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Evaluation of Pavement Surface Friction Treatments

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EVALUATION OF PAVEMENT SURFACE FRICTION TREATMENTS

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16. Abstract

The implementation of a pavement preservation program involves a learning curve with not only a determination to succeed, but also the courage to fail. Also, successful implementation of pavement preservation program requires knowledge of the performance of preservation surface treatments over time, which is critical to the select of candidate projects and the development of performance models for pavement management analysis. In addition, preservation surface treatments, such as chip seal, fog seal, microsurfacing, 4.75 mm thin or ultra-thin overlay, can not only repair certain pavement surface defects, but also change the surface characteristics of pavement and therefore affect pavement surface friction performance. Nevertheless, such information is currently not available but is essential for the Indiana Department of Transportation (INDOT) to evaluate the effectiveness of pavement preservation surface treatments. As a concentrated effort, this study focused on the long-term friction performance of preservation surface treatments, particularly those have been widely used and those have seen increasing use by INDOT.

Based on the selected field pavement test sections, this study aimed to evaluate the surface characteristics, particularly the long-term friction performance for those surface treatments that have been widely used and have seen increasing use by INDOT, including chip seal, fog-chip, fog seal, rejuvenating seal, microsurfacing, ultrathin bonded wearing course (UBWC), 4.75-mm hot mix asphalt (HMA) thin overlay, and profile milling (or diamond grinding). The test sections for each type of surface treatment covered a wide range of traffic volume from light to high. The service life for the selected test sections varied from 6 months to 60 months. Friction testing was mainly conducted using ASTM E 274 locked wheel trailer. Surface texture testing was conducted using either the ASTM E 2157 circular track meter (CTM) or a laser scanner. Pavement roughness and noise tests were also conducted to address the smoothness and noise issues, particularly on microsurfacing. Detailed analysis was provided to evaluate the friction performance of 4.75-mm HMA overlays. It is believed that the test results and findings drawn from this study not only provides timely information for INDOT to improve its pavement preservation program, but also provides the original information for the potential readers to better utilize preservation surface treatments.

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Pavement preservation, surface friction, macrotexture, mean profile depth, chip seal, fog-chip seal, fog seal, rejuvenating seal, microsurfacing, ultrathin bonded wearing course (UBWC), 4.75-mm hot mix asphalt (HMA) thin overlay (UTO), and profile milling (or diamond grinding).	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161

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EXECUTIVE SUMMARY

EVALUATION OF PAVEMENT SURFACE FRICTION TREATMENTS

Introduction

The implementation of a pavement preservation program involves a learning curve with not only a determination to succeed, but also the courage to fail. Successful implementation of pavement preservation program requires knowledge of the performance of pavement preservation surface treatments over time, which is critical to the development of performance models for pavement management analysis. Additionally, preservation surface treatments, such as chip seal, fog seal, microsurfacing, and 4.75-mm thin or ultra-thin overlay, can not only repair certain pavement surface defects, but also change the surface characteristics of pavement and therefore affect pavement surface friction performance. Nevertheless, such information is currently not available but is essential for the Indiana Department of Transportation (INDOT) to evaluate the effectiveness of pavement preservation surface treatments. As a concentrated effort, this study focused on the long-term friction performance of preservation surface treatments, particularly those have been widely used and those have seen increasing use by INDOT.

Based on the selected field pavement test sections, this study aimed to evaluate the surface characteristics, particularly the long-term friction performance for the surface treatments, such as chip seal, fog-chip, fog seal, rejuvenating seal, microsurfacing, ultrathin bonded wearing course (UBWC), 4.75-mm hot mix asphalt (HMA) thin overlay (UTO), and profile milling (or diamond grinding). The test sections for each type of surface treatment covered a wide range of traffic volume from light to high. The service life for the selected test sections varied from 6 months to 60 months. Friction testing was mainly conducted using ASTM E 274 locked wheel trailer. Surface texture testing was conducted using either the ASTM E 2157 circular track meter (CTM) or a laser scanner. Pavement roughness and noise tests were also conducted to address the smoothness and noise issues, particularly on microsurfacing. Detailed analysis was provided to evaluate the friction performance of 4.75-mm HMA thin overlays. It is believed that the test results and findings drawn from this study not only provides timely information for INDOT to improve its pavement preservation program, but also provides the original information for the potential readers to better utilize preservation surface treatments.

Findings

Chip Seal and Fog-Chip Seal

On the newly chip sealed surfaces, surface friction numbers varied between 50 and 70. The greater the friction number on the old pavement, the greater the friction number on the new chip sealed surface. The surface friction decreased after opening to traffic. The greatest friction decrease occurred after 12 months in service. When the chip seals reached the age of about 30 months, the surface friction started to decrease continuously over time. It took 12 months for chip seals to form a stable mosaic. Successful chip seal produced a friction number between 44 and 52 after 12 months in service. Failure was observed in two chip seal sections, which experienced dramatic decrease in friction at the age of 12 months and the resulting friction number was less than 30, commonly around 20.

Applying a fog seal on top of a chip seal resulted in an immediate, temporary decrease in friction by 20%–33%. The surface friction increased due to the material wearing off the tops of the chips tended to reach the maximum value after 6 months or more in service. This indicates that a fog-chip seal may form a stable mosaic faster than a standard chip seal. The surface friction in the fog-chip seals demonstrated a tendency to decrease over time after 12 months in service. The average friction number in the fog-chip seals at the age of 12 months was almost the same as that in chip seals at the age of 12 months. Failure was observed in one fog-chip seal section that demonstrated a friction number of about 20 after 12 months of service. It appears that a fog-chip seal may not necessarily perform as well as a standard chip seal in terms of surface friction.

For new chip seal, the friction number produced by crushed stone was 16.7% greater than that by crushed gravel. After 12 months in services, the difference in friction dropped to 7.0%. If this trend remains in the long term, it is very promising for future use of crushed gravels in chip seals. However, the use of uncrushed aggregate could result in a friction reduction.

Chip seals can be successful even on high traffic volume roads. It seems that truck traffic affected the performance of a chip seal more significantly than AADT. The failed chip seals commonly demonstrated poor surface friction and could easily be identified from the visual appearance, such as insufficient aggregate chips, binder-rich surface or both. The contributing factors to the failures in the test sections were not readily apparent. However, it appears that care should be exercised when applying chip seal to a pavement when its overall condition level or surface roughness is rated fair or worse.

Fog Seal and Rejuvenating Seal

After applying a fog seal, the surface friction decreased immediately and dramatically by more than 50%. However, the variability of friction also deceased by more than 60%. This implies that a fog seal may improve the pavement surface uniformity, and provide more consistent surface characteristics. It took about 18 months for the surface friction to return to the original level. The effectiveness of a fog seal varied with the surface characteristics of existing pavement to a large extent.

After applying a rejuvenating seal, the surface friction experienced a reduction by more than 40% and then increased and peaked in 30 days. Sand blotters resulted in a friction increase by 8 points and became ineffective after 1 or 2 days. The rejuvenating agent on the pavement surface could dry out in about 1 month. The pavement surface friction was unable to return to its original level after the application of a rejuvenating seal. The friction dropped by more than 18% in the passing lane and 35% in the driving lane after 24 months of application.

Microsurfacing

The freshly placed microsurfacing could produce sufficient surface friction between 28 and 57 when opening to traffic. The surface friction of microsurfscing increased significantly in the first six months and peaked after 12 months of service. The microsurfacing surfaces became stable and produced true friction numbers between 40 and 60 after 12 months of service. After 12 months of service, the surface friction tended to decrease continuously over time. However, no friction numbers less than 30 were observed after a service period up to 42 months. The MPD values in the test sections varied between 0.66 mm and 0.94 mm.

The smoothness improvement from microsurfacing depended on the smoothness of existing pavement. The rougher the existing pavement surface the greater the smoothness improvement. However, the improvement of smoothness was limited. The two main distress modes observed in the test sections are delamination and reflective cracking. Delamination commonly occurred at the interface between the microsurfacing and old surface. Care should be exercised when microsurfacing is applied to high traffic volume roads, particularly urban roads. Noise differences between microsurfacing and 4.75-mm HMA overlay were not perceptible on the roadside and in the vehicle.

Ultrathin Bonded Wearing Course (UBWC)

The friction numbers on the fresh UBWC surfaces varied between 48 and 59. The friction numbers in the test sections tended to peak after 6 months of service or less, about 6 months earlier than conventional HMA mixes. UBWC has the potential to provide durable friction performance. UBWC provided coarse pavement surfaces. The measured MPD varied between 0.95 mm and 0.99 mm, much greater than that for conventional 9.5-mm HMA mixes. Significant friction decrease over time was also observed. The surface friction could decrease by more than 34% after 33 months in service. Noticeable polishing occurred to limestone aggregate. The use of steel slag could enhance the long-term friction performance. In order to provide satisfied long-term friction performance, it requires highly durable, highly polish-resistant aggregates.

Thin 4.75-mm Dense-graded HMA Overlay

A fresh 4.75-mm HMA overlay may provide a friction number between 35 and 52, and then decreases quickly and dramatically over time after opening to traffic. In the test sections, the friction decreased by 25% after 6 months of service and by 36% to 48% after 12 months in service. The rate of friction decrease depends on traffic volume. After 12 to 18 months, the 4.75-mm HMA overlays tended to produce steady surfaces. On a steady 4.75-mm HMA surface, the friction number varies between 20 and 30 and was commonly around 20. The 4.75-mm HMA surfaces were very smooth. The majority of MPD measurements varied between 0.20 mm and 0.25 mm.

The 4.75-mm HMA mixes in the test sections were basically fine-graded mixes with too much fine sand and tended to produce significant fluctuation in surface friction over time. It is critical to employ an aggregate gradation to pass below the PCS to enhance the friction properties of 4.75-mm dense-graded mixes. A greater CA ratio may produce larger macrotexture and can be achieved by reducing the percent passing the PCS or increasing the percent passing the HS. Likely, a CA ratio approaching but less than 1.0 will have positive effect on the macrotextrue of a 4.75-mm mix.

Aggregate type affects surface friction. The use of steel slag will produce good texture properties. Measures for enhancing FAA may improve surface friction. A greater SE indicates more coarse particles and may result in better macrotexture properties. The aggregate angularity and abrasion resistance also play an important role in producing and maintaining sound surface characteristics. In addition, it was observed that a greater binder grade could result in better surface texture.

Profile Milling

It can be concluded that the steady-state friction number should not be less than 36 on a freshly profile milled HMA surface and should not be less than 40 on a freshly profile milled concrete surface. The surface friction on the profile milled surface varied over time. The long term friction performance of a ground surface depends on the aggregate properties and grinding texture configuration. It is the aggregate and land area that play an important role in providing durable texture. It appears that longitudinal grinding is capable of providing immediate and long term improvement to surface friction on both light and high traffic volume roads.

Implementation

This study provided INDOT with a comprehensive review of the friction performance of various surface treatments and materials. This knowledge will allow INDOT to better identify applications for various treatments and their expected friction life. Two faced crushed gravel was evaluated in this study, and is now being used routinely to give better performing chip seals. Also, the knowledge will allow INDOT to revise its specifications on mixes to address friction issues.

The 4.75 mm mixture performance issues identified by this study was first addressed by revising the INDOT specification on 4.75 mm mixes in August, 2009, and then two new 4.75 mm thin overlay projects were constructed on SR-227 and US-27 accordingly. The results were reported and the friction issues were further addressed by revising the specification again in early 2011. Consequently, eight contracts were constructed in 2011 using the revised specification. Noise performance was evaluated on microsurfacing projects and the results will be used for future projects.

New 4.75 mm thin overlay projects will be added into the field evaluation list friction test list in April, 2012, including SR-1, SR-32, SR-44, SR-64, US-24, US-40 and US-52. Noise performance will be evaluated on microsurfacing projects on SR-227 and US-421 in 2012. Further field testing and evaluation will be conducted in 2012 and 2013 on the pavement preservation treatments, including chip seals (SR-32, SR-341, US-150, SR-246, and SR-159).

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CHAPTER 1 INTRODUCTION

Problem Statement

Traditionally, after a new pavement is laid and opened to traffic, it is allowed to deteriorate under the combined effects of traffic application and environment over time. After the pavement performance condition or structural condition has reached a certain threshold value, a pavement rehabilitation work, such as structural overlay or reconstruction, is then utilized to restore the pavement surface condition or repair the pavement structural damage. It has been shown that over the past decades, pavement rehabilitation activities are commonly costly and timing consuming. In addition, pavement rehabilitation activities tend to cause significant disruptions and inconvenience to the travelling public and adjacent neighborhood and business. Therefore, the above traditional approach is often labeled as a reactive strategy and referred to as the worst-first policy.

Pavement preventive maintenance is the application of the appropriate surface treatment to preserve the current condition of pavement, retard future deterioration of pavement condition, and improve pavement surface function without significantly increasing the structural capacity of pavement. Pavement preventive maintenance is a planned, proactive strategy to apply cost-effective treatments to the existing pavements in sound structural condition. The key for successful pavement preventive maintenance is to apply the right treatment to the right pavement at the right time. The Foundation for Pavement Preservation (FP^2) has identified the main benefits associated with a successful pavement preservation program, such as higher customer satisfaction, better informed decisions, improved strategies and techniques, improved pavement condition, cost savings, and increased safety (1). In addition, successful pavement preservation program can also result in reduced travel delays due to construction. The Indiana Department of Transportation (INDOT) has a long history of pavement preservation surface treatments, particularly chip seal and fog seal on roadways in light traffic conditions. To date, INDOT has implemented a pavement preservation program and established strategic pavement preventive maintenance goals to keep Indiana's pavements in better conditions. At this time (2), 5–10% of INDOT's major preservation budget is dedicated to pavement preservation. It is estimated that $1 spent on pavement preservation can yield a total saving of up to $10.

