application Rate (C)	lbs / sq. yd	**20.383**	**22.422**	**20.046**	**22.476**	**20.904**	**20.480**	**20.383**	**20.383**	**20.383**	**20.383**
Traffic Factor (*T*)		0.650	0.650	0.650	0.650	0.650	0.650	**0.700**	0.650	0.650	0.650
Surface Condition Factor (*S*)	gal / sq. yd	0.030	0.030	0.030	0.030	0.030	0.030	0.030	**0.060**	0.030	0.030
Aggregate Absorption* (*AA*)	%	2.320	2.320	2.320	2.320	2.320	2.320	2.320	2.320	**2.800**	2.320
Residual Asphalt Content of Binder (*R*)	% as decimal	0.800	0.800	0.800	0.800	0.800	0.800	0.800	0.800	0.800	**0.880**
Binder Application Rate (B) for	gal / sq. yd										
Wheel paths (Flat)		**0.249**	**0.267**	**0.246**	**0.272**	**0.231**	**0.249**	**0.263**	**0.287**	**0.249**	**0.227**
Non-Wheel paths (Not Flat)		**0.314**	**0.338**	**0.314**	**0.344**	**0.288**	**0.314**	**0.332**	**0.351**	**0.314**	**0.285**
Stating Point in the field (Avg.)		**0.281**	**0.302**	**0.280**	**0.308**	**0.260**	**0.281**	**0.297**	**0.319**	**0.281**	**0.256**

* When aggregate absorption (*AA*) is less than 1.5%, aggregate absorption factor (**A**) is 0, which means no adjustment, and when aggregate absorption is greaer than or equal to 1.5%, the factor (**A**) is applied.

** Instead of 10% increament, Tables are used.

CHAPTER 5. LOCAL AGGREGATE FOR MICRO-SURFACING

Researchers coordinated an effort by local aggregate producers and Koch Materials, Inc., to develop a micro-surfacing mix design using local aggregate. It was expected that the mix would first be used for filling ruts and for scratch courses (the first course of a two-course application). Use could possibly be extended to surface courses of lower volume primary roads.

Quality Tests

In "Recommended Performance Guidelines for Micro-Surfacing" (1991), the International Slurry Surfacing Association specifies quality tests for aggregates as shown in Table 37.

Table 37. Guidelines for Quality Tests of Aggregate Used for Micro-Surfacing

Test	Quality	Specification
AASHTO T176/ASTM D2419	Sand Equivalent	60% minimum
AASHTO T104/ASTM C88	Soundness	15% maximum using NA_2SO_4 or 25% maximum using $MgSO_4$
AASHTO T96/ASTM C131	Abrasion Resistance	35% maximum

Currently, micro-surfacing aggregate for use by the Iowa DOT is restricted to Type 2 or Type 3 friction classification that excludes all limestone (Iowa Supplemental Specification 95024M). This restriction was made because it provided an expedient way to ensure that a uniform aggregate was provided to work with the highly reactive micro-surfacing emulsion. Specifiers wanted to ensure the technical success of early micro-surfacing projects, and it was felt that prohibiting the use of limestone would make the micro-surfacing more reliable. At the time of this study, it was felt that micro-surfacing had established a track record and some risks could be taken to reduce costs by identifying locally produced aggregate. It should be noted that limestone has been used for micro-surfacing aggregate elsewhere, including Ontario, Canada.

Two meetings were held that included representatives from the Iowa DOT, Iowa Limestone Producers Association, and Koch Materials, Inc. Based on Koch's experience, it was determined that aggregate sources with low clay content are considered the best candidates for micro-surfacing use. This is because micro-surfacing emulsion tends to react quickly and break on clay particles. The industry standard test to detect clay content of aggregates is the Sand Equivalent Test. Iowa requires that the Sand Equivalency Test (AASHTO T176) have a minimum of 60 percent. The result of this test provides an index for the amount of clay in a sample, not a direct measure.

Recently Iowa DOT had developed other testing methods for limestone in attempt of select better aggregates to PCC pavement construction. One of these tests, Iowa Test Method 222 (X-Ray Fluorescence Test) provides a better inference of the clay content. A list of previously conducted tests was made available for a number of different quarries.

Identification of Aggregate Sources

After discussion, a procedure for identifying promising aggregate sources was proposed:

1. If the X-Ray Fluorescence Test had been conducted on a sample and the alumina percentage was less than 0.15 percent, the material would be considered a candidate. This proposed limit was the upper bound for material produced in Martin Marietta's Fort Dodge Mine; this material has an excellent track record as a slurry seal aggregate. Based on the test results the following locations are likely candidates:

   - Fort Dodge Mine, Martin Marietta
   - Cedar Rapids (formerly Beverly), Wendling
   - Milan, Moline Consumer Co.
   - Tripoli, Paul Newman Construction Co.

   The following are also good candidates, as their geology is favorable:

   - Cedar Rapids South, Martin Marietta
   - MacGuire, Wendling
   - Wyoming, Wendling
   - Le Clair, Moline Consumer Co.
   - Moscow, Wendling
   - Wexford, Bruning

   Other possibilities are a number of quarries that have tested sand equivalents greater than 60 percent:

   - Shaffton Quarry (dolomite), Clinton County
   - Ballou Olin Quarry (dolomite), Jones County
   - White Quarry, Beds 1-2 (dolomite), Delaware County
   - Hawarden (crushed gravel), Sioux County

2. Producers who wish to have their material considered could send a sample to the Iowa DOT for clay content testing.

3. Koch Materials would select three aggregates for mix design development. The limit of three was established because the testing procedure is complex and time consuming; Koch representatives felt this was the maximum number to mix designs that could be produced given other demands on their laboratory. The sources would be geographically distributed as much as possible to limit transportation costs for potential projects.

If the mix design procedure is successful for one of the materials, micro-surfacing contractors using Koch emulsion could consider that producer as an aggregate source for future jobs. It would be likely that other emulsion suppliers would develop mix designs for the selected aggregate sources, thus providing competition for emulsion suppliers.

Micro-Surface Program/Investigation Status

Shortly after identifying candidate sources, the Iowa DOT suspended its micro-surfacing program. Koch Materials determined that the expense of developing the mix designs could not be justified if Iowa DOT was not going to continue their micro-surfacing program. Therefore, this part of the investigation was ended. In the summer of 2000 Pocahontas County micro-surfaced an 11 mile stretch of County Road N-28 (between Laurens and Fonda). Koch Materials developed mix designs using limestone aggregate from Martin Marietta's Fort Dodge mine. Now that the mix design has been developed, it can be applied to future projects.

Note: It is possible that results from Iowa Test Method 222 could also be used to predict aggregate compatibility with cationic emulsion instead of Iowa Test Method 630-B. An upward adjustment in the allowable alumina content may be appropriate for seal coating.

CHAPTER 6. GUIDELINES FOR WINTER MAINTENANCE ON THIN MAINTENANCE SURFACES

Introduction

Thin maintenance surfaces are an important part of a preventive maintenance program that allows the life of a pavement to be extended considerably, at relatively low cost. However, in considering application and use of TMS in Iowa, consideration must be given to their performance in winter conditions. In this regard, there is anecdotal evidence (Jahren et al. 1999) that TMS may be susceptible to plow damage during winter maintenance activities. This susceptibility appears to be greater for TMS pavements than for other pavements.

There is also concern that the open graded nature of TMS may be a complicating factor in winter maintenance activities. The open structure of the pavement surface may allow the creation of a stronger bond between ice and pavement and thus make snow and ice removal operations more difficult. The greater effort needed to remove the snow and ice mechanically may then result in added mechanical damage to the treatment.

There is also some indication that such open surface structures may allow chemicals to concentrate unduly on the pavement and give rise to a condition termed “chemical slipperiness” (SICOP 1999). Clearly, there are a number of unresolved issues with regard to winter maintenance activities on pavements with TMS. The aim of this chapter is to address these issues.

This chapter presents results of a literature review on the effects of winter maintenance on TMS performance. Since not much literature was found pertinent to this issue, the winter maintenance community was asked for input directly (details of this are given below). The results of this input are given below and provide some useful direct anecdotal information. Four sites in eastern Iowa were visited and evaluated. The sites and the findings of the site visits are given in the section titled “Observations from the Field.” Finally some preliminary guidelines for using TMS in regions where winter maintenance is required are presented in the section titled “Preliminary Guidelines for Practice.” Conclusions are given near the end of the chapter.

Literature Review

There is very little formal literature available that relates to winter maintenance on asphalt treatments. Because of this, the literature search was widened to address other related topics. For example, the NCHRP Synthesis 260 on “Thin-Surfaced Pavements” (Geoffroy 1998) was reviewed, but made no mention at all of winter maintenance issues. To a degree, this scarcity of literature was somewhat expected. Much of the data relating to winter maintenance issues tends to be anecdotal (and is reviewed in the section “Input from Community” below).

Nonetheless, one interesting paper was found (Noort 1996) relating to winter maintenance on porous, or open-graded, asphalt. The first phenomenon noted by Noort about open-graded asphalt was that it cooled more rapidly than "regular" asphalt. Further, the open-graded asphalt stayed below freezing longer also. Noort also reported specifically on the behavior of open-graded asphalt in three specific conditions: when a wet road starts to freeze, when freezing occurs because of fog or mist, and when frozen rain falls on the road.

In the case of a wet road freezing, the open nature of open-graded asphalt means that there is a lot of moisture both on and "in" the road surface. Thus, the amount of salt applied to the road must be increased in comparison with "regular" asphalt or dilution and refreeze of the moisture will occur.

There appears to be some benefit to porous asphalt in situations where small amounts of freezing precipitation occur. In such cases, the residual or "buffer" salt that remains in the asphalt pores after a winter storm can actually keep the road from freezing. This "buffer" salt is drawn to the surface by traffic and causes melting of the ice or frost there, thus maintaining good traveling conditions.

Freezing rain appears to be a very serious condition for open-graded asphalt. Under such conditions, the roads become much more slippery than "regular" asphalt and are much harder to return to a safe condition, perhaps because they drain so well, and thus any chemicals are rapidly flushed.

Noort's paper (1996) suggests not only specific problems with open-graded asphalt, but also that pavement surface type can have significant effects on the ease with which a road can be maintained (kept clear of snow and ice) in wintertime.

Another reference refers to this phenomenon, but does not provide much information. Ichihara, Sakagami, and Tanifuji (1977) report that in Japan, gap graded dense asphalt concrete is used in snowy areas, because it has comparatively high skid resistance. However, no information is provided as to the ease with which this pavement can be plowed, or whether snow adheres to it more or less readily than to "regular" asphalt.

Guiliani (2002) notes again that open-graded asphalt pavements appear to require significantly more salt to keep them free of snow and ice during winter road maintenance, in comparison with regularly graded asphalt. According to Guiliani, the problem is so severe that heated road elements become economically feasible for roads with this sort of pavement (although the economic arguments may not be valid in US situations).

No reports could be found of de-icing chemicals having any adverse effects on asphalt pavements, although there are studies that suggest certain aggregates may be susceptible to damage from certain de-icing chemicals. For example, dolomitic limestone is susceptible to damage from calcium magnesium acetate (Cody et al. 1997).

However, aside from these very scarce reports, the majority of the information about winter maintenance aspects of thin maintenance treatments is anecdotal. This is discussed further in the next section.