The implementation of a pavement preservation program involves a learning curve with not only a determination to succeed, but also the courage to fail. In reality, it was approximately in 2007 when INDOT started to formalize its pavement preventive maintenance activities and accelerate the process to adopt pavement preservation surface treatments into pavement preservation program and integrate pavement preservation surface treatments into pavement management activities. It is well known that successful integration of pavement preservation surface treatments into pavement management activities requires knowledge of the performance of preservation surface treatments over time, which is critical to the development of performance models for pavement management analysis. Also, preservation surface treatments, such as chip seal, fog seal and microsurfacing, can not only repair certain pavement surface defects, but also change the surface characteristics of pavement and therefore affect pavement surface friction performance. Nevertheless, such information is currently not available but is essential for INDOT to evaluate the effectiveness of pavement preservation surface treatments and establish an approval list for those treatments. Therefore, it is a pressing need for INDOT to address these issues so as to accomplish its pavement preservation strategic goals.

Objectives

As a concentrated effort to improve INDOT pavement preservation program, this study focused on the long-term friction performance of preservation surface treatments, particularly those that have been widely used and those that have seen increasing use by INDOT. The objectives of this study were threefold. First, this study aimed to compile the first-hand information on the variables that affect the friction performance of pavement preservation surface treatments. Second, this study aimed to provide INDOT engineers with original data on novel pavement preservation surface treatments and materials. Third, this study was designed to monitor the long-term friction performance of pavement preservation surface treatments. It is believed that after these objectives have been accomplished, INDOT will be able to shorten the learning curve and better apply preservation surface treatments.

Study Scope and Main Tasks

In order to fulfill the study objectives, the scope of study and main tasks are listed below:

Selection of Preservation Surface Treatments

After a state-of-the-practice synthesis conducted on pavement preservation surface treatments nationwide and after consulting with the Study Advisor Committee (SAC) members and INDOT pavement preservation engineers, the following preservation surface treatments were identified to have potential applications by INDOT and be able to provide benefits to both INDOT and public travelers:

- Chip Seals, and fog-chip seal
- Fog seal and rejuvenation
- Microsurfacing
- Ultrathin bonded wearing course (UBWC)
- Ultrathin 4.75-mm hot mix asphalt overlay (UTO), and
- Profile milling (asphalt and concrete pavements)

Selection of Preservation Surface Treatment Test Sections

This task was completed after working closely with SAC members, INDOT pavement preservation engineers, and INDOT districts. The selection of test sections was made by taking into account the type of surface treatment, applied materials, existing pavement condition, and traffic level. A minimum of three sections were selected for each type of surface treatment if available. Preference was given to those surface treatments under different traffic levels, particularly on high traffic volume roads.

Field Testing

Field testing was conducted to evaluate the surface characteristics related to friction performance. For most surface treatments, such as chip seal, fog seal, UBWC, and profile milling, the friction of pavement surface was measured using a locked wheel trailer in accordance with American Society for Testing and Materials, ASTM E 274 (3). For ultrathin HMA overlay and microsurfacing, surface macrotexture was also measured using the circular track meter (CTM) in accordance with ASTM E 2157 (4) in some test sections in addition to the friction testing. In some cases, the so-called dynamic friction tester (DF-Tester) (5) testing was conducted to verify the friction performance and make comparison between the test results measured using the locked wheel trailer and the DF-Tester. It was pointed out that by Henry (6), while conducting the locked wheel friction testing, the standard smooth (7), instead of the standard rib tire (8), should be used. The rib tire provides six 0.2-in wide grooves that are much larger than the macrotexture of pavement surfaces in many situations. As a result, the friction measured using a rib tire depends mainly on the microtexture of pavement surface. However, the friction measured using a smooth tire depends on both the surface microtexture and macrotexture.

Analysis of Test Results

In the analysis of test results, great effort was made to determine the long-term friction performance associated with the preservation surface treatments selected in this study in terms of the following:

- Surface friction properties: The surface friction was evaluated primarily by the friction number (FN) from the locked wheel testing. A range of possible FN values was identified for each preservation surface treatment based on statistical analysis. DF-Tester friction coefficient was also provided to validate surface friction when concerns arose about the FN values in some special situations.
- Surface texture characteristics: For the ultrathin overlay of 4.75-mm hot mix asphalt (HMA), surface texture characteristics were evaluated by macrotexture measurements, particularly the so-called mean profile depth (MPD) of macrotexture (9). The effect of macrotexture on surface friction prevails on wet pavement surface. Also, macrotexture plays an important role in preventing hydroplaning in rainy seasons.
- Long-term friction performance: The long-term friction performance was measured for each type of preservation surface treatment in terms of the FN values and associated variations over time. This information was utilized to evaluate the durability of surface friction under the application of traffic.
- Effects of materials: The effects of applied materials were evaluated so as to make comparison of the effectiveness of different materials if available for each preservation surface treatment. This information is particularly useful to better utilize local materials, such as aggregate chips of different sources.
- 4.75-mm dense-graded HMA ultrathin overlay: The use of 4.75-mm HMA mixtures is receiving increasing popularity nationwide. However, very little information is available on the performance of such mixtures. A comprehensive analysis was conducted to evaluate the friction performance of 4.75-mm dense-graded HMA mixtures, particularly the effects of aggregate and mixture properties.

Documentation

The final report is provided to document the entire research effort made by this study, including research procedures and methodologies, test data, results and analysis, findings, and recommendations.

Implementation Benefits

The study results will be implemented by INDOT districts, Division of Planning and Production, and Division of Highway Operations. The anticipated benefits are summarized below:

- First-hand data on long-term friction performance of preservation surface treatments that have potential applications in INDOT.
- Accurate field data on novel pavement preservation surface treatments and materials used by INDOT.
- Realistic effectiveness data for the INDOT divisions and districts to formalize pavement preservation surface treatments.
- Fundamental information to incorporate pavement preservation surface treatments into pavement management activities.
- Accelerated learning curve for INDOT engineers, and
- Validated data, procedures, test results and findings for INDOT to fully benefit from pavement preservation surface treatments

CHAPTER 2 CHIP SEALS AND FOG-CHIP SEALS

General Description

Chip Seal

Chip seal is an asphalt surface treatment in which asphalt binder (commonly asphalt emulsion) is applied to the existing asphalt pavement surface followed by the immediate application of aggregate chips that are rolled using pneumatic-tired roller to achieve the anticipated aggregate embedment and increase the retention of aggregate chips, and therefore enhance chip seal performance. The primary use of chip seal is to seal pavement surface and provide a new surface with enhanced surface friction performance. Chip seals are also used as a wearing course on low volume roads. It has been proved that chip seals are effective in improving surface friction, preventing surface material oxidization, inhibiting raveling, and correcting surface defects, such as bleeding and minor roughness. Also, chip seals can correct minor cracking and rutting. Based on the many year experiences from state highway agencies (SHAs) (10,11), chip seals are an economic, durable surface treatment that performs well in many climates, particularly if applied early in a pavement's life. Currently, chip seals constitute a large portion of preservation surface treatment activities in many SHAs.

There are different types of chip seals in use nationwide, such as single chip seal, multiple (usually double) chip seal, stress absorbing membrane (SAM) seal, and stress absorbing membrane inter-layer (SAMI). A single chip seal is an application of asphalt binder followed by an application of aggregate chips. It is commonly used as a pavement preservation surface treatment to seal minor cracking, arrest surface raveling, and restore surface friction. A multiple chip seal (armor coat) consists of multiple applications of asphalt binder and aggregate chips. For instance, a double chip seal consists of two single chip seal applications: spraying asphalt binder, spreading a layer of aggregate chips, rolling the applied aggregate chips, and then repeating the above steps one more time. The first chip seal usually utilizes more asphalt binder and larger aggregate chips than the second chip seal. Therefore, a double chip seal can apply to pavements in poor conditions, where a single chip seal may not perform well, and provide a quieter and smoother riding surface (12). Currently, a single chip seal is commonly used by INDOT. Figure 2.1 shows a typical single chip seal pavement in Indiana.

Figure 2.1 Photo of a Typical Single Chip Seal Pavement after 36 Months of Service

Fog-Chip Seal

As a variation to the standard chip seal, the so-called fog-chip seal in Indiana is a combination of a chip seal with a fog seal. A fog-chip seal applies a fog seal (0.11 gal/yd^2 to 0.15 gal/yd^2) to the top of a chip seal clean of excessive aggregate chips. A fog seal overlying a chip seal may provides many benefits. The primary benefit is that the fog seal holds the top layer of the stones in the chip seal and prevents possible aggregate loss and fly-rock that appears to be the main problem associated with the standard chip seal. Therefore, the application fog seal may not only prevent potential vehicle damage, but also protect the chip seal pavement from winter snowplow operations. The fog seal can fill the voids in the chip seal. In addition, the fog seal can darken the chip seal pavement, and therefore create distinct demarcation and improve driveway aesthetics and delineation. It has been widely accepted that a fog-chip seal is capable of providing better performance in terms of longer lasting pavement and enhanced customers' satisfaction, compared to a standard chip seal. Presented in Figure 2.2 is a photo of a typical fog-chip seal pavement in Indiana.

Test Sections

Chip Seal Sections

The selection of test sections for evaluating both chip seals and fog-chip seals was based on the availability of the test sections and by taking into account the

Figure 2.2 Photo of a Typical Fog-Chip Seal Pavement after 24 Months of Service

TABLE 2.1
Summary of Selected Chip Seal Test Sections

Category	Road	Agg. Type	Location (RPs)	AADT	Truck %	Year Sealed
1	SR-32	Crushed stone	17.73–25.97	732	8.9	08/2008
	SR-14	Crushed stone	32.08–46.98	1,530	18.3	07/2008
	SR-10	Crushed stone	3.99–10.25	3,372	47.2	07/2007
	US-421	Crushed stone	197.81–206.83	4,527	23.5	08/2007
	SR-129	Crushed stone	36.3–42.79	5,892	4.8	07/2007
	SR-19*	Crushed stone	17.93–26.82	6,129	9.7	07/2007
2	SR-341	Crushed gravel	10.68–23.56	955	8.1	07/2009
	US-150	Crushed gravel	0–12.74	2,450	6.4	07/2009
3	SR-246	Crushed gravel	1.0–5.0	902	8.5	06/2010
	SR-159	Gravel	15.0–19.0	2,467	9.7	06/2010

aggregate type applied. For chip seals, a total of ten test sections were selected as shown in Table 2.1. All these ten test sections were located respectively on different two-lane highways, and were divided into three different categories according to their aggregate types applied. Category 1 includes six sections chip sealed using crushed aggregate chips (commonly limestone in Indiana). Category 2 includes two sections chip sealed using crushed gravel chips. Category 3 includes two sections chip sealed using the aggregate chips from the same source, of which, one section (SR-246) consisted of angular chips and the other (SR-159) but contained a few truckloads (by mistake) of naturally formed chips. Most test sections were chip sealed in 2007 and 2008. Also presented in Table 2.1 is the traffic data in 2007, including traffic volume in terms of annual average daily traffic (AADT) and associated truck percentage for each test section. It is shown that the AADT varied from 732 to 6,129, and the truck percentage varied from 8.9% to 47.2%.

Fog-Chip Seal Sections

For chip-fog seals, a total of eight test sections were selected as shown in Table 2.2. All test sections were chip sealed using crushed aggregate chips. The construction was completed in either 2008 or 2009. Two test sections, i.e., SR-9 and US-52(a), are located on four-lane highways, respectively. All other test sections are located on two-lane highways. The greatest AATD was 6,307, which was observed in the test section of US-52(a), and the lowest AADT was 1,047 and occurred in the test section of SR-101. The greatest truck percentage was 39.6% occurred in the test section on SR-67, and the lowest truck percentage was 4.5% observed in the test section on SR-11. It appears that both the test sections selected for both chip seals and chip-fog seals covered similar traffic levels.

TABLE 2.2
Summary of Selected Fog-Chip Seal Test Sections

Road	Agg. Type	Location (RPs)	AADT	Truck %	Year Sealed
SR-9	Crushed Stone	121.6–127.6	3161	10.8	08/2009
SR-11	Crushed Stone	0–18.44	1656	4.5	09/2009
SR-48	Crushed Stone	0–6.96	1410	8.5	08/2009
SR-67	Crushed Stone	190.8–199.3	2110	39.6	05/2009
SR-101	Crushed Stone	34.41–36.52	1047	19.4	08/2008
US-36	Crushed Stone	0–7.66	2085	15.8	10/2008
US-52(a)	Crushed Stone	27–37.29	6307	7.2	10/2008
US-52(b)	Crushed Stone	137–145.7	1531	9.8	09/2009

Surface Friction Performance

Surface Friction on Chip Seal

Friction testing was conducted right after the construction was completed and then after right after the pavement was opened to traffic on each test section. In the following years, friction testing was conducted twice a year, one in spring (commonly in May) and one in fall (commonly in September). In general, chip seals are a common surface treatment to restore pavement surface friction, particularly on low volume roads. This is mainly due to the contribution of large macrotexture depth produced by the cover aggregate chips. Presented in Table 2.3 are the friction numbers measured in accordance with ASTM E 274 using the standard smooth tire before and after chip seals. Apparently, chip seals can improve pavement surface friction performance significantly. For most newly chip sealed pavements, surface friction numbers were no less than 50. The greatest friction number was 67 observed in the test section on SR-32, and the lowest friction number

TABLE 2.3
Friction Numbers Measured before and after Chip Seal

Test Section	FN (before)	FN (after)	FN Increase%	Agg. Type
SR-32	49	67	26	Limestone
SR-14	40	56	28	Limestone
SR-101	43	65	33	Limestone
US-421	33	50	34	Limestone
SR-341	42	50	16	Gravel
US-150	28	49	43	Gravel

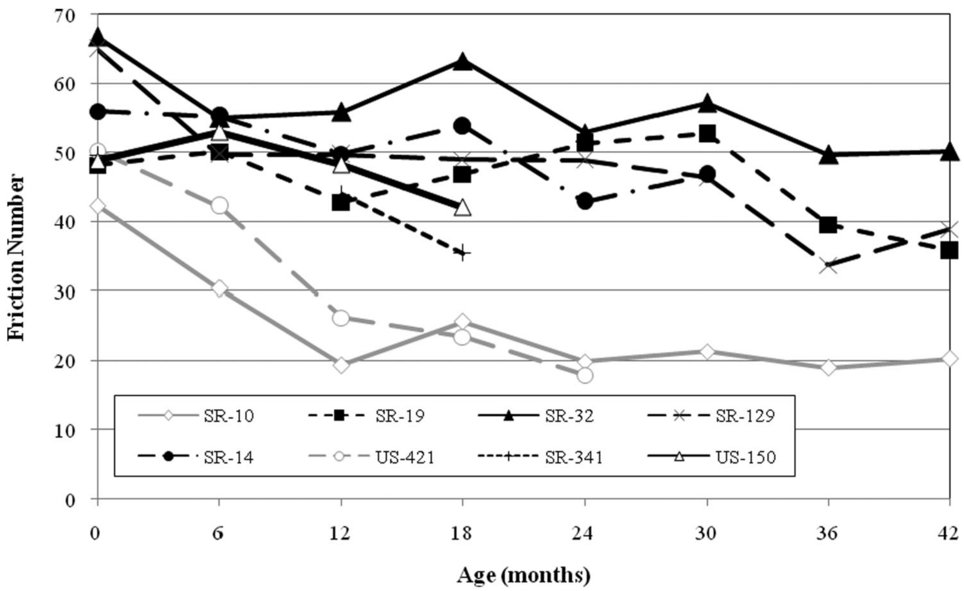


Figure 2.3 Friction Measurements on Chip Seals over Time

was 49 measured in the test section on US-150. Also, it appears that the greater the friction number on the old pavement, the greater the friction number on the new chip sealed surface.

Presented in Figure 2.3 are the friction numbers measured over time in eight chip seal test sections. Two test sections, including the one on SR-159 and the other on SR-246 are not presented since their lives of service are only 6 months and the surfaces may not be stable. In Figure 2.3, the x-axis indicates the age of chip seal at the time of friction testing and the y-axis indicates the average friction number in the test section. It was shown that the friction variations in all test sections roughly followed a similar trend. The surface friction decreased after opening to traffic. The greatest friction decrease occurred at the age of 12 months. Afterwards the surface friction fluctuated. When the chip seals reached the age of 30 months, the surface friction started to decrease over time. Also, two test sections, including SR-10 and US-421 (see the two grey color curves), experienced dramatic decrease in surface friction at the age of 12 months and demonstrated low surface friction subsequently, due to too much oil applied and testing different application rate to develop a chip seal design method, respectively. Two implications can be drawn from these observations. First, chip seal failures may occur due to many different factors that will be discussed later. A successful chip seal arises not only from the rationale science, but also from the lessons learned from failures in the field. Second, it usually takes approximately 12 months for chip seals to form a stable mosaic. This confirms why the texture depth of a chip seal after 12-month service is utilized to evaluate the performance of the chip seal (13). The test data indicated that in Indiana, successful chip seals could produce friction numbers vary between 44 and 52 after 12 months of service at a confidence level of 95% during the 3-year study period. However, a failed chip seal tended to provide a friction number of less than 30 (commonly 20) after 12 months of service.

Surface Friction on Fog-Chip Seal

One of the most immediate effects caused by applying a fog seal on top of a chip seal is the decrease in surface friction due to the presence of asphalt emulsion on the pavement surface. This is because the asphalt emulsion on the pavement surface will produce lubricating action at the interface between tire and pavement, resulting in a decrease of adhesion force in the tire-pavement contact areas. Presented in Figure 2.4 are the friction numbers measured in three fog-chip seal test sections before and after applying asphalt emulsion onto the chip seals. A decrease of more than 20% in

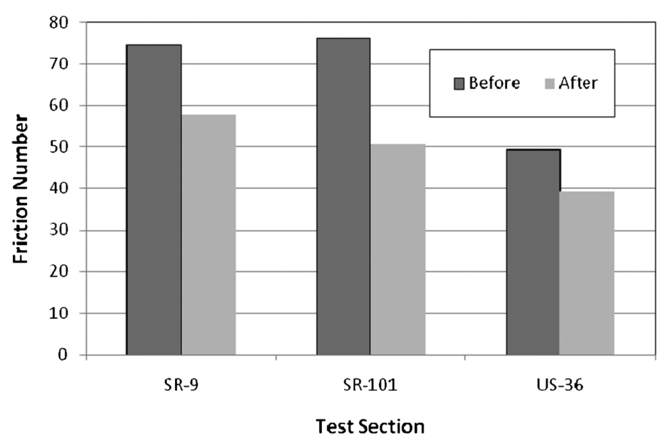


Figure 2.4 Friction Measurements before and after Fog Seal

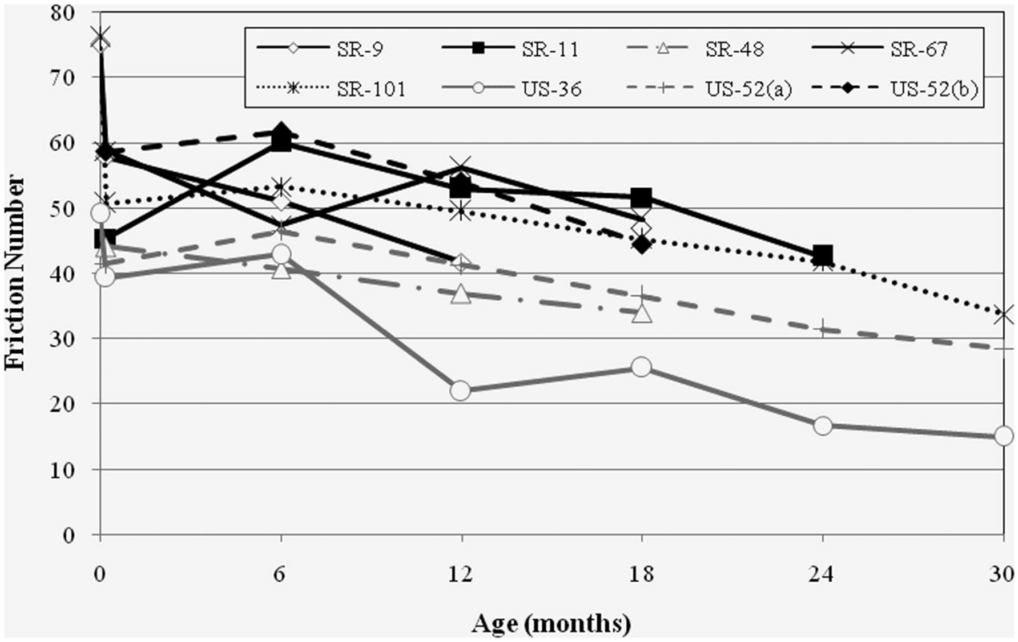


Figure 2.5 Friction Measurements on Fog-Chip Seals over Time

surface friction was observed in all three test sections. The greatest decrease was up to 33%, which occurred in the test section on SR-101. It appears that the greater the surface friction before applying the asphalt emulsion, the greater the decrease in surface friction after applying the asphalt emulsion. Even with the decrease, the friction numbers were excellent.

Presented in Figure 2.5 are the friction numbers measured in the fog-chip seal test sections over time. It is shown that after applying asphalt emulsion onto chip seals, the surface friction experienced a significant reduction. Afterwards, the surface friction went up due to the drying out of asphalt emulsion on the pavement surface. The surface friction reached its maximum value at the age of 6 months in most test sections, and then started to decrease. In these two test sections on SR-9 and SR-67, however, it took more than 6–12 months for the surface friction to reach its maximum value. This implies that a fog-chip seal may form a stable mosaic faster than a stand chip seal. The average friction number was 47 after 12 months in service, which was very close to that observed in the chip seal test sections. It is also shown that the test section on US-36 not only experienced a dramatic decrease in friction in 12 months, but also demonstrated poor friction numbers around 20 subsequently due to the use of the wrong oil. More importantly, it is shown that the surface friction in the fog-chip seal test sections demonstrated a trend to decrease continuously over time after 12 months in service. If this trends remains, concerns may arise over the long-term friction performance of a fog-chip seal. Since the ages of the fog-chip seal test sections were only 2 years old or less, it is a tentative conclusion that a fog-chip seal may not necessarily outperform a standard chip seal.

Factors Affecting Friction Performance

Effect of Aggregate

Material properties, such as the type and grade of asphalt emulsion, the type, gradation, and shape of aggregate, and the application rates of asphalt emulsion and aggregate chips, have significant effect on the performance of a chip seal. It is well known that the embedment of aggregate chips plays a critical role in the long-term performance of a chip seal. In Indiana, the requirements for the materials of chip seal are summarized in Table 2.4 and the detailed information can be found elsewhere (14,15). It should be pointed out that Indiana is an anionic emulsion state with two asphalt emulsion suppliers (16). Three sizes of aggregate, including No. 11, No. 12, and SC 16 (in some situations), are currently used by INDOT. No. 11 is coarser than No. 12 and SC 16. The type of aggregate that has been commonly used in Indian is the crushed limestone aggregate. Effort has been made by INDOT to explore the use of other aggregate types, such as crushed gravel, so as to utilize local aggregate materials.

Presented in Figure 2.6 are the friction numbers measured in the chip seal test sections using crushed stones and crushed gravels, respectively. The friction numbers were averaged with respect to the time in service. For new surfaces, i.e., 0-month in service, the average friction number in the crushed stone sections was 9 points (about 16.7%) greater than that in the crushed gravel sections After 12 months in service, the average friction number in the crushed stone sections was greater than that in the crushed gravel sections by 3 points (or 7.0%). If this trend remains in the long term, it is very promising for the future use of crushed gravels

TABLE 2.4
Specifications for Chip Seal (14,15)
(a) Materials

Material	Type
Emulsions	AE-90, AE-90S, RS-2, HFRS-2
Aggregate	Size 11, Size 12

(b) Aggregate Gradations

Aggregate	Percent Passing (%)						
Size	1/2"	3/8”	No. 4	No. 8	No. 16	No. 30	Decant
11	100	75–95	10–30	0–10	–	–	0–1.5
12	100	95–100	50–80	0–35	–	0–4	0–1.5
16	100	94–100	15–45	–	0–4	–	0–1.5

(c) Recommended Rates of Material Application

Aggregate Type	Aggregate Application Rate (lb/yd^2)	Emulsion Application Rate (gal/yd^2)
11	16 – 20	0.36 – 0.4
12	14 – 17	0.29 – 0.33

in chip seals. Presented in Figure 2.7 are the friction numbers measured in the two test sections located on SR-159 and SR-246, respectively. Both sections used two faced crushed gravel chips of SC 16. However, the crushed gravels were mixed with several loads of uncrushed gravel chips in the test section on SR-159 due to aggregate delivery error. Apparently, the uncrushed gravel chips resulted in a reduction in surface friction probably by 5 points or more. Commonly, aggregates with more fractured faces tend to provide better surface friction performance.

Effect of Traffic

Most preservation treatment, particularly chip seal, has occurred on low traffic volume roads nationwide and the concerns on pavement preservation on high

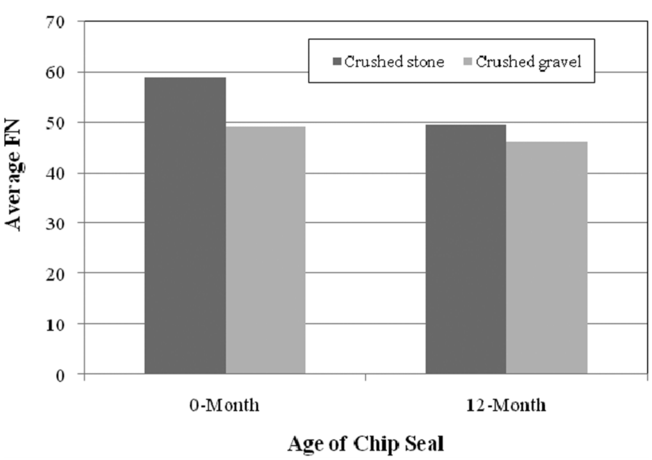


Figure 2.6 Comparison of Friction Numbers by Aggregate Type

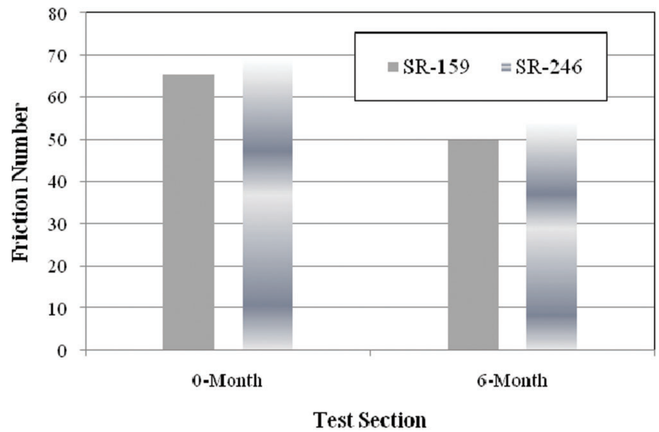


Figure 2.7 Comparison of Friction Numbers by Aggregate Shape

traffic volume roads include durability, performance, and negative public perception (17). Traffic wearing will cause chip orientation and affect chip embedment. The loose chips on top of a fresh chip seal will be removed by traffic until a stable mosaic is formed. This study investigated the possible effect of traffic level based on the five chip seal test sections with different traffic levels in terms of AADT. Four sections, including SR-10, SR-19, SR-32, and SR-129, have a service life of three and half years, and one section, i.e., SR-14, has a service life of two and half years. Figure 2.8 shows the friction numbers measured over time in these five sections. It is shown that the two test sections on SR-14 and SR-32 experienced lowest AADT and demonstrated good friction performance. However, both the two test sections on SR-19 and SR-129 experienced more than 5,000 AADT, respectively, and also demonstrated good friction performance after 3 years in service. The AADT was 3,372 in the test section on SR-10, which demonstrated a poor friction performance due mainly to bleeding. The above observations indicate that traffic may affect the performance of a chip seal and chip seals can be successful even on high traffic volume roads. However, it seems that the truck traffic might affect chip seal more significantly. More data is needed to validate this.