Input from Community

Given the absence of published information on winter maintenance effects on thin maintenance treatments, the collection of anecdotal information became even more importance. To obtain such information, the Snow and Ice List-Serve (snow-ice@uiowa.edu), which has more than 600 subscribers, was used to collect data. The following message was sent to the list:

> Greetings:
>
> I'm looking for any information available on effects of thin maintenance treatments on winter maintenance. To set this in context, I'm interested in three particular types of thin maintenance treatments: micro-surfacing, seal coats, and slurry seals. Three issues in particular are of interest.
>
> We have anecdotal information that micro-surfacing may produce an extremely hard surface that wears down cutting edges very quickly. We'd be interested in hearing any information related to this phenomenon.
>
> The second issue concerns the open nature of the pavement surface created by these treatments. There is concern that this open surface will allow snow and ice to bond more effectively to the pavement. Again, information on this would be very welcome.
>
> The third issue is whether the seal coat and slurry seal treatments are especially susceptible to damage from plows.
>
> In all three cases described above, I'd be delighted with published reports or papers, but I'm expecting that such information will be primarily anecdotal. If you have such information, I'd welcome the chance to discuss it with you. If you know someone with such information, it would be marvelous if you could let me know who they are, and I'll contact them.

Appendix B gives the responses to this query. In total, 13 relevant responses were received. A number of responses (not included in the appendix) indicated whom to contact for additional information. The results of those further contacts are included (as some of the 13 messages) in Appendix B. It should be noted that these samples do not in any way constitute a statistically significant sample; nor were they intended to do so. They simply provide an insight into some of the experiences (both positive and negative) of winter maintenance on pavements with thin treatments.

Of the 13 responses two (numbers 2 and 4 in Appendix B) provided no useful information for the project. Three responses (numbers 8, 9, and 13) indicated no problems of any sort with winter maintenance of thin maintenance treatments. Of these three, response number 8 had only used slurry seal coats in winter maintenance regions. Response number 9 provided experience with slurry seals, chip seals, and micro-surfacing. While problems were noticed with slurry seals, these were probably not winter maintenance related, although part of the de-bonding problems observed may be due to plowing activity. Chip seals and micro-surfacing did not exhibit any problems. Response number 13 provided information about the use of slurry seals. This respondent reported no wear problems on cutting edges. The only minor issue raised concerned the darker treatment melting snow and ice more quickly than the rest of the highway, which is probably an asset rather than a drawback.

Five respondents (numbers 1, 3, 5, 7, and 11) noted problems with chip seal material being removed during normal plowing operations. Respondent number 3 gives significant details of one storm that may have contributed significantly to this material loss. In this case, rain fell first, followed by a lengthy, steady snowfall. During the storm, all resources were devoted to keeping emergency routes clear, so that by the time residential streets were plowed the snow had been hard-packed by traffic and required clearing with motor graders, which may have loosened the material. Respondent number 5 indicates that both chip seals and slurry seals have performed poorly from a material retention perspective. Respondent number 7 indicated that if a chip seal is peeled off by plowing, then it is due to problems with the application of the seal coat itself. Three specific problems were noted in this regard: dirty stone, bad emulsion, and damp pavement. Of the three, the respondent indicated damp pavement was the most likely cause, and their specifications now require careful monitoring of humidity and temperature during seal application. Respondent number 11 notes that maintenance workers treat their chip seal sections with extra care to avoid removal of material.

Five respondents (numbers 1, 6, 10, 11, and 12) reported that thin maintenance treatments wore plow blades more quickly than other surfaces. Of these respondents number 1 and number 10 indicated the wear problem occurred with chip seal, while the other three respondents (6, 11, and 12) indicated that the micro-surface treatment caused significant wear problems.

Two respondents (numbers 1 and 6) indicated problems of ice sticking more readily to overlaid surfaces than regular surfaces. Respondent number 1 indicated that snow and ice bound much harder to a chip seal treatment than to other pavement types, but suggested that by using a less coarse chip seal, this problem could be alleviated. This would be consistent with some of the problems (see literature review) noted with open graded asphalt pavements and ice adhesion. Respondent number 6 noted that a micro-surface treatment required more salt than regular pavement surfaces, presumably (although this was not stated) because the snow and ice bound more to the pavement.

The findings of this informal survey can be summarized as follows. Chip seal coatings can be prone to excessive material loss under plowing conditions, but this appears to be related to the conditions under which the treatment was placed. A well-placed chip seal will not exhibit material loss. It appears that micro-surface treatments may well result in more rapid snowplow blade wear than other pavement surfaces, which should not surprise given the nature of such treatments. Open graded chip seals may create a situation in which snow and ice bond more effectively to the pavement. Few problems relating to slurry seals were noted, and those that were seem related more to issues other than winter maintenance.

Observations from the Field

Four different sites in Iowa were visited (see Table 38). The sites gave examples of three different thin maintenance surface treatments: micro-surfacing, slurry seal, and seal coat (or chip seal). Each site is described below.

Table 38. Thin Maintenance Surface Observation Sites

Roadway	Location	AADT	Treatment	When Treated
Highway 70	From West Liberty to Highway 22	2,700	Micro-surfaced	1999
Highway 927	From Y-40 to I-280	4,360	Micro-surfaced	1999
Highway 131	From Belle Plaine to Route 30	1,060	Seal coat	1999 (rehabilitated in 2001)
Highway 965	From railroad tracks in North Liberty to Mile Post 103	3,340	Slurry seal	1999

Highway 70

Figure 30 shows the section of Highway 70 that was micro-surfaced in 1999. The road runs north-south from West Liberty to Highway 22. The site was visited and photographed on May 5, 2000. Figure 31 shows a typical view of the road surface. From this, it is apparent that some sort of vibratory scraping has occurred on the road surface, possibly due to a plow blade scraping the surface. This is shown more clearly in Figure 32. Figure 33 shows in close up a region where such scraping has occurred. Again, the scraping is consistent with being formed by a scraping plow blade during winter maintenance operations.

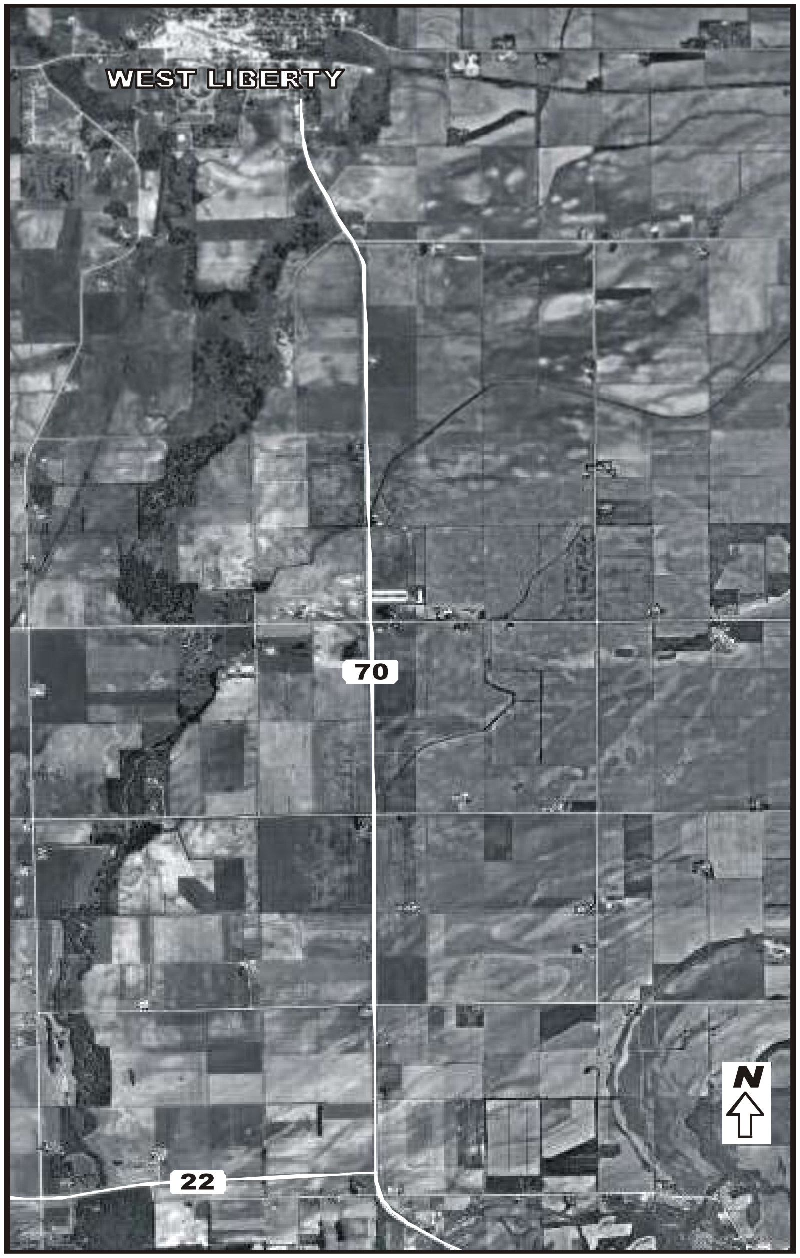


Figure 30. Aerial View of Micro-Surfaced Section of Highway 70

Figure 31. Typical View of Roadway Surface

Figure 32. View of Vibratory Scraping that Has Already Occurred on the Roadway

Figure 33. Close-up View of Vibratory Scraping on the Roadway

Highway 927

The resurfaced section of Highway 927 visited in this study runs east-west between Walcott and I-280, as shown in Figure 34. This site was also visited and photographed on May 5, 2000. Figure 35 shows the same sort of vibratory scraping marks that were seen on Highway 70, although these are perhaps more pronounced. Figure 36 shows a close-up view of the road surface, and there is some evidence of scraping wear on the surface (especially toward the top of the photograph). This scraping would be consistent with having been caused by a plow during winter maintenance operations.

Figure 34. Aerial View of Resurfaced Section of Highway 927

Figure 35. View of Vibratory Scraping on Highway 927

Figure 36. Close-up View of Some Sort of Scraping Wear on the Roadway Surface

Highway 965

The section of Highway 965 that had been treated with Slurry Seal runs approximately northwest-southeast, as shown in Figure 37. This site was visited and photographed on May 31, 2000. Figure 38 shows the typical condition of the road surface. As can be seen in Figure 39 the road surface is not in particularly good condition. Cracks have come through the treatment, and in places, it appears to have come away from the pavement completely. Although it cannot be clearly stated that this damage was due to scraping by a plow, Figure 40 suggests some scraping damage in particular close to the fog line. However, such scraping damage is minor in comparison with the otherwise poor shape of the treatment.

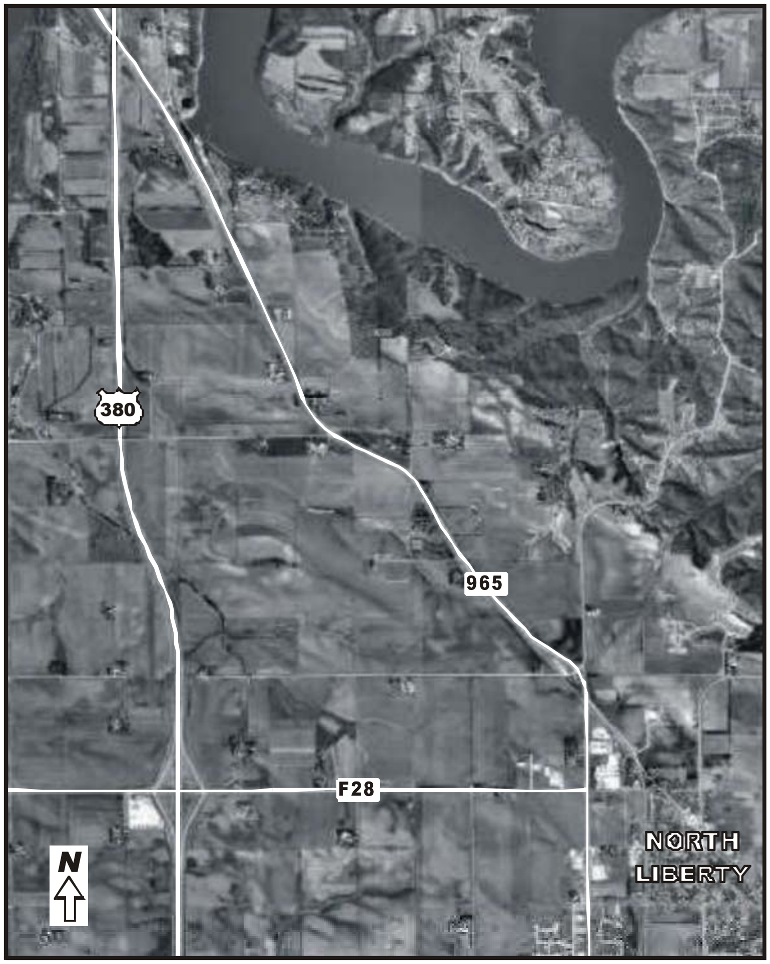


Figure 37. Aerial View of Section of Highway 965 Treated with Slurry Seal

Figure 38. Typical View of Roadway Surface

Figure 39. Closer View of Roadway Surface Showing It Is Not in Good Condition

Figure 40. Close-up View of Some Scraping of the Roadway Surface and Overall Poor Condition of the Roadway

Highway 131

Figure 41 shows the section of Highway 131 that had been treated with seal coat, which runs south from US 30, before turning east into Belle Plaine. This site was also visited and photographed on May 31, 2000. A typical section of the road is shown in Figure 42. Significant cracking is apparent, especially close to the fog line and into the shoulder area. Figure 43 shows the cracking in this edge region more closely. Again, it does not appear that this cracking is due to scraping by a plow, although there are what appear to be scrape marks present. However, such scraping damage is minor in comparison with the damage due to cracking.

Figure 41. Aerial View of Section of Highway 131 Treated with Seal Coat

Figure 42. Typical View of Roadway Surface

Figure 43. Close-up View of Cracking Along Edge

Preliminary Guidelines for Practice

On the basis of the literature survey, and informal responses received from the snow and ice community, and field observations, some preliminary recommendations can be made concerning how thin maintenance treatments perform with respect to winter maintenance activities. It should be noted that these guidelines relate only to the effects of winter maintenance activities on thin maintenance treatments. They do not include other performance factors for these treatments, and in any design choice for a treatment, such factors would of course be critical and would have to be considered.

The guidelines are presented in terms of the three types of thin maintenance treatments considered in this study: micro-surfacing, slurry seals, and seal coats.

Micro-Surfacing

Micro-surfacing treatments tend to be much harder than conventional pavement surfaces. Thus it is recommended that care be taken when plowing such surface treatments to ensure that cutting edges on plows do not get worn down too quickly. Particular care should be taken when underbody plows are used as this type of plow can exhibit considerable down-force and thus cause very rapid blade wear as a result.

Slurry Seals

Very few problems have been reported with winter maintenance activities on slurry seal treatments. However, there is the possibility that such treatments might be easily damaged, and in particular, they might de-bond, as a result of plowing operations. Care should therefore be taken when plowing such roads not to use excessive down-force. Further such roads should be carefully observed after plowing to monitor any possible de-bonding of the treatment.

Seal Coats

Seal coats appear to be vulnerable in several ways to winter maintenance. First, if not applied properly (and in particular if humidity is too high when applied), they are subject to being significantly degraded when plowed, to the extent that regular plowing essentially removes the treatment. Application of such treatments should not be done under damp or very humid conditions. Second, if an open-graded treatment is used, snow and ice may adhere more firmly to the road surface and require greater levels of chemical application for removal. Such treatments should be carefully monitored for such ice retaining behavior, and if it is present, chemical application rates should be increased for those stretches of highway affected. This behavior can be avoided by using a less coarse gradation in the treatment. Third, well-applied chip seal treatments may cause high levels of cutting edge wear. Thus care should be taken when plowing not to apply excessive levels of down-force.

Conclusions

This study has collected information on the performance of thin maintenance surfaces under winter maintenance conditions. The study included a review of the literature, which is very sparse in this area, the collection of anecdotal information from maintainers about the performance of such treatments, and site visits to four locations in eastern Iowa where such treatments have been used. On the basis of the information gathered in the study, some simple recommendations have been made on the use of three types of treatments in conditions where winter maintenance is regularly conducted.

CHAPTER 7. GUIDELINES FOR USE OF THIN MAINTENANCE SURFACES

Phase One Interim (Qualitative) Guidelines

The guidelines for thin maintenance surfaces proposed in the phase one project were qualitative (see Appendix A). An example of such a qualitative guideline is as follows: Slurry seal and micro-surfacing are not recommended for badly cracked pavements; however, those treatments can be used to address a small amount of light cracking. The judgment may vary between decision makers about what is a “badly cracked pavement” and what constitutes “a small amount of light cracking.” Since no quantitative standards exist, part of this project was to develop a framework for guidelines that are more quantitative. The framework is based on the surface condition index (pavement condition index) as described by Shahin (1994) and the principal investigator’s experience accumulated while executing both phases of this research project. The result is a set of guidelines that could be improved with further research, but the guidelines are more quantitative than the ones developed in Phase One.

Phase Two Refined (Quantitative) Guidelines

The allowable quantity of each type of distress was selected by considering an appropriate SCI value for given treatments, traffic levels, and distresses. After the SCI level was selected, a permissible amount of distress was back calculated. Three levels of traffic were considered:

- **5,000 AADT.** This traffic level was considered because it is typical of a high volume, two-lane, rural primary highway that may be a candidate for conversion into a four-lane highway.
- **2,000 AADT.** This traffic level was considered because it represents a transition from a high volume primary rural highway to a low volume primary rural highway. Traditionally, Iowa DOT has had different maintenance practices for highways above and below this traffic level.
- **200 AADT.** This traffic level was considered because it represents a transition between rural roads that are usually paved to ones that are usually graveled.

The guidelines were developed with the expectation that users will use their judgment and interpolate or extrapolate to investigate treatment selection for a particular traffic counts. In general, treatments that are the most appropriate for particular types of distress will be recommended at lower SCI values than treatments that are less appropriate.

The guideline for cracks serves as an example. First, notice that the recommended SCI values for routine maintenance range from 60 to 95, for preventive maintenance range from 50 to 75, for rehabilitation range from 25 to 60 and for rebuilding range from 0 to 60 (Table 39). It is expected that a TMS will be used for preventive maintenance, so the expectation is that the SCI value will range from 50 to 75 at the time of treatment.

Table 39. SCI Values for Maintenance Activity Types

Maintenance Activity	SCI Value	Deduct Value
Routine	60–95	5–40
Preventive	50–75	25–50
Rehabilitation	25–60	40–75
Rebuilding	0–40	60–100

Table 40 was developed for four surface treatments (micro-surfacing, 1/4-inch seal coat, 1/2-inch seal coat, and double seal coat) and various crack lengths on a 24-foot-wide by 100-foot-long section of roadway. Crack lengths ranged from 300 to 1,500 feet in increments of 150 feet, except for a final 300-foot increment. SCI and deduct values were calculated as described by Shahin (1994), with the assumption that light L&T cracking was the only distress present. Note that Shahin's method does not provide SCI calculations for L&T crack lengths that exceed 720 feet (30 percent distress). It may be that distress densities that exceed this amount are considered block cracking or some other type of distress in this method; no further explanation was found.

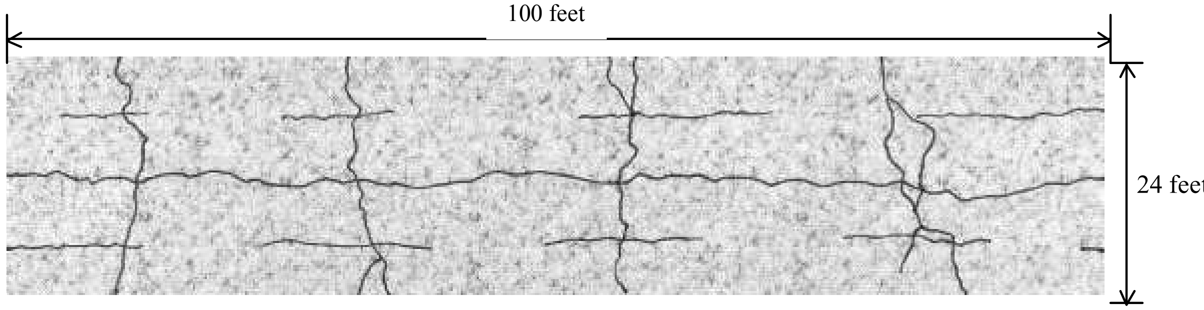
Table 40. Thin Maintenance Surface Guidelines Based on Amount of Cracking and Annual Average Daily Traffic

Feet of Cracking*	300	450	600	750	900	1,050	1,200	1,500
SCI basis**	80	78	73	71	***	***	***	***
Deduct basis**	20	22	27	29	***	***	***	***
	AADT							
Micro/slurry	5,000		2,000		200			
Seal coat (1/4 inches)		5,000		2,000		200		
Seal coat (1/2 inches)			5,000		2,000		200	
Double seal coat				5,000		2,000		200

Note: Based on 100 feet of road 24 feet wide.
* Medium intensity cracks require joint sealing or slurry strip repair before surface treatment is placed. Likely long-term result is two closely spaced light intensity cracks. Therefore, consider 1 foot of medium intensity crack equal to 2 feet of light intensity crack. High intensity cracks require patching before treatment is placed. The likely long-term result is two closely spaced light intensity cracks. Therefore, consider 1 foot of high intensity crack equal to 2 feet of light intensity crack. Utility cuts and patches are considered low intensity cracks around the perimeter of the repairs.
** Based on light L&T cracking.
*** SCI basis and deduct are not given for more than 750 feet of light L&T crack.

For the purposes of these guidelines all cracks (except alligator cracks) are converted into an equivalent length of light cracking. Medium and heavy intensity cracks are considered to be equivalent to light density cracks at twice the length of the original crack. It is assumed that both types of cracks will be repaired before the treatment is placed: medium intensity cracks with joint sealer or slurry strip and high intensity cracks with patches. The likely result in both cases is two light intensity parallel cracks, one on each side of the repair. The perimeter of any patches or utility cuts is also considered to be the genesis of a light intensity crack.

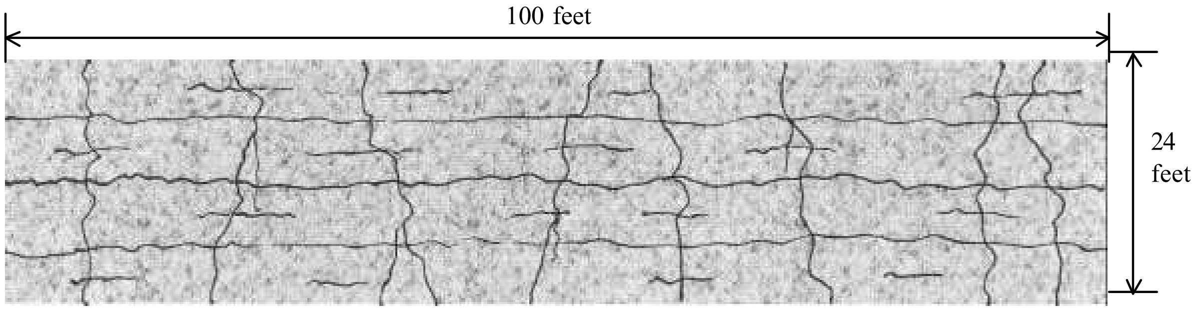
The possible use of slurry seal or micro-surfacing was considered to establish a lower bound on the amount of cracking distress that would be addressed by thin maintenance surfaces. Since these techniques do not address cracking as well as other techniques, the required SCI is set somewhat above the usual preventive range at 80 (preventive range is 50–75) for high volume primary roads (AADT = 5,000). If light L&T cracking is the only distress, the maximum allowable percent of distress is 12.5 percent for a deduct value of 20. For a 100-foot section of road 24 feet wide (2,400 ft^2), the maximum allowable feet of length of cracking is 12.5 percent of 2,400 ft^2, or 300 feet. A road with four transverse joints in 100 feet, a completely cracked longitudinal joint at the centerline of road, and a partial (50 percent) crack in each mid-lane would yield slightly less than 300 feet of crack (Figure 44). In the principal investigator's experience, this represents a reasonable amount of cracking to be addressed by micro-surfacing on a high volume road.