Based on David Peshkin’s survey (17), high traffic volume roads in rural areas may be as those roads with AADT greater than 5,000. INDOT maintains approximately 10,000 centerline miles of non-interstate roads. The lowest AADT was 15 on SR-166 and the highest AADT was approximately 80,000 on US-30. Figure 2.9 shows the percent distribution of the road length by AADT. Apparently, a large portion of the non-interstate roads carried 1,000 to 6,000 AADT, and more than 90% of non-interstate roads carried an AADT less than 12,000. In order to facilitate pavement engineers to select surface treatment and materials, the non-interstate roads are further grouped into three traffic levels, such as light, medium and high. Table 2.5 shows the total length of roads in each traffic level and

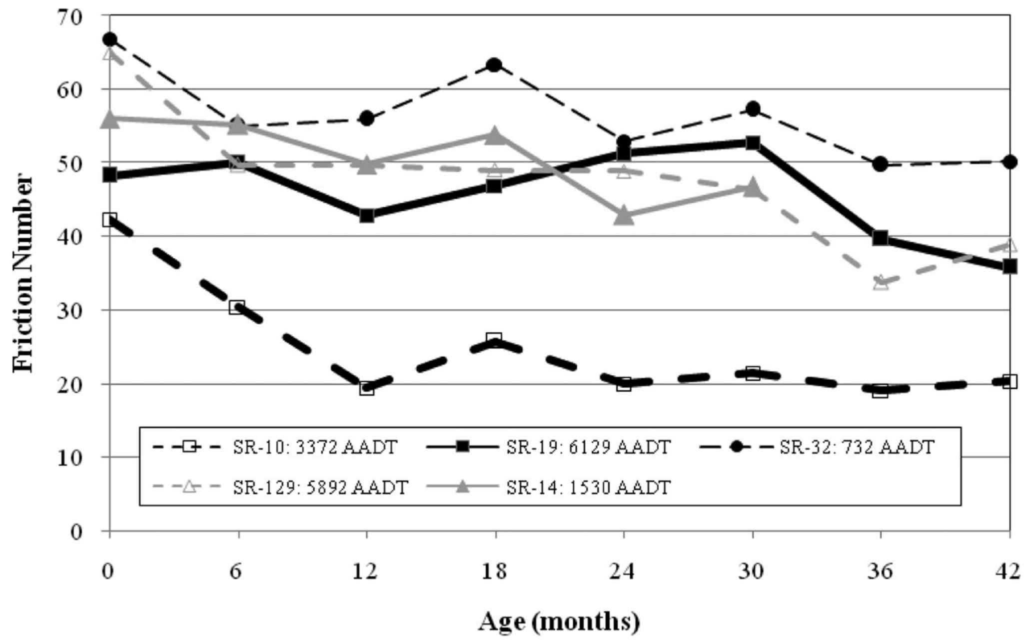


Figure 2.8 Friction Measurements in Chip Seal Test Sections with Different AADT

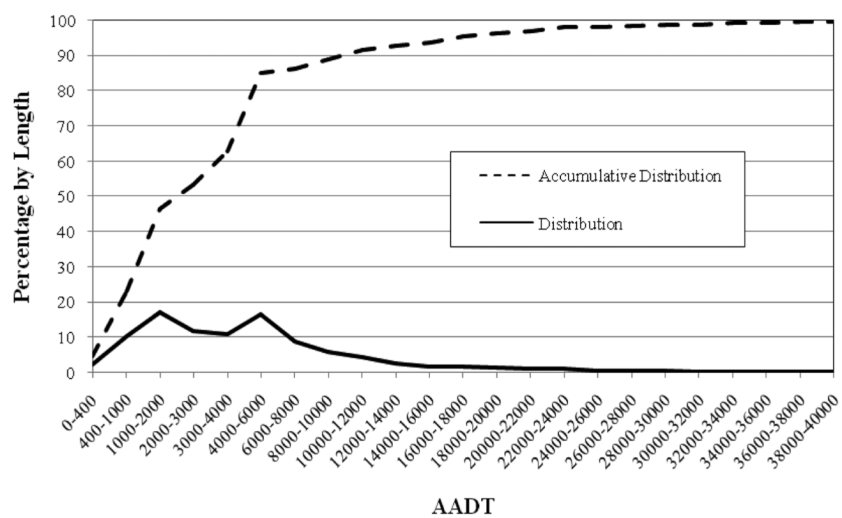


Figure 2.9 Percent Distribution of Total Road Length by AADT

associated percentage. It should be pointed out that the definition of traffic level is not a pure science and involves engineering judgment. However, the total length of the roads in each traffic level as shown in Table 2.5 accounts for approximately one third of the total length of all non-interstate roads.

Effect of Existing Pavement Condition

The ultimate consequence of chip seal or fog-chip seal failure is the significant decrease in surface friction that commonly occurs after 12 months in service. The typical friction number on failed chip seals or fog-chip seals is around 20 or less. The failure of chip seal or fog-chip seal is mainly due to the loss of a large amount of aggregate chips applied and bleeding arising from the asphalt emulsion retained on the pavement surface. The main causes for loss of aggregate chips include insufficient asphalt emulsion, inadequate binder adhesion, dirty aggregate chips, and fast traffic before the curing of asphalt emulsion is completed. Bleeding is mainly caused due to use of too much asphalt emulsion. The excessive asphalt emulsion will rise onto the surface

TABLE 2.5
Traffic Levels on INDOT Non-Interstate Roads

Traffic Level	AADT	Length (miles)	Percent by Length (%)
Light	0–2,000	2,965	29.6
Medium	2,000–5,000	3,226	32.2
High	>5,000	3,818	38.1

Figure 2.10 Photo of a Chip Seal on SR-10 after 36-Months of Service

in hot weather and accumulated on the pavement surface over time. Also, the loss of aggregate chips or insufficient aggregate coverage may result in a binder-rich surface, and therefore aggravate bleeding. Some researchers indicated that the better existing pavement condition, the longer the chip seal will last (18). Also, aggregate gradation also affects the friction performance. In general, the failure of a chip seal can be easily identified from the visual appearance of the chip seal as shown in Figure 2.10.

Presented in Figure 2.11 are the photos of the pavements before chip seal and the close-ups of surfaces in the wheel paths after chip seal for three test sections, including two chip seals on SR-10 and US-421, respectively, and one fog-chip seal on US-36. All the three test sections have experienced poor surface friction. As indicated in the close-up view on SR-10, the chip seal experienced significant loss of aggregate and bleeding. For the chip seal on US-421, there was no sufficient coarse aggregate, resulting in a smooth surface texture. Due to the use of wrong fog seal material in the test section on US-36, the fog-chip seal demonstrated not only insufficient cover aggregate, but also binder-rich surface. Table 2.6 shows the pavement conditions before chip or fog-chip seal in these three sections, and associated traffic volumes.

(a) SR-10 before Chip Seal

(b) SR-10 after Chip Seal

(c) US-421 before Chip Seal

(d) US-421 after Chip Seal

(e) US-36 before Fog-chip Seal

(f) US-36 after Fog-chip Seal

Figure 2.11 Photos and Close-ups of Three Chip Seals with Low Friction

TABLE 2.6
Pavement Conditions before Chip or Fog-chip Seal

Test Section	AADT/Trucks	IRI (in./mi)*	PCR*	Rut* (inch)
Chip seal, SR-10	3372/1592	144	89.6	0.13
Chip seal, US-421	4527/1062	93	93.0	0.08
Fog-chip seal, US-36	2085/329	111	80.8	0.18

*Taken from video log.

The old pavement on SR-10 demonstrated moderate roughness and light rutting and its overall condition was rated good. On US-421, the old pavement experienced fair roughness and very light rutting and its overall condition was rated excellent. The old pavement in the fog-chip seal section on US-36 demonstrated fair roughness and light rutting, and its overall condition was rated fair. Seemingly, chip seal may not be a right treatment for SR-10, taking into account the combined effect of old pavement condition and traffic. For the chip seal on US-421, it appears that the aggregate gradation contained only a small percentage of coarse particles. Therefore, the surface did not have sufficient resistance to traffic grinding. While the traffic was not high on US-36, the old pavement condition that was rated fair might have affected the performance of the fog-chip seal. While the contributing factors to the failures in these three sections were not readily apparent, care should be exercised when applying chip seal to a pavement when its overall condition level or surface roughness is rated fair or worse, or there is rutting >0.25".

CHAPTER 3 FOG SEAL AND REJUVENATING SEAL

General Description

Fog Seal

Fog seal is defined as a light spray application of diluted asphalt emulsion used to primarily to seal an existing asphalt surface to reduce raveling and enrich dry and weathered surfaces (19) (see Figure 3.1). The primarily goal of a fog seal is to coat, protect, and rejuvenate the existing asphalt pavement surface. The applied asphalt emulsion can also fill the voids and seal small cracks in the surface of asphalt pavement, and therefore improve the waterproofing and reduce permeability of the surface to air and water. Consequently, a fog seal can postpone major pavement rehabilitation and increase pavement service life. In addition, fog seal is frequently applied to the top of a fresh chip seal to hold the top layer of the stones in place and prevent possible aggregate loss (see chapter 2). During for seal application, slow setting, diluted asphalt emulsion is commonly used and applied to the asphalt pavement surface without adding any aggregate.

The asphalt emulsion used in fog seals can be positively charged (cationic) or negatively charged (anionic) (20). Cationic emulsions replace water from the aggregate surface or aged asphalt film and break due to loss of water and due to chemical action. Anionic emulsions, however, have no interaction with the aggregate surface and break purely due to the loss of water by evaporation and absorption through voids in the pavement. As indicated in Chapter 2, Indiana is an anionic emulsion state with two asphalt emulsion suppliers. Several disadvantages have been identified in connection with fog seals (21). First, slow setting emulsions are commonly used in fog seals, resulting in longer traffic delays. Second, asphalt emulsions wet the pavement surface, resulting in immediate reduction in surface friction. Therefore, the use of fog seals has been commonly limited to pavements with rough surface texture or high air voids so that sufficient friction and penetration of the asphalt emulsion can be provided.

Figure 3.1 Application of Fog Seal on US-36

Rejuvenating Seal

Rejuvenating seal is an application of rejuvenating agents, such as recycling agents, special chemicals or asphalt emulsions, to the asphalt pavement surface to adjust the properties of the aged or oxidized asphalt binders so as to restore the elasticity and flexibility of the hardened surface mixes (see Figure 3.2). Notice that while a fog seal can also soften the stiffness of aged asphalt pavement surface, the primary purpose of the fog seal is to prevent surface water penetration and repair minor to median surface distresses, such as raveling. The rejuvenation effect may be lost once the asphalt emulsion used in a fog seal breaks. However, the primary goal of a rejuvenating seal is to restore the elasticity and flexibility of the aged asphalt surface mixes. A rejuvenating seal can to some extent seal pavement surface cracking, increase the retention of surface aggregates, and retard block cracking or thermal cracking. As a result, rejuvenating seals can extend the service life of asphalt pavement surface.

The aging of asphalt materials occurs commonly in the top half inch of the surface layer (22). The maximum penetration depth by a typical rejuvenating

Figure 3.2 Application of Rejuvenating Seal on US-40 (Courtesy of Jusang Lee, INDOT)

agent that is commercially available may reach 3/8~1/2 inches (23). Therefore, the success of a rejuvenating seal depends to a large extent on the penetration depth of the rejuvenating agent into the asphalt pavement surface. The greater the penetration depth, the greater the effectiveness of the rejuvenation seal. If the rejuvenating agent cannot penetrate the pavement surface as expected, the whole rejuvenating seal will become ineffective and a slippery surface may be produced. Therefore, special care should be exercised to ensure surface friction before opening traffic after applying rejuvenating seals, particularly when conventional asphalt emulsions are utilized as a rejuvenating agent. Currently, the use of rejuvenating seals is limited to asphalt pavements in the age of two or more years. The pavements should be in good structural conditions and have experienced fairly oxidized surfaces. The pavement surfaces may demonstrate the signs of minor to medium distresses, including cracking, raveling, and pitting.

Test Sections

Very few fog seals have been directly applied onto existing asphalt pavements in Indiana and most of the fog seals were applied to fresh chip seals to hold aggregate chips in place. Some fog seals were applied to roadway shoulders. The application of a fog seal to a fresh chip seal is referred to as a fog-chip seal and has been discussed in Chapter 2. Similarly, there have been very few rejuvenating seals in Indiana over the past years. The primary reason is that improperly applied fog seals or rejuvenating seals may result in possible safety problems. It has been reported that the surface friction number may drop by 10–20 points for the first 72 hours after applying a fog seal or a rejuvenating seal (24). In addition, chip seals are a proven method and have been widely used to extend pavement service life. The secondary reason is probably due to lack of knowledge and experience on the effectiveness and potential benefits from fog seals and rejuvenating seals.