1 longitudinal joint	100 feet
50% crack at 2 mid-lanes	100 feet
4 transverse joints	96 feet
Miscellaneous	4 feet
Total	300 feet

Figure 44. 2,400 ft^2 Section of Roadway with about 300 Feet of Cracking

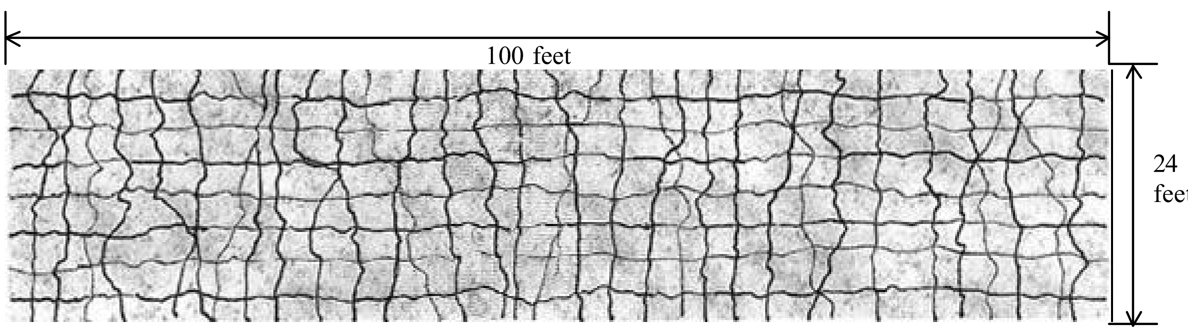
Table 40 indicates that if length of crack doubles, micro-surfacing would only be recommended if traffic is 2,000 or less AADT. This calculates to a SCI value of 73, which is inside the preventive range. Six hundred feet of crack could occur in a 100-foot section of 24-foot-wide road, if there are eight transverse cracks, the centerline and both mid-lanes were cracked and 25 percent of the wheel paths is cracked (see Figure 45). Although the start of wheel path cracks may suggest incipient fatigue failure, at 2,000 AADT, it is possible that the pavement may retain sufficient structural strength to last the life of the maintenance treatment—about seven years. Note that caution should be used when applying TMS to pavements that may be suffering fatigue failure, because TMS will do little to mitigate this failure. Note that for 600 feet of light intensity cracks on a higher volume road (5,000 AADT), 1/2-inch seal coat would be suggested, if the agency had a policy of seal coating such high volume roads.



1 longitudinal joint	100 feet
2 mid-lane	200 feet
25% of 4 wheel paths	100 feet
8 foot × 24 foot transverse	192 feet
Miscellaneous	8 feet
Total	600 feet

Figure 45. 2,400 ft^2 Section of Roadway with about 600 Feet of Cracking

To establish an upper bound, for the amount of cracking distress that could be addressed with TMS, a 3 foot by 3 foot crack pattern similar to block cracking was considered (Figure 46) and a double seal coat was selected as a satisfactory treatment for roads with 200 or less AADT. This was selected on the basis of anecdotal evidence that the first author collected where a road with a similar crack pattern was successfully treated in this way. Note that the cracks could not be cracks that "work" under load and that the road may not meet the usual standards for ride and appearance. However, the treatment might successfully preserve a road with such light traffic.



3 foot × 3 foot crack pattern (similar to block cracking):

7 longitudinal		700 feet
1 centerline	100 feet	
2 mid-lane	200 feet	
4 wheel paths	400 feet	
33 foot × 24 foot transverse		~800 feet
Total		1,500 feet

Figure 46. 2,400 ft^2 Section of Roadway with about 1,500 Feet of Cracking

Guidelines were also developed to address alligator cracking with TMS. Alligator cracking usually indicates that the pavement is experiencing a fatigue failure. Again, since TMS does very little to address fatigue problems, the strong possibility exists that the pavement will experience continued structural failure and an investment in preventive maintenance would be wasted. However, a TMS may reduce the amount of moisture entering the base and subgrade through the pavement, thus stiffening the subgrade and reducing pavement stress, which would provide modest benefit. Also, the principal investigator has anecdotal evidence that low volume roads, especially urban residential streets can also be candidates for thin maintenance surfaces, if they have light alligator cracking due to small deflection fatigue (the pavement may fail in fatigue after it has lost flexibility with age and has experienced many small fatigue cycles). For low volume road, the thin maintenance surface may be sufficient to "glue" the alligator blocks in place and reduce crack width so as to prevent spalling for a time.

Table 41 was developed to provide a guideline for using TMS for addressing alligator cracking distress. Thin maintenance surfaces are not recommended for a pavement that is experiencing medium or heavy intensity alligator cracking; any such areas that exist should be patched before the TMS is applied. Table 41 indicates that zero percent distress is allowed for medium and heavy intensity cracking and for roads with traffic volumes of 5,000 AADT. The SCI requirement for micro-surfacing and 2,000 AADT was set at 75, which is the upper limit of the usual range for preventive maintenance. Thus the maximum allowable alligator cracked area would be 5 percent. This was chosen because micro-surfacing/slurry seal is not a preferred treatment for addressing cracking distress. The required SCI for 2,000 AADT and 1/4-inch seal coat, 1/2-inch seal coat, and double seal coat are 70, 65, and 60, respectively, based on the principal investigator's judgment. For each treatment, compared to the requirement for 2,000 AADT, the SCI requirement is 10 points less for 200 AADT.

Table 41. Thin Maintenance Surface Guidelines Based on Amount of Alligator Cracking and Annual Average Daily Traffic

	Micro/Slurry			**Seal Coat (1/4 inches)**		
AADT	5,000	2,000	200	5,000	2,000	200
SCI basis	*	75	65	*	70	60
Deduct basis	*	25	35	*	40	50
Light cracking**	*	5%	12%	*	8%	1%
Medium cracking	*	***	***	*	***	***
Heavy cracking	*	***	***	*	***	***
	Seal Coat (1/2 inches)			**Double Seal Coat**		
AADT	5,000	2,000	200	5,000	2,000	200
SCI basis	*	65	55	*	60	50
Deduct basis	*	35	55	*	40	50
Light cracking**	*	12%	22%	*	18%	40%
Medium cracking	*	***	***	*	***	***
Heavy cracking	*	***	***	*	***	***

Note: Based on 100 feet of road 24 feet wide.
* TMS are not recommended to address any alligator cracking on roadways with 5,000 or greater AADT.
** Applies to alligator cracking caused by fatigue due to advanced age combined with moderate deflection on firm subgrade. Do not use TMS for fatigue cause by severe deflections on soft subgrade.
*** TMS not recommended for medium or heavy alligator cracking.

Bleeding is the last type of distress for which guidelines were refined (Table 42). Separate guidelines were developed for slurry seal and micro-surfacing. The minimum SCI requirement for 5,000 AADT and micro-surfacing was set at 80, while for the same traffic and seal coat, the SCI was set at 60. As traffic decreases, 10-point increments are allowed between each category. The SCI requirement was set high for micro-surfacing and slurry seal because it is difficult to change the mix design to use less binder to compensate for bleeding from the substrate. For seal coat, a SCI requirement of 60 was selected because the amount of binder can be adjusted downward to compensate for bleeding. The SCI of 60 is near the middle of the preventive maintenance range (Table 39). If seal coat is used, the chances of success can be increased by using one-size aggregate that will allow excess void space to accommodate additional oil from the bleeding surface. Compared to smaller sized aggregate, larger sized aggregates will provide more void space for excess oil.

Table 42. Thin Maintenance Surface Guidelines Based on Amount of Bleeding and Annual Average Daily Traffic

	Micro/Slurry			Seal Coat*		
AADT	5,000	2,000	200	5,000	2,000	200
SCI basis	80	70	60	60	50	40
Deduct basis	20	30	40	40	50	60
Light bleeding	100%	100%	100%	100%	100%	100%
Medium bleeding	23%	55%	100%	100%**	100%**	100%**
Heavy bleeding	8%	15%	25%	25%**	40%**	60%**

Note: Based on 100 feet of road 24 feet wide.

* Consider using clean, one-size cover aggregate to provide more void space for excess oil and reducing binder application rate (especially for medium to heavy bleeding).

** Consider using 1/2-inch cover aggregate (more void space for excess oil).

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The conclusions for this project were drawn from four sets of test sections placed over three years, as well as the literature review, and anecdotal evidence from conversations with government and industry employees and observations by the authors. Statistical analysis was not undertaken to interpret test section results or to compare results between test sections. Each set of test sections stands on its own as a separate case study.

When thin maintenance surfaces are properly selected and applied, they can improve the surface condition index and the skid resistance of pavements. Note that for success to occur, several requirements must be met, including proper material selection, design, application rate, workmanship, and material compatibility, as well as favorable weather during application and curing. Conversely, deficiencies in any of the previously listed items may result in degradation of the surface condition index or skid resistance. Therefore, good decisions and careful quality control are necessary from initial concept to acceptance of the completed project. Many references in the literature claim that thin maintenance surfaces can be an important part of a cost effective preventive maintenance program that can improve the overall condition of a road network at a low cost.

Other strategies aside from the use of thin maintenance surfaces could also be considered such as thin lift hot mix overlays, fog sealing, and crack sealing and crack filling. Generally, the study of such treatments was outside the scope of this project. However, the thin hot mix treatments that were applied as part of this project performed well. Although their initial costs were higher than thin surface treatments, those costs could be recovered if the benefits of these treatments outlast the benefits of thin maintenance surfaces for a sufficient amount of time. The period of observation for this study has not been long enough to develop a conclusion on that point.

It can be concluded that more effort with respect to concept selection, material selection and construction quality is necessary when thin maintenance surfaces are used on high volume roads and roads where the pavement condition is relatively good. In either case the risks of poor result is greater. On higher volume roads, more road users are exposed to problems during and after construction. For roads with good pavement condition, smaller problems with the thin maintenance surface are more likely to lead to degradation in pavement condition as a result of thin maintenance surface application.

It can also be concluded that thin maintenance surfaces can improve the surface condition index and the skid resistance of roads that by usual standards would be judged to have passed beneath the surface condition index range where such treatments are usually recommended. This is especially true if the main distress is rutting (not due to pavement instability), bleeding, or raveling. Micro-surfacing can be used to address rutting. High quality slurry seal mixes can address rutting less than one inch deep on lower volume roads. Micro-surfacing, slurry seal, or seal coating can address many problems with bleeding or raveling. However, in some cases, the treatment may not have the life that is normally expected.

Designed seal coats appear to be effective. Test sections that had seal coats that were designed using Minnesota DOT's (Janisch and Gaillard 1998) method performed well and used two-thirds to three-fourths of the materials that are normally used under current (2002) Iowa specifications. Since much of the cost of seal coat construction is purchasing materials, it can be expected that much of the savings will be passed on to the contracting authorities.