In order to fill the void of knowledge on fog seals and rejuvenating seals, INDOT started to experiment with fog seal and rejuvenating seal in its pavement preservation program in 2007. The experiment of fog seal was initially conducted on the shoulders on US-231 and US-36 in 2007. The experiment of rejuvenating seal was conducted in the driveway pavement on US-40. Presented in Table 3.1 is the general information on these three test sections. The shoulders are 10 feet wide on both US-36 and US-231, and their surfaces experienced both cracking and weathering. In the test section of rejuvenating seal on US-40, the pavement was resurfaced with a 1.5" HMA overlay in October, 2006. However, it was showing the early signs of distresses, such as premature oxidation, hardening and cracking. Due to low asphalt content, high dust content, and high absorption of dolomite aggregate, the surface became dry. The observed AADT was 2,362 with a truck percentage of 7.5% in 2007. The typical emulsion application rate is 0.10~0.15 gal/yd^2. The accuracy of emulsion application rate is $\pm$ 0.02 gal/yd^2.

Surface Friction Performance

Surface Friction on Fog Seal

After a fog seal is applied to an existing asphalt pavement, a liquid emulsion film will be formed on the pavement surface (see Figure 3.3). Even after the asphalt emulsion is thoroughly cured, the pavement surface may still be coated with a thin film of asphalt binder. As a result, the pavement surface friction is expected to decrease after applying a fog seal. Presented in Table 3.2 are the friction numbers measured before and after the application of a fog seal in the two fog seal test sections. It is shown that after applying a fog seal, the pavement surface friction experienced dramatic decreases in all three test sections. On US-36 westbound, the average friction number was about 61 before fog seal and decreased to 28 after fog seal. On US-231, the average friction number was 58 in northbound, and 56 in southbound before fog seal. After fog seal, the average friction number decreased to 23 and 25

TABLE 3.1
Summary of Fog Seal and Rejuvenating Seal Test Sections

Road	Application	Material	Lane	Length	AADT	Truck	Year Sealed
US-36	Fog seal	Emulsion	Shoulder	5.0 miles	15,635	8.0%	08/2007
U-231	Fog seal	Emulsion	Shoulder	3.1 miles	10,815	14.2%	10/2007
US-40	Rejuvenation	Reclamite	Pavement	1000 feet	2,362	7.5%	08/2007

Figure 3.3 Close-up of Pavement Surface after Fog Seal

in northbound and southbound, respectively. The friction number dropped by more than 50% in all three test sections, which exceeded more than 20 points in terms of the friction number. However, the standard deviations of the friction numbers also decreased by more than 60% in all three test sections. This may indicate that the variability of surface friction was significantly reduced after applying a fog seal. In other words, a fog seal may improve the pavement surface uniformity and provide more consistent surface characteristics.

Presented in Figure 3.4 are the surface friction measurements, including average friction number and standard deviation, made over time in the three fog seal test sections. It is shown that the average friction numbers follow a similar trend within the first two years in service in all three test sections. It took about 18 months for the surface friction to return to the original level. On US-231, however, the average friction number increased in northbound and decreased in southbound after two years in service. While the friction numbers on US-36 eastbound and westbound decreased after the first 18 months, the standard deviations demonstrated different trends after 24 months. This is probably due to the surface characteristics of existing pavement before fog seal. The shoulder surface on US-36 demonstrated greater surface friction than that on US-231. This indicates that the shoulder surface on US-36 is rougher than that on US-231. The above observations may imply that the maximum

TABLE 3.2
Friction Measurements before and after Fog Seal

	Before		After	
Test Section	Ave	Stdev	Ave	Stdev
US-36, EB	61	10.8	28	4.0
US-231, NB	58	9.9	23	3.5
US-231, SB	56	11.0	25	3.8

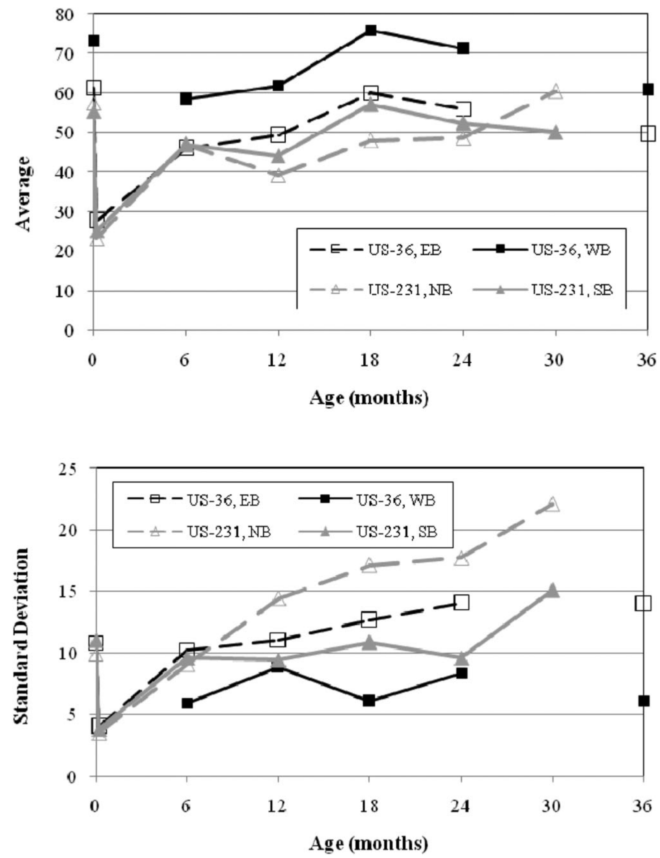


Figure 3.4 Friction Numbers Measured on Fog Seals over Time

friction life of a typical fog seal is around 2 years. The performance and effectiveness of a fog seal vary with the surface characteristics of existing pavement to a large extent.

Surface Friction on Rejuvenating Seal

After a rejuvenating seal is applied to an existing asphalt pavement and is thoroughly cured, the residue of rejuvenating agent will form a surface sealer on the existing pavement, and therefore, the surface friction is expected to decrease. While conventional asphalt emulsions can be used as rejuvenating agents, the rejuvenating chemicals commercially available, such as Reclamite®, have become popular in rejuvenating seals due to greater penetration depth and better effectiveness. As pointed out earlier, INDOT experimented with the rejuvenating seal of Reclamite® on US-40 in 2007. The Reclamite® diluted one part product to one part water (1:1) was applied at an application rate of 0.10 gal/yd^2. Sand blotters were applied at 1.0 lb/yd^2 to allow early opening to traffic. The Reclamite® sealed pavement (see Figure 3.5) was opened to traffic 1.5 hours (driving lane) and 2 hours (passing lane) after application. In order to evaluate the variation of surface friction on the freshly rejuvenated surface, friction testing was conducted right before opening to

Figure 3.5 Close-up of a Cured Rejuvenating Seal on US-40 (Courtesy of Jusang Lee, INDOT)

traffic, 0.5 hours after opening to traffic, and 24 hours after opening to traffic, respectively.

Plotted in Figure 3.6 are the friction test results on the existing pavement before application and on the freshly sealed pavement within 24 hours after applications, including average friction numbers and standard deviations. It is shown that the pavement surface friction experienced dramatic reduction after the application of Reclamite®. The average friction number dropped approximately from 74 to 39 in the passing lane and from 52 to 30 in the driving lane in the first two hours. However, the standard deviation experienced a dramatic increase in both the driving and passing lanes probably due to that the curing was not uniform in the test section and the effect of sand blotters. The surface friction increased slightly after the first 2 hours probably due to the combined effect of sand blotters and drying of the rejuvenating agent. Afterwards, the surface friction decreased by about 8 points within 24 hours due to the loss of sand blotters when the sealed surface became dry. The standard deviations also tended to decrease after two hours of application. The driving lane demonstrated greater standard deviation than the passing lane due to the additional application of sand blotters to prevent pick-up.

Figure 3.7 shows the friction variation of the sealed pavement over a 24-month period. The surface friction reduced immediately after application, and then increased to a peak value in 30 days. Afterwards, the surface friction tended to fluctuate over time. The above observations indicate that sand blotters can increase the surface friction of a fresh rejuvenating seal (by 8 points in this case), but may become ineffective in 1 or 2 days. The rejuvenating agent will dry out in about 1 month. The pavement surface friction is unable to return to its original level after the application of a

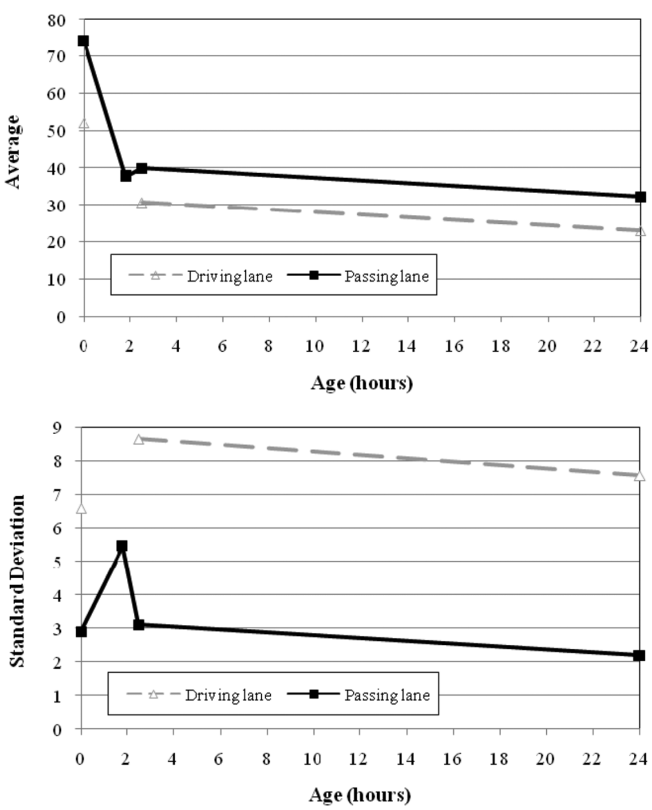


Figure 3.6 Friction Measurements on Reclamite® Seal within 24 Hours

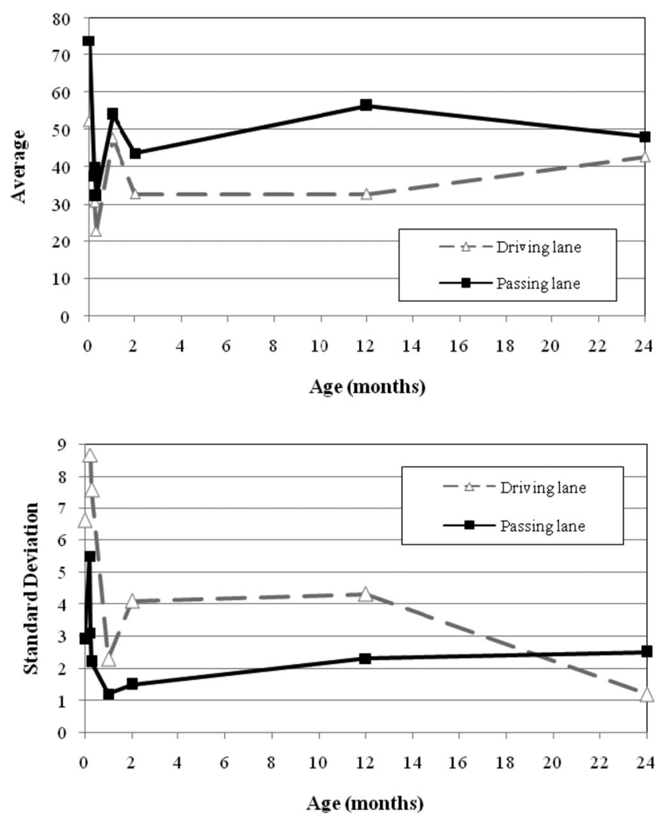


Figure 3.7 Friction Variation over a Two Year Period

rejuvenating seal. The friction dropped by more than 18% in the passing lane and 35% in the driving lane after 24 months of application. However, the standard deviation of the surface friction decreased in both the driving and passing lanes. A smaller standard deviation indicates less variability. Therefore, a rejuvenating seal may improve the uniformity of pavement surface.

CHAPTER 4 MICROSURFACING

General Description

Microsurfacing is the application of a mixture of polymer modified asphalt emulsion, crushed and graded aggregate, mineral filler (commonly Portland cement to improve strength), water, and other additives, which have been properly proportioned and mixed, onto an existing HMA pavement. Microsurfacing provides many benefits: sealing the existing pavement from further water intrusion; restoring surface friction; no adjustment of the curb line or manholes are required due to the thin lift height; rut filling; minor re-profiling and a desirable black appearance to the public. While its mix is prepared and paved using a slurry seal machine, microsurfacing differs from slurry seal in that slurry seal uses a standard, conventional asphalt emulsion, but microsurfacing uses a polymer-modified asphalt emulsion. As a result, slurry seal requires more curing time (several hours) depending on the weather and pavement conditions for water evaporation and the asphalt emulsion to break and to be fully cured. Unlike a standard asphalt emulsion, a polymer-modified asphalt emulsion produces chemical action to drive water out, resulting in less curing time (usually than one hour) and faster development of strength. Also, microsurfacing commonly uses higher quality aggregates. As a result, microsurfacing may provide durable performance on high traffic volume roads.