Graded cover aggregate for seal coats has performed well producing a tight, quiet surface. Such tight surfaces seem to be beneficial for reducing snowplow damage. Although some problems with bleeding were noted, this issue can be mitigated with proper design and application. It is likely that the use of polymer modified binder will also be helpful. The polymer makes the binder stiffer at high temperatures, therefore less likely to flow and bleed. The Minnesota seal coat design method was used to design these graded seal coats. The previous conclusion runs counter to advice that is typically provided in the literature, which contends that one-size aggregate bonds better to the road surface because there is more void space for binder and because it is easier to spread one-stone thick, thus promoting direct adhesion to the pavement. Also there is more room for error in application rates because the extra void space provides additional capacity for extra binder in case the binder application rate is too high. Apparently, application rates can be sufficiently well controlled to prevent bleeding problems and the various size pieces of aggregate can be bound well enough to prevent aggregate loss problems.

Smaller sizes of seal coat aggregate perform well in the short term according to test section results. They provide a tighter surface texture and require less weight of aggregate per square area to provide adequate coverage, thus reducing material cost. Also, less binder is required to bind the aggregate to the surface. Generally the literature suggests that seal coats constructed with smaller cover aggregate sizes will wear more quickly than larger sizes, especially under heavier traffic. The test sections have not been observed long enough to confirm or dispute this assertion. Also, the literature asserts that there is less room for error in the binder application rate for smaller size aggregates because the design amount is usually lower, yet the ability to control the application rate stays fixed in terms of volume per square area. This is undoubtedly true. However, the test sections show that it is possible to control application rates with sufficient accuracy to bind the aggregate without bleeding. As with graded cover aggregate, polymer modified binder may be more forgiving, if the application rate is off slightly. If too much binder is applied, it is less likely to flow and bleed in hot weather. Since it likely retains aggregate better, it might retain aggregate better, even when too little is applied.

HFRS is an alternative binder that can be used in attempt to improve compatibility between the aggregate and the binder. In this study it performed comparably or better than CRS-2P.

Treatments that use quartzite aggregate provided the best skid resistance of those used in this project. Included was quartzite-manufactured sand in the thin hot sand mix, which had very high skid resistance despite the small aggregate size and smooth, tight surface. However, except for the hot sand mix, quartzite surfaces also appear to have the greatest vulnerability to snowplow damage. There are a number of possible reasons for vulnerability to snowplow damage:

- Because the aggregate is hydrophilic, the aggregate may not remain well adhered to the binder (stripping)
- If the aggregate gradation lacks fine particles, the larger particles may stand up taller compared to the rest of the surface so they are more vulnerable to being hit by the snowplow blade.
- If the aggregate gradation lacks fine particles, aggregate may lack stability and be easily removed when hit from the side by snowplows.
- If the aggregate particles are hard, they may be plucked out of the surface rather than being sheared off, when hit by a snowplow blade.

Gradual loss of quartzite manufactured sand aggregate on hot sand mix may provide a renewed surface with considerable micro texture, which may account for its good performance with regard to skid resistance. Since the aggregate particles are small, their loss is not particularly problematic.

Snow removal operations must be performed with care on thin maintenance surfaces to limit damage to the roads and the snowplow blades. Down-pressure should be limited. Anecdotal evidence suggests that open surfaces that are typical of some thin maintenance surfaces retain more snow and ice when compared to tight surfaces.

Recommendations

Several recommendations follow from the findings and conclusions of this study. They include possible changes in policy, development of new specifications, use of materials developed under this project, and targeted areas for additional research.

If the use of thin maintenance surfaces is to reach its full potential, those involved with concept selection, specification selection, construction and inspection must strive to improve quality. This is especially true when treatments are used for preventive maintenance on pavements that are in good condition. For such pavements, even seemingly small lapses in quality may degrade the surface condition index and road user experiences. Particular attention should be paid to material selection including aggregate gradation and binder compatibility. Also workmanship should be monitored to ensure that materials are applied evenly and at the proper rate. Work should be performed in favorable weather and early in the season to ensure good curing.

Seal coats should be designed using method described in the *Minnesota Seal Coat Design Handbook* (Janisch and Gaillard 1998), which is based on McLeod's method. The use of Iowa's current aggregate gradations should be continued; however, further investigation would be desirable to identify possible uses of one-size aggregates. The use of smaller aggregate sizes should be considered to limit material use and provide tighter road surfaces. Double seal coats should be carefully designed and constructed to avoid the possibility of bleeding. Use of current Iowa DOT specifications for double seal coats should be discontinued, because the risk of bleeding is too great as shown by the 1997 (US 151 and US 30) test sections. Additional research to identify a more quantitative, yet practical design method for double seal coats may be desirable. Also, consideration should be given to developing a strict protocol for applying seal coats to higher volume roads.

Consideration should be given to developing a protocol to utilize the x-ray fluorescence test to predict the compatibility of limestone with reactive binders such as CRS-2P (seal coat) and CSS-1H (micro-surfacing). The clay content of the aggregate can be inferred from the x-ray fluorescence test and the presence of clay makes the breaking time of reactive binders hard to control. Such testing might be used in addition to Iowa Test Method 222 (Aggregate Emulsion Compatibility).

Micro-surfacing should be improved in several ways:

- Consider the use of Type 2 (1/4-inch top size) rather than Type 3 (3/8-in top size) aggregate to reduce vehicle noise, snow retention and snowplow damage.
- Tighten the gradation specification to ensure that the mix will be well graded.
- Consider using local limestone in order to reduce transportation expense and possibly reduce aggregate loss due to stripping and snowplow damage. (In July of 2000 an 11 mile stretch of Pocahontas County Road N28 [between Laurens and Fonda] was successfully micro-surfaced with the use of limestone from the Martin-Marietta pit in Fort Dodge, Iowa.)
- Conduct further research to investigate whether or not stripping of quartzite aggregates plays a role in aggregate loss during snowplow operations. The investigation should also find ways to mitigate stripping if that is the cause of the problem (this would be helpful for quartzite seal coats also).
- Consider the use of new micro-surfacing binder specifications that are currently under development at the national level.

Consider the use of hot sand mix using manufactured sand in areas where high skid resistance and smooth surfaces are necessary. Further research may also be desirable to investigate the use of manufactured sand in hot mix asphalt to increase skid resistance. The use of Nova Chip should be considered for areas where it would be desirable to seal the existing pavement, provide a thin (3/4-inch) lift, improve skid resistance, and provide an open textured, drainable, non-glare surface. Thin lift overlays should be considered for higher volume roads with more severe defects.

Further investigation of snowplowing and deicing operations on thin maintenance surfaces would be desirable. Additional knowledge regarding the advantages and disadvantages to tight and open surfaces would be helpful in making concept selections. It would be desirable to provide additional guidance based on the finding of the research project for snow removal operators who work on thin maintenance surfaces.

Transportation officials should use the guidelines for selecting thin maintenance surfaces that were developed in this project. The interim guidelines may be used if a non-quantitative approach is desired and the refined guidelines may be used if a more quantitative approach is desired. The guidelines should be further refined as more experience is collected.

New documents should be adopted as follows:

- Materials instructional memorandum for seal coat design
- Materials instructional memorandum for aggregate spreader calibration
- Specification for high float rapid set emulsion
- Specification for 1/4-inch seal coat cover aggregate
- Specifications to accommodate the design of seal coats

The test sections constructed under this project should be periodically observed until the end of their service lives. Given sufficient interest by local jurisdictions and or the Iowa DOT additional test sections should be constructed to demonstrate maintenance treatments that have not been observed as part of this project:

- Crack sealing
- Crack filling
- Fog sealing and or pavement rejuvenators
- Limestone aggregate micro-surfacing
- Thin lift hot mix products
- Products and processes that have been recently introduced or will be introduced in the near future
- All types of maintenance treatments in an urban setting

In general, thin maintenance surfaces should be considered as some of the many tools available in a tool kit for maintaining, upgrading, and building highway and road networks. They should be used in cases where they provide economic benefit by preserving roads and where they increase road user safety and satisfaction. For successful use, they must be properly selected at the concept level and constructed with an emphasis on quality.

APPENDIX A. PHASE ONE INTERIM (QUALITATIVE) GUIDELINES FOR USE OF THIN MAINTENANCE SURFACES

The interim (qualitative) guidelines provide a five-step TMS decision procedure (see Figure 47):

- Step 1. Collect information on candidate roads (conduct performance/distress survey)
- Step 2. Identify feasible treatments (see Table 43, or Table 44 if rutting is the primary distress)
- Step 3. Consider other factors (see Table 45)
- Step 4. Consider timing (see Table 46)
- Step 5. Consider cost (see Table 47)

Step 1. Collect Information on Candidate Roads

A performance survey should be conducted to assess the amount and type of distress that the road is suffering. The survey could be a detailed distress survey to provide input for SCI calculations. If a pavement management system is in place, the SCI has been calculated and tracked for a number of years. Thus additional helpful information regarding the rate of deterioration is available. At least a visual assessment should be made and rut depths should be noted. The traffic count should also be obtained and areas that must withstand many turning and stopping movements should be noted.

Step 2. Identify Feasible Treatments

Table 43 makes recommendations for the use of seal coats, slurry seal, and micro-surfacing (Al-Hammadi 1998). It is based on the results of literature reviews, interviews with Iowa transportation officials, review of survey results, and experience with test sections. A detailed explanation of the entries of Table 43 is given after the following tables.

Table 44 provides additional guidance for selecting treatments for roads where rutting is the primary distress (Celik 1998). It should be noted that rut filling will serve as only a temporary remedy for ruts that are caused be instability of the ACC or subgrade. Information about micro-surfacing is based on that provided by the International Slurry Seal Association. Information about slurry seal represents current practices in Iowa. It should be noted that proper mix design and proper application technique are especially important when slurry seal is used to fill ruts.

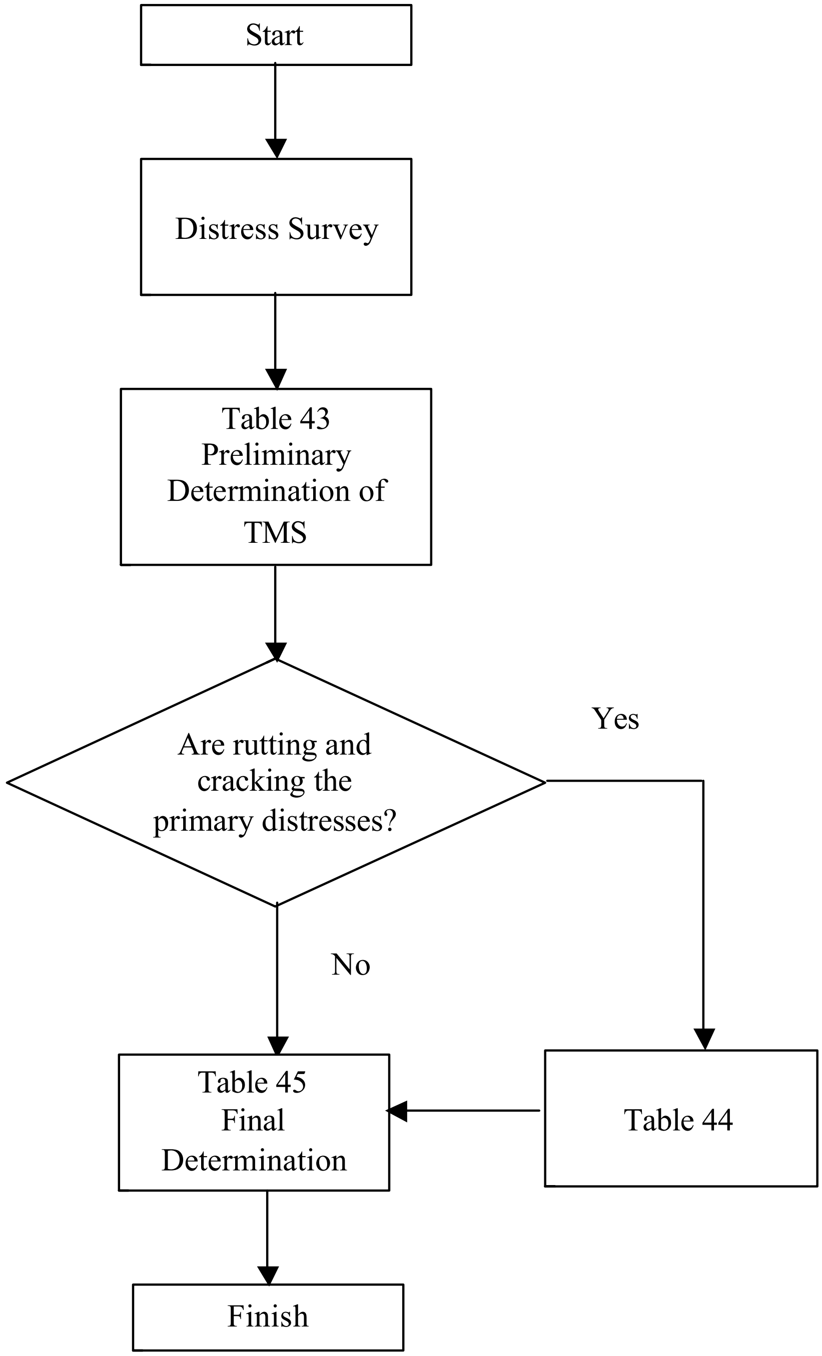


Figure 47. TMS Selection Flowchart

Table 43. Thin Maintenance Surfaces for Various Traffic Volumes and Distress Types

	Seal Coat	Slurry Seal	Micro-Surfacing
Traffic volume:			
AADT < 2,000	Recommended	Recommended	Recommended
2,000 > AADT < 5,000	Marginal*	Marginal*	Recommended
AADT > 5,000	Not Recommended	Not Recommended	Recommended
Bleeding	Recommended	Recommended	Recommended
Rutting	Not Recommended	Recommended	Recommended
Raveling	Recommended	Recommended	Recommended
Cracking:			
Few tight cracks	Recommended	Recommended	Recommended
Extensive cracking	Recommended	Not Recommended	Not Recommended
Low friction	May improve	May improve	May improve**
Snowplow damage	Most susceptible	Moderately susceptible	Least susceptible

* There is a greater likelihood of success when used in lower speed traffic.

** Micro-surfacing reportedly retains high friction for a longer period of time.

Table 44. Thin Maintenance Surfaces for Medium/High Traffic Volumes and Rutting

	Rut Depth			
	Less than 1/4 inches	1/4 to 1/2 inches	1/2 to 1 inches	Greater than 1 inch
Micro-surfacing*	One course	Scratch course and final surface	Rut box and final surface	Multiple placements with rut box
Slurry seal**	One course	One course	Micro-surfacing Scratch course and final surface	***

* As recommended by International Slurry Seal Association.

** Current practice in Iowa.

*** Sometimes successful (anecdotal evidence).

Traffic Volume

Seal coats and slurry seals are usually recommended for lower traffic volumes while micro-surfacing is usually recommended for higher traffic volumes. What constitutes low volume and what constitutes high volume is a matter of judgment and may depend on the expectations of transportation officials and highway users. Current Iowa DOT policy is to use seal coats for traffic volumes up to 2,000 vehicles per day (VPD). Researchers and transportations officials are working to improve seal coats so they can be used for higher traffic volumes by controlling the gradation and shape of aggregates, executing designs to determine application rates, and using polymer modified binders. Thus, in the near future it may be possible to extend seal coat usage in road with higher traffic volumes. Since it seems likely that the traffic volume for seal coat application will likely increase in the future, it is recommended that the cutoff be set at 2,000 VPD or higher. Therefore, it is recommended that the current 2,000 VPD cutoff be retained.

Although 1,000 to 2,000 VPD seems like low volume traffic on state highways, it is a relatively high volume for county roads. Therefore, expectations may be different for a local jurisdiction. In such cases a lower cutoff (possibly 1,000 VPD) may be more appropriate to match the expectations of road users and transportation officials.

For TMS, the break between medium volume and high volume traffic was set at 5,000 VPD. This is the same as one used by Hicks, Dunn, and Moultrop (1996) for a series of decision trees for thin maintenance surface selection. Because of expectations for durability, high friction, short construction time, and reduction of fly rock, only micro-surfacing is recommended for these roads.

Slurry seal is not recommended for high volume roads because a longer time is required before traffic can be placed on the newly constructed surface. A Strategic Highway Research Program study (Raza 1994) included slurry seal on comparative test sections located on highways throughout the United States and Canada. Fewer than half of the slurry seal sections outperformed the control sections. The authors have found considerable anecdotal evidence that slurry seal is effective on low volume roads in Iowa. Therefore, it is recommended for low volume roads.

Both seal coat and slurry seal are shown as marginal for medium volume roads (between 2,000 and 5,000 VPD). As discussed previously, it appears that it is possible to extend seal coat use for medium volume traffic when application rates are designed, premium materials are used, and quality control is carefully maintained. Researchers assigned slurry seal to the marginal category for medium volume roads because there was not enough evidence available to select a more definite dividing line. Seal coat and slurry seal are likely to be more effective on medium volume roads with low traffic speeds because such roads suffer lower impact loads.

Bleeding

All types of surface treatments are effective in addressing light to moderate bleeding or flushing. For seal coats, success is increased if the amount of binder is reduced slightly when covering areas that are bleeding. Often it is not possible to correct heavy bleeding with surface treatments because the excess binder seeps through the surface treatment.

Rutting

Micro-surfacing is the most effective surface treatment for correcting rutting problems. The heavily polymer modified binder is stiff enough to maintain stability, even when layers as thick as one inch are placed to fill ruts. Deep ruts require multiple passes and special equipment as shown in Table 44. Slurry seal can certainly be used to fill ruts up to 1/2-inch deep on low volume roads. Anecdotal evidence suggests that properly formulated and applied slurry seal can fill deeper ruts on higher volume roads. Compaction of the slurry material may cause the ruts to partially return in time. Seal coat applications follow the profile of the original road; therefore, chip seals cannot address rutting.

Raveling

Raveling is a surface defect; therefore thin surface treatments are extremely effective in correcting this problem. Surface treatments are especially effective in correcting raveling due to end load segregation.

Cracking

Working cracks reflect through slurry seal and micro-surfacing quickly because both mixes are relatively stiff and brittle when compared to hot mix or chip seal. However both treatments reduce the width of the cracks. Since micro-surfacing is stiff and tough, the cracks on treated pavements widen more slowly than those treated with slurry seal. Seal coats are more flexible when compared to slurry seal and micro-surfacing. There fore seal coats are more effective in sealing cracks. Double seal coats are especially effective because this technique allows the placement of two layers of binder.

Low Friction

All surface treatments can be effective for increasing friction if properly applied. In cases where friction is important, extra care should be taken during seal coat application to ensure that bleeding will not result. If non-polishing aggregate is used, the increase in friction can last for as long as the surface treatment remains on the surface of the pavement. Since micro-surfacing aggregate is usually non-polishing, it tends to maintain high friction throughout its life.

Snowplow Damage

Thin surface treatments are often susceptible to snowplow damage as the plow blade removes aggregates from the road surface. This is especially true for rutted pavements where the plow blade rides hard on the high surfaces. Taking steps to fill ruts will minimize plow damage (see Table 44). Operators can place more down pressure on underbody plow blades, so they are likely to cause greater damage.

Micro-surfacing has a hard dense surface that is most effective in resisting plow damage. Seal coats tend to have a more open surface and aggregate particles may not be securely bound; therefore they are most susceptible to plow damage. Slurry seals generally perform better than chip seals, but not as well as micro-surfacing, in resisting plow damage. Researchers received anecdotal evidence that durable surface treatment aggregate is associated with accelerated plow blade wear.

Step 3. Consider Other Factors

Table 45 provides a list of other factors that should be considered before making a final selection regarding seal coats, slurry seals, and micro-surfacing (Al-Hammadi 1998). This table was developed after reviewing the literature, conducting interviews of Iowa transportation officials, and examining survey results. If previous investigation shows that more than one treatment is feasible, this table could be used to determine the preferred method. A detailed explanation of the entries in Table 45 is given below.

Table 45. Other Factors Impacting Thin Maintenance Surface Decisions

	Seal Coat	Slurry Seal	Micro-Surfacing
Past practices	Most officials prefer not to change successful past practice unless there is definite reason for a change. These reasons could be positive or negative changes in funding, neighbor complaints, user complaints, or an opportunity to use better product.		
Funding and cost	Least expensive option → less funding is required.	More expensive than SC and less expensive than micro-surfacing.	Most expensive option → more funding is required.
Durability	Dependent of aggregate type, binder type, and application technique.	Less durable than micro-surfacing.	More durable than slurry seal.
Turning and stopping traffic	Can be flushed by turning and stopping traffic.	Can hold turning and stopping traffic.	Best wear in turning and stopping traffic.
Dust and fly rock	Considerable dust possible during construction.*	Little dust possible during construction.	
Curing time**	Road can be opened after rolling is completed and speed should be limited to about 20 mph for 2 hours.	Road can be opened after 2 hours in warm weather and 6–12 hours in cold weather.	Road can be opened after 1 hour.
Noise and surface texture	Fairly noisy surface, open surface texture, and many loose rocks immediately after construction.	Less noise and dense surface texture (close to hot mix surface).	
Availability of contractors	13 contractors in Iowa.	3 contractors in Iowa.	2 contractors in Iowa.
Use of local aggregates	Maximum flexibility: - Can use somewhat dusty aggregates with cutback binder. - Can use emulsion or cutbacks. - Rock chips, pea gravel, and sand may be used.	Less flexibility.	Least flexible. The binder is highly reactive (break time is affected by clay content).

* Dust is mitigated by using washed, hard, or pre-coated aggregate.
** Federal Highway Administration.

Past Practices

Most transportation officials prefer to continue successful past practices for as long as possible. Changes may possibly affect the staff, politicians, contractors, road users, and property owners; therefore, it is desirable to communicate with all these groups before, during, and after the change. When a change is made, there is a risk that the change may not be successful. However, there are good reasons for considering changes. These include the need to live within funding limits and opportunities to serve the public better with a better product. When the likely benefits of the change exceed the risk and effort, conditions are favorable for making the change.

Funding and Cost

Seal coats are usually the least expensive surface treatment; therefore, they are attractive to jurisdictions that have limited funding. Requirements for premium materials cause micro-surfacing to be the most expensive option. The cost of slurry seal is between micro-surfacing and seal coat; in some cases it is only slightly more expensive than seal coats.

Durability

Often the more expensive treatments are more durable. Therefore the life cycle costs of the more durable treatments may be advantageous, even though they have higher first cost. Micro-surfacing is more durable than slurry seal. Seal coat durability depends on the choice of materials and the application technique. Harder aggregates and polymerized binders often result in greater durability and cost.

Turning and Stopping Traffic

Turning and stopping traffic can cause seal coats to flush as tires work the aggregate around in the binder and push it into the substrate. Slurry seal and micro-surfacing tend to be stiffer and therefore less likely to flush.

Dust and Fly Rock

Seal coat construction tends to be dusty and produce fly rock. Using hard washed aggregate, controlling the aggregate application rate, and sweeping promptly can mitigate dust. Controlling the aggregate application rate and sweeping promptly reduces fly rock. Since slurry seal and micro-surfacing are placed after the emulsion and aggregate have been mixed, the construction process is almost free of dust and fly rock.

Curing Time

Micro-surfacing can be returned to traffic after one hour of curing on warm days, while slurry seal requires two hours in warm weather and six to twelve hours in cold weather. For seal coats, traffic can be returned to the road at low speed after rolling. Curing time is usually two hours, but this varies with climactic conditions.