On the one hand, microsurfacing uses NMAS 4.75-mm dense-graded fine aggregates, which allows an application as thin as 3/8 inch without compaction (25). On the other hand, microsurfacing uses higher quality aggregates and poly-modified asphalt emulsion and produces fast setting and greater strength, which allows thicker (up to 1 inch) application on high traffic volume roads to correct wheel path rutting that may exceeds ¾ inch and enhances long-term pavement surface friction performance. Particularly, the use of polymer modified asphalt emulsion can not only improve aggregate retention and enhance resistance to cracking and traffic wearing, but also reduce thermal susceptibility, which may result in better long-term performance. The application of microsurfacing can be combined with other pavement preservation treatment such as chip seals to reduce aggregate loss, improve surface smoothness and provide desired pavement appearance. However, microsurfacing is currently used as a stand-alone preservation surface treatment by INDOT. INDOT started to formally experiment with microsurfacing in 2007 and is now still on the learning curve to fully utilize microsurfacing as a preservation treatment.

The Pilot Microsurfacing Projects

The Selected Test Sections

INDOT started to formally experiment with microsurfacing in 2007. Afterwards, several more microsurfacing projects have been completed for the purpose of field assessment. Presented in Table 4.1 is the information, including road, approximate length, traffic volume and construction completion date, on six test sections that were selected for evaluating the surface characteristics of microsurfacing. All these six test sections are located on two-lane highways. The first two pilot microsurfacing projects are these two projects, one on SR-22 and the other on SR-3. The test section on SR-3 is a resurfacing project running through the City of Rushville, consisting of 15 junctions with local town streets. The test section on SR-56 is 11.5 miles long, of which, one portion (about six miles long) is located out of the City of Madison, and the remaining portion is located in the City of Madison. The former experienced an AADT of 2,267 with 246 trucks and the latter experienced an AADT of 10,320 with 679 trucks. The greatest AADT of 15,596 was observed on SR-22 and the greatest truck traffic of 1,501 was observed on SR-3. Basically, these six test sections covered a wide range of AADT on non-interstate highways.

Requirements for Microsurfacing Mixes

The polymer modified asphalt emulsion specified by INDOT is a quick-set, CSS-1H emulsion. The minimum polymer solids content is 3.0% based on the residual of the emulsion. Special additives are required to provide control of the quick-set properties. The coarse aggregates of Class B or higher is required for microsurfacing mixes. For rut filling, the required coarse aggregates include limestone, dolomite, crushed gravel, sandstone, SF, or ACBF. The fine aggregates for microsurfacing are the same as those for HMA surface mixes, including limestone, dolomite, crushed gravel, sandstone, SF, ACBF or Polish resistant aggregate. When used for leveling application, the selection of fine aggregate type is based on the ESAL category. Summarized in Table 4.2 are the main quality requirements for both coarse and fine aggregates used in microsurfacing. Table 4.3 shows the requirements for aggregate gradations used in microsurfacing, including leveling and rut filling applications. Portland cement of

TABLE 4.1
Summary of Six Microsurfacing Test Sections

Road	Length	AADT (2007)	Truck	Completion
SR-22	0.8 mi.	15,596	500	10/2007
SR-3	1.1 mi.	11,837	1,501	07/2007
SR-28	0.4 mi.	6,578	732	09/2007
SR-70	9.3 mi.	1,744	206	08/2008
SR-56	11.5 mi.	2,267/10,320	246/679	09/2008
SR-227	7.0 mi.	1,964	77	10/2009

TABLE 4.2
Aggregate Properties for Microsurfacing

Requirements	Coarse Aggregate	Fine Aggregate
Aggregate Class	B or Higher	–
Los Angeles Abrasion, %, Max.	40	–
Freeze and Thaw Soundness, %, Max.	12	10
Sodium Sulfate Soundness, %, Max.	12	10
Brine Freeze and Thaw, %, Max.	30	12
Crushed Particles, %m Min.	70	–
Aggregate Angularity, %, Min.	95	45
Sand Equivalency, %, Min.	–	60

Type I is required to be used as the mineral filler. The detailed information on the quality requirements for microsurfacing materials can be found elsewhere (14,15).

Surface Friction and Texture

Friction on Freshly Placed Microsurfacing

A freshly placed microsurfacing can be opened to traffic after adequate cohesion has been developed to resist traffic abrasion. This commonly occurs when the microsurfacing surface has turned black (26). Rolling with pneumatic rollers is not necessary but may be utilized to reduce aggregate loss. Presented in Figure 4.1 are the friction numbers measured on four of the six test sections right after opening to traffic. Two main observations can be made from Figure 4.1. First, it is apparent that the freshly placed microsurfacing pavements in the test sections produced sufficient surface friction. The lowest friction number was 28 on SR-22 westbound and the greatest friction number was 57 on SR-56 eastbound. Figure 4.2 shows a photo of a fresh microsurfacing pavement. While the surface had not dried out completely, the surface friction was sufficient to withstand traffic when opening to traffic.

Second, the surface friction on the freshly placed microsurfacing pavement varied significantly from test section to test section. This is probably due to the effect of curing process. The state of curing not only affects the development of mix strength, but also affects the surface properties. In reality, it has been pointed out that a microsurfacing pavement will not lose all water

TABLE 4.3
Aggregate Gradation for Microsurfacing

Sieve Size (mm)	Leveling	Rut Filling
9.5	100	100
4.75	85–100	70–90
2.36	50–80	45–70
1.18	40–65	28–50
0.6	25–45	19–34
0.3	13–25	12–25
0.15	7–18	7–18
0.075	5–15	5–15

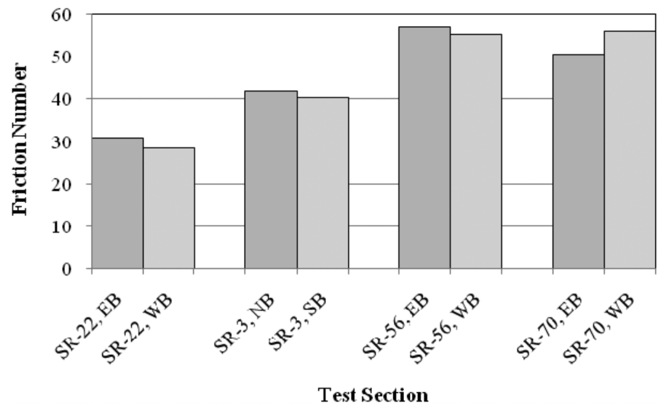


Figure 4.1 Friction Numbers on Freshly Placed Microsurfacing Pavements

in the first hours after placement (26). It may take up to several weeks for the total water loss process to end, depending on the weather and existing pavement conditions. While the placement of the microsurfacing in one direction is always earlier than that in the other direction, the friction numbers on the freshly placed microsurfacing were very consistent in both directions in each test section. Therefore, a freshly placed microsurfacing pavement can not only provide sufficient surface friction, but also produce consistent surface properties and early opening to traffic. This confirms the benefits to use polymer in microsurfacing application.

Friction Variation over Time

As curing proceeds, the strength of microsurfacing mix develops and the asphalt emulsion on the surface dries out. Presented in Figure 4.3 are the friction numbers measured in five of the six test sections over time. No friction variation was measured on SR-3. As mentioned earlier, this section consists of 15 junctions. It was very hard to conduct the locked wheel friction testing without traffic control. In addition, some data

Figure 4.2 Photo of a Fresh Microsurfacing Pavement

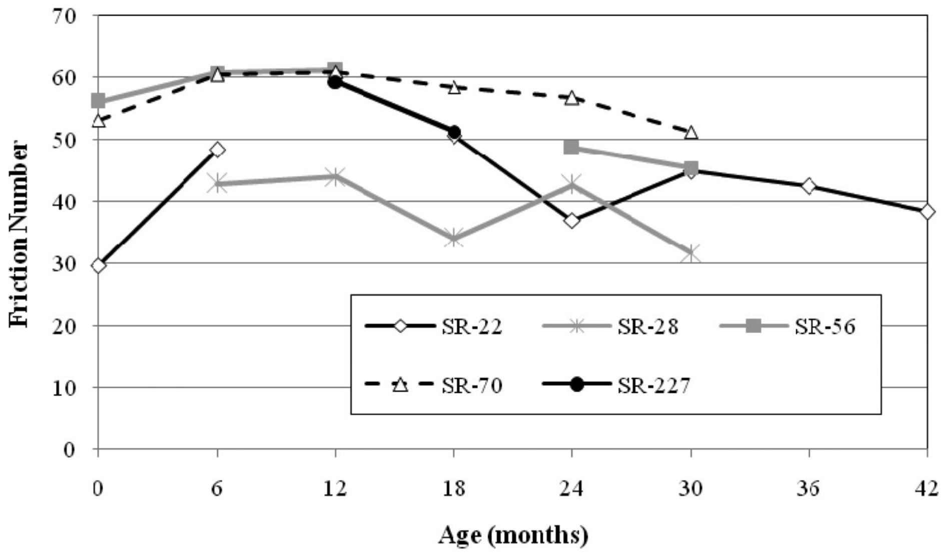


Figure 4.3 Friction Variations on Microsurfacing Pavements over Time

such as the data on SR-22 after 12 months of service and on SR-56 after 18 months of service, respectively, is not available because the research team was unable to conduct testing due to other on-going major road works. It is illustrated that in Figure 4.3, the surface friction of microsurfscing pavement increased significantly in the first six months, and reached the maximum number approximately after 12 months of service. This indicates that the surface of a typical microsurfacing pavement became stable and produced true friction numbers around 40–60 after 12 months of service.

Afterwards, the surface friction number decreased over time. Traffic volume had an impact on the variation of surface friction. The surface friction in the test sections with high traffic volumes, particularly truck traffic, decreased more than that with light traffic volume. Also, it appears that the surface friction in the microsurfacing pavements has always decreased after 12 months of service and tended to decrease faster over time. However, no friction number less than 30 occurred in all of the five test sections. A microsurfacing pavement commonly uses a 4.75-mm dense-graded fine aggregate mix. In reality, the aggregate gradation for the microsurfacing leveling mix is very similar to that for a conventional 4.75-mm HMA dense-graded fine aggregate mix. However, the use of polymer modified asphalt emulsion and special additives provides the microsurfacing mix enhanced surface properties.

Surface Macrotexture

A 4.75-mm dense-graded fine aggregate mix usually lacks of coarse aggregates. One of the most typical characteristics associated with this type of mix is probably the poor surface macrotexture properties. However, the strength mechanism for a microsurfacing mix is different from that for a conventional HMA mix. For a microsurfacing pavement, the rheological properties of asphalt emulsion residue improve significantly. The microscopic honeycomb structure of flexible cement-polymer formed in a microsurfacing pavement plays a critical role in early strength and rutting resistance (27). Shuo et al. observed that HMA surfaces with rutting issues also tended to experience low friction performance (28). Also, the polymers adhering to the aggregate surface may also improve the properties of pavement surface texture.

Presented in Figure 4.4 are the close-up views of the microsurfacing pavements in two test sections. The surface on SR-227 demonstrated aggregate particles protruding from the surface. The surface on SR-70 demonstrated angular aggregate particles. Both surfaces produced coarse textures. Presented in Table 4.4 are the MPD of surface macrotexture measured in the right wheel path in these four test sections, respectively. The greatest MPD is 0.94 mm that occurred on SR-70. The lowest MPD is 0.66 mm that was witnessed on SR-56. When compared to the macrotexture depths of conventional HMA mixes presented in Chapter 6, the microsurfacing pavements in these test sections produced surface texture much greater than those on conventional 4.75-mm dense-graded HMA pavements and greater than those on conventional 9.5-mm HMA pavements.

Smoothness, Distress and Noise Performance

Pavement Smoothness

Field testing was conducted to measure surface longitudinal profiles (29) using an inertial profiler system (30) and the international roughness index (IRI) (31) was computed to assess the smoothness of the surface in each test section. During testing, the

(a) SR-227 (b) SR-70

Figure 4.4 Close-up Views of Microsurfacing Pavement Surfaces

TABLE 4.4
Macrotexture Measurements

Test Section	Pavement Age	MPD (mm)
SR-28	30 months	0.86
SR-56	30 months	0.66
SR-70	30 months	0.94
SR-227	18 months	0.67

longitudinal profiles were measured in both the right and left wheel paths in each direction. Figure 4.5 shows the IRI values measured before and after placing microsurfacing in three test sections located on SR-28, SR-70, and SR-227, respectively. On the x-axis marked using three letters, the first letter indicates the direction of road, the second letter the wheel path, and the third letter the testing time. As an illustration, ERB indicates eastbound, right wheel path and before microsurfacing. On the freshly placed microsurfacing, the surface smoothness varied between right and left wheel paths and between different directions. The IRI increased in two situations, one in the right wheel path on SR-28 eastbound and the other in the right wheel path on SR-227 northbound. However, the surface smoothness improved in most situations after placing microsurfacing. The greatest improvement occurred on SR-70 westbound with a decrease in IRI by 26%. Table 4 shows the IRI measurements made right after opening to traffic and in 2011. The test section on SR-28 experienced the greatest change in smoothness and the IRI increased by approximately 9 points each year. IRI increased by 1 to 6 points each year in other test sections. The IRI increased in proportion to traffic volume.

The above observations indicate that the smoothness improvement from microsurfacing depended to some extent on the smoothness of existing pavement. The rougher the existing pavement surface the greater the smoothness improvement after placing microsurfacing. However, the effectiveness of microsurfacing in enhancing surface smoothness was limited, particularly when the smoothness of existing pavement was in good condition. This may be due to that microsurfacing in Indiana was commonly placed in two courses, including leveling and surface courses. The thickness of microsurfacing was ultrathin, not greater than 3/8” (9.5 mm). In addition, the application of microsurfacing was commonly accomplished without compaction, which might also affect surface smoothness. It is also indicated that the surface smoothness in the left wheel path was always better than that in the right wheel path after microsurfacing. Notice that IRI was not obtained on other surface treatments, only on microsurfacing,

Surface Distress

Visual inspection was conducted twice a year to identify surface distress throughout the study period. It has been observed that in the early stage, one of the main distress modes is delamination in four of these six microsurfacing test sections on SR-22, SR-28, SR-70, and SR-227 (see Figure 4.6). The greatest delamination size was 26×16 inches on SR-70. While the exact cause of surface delamination in the early stage is not fully understood, the evidences identified in the field visit point to traffic wear and bonding. The two test sections on SR-22 and SR-28 have experienced high traffic volume. In particular, these two test sections are located in downtown Kokomo and Tipton, respectively, and consist of several signalized intersections. As a result, great horizontal forces might arise due to frequent starting and braking of vehicles and cause damage to the pavement surface. In addition to delamination, cracking was also observed in these two sections. Care should be exercised when microsurfacing is used on high traffic volume roads, particularly in urban areas.

The test sections on SR-70 and SR-227 are both carrying light traffic. However, delamination occurred approximately in July 2011, after 36 months of service on SR-70 and 22 months of service on SR-227. It was observed that the microsurfacing delamination commonly started at the interface between the old pavement surface and microsurfacing and the old pavement surface was exposed in the affected area after surface delamination occurred. Seemingly, it is probably the bonding at the interface that might have caused surface delamination. Reflective cracking was also observed in the microsurfacing test sections, particularly on SR-56. The test section on SR-56 is divided into two segments

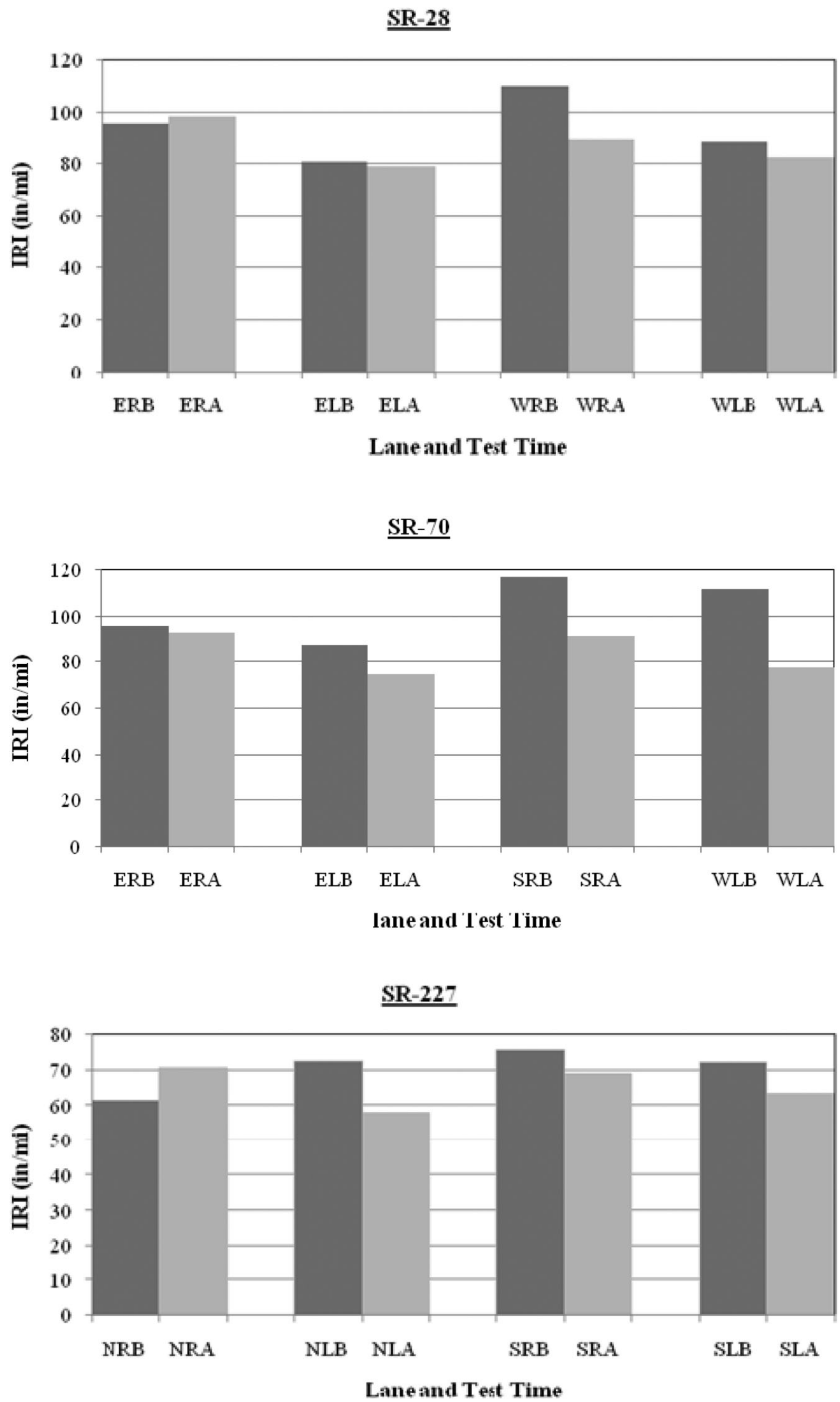
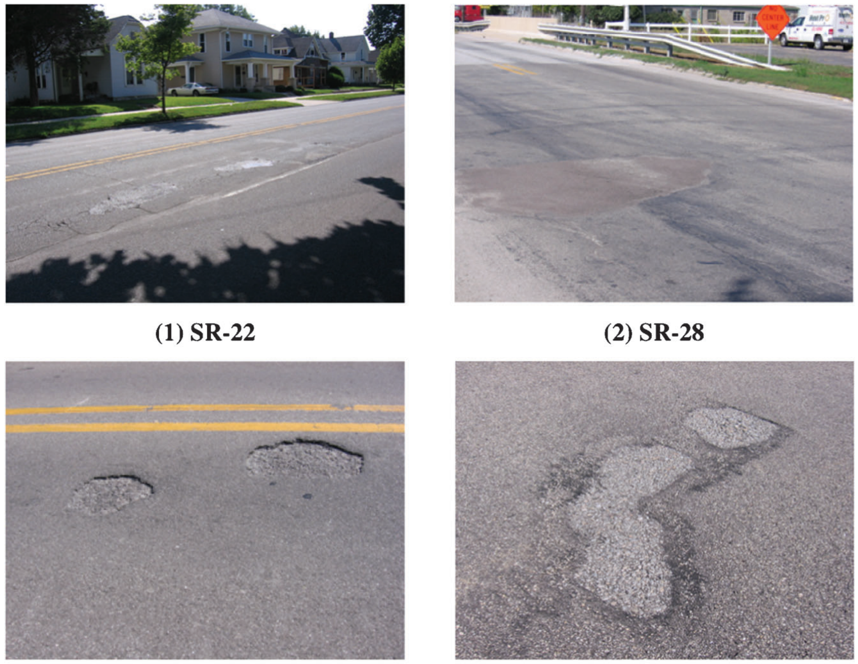


Figure 4.5 IRI Measurements in Three Test Sections

by SR-62. The first segment between RP 125.7 and SR-62 carries light traffic and the second segment from SR-62 to RP 137.8 carries high volume traffic. It was observed the microsurfacing in the first segment was in excellent condition. However, the microsurfacing n the second segment demonstrated many transverse cracks with uniform spacing. It seems that microsurfacing could not resist reflective cracking, in particular on high traffic volume roads.

Noise Level

One possible concern with microsurfacing is the increased traffic noise that has become an ever-increasing problem. As demonstrated earlier, a micro-surfacing pavement tends to produce much greater surface texture than a typical conventional HMA pavement. Therefore, the tire-pavement interaction becomes more intense and may result in greater tire-



(1) SR-22 (2) SR-28

(3) SR-70 (4) SR-227

Figure 4.6 Delamination in the early Stage in Microsurfacing

(1) RP 125.7-SR-62 (2) SR-62-RP 137.81

Figure 4.7 Photos of Microsurfacing Test Section on SR-56

pavement noise. One study by Georgia Department of Transportation concluded that microsurfacing pavement produced a minimal increase in pavement noise levels (32). In order to further evaluate the acoustic performance of microsurfacing pavements, noise testing was conducted to measure both pass-by on the roadside and in-vehicle noise levels in the microsurfacing test section on SR-227. For comparison purpose, noise testing was also conducted in the adjacent test section of ultrathin 4.75-mm HMA overlay on SR-227. A sound level meter of ASNI Type I was utilized to record nose measurements. While measuring the pass-by noise levels on the roadside, the microphone was positioned at a height of 1.5 meters and located at a distance of 2.3 meters from the pavement edge marking.

Presented in Tables 4-5 and 4-6 are the three noise descriptors, including the equivalent sound level (L_{Aeq}), the sound exposure level (L_{AE}) and the maximum A-weighted sound level (L_{AFmax}), taken in the microsurfacing and 4.75-mm HMA pavement test sections. In general, L_{Aeq} is used to describe continuous sounds and L_{AE} or L_{AFmax} is used to describe the sound of individual vehicle pass-bys (33). L_{Aeq} and L_{AE} on microsurfacing are very close to those on 4.75-mm HMA overlay. The greatest difference is 1.9 dB, which arose in L_{AFma}. The in-vehicle noise testing was conducted using a passenger car, 2007 Chevrolet Malibu LS. Presented in Table 5.5 are the in-vehicle noise levels measured at 40 mph and 55 mph, respectively. The noise level increased as speed increased. At the same speed, the noise levels measured on the microsurfacing and 4.75-mm HMA overlay are very close. The greatest difference is 1.4 dB, which occurred in L_{Aeq} at 40 mph. Apparently, the noise changes between microsurfacing and 4.75-mm HMA overlay are not perceptible on the roadside and in the vehicle.

TABLE 4.5
Roadside Noise Levels

Surface Type	Noise Measurement, dB(A) L_{Aeq}	L_{AE}	L_{AFmax}
Microsurfacing	69.7	98.7	93.3
4.75-mm HMA	69.2	98.3	91.4

TABLE 4.6
In-Vehicle Noise Levels

Surface Type	Speed, mph	Noise Measurement, dB(A) L_{Aeq}	L_{AFmax}	L_{AFmin}
Microsurfacing	40 mph	63.6	68.5	55.5
	55 mph	66.1	70.9	60.3
4.75-mm HMA	40 mph	62.2	68.9	55.9
	55 mph	64.8	71.8	59.1

CHAPTER 5 ULTRATHIN BONDED WEARING COURSE

General Description

Ultrathin bonded wearing course (UBWC) is a high-performance surface course placed on an existing asphalt pavement. A UBWC can be utilized for preventative maintenance or new construction. When used for preventative maintenance, a UBWC can seal the existing pavement, fix minor to moderate surface defects such as cracking, bleeding, rutting and raveling, restore surface friction, enhance ride quality, improve noise performance, and reduce back-spray in wet weather. A typical example of UBWC is the so-called NovaChip® system (34) that is a thin layer of high quality aggregate, gap-graded HMA over a heavy tack coat as shown in Figure 5.1. The thickness of UBWC varies from 3/8 inch to 3/4 inch depending on the maximum size of the aggregate used. The tack coat is a polymer modified emulsion membrane called NovaBondⓉⓜ that is applied at a rate of 0.13–0.3 gal/yd^2. It is claimed that UBWC combines the benefits of both stone matrix asphalt (SMA) and open-graded friction course (OGFC). The main advantages associated with UBWC are durable surface, rapid construction, and quick opening to traffic. Other reported advantages include superior bonding to the underlying surface, and lower life-cycle cost.

Figure 5.1 Photo of a Freshly Placed UBWC

It is well known that UBWC was originally developed as an ultrathin friction course in France in 1986 (35) and first introduced to US in Texas and Alabama in 1992 (36,37). UBWC is placed using a special paving machine that applies both asphalt emulsion and hot mix in a single pass. The hot mix is produced at the asphalt mix production plant and delivered to the UBWC paving machine. Immediately after the NovaBondⓉⓜ is sprayed onto the existing pavement surface, the hot mix is distributed over the NovaBondⓉⓜ membrane. The compaction is accomplished with the vibratory screed of the paver, followed by one or two static passes from steel drum roller of ten tons. Notice that the ultimate goal of compaction is to embed the aggregate into NovaBondⓉⓜ membrane. Also, it is the authors' opinion that the compaction can reorient the aggregate particles to ensure surface smoothness. It has been claimed that a UBWC pavement can resist wear and rutting for 10 years or longer (34).