Noise and Surface Texture

Chip seals have an open surface texture that can be noisy and rough. In residential areas, property owners often prefer a dense surface so children can more easily use bicycles, roller blades, and skateboards. Micro-surfacing and slurry seal provide a more dense, quiet surface, although it is not as dense and quiet as hot mix asphalt.

Availability of Contractors

When several contractors are available to perform work, competition increases and costs are reduced. Also scheduling is easier. In the summer or 1998, there were 13 contractors in Iowa who construct chip seals and seal coats, three who do slurry seal work, and two who perform micro-surfacing. In addition, two out-of-state contractors performed micro-surfacing work.

Use of Local Aggregates

Chip seals and seal coats offer the most flexibility with regard to aggregate usage. Although emulsion binders require the use of clean, washed aggregate, dusty aggregate can be used when cutback binder is used. High float emulsion binders are more forgiving with regard to coating dusty aggregate than cationic emulsions. Pea gravel and sand can be used as cover aggregate on low volume roads. For micro-surfacing, there is little flexibility with regard to aggregate selection. The micro-surfacing binder is highly reactive and will bind quickly and set if clay is present. Therefore, the aggregate may have little clay. High durability is also desired for micro-surfacing aggregate. Locally produced aggregate often does not have these attributes; therefore, it is often necessary to import aggregate. Slurry seal binder is less reactive and, since it is usually used for lower volume roads, durability is less important for slurry seal aggregate when compared to micro-surfacing aggregate. With regard to aggregate selection, slurry seal has more flexibility than micro-surfacing and less than seal coats.

Step 4. Consider Timing

Properly timing the construction of TMS is extremely important. If the treatment is applied too soon, funds are being expended on roads that do not require treatment. If the treatment is applied too late, the road may have deteriorated to the point that TMS are ineffective.

Of the 1997 test sections, it is likely that TMS were applied too late to be effective on US 151. This road had been overlaid in 1965 and then 22 years later in 1987. Deterioration of the 1987 overlay may have been accelerated because it was applied over pavement that was exposed for 22 years. So far, the results on US 30 are more promising. It was overlaid in 1965, 1977, and 1990. A longer period of observation will be required to determine the service life of these treatments. However, preliminary guidelines may be developed based on the results of literature reviews, interviews with transportation officials, and field observations.

Most experts suggest that TMS be applied to a road seven to ten years after it is first constructed. The expected life of the treatment is five to ten years. Geoffroy (1994) surveyed 60 state, provincial, and local transportation agencies and reported the results shown in Table 46. During interviews and field observations, researchers have obtained anecdotal evidence that confirms the findings shown in Table 46. Transportation officials who have successful thin maintenance surface programs for hot mix asphalt pavements usually time the first surface treatment when fine aggregate begins to ravel from the road surface; in most cases this is seven to twelve years after the pavement was initially constructed. Roads that consist of several layers of seal coat may require maintenance more often because less pavement structure is available to support loads.

Table 46. Service Life of Thin Maintenance Surfaces

Treatment	Pavement Age at Time of First Application (years)	Frequency of Application (years)	Observed Increase in Pavement Life (years)
Crack filling	5 to 6	2 to 4	2 to 4
Single seal coat	7 to 8	5 to 6	5 to 6
Multiple seal coat	7 to 8	5 to 6	5 to 6
Slurry seal	5 to 10	5 to 6	5 to 6
Micro-surfacing	9 to 10	5 to 6	5 to 6
Thin lift	9 to 10	9 to 10	7 to 8

Step 5. Consider Cost

Construction costs for maintenance treatments are given in Table 47. These costs are averages from Iowa DOT Offices of Maintenance Operations and include mobilization and traffic control. Overlay costs include the cost of adding shoulder aggregate. Costs range from \$0.11/yd^2 for fog seal to \$3.91/yd^2 for 2-inch ACC overlays. The average costs for surface treatments are less than half the average costs for two inches of ACC overlay.

Table 47. Costs of Thin Maintenance Surfaces

Treatment	1996 Cost/yd^2	1997 Cost/yd^2	Comparison to Seal Coat Cost
Micro-surfacing	$1.29	$1.48	1.82
Fog seal	$0.11	$0.11	0.13
Seal coat	$0.71	$0.81	1.00
Slurry seal	$0.92	$0.92	1.14
1-inch ACC*	$2.27	$2.50	3.09
2-inch ACC*	$3.55	$3.91	4.82

Note: Includes the cost of traffic control and mobilization. Local system costs may be lower.
* Includes the cost of adding shoulder aggregate.

Al-Hammadi (1998) calculated the proportion of cost that is associated with binder, aggregate, labor, and equipment for seal coats, slurry seals, and micro-surfacing (Figure 48, from Al-Hammadi 1998).

Al-Hammadi used crew sizes and equipment fleets that are typical of construction projects in Iowa. Labor rates were based on Davis Bacon minimum wage rates plus a 30 percent labor burden. Hourly equipment rates, production rates, and material costs were estimated after interviewing contractors and suppliers. In each case, binder accounted for the highest percentage of costs, ranging from 36 to 42 percent. Aggregate had the next highest percentages for seal coat (29 percent) and micro-surfacing (30 percent), while labor was the next highest percentage for slurry seal (35 percent). Equipment comprised a larger proportion of cost for micro-surfacing and seal coat and the smallest proportion of cost for slurry seal. Since the majority of costs are materials, efforts to reduce materials usage will reduce the costs of TMS. Reduced materials usage will also reduce equipment and labor costs because much of the equipment and labor costs are related to the amount of material used. Using seal coat designs is one possible approach to reducing materials usage and costs.

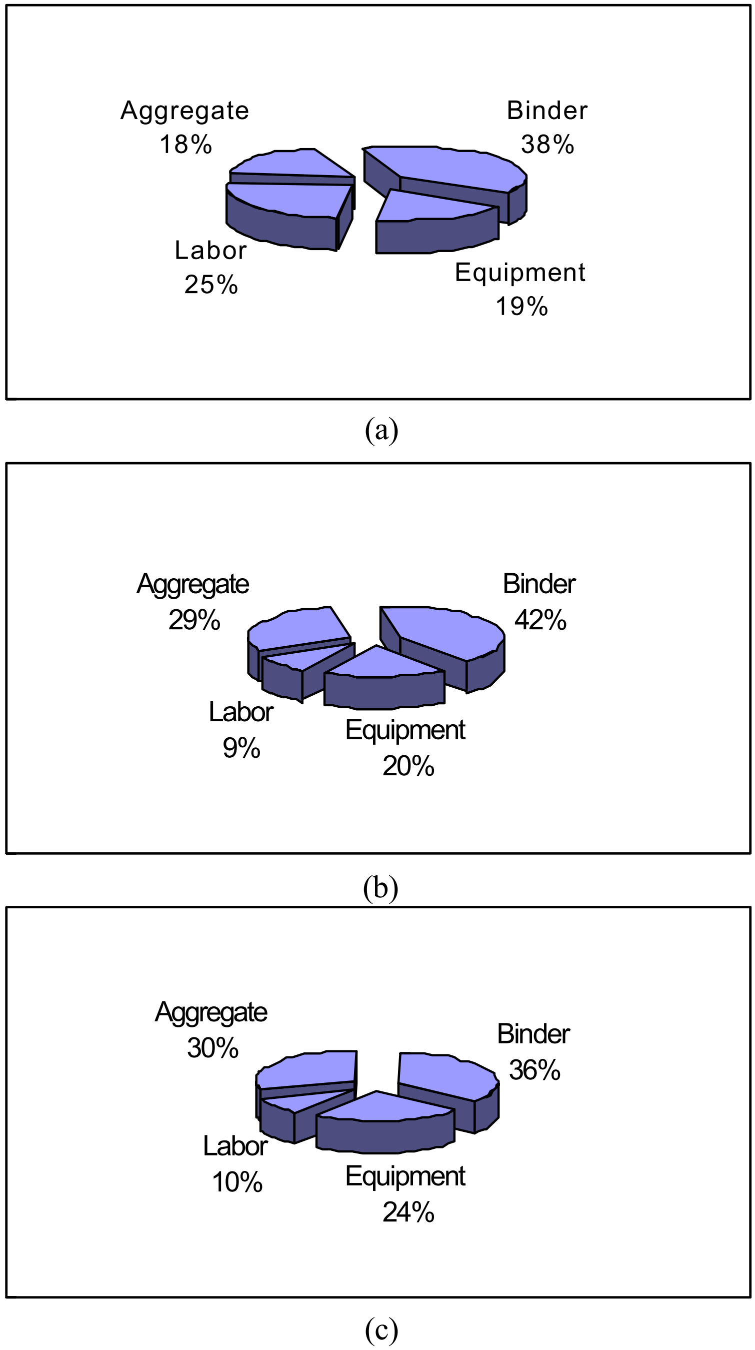


Figure 48. Thin Maintenance Cost Proportions: (a) Slurry Seal Cost Breakdown Using Local Aggregates, (b) Seal Coat Cost Breakdown Using Local Aggregates, and (c) Micro-Surfacing Cost Breakdown Using Imported Aggregates

APPENDIX B. RESPONSES TO QUERY REGARDING WINTER MAINTENANCE ON THIN MAINTENANCE SURFACES

The following replies were received in response to the e-mail query sent to the snow and ice mailing list (as described in Chapter 6). Each response identifies the correspondent by job title and organizational affiliation, rather than by name. Other identifying information has been removed.

Response 1 from a Maintenance Engineer in Alaska DOT

Good morning . . . Just outside of Anchorage, Alaska we applied a seal coat to the Seward Highway. We applied a single shot "C Chip" leaving a relatively course surface. The foreman in the area says that this caused the snow/ice to bind harder to the pavement. He suggested that a double shot utilizing a smaller "E Chip" would fill in some of the voids making a more user-friendly pavement. After a rain or melt period ice would form filling in all the voids and he wasn't able to get it all off. Sand wouldn't stick very well to the smooth icy surface. Liquid mag chloride wasn't as effective as normal due to the fact that the ice was thicker. The mag would only melt the surface layer and refreeze. On a smoother asphalt surface it was easier in the past to get the liquid mag chloride to penetrate and undercut the ice. The foreman also noted that his cutting edges wore down more quickly. It was very noticeable to him. Lastly, he noticed that normal snowplowing removed much of the chip seal material.

I do have some experience with a "stone mastic" pavement that we've been using to try and counter the rutting that typically occurs here. There is a stretch of the Seward Highway here in town that was overlaid to the shoulder stripes with stone mastic, leaving an area of existing pavement on the shoulders. I've noted that during periods of rain or melting the stone mastic dries out quicker due to the fact that it is a courser, more porous material. There is typically very little standing water in the driving lanes whereas on the shoulders of the highway the old pavement will still be very wet. It seems that this is a good thing for traction and to prevent hydroplaning. I haven't noted any excessive snow/ice bonding although it makes sense that there may be a little more.

Response 2 from a University Faculty Member in Michigan

I am (and have been) working with several aspects of polymer concrete bridge deck overlays and similar coatings as wear surfaces and am now getting into some aspects of coatings for deicing and anti-icing applications. I have only seen minor damage to these overlays that can be directly linked to plows but have done quite a bit of analysis on bond strength, durability, etc for and with Michigan DOT. They have been using these coatings along with studying them on lots of bridges as well as at snowmobile crossings. If you would like to give me a call, I can fill you in on what I've done and am working on as well as steering you to the knowledge base at MDOT.

Response 3 from a City Engineer in South Dakota

Speaking anecdotally, this past winter we had a significant problem with excess seal coat rock being left in the boulevards after the snow melted. I don't know if it was a result of a problem with the seal coat process or something else. In some cases we actually went out with a Holder machine and a broom attachment and had to blow it off of resident's property because it was so thick.

We had one storm event that may have contributed to what happened. It went like this - Warm temperatures in the early a.m. with rain changing to snow into the afternoon. The precipitation ended and the temps. Dropped immediately into the lower 'teens. Because the snow fell steadily in the morning we had to commit all of our resources to the Emergency Snow Routes and by the time we got to the residential streets (where the seal coat areas had been) everything was packed down and frozen.

We spent the next several days cutting ice with motor graders (some with serrated cutting edges and some with an ice-buster attachment). The ice came up OK, but it might have brought up some of the seal coat material along with it. I haven't gone out recently to see how those streets look, but am planning to sometime soon. Otherwise, I'm sort of at a loss to explain it. As far as I know, we haven't experienced any significant wear problems with the cutting edges of equipment in the seal coat areas.

This is the first year that I can remember having this much rock left over. The seal coat areas are typically gone over twice by our sweepers in the fall so the excess material should have been picked up. I'll find out the name of the firm who had the seal coat contract last year if you're interested. Let me know if you have any other questions.

On another note, the City and DOT both tested a new thin asphalt overlay material (Nova Chip) on a couple of interstate sections and local streets. I'll check and see how they held up. With the overlay mild winter though, I'd say give us another year to observe the success/failure.

Response 4 from a Virginia DOT Research Engineer

I have seen your notice only on the snow-ice list. Have you posted on the general maintenance list or a pavement management list?

Two synthesis reports that might give you contacts and/or references are NCHRP Synthesis 260, "Thin Surfaced Pavements" and NCHRP Synthesis 284, "Performance Survey on Open Graded Friction Course Mixes." My quick review of 260 does not indicate they asked agencies about snow removal on these surfaces, but the references appear to be a good source. Reference number 19 in the Proceedings of the 6th Conference on Low Volume Roads looks interesting. Synthesis 284 does not deal with the thin surface treatments, but their survey did ask questions about snow removal problems on OGFC mixes. Again, the references are extensive.

Response 5 from a Vermont DOT Engineer

Vermont has had very little or no experience with these treatments. From what we hear, your fears may have some foundation. Slurry seals and chip seals have not performed well here form a stone retention standpoint. There may be some sealing benefits.

Response 6 from a Minnesota DOT Maintenance Engineer

I have forwarded your inquiry to our lab people for consideration. From a maintenance operation perspective, I have not been made aware of this issue from our people.

On a side note, when I was at the Midwest snow and ice workshop in Hannibal, Missouri (this past March), I wrote down the following note: Michigan - they are using "micro sealing" - this is taking on more salt. further comment - "uses blades up" 2 to 3 times faster.

Response 7 from a County Engineer in Michigan

I'm happy to share our experience with two of the three types of thin maintenance overlays, micro surfacing and chip seals. We had a bad experience with a slurry seal about 10 years when MI DOT used it on a state highway locally (we are their contact agency for our county). It was the first & last time slurry seal was used in our county (It wouldn't set up and we had to flag traffic for hours on a major 2 lane highway for hours till it finally set up).

We have been using micro surfacing on residential streets instead of chip seals for about 4 years due to citizen complaints about loose stone and the rougher surface. There is no doubt it is a harder, more brittle material than a chip seal. However, we have not had any comments from our drivers on wearing the cutting edges faster. This may be due to the fact most of our plowing is done with underbody scrapers. My observation has been that down pressure is a bigger factor on cutting edge life than the material being scraped. Another observation is the surface is smoother than our chip seal, perhaps smoother than some of our bituminous mixes (MI DOT Specs.)

We have used chip seals for at least 40 years. About 20-25 years ago we switched fromnatural aggregate (pea stone) to slag aggregate of a similar gradation. We use blast furnace slag, not steel slag. One of the properties of the slag material is that keeps getting sharp edges as it wears. This keeps the coefficient of friction up as opposed to our natural aggregate pea stone, which polishes. We have observed that our slag chip seal roads do not get icy wheel tracks and do not get slippery as quick when it snows as our roads surfaced with bituminous mixes. We even suggested that MI DOT slag seal a portion of the Interstate with bituminous surfacing that is susceptible icy wheel tracks in a light snowfall. They didn't want to do it because of concerns about high AADT and loose stone.

As for plows damaging seal coats, we have found if an underbody peels the stone off, it is due to problems with the seal, either dirty stone, bad emulsion, but most likely damp pavement. We have tightened our specs to require sealing be done in June, July and August. Even then we watch the humidity and pavement temperature quite carefully. We have slag seal surface treatments 10 years old that are still in good shape.

Response 8 from an Arizona DOT Maintenance Engineer

Arizona DOT has not used slurry seal in snow country for about 15 years. The last time it was used was on a 5-lane urban section in an area of the White Mountains. There was no discernable difference in snow removal operations on or off the slurry seal.

Chip seals are still used on low volume roadways. The method now used in snow country is to use a 3/4" minus chip with a 0.45+ gal/sy shot of emulsified asphalt, cure for 6 hours, lightly broom, and then choke the surface with a 0.10 gal/sy shot of emulsified asphalt and a light coating of sand. This double application chip seal has held up well under snowplows for up to 15 years in some areas. We are now experimenting with other treatments but have not had a chance to really test them as yet.

Micro-surfacing has been used mostly in the southern latitudes of Arizona and we have not tested them extensively in snow country.

We haven't noticed any extra snow or ice buildup on the chip seal sections.

Response 9 from a Wisconsin DOT Maintenance Engineer

Wisconsin DOT has been using a thin flexible overlay as one of maintenance treatment for years.

Two years ago we tried slurry seal on Highway 11 (moderate-high AADT) using fine crushed aggregate (type 2 natural material # 200x 1/4"), and AC-20. Our experience demonstrated that slurry seals are not effective treatments for cracked pavements. For a successful slurry seal application, the existing pavement should not have a large cracks that displace under traffic. Pavement has to be stable with no excessive rutting or shoving. So far debonding and delamination (minor) are the only problem we noticed with slurry seal last year, part of this debonding is probably from plows.

WisDOT is using seal coat (chip seal, or aggregate seal) as usual with low-medium volume roads. Chip seal is a single spray operation, usually of a liquid or emulsified asphalt, followed immediately by a single layer of aggregate of as uniform a gradation as practical. We noticed no excessive icy wheel tracks or slippery roads when it snows.

We also have used Micro-surfacing as rut treatments to fill the wheel path rut with success. So far we have couple of projects are exceeding the five to seven years of expected performance with no major distress deterioration.

Response 10 from a Colorado DOT Maintenance Engineer

CDOT has been using chip seals for at least 30 years. We have used several different types, 3/8", 1/2", and light weight. The light weight chips are a blast furnace expanded shale type material. We use primarily emulsion asphalt's, we used to use cut backs but they are more difficult to use and purchase.

It is critical if you want to get a good chip job to have the chips tested for compatibility to the asphalt. We often have to add additives like anti-strip. We do most of the chip jobs with our own forces. Another item that causes grief is if the ruts are not filled ahead of time.

As for plows damaging seal coats, we have found that if you get a good chip job we have little damage from plows. All of our new chip jobs are extremely hard on plow blades the first year.

Response 11 from a Minnesota DOT Maintenance Engineer

We have a section of chip seal and micro surfacing within a stone's throw of each other. My staff agrees that the micro surfacing does wear down cutting edges faster than regular bituminous. However, the benefits do not seem to outweigh these costs.

While micro surfacing and chip seals provide a rougher surface that might seem to trap water, snow, and ice the rough surface also provides enough bare surface to help travelers with traction. Our snow fighters know they need to treat the chip seals with a little extra care so they keep the downward pressure of their underbodies at a minimum. They have been able to extend the life of the chip seal while keeping the pavement acceptably bare during snow events.

We like the micro surfacing for its ability to keep a darker shade of gray or black. This darker color noticeably affects the thawing of any frozen liquid.

We are positive on both micro surfacing and chip seals. We find they are an asset to snow fighting.

Response 12 from a Minnesota DOT Maintenance Engineer

We have Micro down on several roads in our area, however, not on one entire snow and ice route. The past two winters have been very light for us, so I can't tell you from experience that the cutting edges will wear out sooner on Micro. The snowplow operators do report a definite pull or drag when they plow on the micro-surfacing, which I am sure would lead to extra wear on the cutting edge. Also, given the fact that generally the Micro-surfacing is done with a high quality aggregate such as in our case, granite and quartzite, which is very hard and sharp edged, it would generally create a high wear situation for cutting edges. We have experienced extremely high wear on cutting edges where we have planed concrete to improve the ride, almost double the normal wear rate, so it stands to reason from my perspective that you will experience a higher level of wear on your Micro-surfacing.

As far as the ice bonding on Micro goes, we have not found it to be a problem of any significance, if anything, the coarseness seems to retain the salt better.

The damage from plows on the seal coat and slurry coated roads have not materialized for us. The winter of '96/97 in which we did an extreme amount of plowing did not show any severe effects of aggregate loss etc. on our seal coated roads. Underbodies may create a problem depending on the type of cutting edge used, but that also would hold true of a regular blacktop road. I have seen on some of the county roads where the plows have worn the aggregate off of the high spots in the roadway or aggregate loss was created by poor workmanship by the contractor in placing the aggregate on the oil in a timely manner or rolling it properly.

I would be more than happy to discuss these issues with you in more detail if you are interested. This is pretty general, hope this helps.

Response 13 from a Virginia DOT Maintenance Operations Manager

This is in response to your questions of the effects of thin maintenance overlays/pavements on the cutting edges of plow blades, snow/ice bonding effects, pavement damage from snow.

BACKGROUND: I am the Maintenance Operation manager for the Virginia Dept. of Transportation Northern Virginia District Interstate Maintenance/Arlington Primary Road Section. The Interstates (I-95/I-395/I-495/I-66) maintained by our Section are a major transportation link for the Washington Metropolitan Area, border Maryland and DC and include all of Arlington, Fairfax and Prince William Counties. Traffic volumes in the area run over 350,000 AADT in some parts of the system. We have approximately 1424 lane miles of Chemical Treated lanes and approx. 1938 lane miles of plowed lanes in our snow removal program.

Although we do not use a lot of slurry on the Interstate we do have some locations that have been slurry treated:

I-66 Shoulder/HOV Lanes: This section of roadway has concrete pavement overlaid with slurry to designate shoulder(s) used during rush hour time frames as travel lanes. The slurry has been on this section of roadway for close to 7 years with little to no impact from the snow operations. Since the pavement is black compared to the white concrete we may actually see less bonding or quicker melting due to the heating effects. The overall difference is hardly noticeable. The biggest difference on this section of roadway is the effect of traffic only using them during peak traffic flows. This creates a problem since the chemicals are much more effective when vehicles are churning the snow/chemicals to keep them as a brine.

I-66 Gainesville (Old Open graded mix): This section of road was slurry sealed with a latex modified mix around 1993 to prevent the open graded mix from scaling. The old mix was approximately 11 years old and beginning to break loose from the pavement structure. The slurry seal worked well in this instance and we have not experienced any pavement/snow related problems.

We use carbide tipped blades and have not experienced any unusual wear between these road sections compared to other sections that do not have slurry seal.

We have noticed some minor differences in snow accumulating earlier/quicker in the early stages of snow operations with some of the SMA (Stone Matrix) or Superpave mixes which are more open graded in design. We believe this may be the result of the air flow through the open graded surface and cooling the pavement quicker. Once chemical treatment is applied we notice very little difference.

We have also seen some moisture and winter freezing effects on these types of mixes in cases where the shoulders were not resurfaced and have a denser mix. The water tends to flow into the new surface and then resurface at the shoulder joint, which causes water/ice related problems at the joint.

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