The UBWC Test Sections

The Test Sites

The first UBWC experimented by INDOT was placed as surface treatment on a conventional road, i.e., US-40 in 2007, and then, three more UBWC pavements were placed on three different state roads, including SR-3, SR-11, and SR-114 in the subsequent years for further evaluation. Provided in Table 5.1 is the general information, including AADT and truck traffic volume in 2007, existing pavement condition, and construction date for the four UBWC test sections. It is shown that all test sections except for the one on SR-11 carried high volume traffic. The greatest AADT is 17,401 that occurred in the test section on US-40. The greatest truck traffic volume is 700 that were observed in the test section on SR-3. As shown in Table 5.1, the existing pavements on SR-3, SR-11, and US-40 were in either good or very good condition prior to the placement of UBWC. The best pavement condition rating (PCR) was 94 in the test section on SR-114,

TABLE 5.1
General Information on UBWC Test Sections

Road	Length	AADT (2007)	Truck	PCR	Completion
US-40	0.5 mi.	17,401	300	87	04/2007
SR-114	0.9 mi.	8,755	498	94	07/2008
SR-3	12.5 mi.	9,721	700	92	06/2009
SR-11	8.0 mi.	1,047	44	80	05/2010

TABLE 5.2
Job Mix Formulas for the UBWC Mixes in the Test Sections

Sieve Size (mm)	US-40	SR-114	SR-3	SR-11
12.5	100	100	100	100
9.5	90	91	86.1	87
4.75	39	36	34.5	37
2.36	20	23	27.8	20
1.18	15	17	19.3	16
0.6	11	12	12.6	11
0.3	9	9	8.1	8
0.15	8	7	5.8	7
0.075	5.4	4.9	4.8	5.4
AC Content	5.2	5.2	5.3	5.9
PG Grade	70–28	70–28	70–28	70–28

which received a Partial 3R HMA overlay in 2004. The PCR for the test pavement on SR-11, which received a Partial 3R HMA overlay in 1999, was 80 that is the dividing point between fair and good conditions. No rutting severer than medium rating was observed in all four test sections. IRI varied between 70 and 117 inches per mile. Cracking, including transverse cracking and longitudinal cracking, was observed particularly in the test section on SR-11.

Mix and Material Properties

Presented in Table 5.2 are the job mix formulas for the UBWC mixes in the four test sections. It is shown that these four UBWC mixes shared some common characteristics. First, the nominal maximum aggregate size (NMAS) is 9.5 mm for all four mixes. Second, all mixes used PG 70-28 binders that seem to emphasize the low-temperature performance. Third, coarse aggregate accounted for more than 60% of the total mix weight. Figure 5.2 shows the aggregate gradations of these four mixes. Apparently, the aggregate gradations are very similar for the four mixes. It should be pointed out that the aggregate gradations are not simply gap-graded or open-graded. The gradation for aggregate greater than 4.75 mm is more like aN SMA mix but contains less aggregate above the 9.5-mm sieve. The gradation for the aggregate smaller than 2.36 mm is more like the combination of SMA and OGFC mixes. All these gradation curves are concave curves passing far below the maximum density line (MDL). A just-noticeable difference arises around the intermediate sieve (2.36 mm). The gradation curve between the 2.36-mm and 4.75-mm sieves for the mix on SR-3 is flatter than those for the other mixes.

UBWC Friction Performance

Friction on Fresh UBWC

As pointed out earlier, one of the reported main advantages associated with UBWC is quick opening to traffic. To allow the road quick reopening to traffic, the pavement surface has to provide not only sufficient stability and strength, but also sufficient surface skidding resistance to ensure travel safety. This is because the surface may be coated with a thin film of asphalt binder, resulting in low friction number when opening to traffic. This study conducted friction testing on the freshly placed UBWC pavements and the results are presented in Table 5.3. The lowest friction number is 48 in the test section on US-40, eastbound (EB), and the greatest friction number is 63 in the test section on SR-3, northbound (NB). Also, all test sections except

TABLE 5.3
Friction Numbers on Fresh UBWC Pavements

Test Section	US-40	SR-114	SR-3	SR-11
EB or NB	48	56	63	58
WB or SB	49	52	62	59

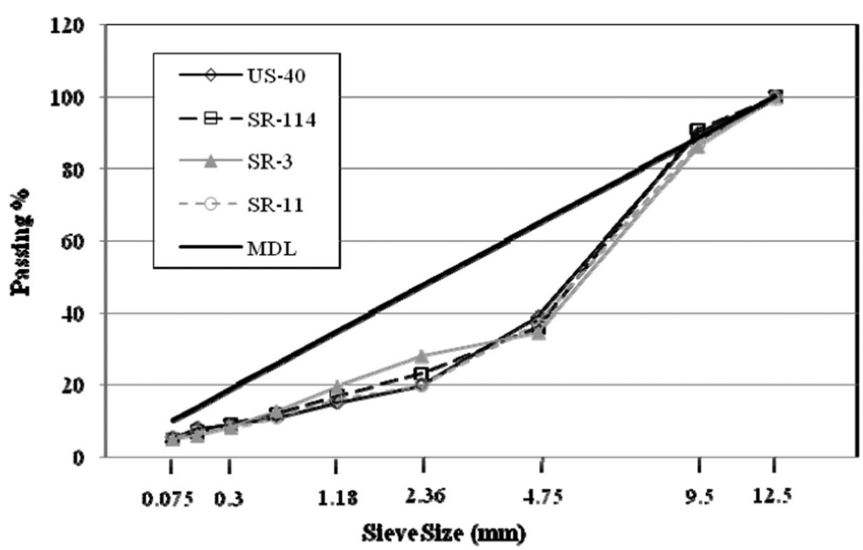


Figure 5.2 Aggregate Gradations of the UBWC Mixes in the Test Sections

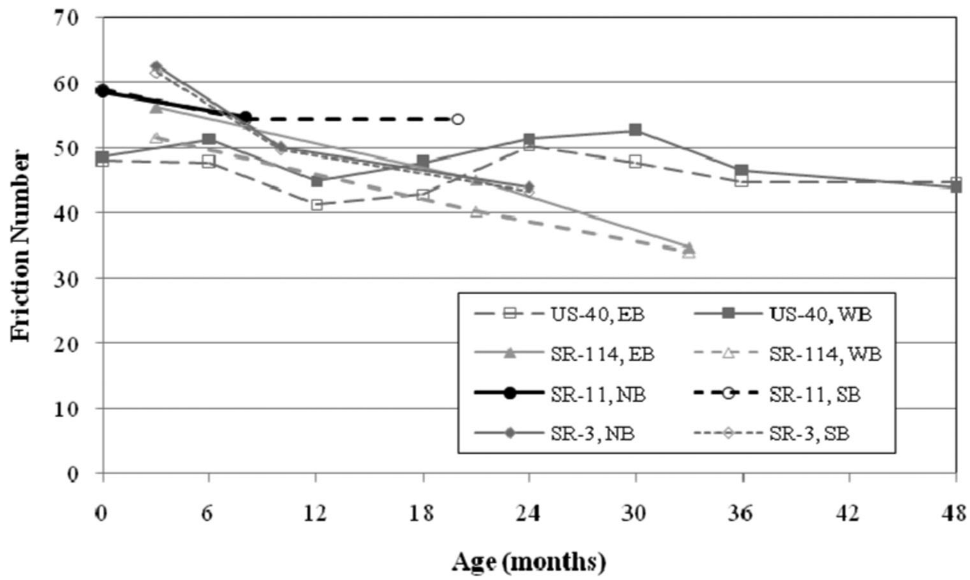


Figure 5.3 Friction Variation on UBWC Pavements over Time

for the one on SR-114 provided consistent friction performance in both directions. This indicates that UBWC is capable of providing sufficient and consistent skidding resistance to allow quick opening to traffic.

Friction Variation over Time

Presented in Figure 5.3 are the friction numbers measured over time in the four test sections. Once again, it is demonstrated that the friction numbers were close to 50 or greater on the new UBWC pavements. Further observations can be made on the variation of friction over time through careful inspection of the friction numbers. First, the friction numbers tended to peak after 6 months of service or earlier. This is approximately 6 months earlier compared to conventional HMA mixes. Second, it appears that UBWC has the potential to provide durable friction performance. As illustrated by the variation of friction on US-40, the friction number peaked after 6 months of service, and then fluctuated over time. After 4 years of service, the friction number decreased by about 8.3%, from 48 to 44. To the authors' knowledge, this decrease is almost negligible compared to the magnitude of friction number produced by UBWC pavements. Third, the surface friction demonstrated an ever-decreasing trend on SR-114, SR-3, and SR-11. It decreased by more than 34% on SR-114 after 33 months in service and 30% on SR-3 after 24 months in service. If this trend continues, concerns over the long-term friction performance may arise. Apparently, this contradicts the observation made over the friction variation on US-40.

Factors Affecting UBWC Surface Friction

On the one hand, UBWC demonstrated a durable friction performance surface friction on US-40. On the other hand, UBWC demonstrated a noticeable decrease in friction on SR-114 and SR-3. In order to investigate the contradicting results, the authors examined the UBWC job mix formulas in these four test sections. It was found that the type of coarse aggregate might play a critical role in the long-term friction performance. Presented in Table 5.4 are the types of coarse aggregate used in these four test sections. For the UBWC mix on US-40, the coarse aggregate consisted mainly of steel slag that accounted for 72% of total aggregate by weight. However, the coarse aggregate consisted mainly of limestone or dolomite for the UBWC mixes on SR-114, SR-3 and SR-11. Noticeable polishing had already occurred to the coarse aggregate on SR-114 as shown in Figure 5.4. In reality, it was pointed out that by Li et al. (38), limestone and dolomite demonstrated low British pendulum number (BPN) after polishing for 10 hours even though dolomite presented good initial BPN. The use of steel slag could enhance the long-term friction performance for HMA mixes.

As shown in the preceding sections, the aggregate gradation for UBWC mixes is very close to those for SMA and OGFC mixes. One of the common features for this type of aggregate gradation is that there is a large amount of coarse aggregates. Consequently, there are no enough fine aggregates to fill the voids created by coarse aggregates and the resulting mix tends to have large air voids. For example, the air voids in the UBWC mix on SR-11 is about 10%. As shown in Figure 5.5 is the surface of a freshly placed UBWC.

TABLE 5.4
Aggregate Sources in UBWC Test Sections

Agg. Type	US-40	SR-114	SR-3	SR-11
Steel Slag	72%	–	–	–
Dolomite	10%	–	77%	–
Limestone	–	80%	–	73%

Figure 5.4 Close-up of Polished UBWC Surface on SR-114

Figure 5.5 Close-up of Fresh UBWC Surface

Apparently, the UBWC provides coarse surface. This study conducted pavement texture testing on the UBWC surface on SR-3. The measured MPD varied between 0.95 mm and 0.99 mm, which is much greater than that for conventional 9.5-mm HMA mixes and equal to that for SMA mixes (see Chapter 6). Therefore, a fresh UBWC surface usually has good surface friction.

However, a UBWC pavement requires durable, highly polish-resistant coarse aggregate, particularly on high traffic volume roads. This is because when a vehicle tire applies onto a UBWC pavement, the contact area between the tire and pavement surface is smaller than that on a conventional HMA pavement due to a larger air voids in the UBWC mix. The contact interaction which mainly occurs between the tire and coarse aggregate surface also becomes more intense. Therefore, the aggregate surface may become polished quickly under the wear of vehicle tires. As a result, the pavement surface texture may be reduced, which will result in a decrease in surface friction as demonstrated in the UBWC test sections on SR-114, SR-3, and SR-11. In reality, the friction test results presented in the preceding sections have already indicated that a UBWC mix does not necessarily provide friction advantage over conventional gap- or open-graded HMA mixes. In order to provide satisfied long-term friction performance, it requires highly durable, highly polish-resistant aggregates.

CHAPTER 6 THIN 4.75-MM DENSE-GRADED HOT MIX ASPHALT OVERLAY

General Description

Thin overlay is referred to as a hot-mix asphalt (HMA) overlay with a thickness of 1.5 in. or less over an existing HMA. A thin HMA overlay not only addresses pavement surface defects, such as cracking and rutting, but also enhances ride quality (smoothness), reduce surface permeability and improve noise performance. In recent years, thin overlays with 4.75-mm dense-graded HMA mixes are increasingly catching the attention in pavement preservation as an effective alternative to preservation surface treatments, such as microsurfacing or slurry seal. Thin 4.75-mm dense-graded mixes utilize both conventional manufacturing facilities and construction equipment. In addition, the use of 4.75-mm dense-graded mixes promotes the use of aggregate screenings treated as waste materials and helps mitigate the environmental issues due to the disposal or stockpiling problems (39,40).

In the past, many DOTs and contractors have been reluctant to use 4.75-mm HMA mixes simply because small aggregate size mixes are more prone to rutting than coarse aggregate size mixes. However, the use of thin HMA overlays in pavement preservation requires fine aggregate size mixes due to the limit of lift thickness. While the test track studies by several organizations (41,42) indicated that 4.75-mm mixes can serve well and provide good rutting performance, reluctance remains due to that there is very little successful experience with the use of 4.75-mm dense-graded mixes, particularly on high traffic volume roadways. The current practice of 4.75-mm mixes by several DOTs has been limited to the applications on low traffic volume roads or with higher grade asphalt binders modified using polymers (39,40,43–45). Many issues have not been well addressed so far.

It was believed that 4.75-mm dense-graded mixes tend to exhibit low surface texture due primarily to their small aggregate sizes (46). However, only one study by Stacy et al. has been reported to evaluate the friction performance of 4.75-mm HMA mixes (47). Little effort has been made nationwide to investigate the friction performance of 4.75-mm HMA mixes. A pavement surface consisting of such a mix may not only experience low surface friction, but also produce opportunity for hydroplaning in wet weather. In order to evaluate the long-term friction performance of conventional 4.75-mm mixes, INDOT placed an experimental pavement section on I-465 in 2006. The friction testing conducted 12 months later indicated that this experimental section had experienced low friction numbers. Consequently, INDOT revised the 4.75-mm mix specifications and laid three more test sections on three different conventional highways, two

TABLE 6.1
General Information on Four 4.75-mm HMA Pavement Sections

Road	No. of Lanes	Length	AADT	Trucks	Construction Time
I-465	6	15.3 miles	104,593	19,475	08/2006
US-27	2	4.0 miles	7,735	741	09/2009
SR-227	2	3.0 miles	1,964	77	09/2009
SR-29	2	9.0 miles	5,552	1,215	08/2010

in 2009 and one in 2010. Pavement friction testing has been conducted over a period of up to five years to monitor the surface friction performance. Field testing has also been conducted to evaluate the surface macrotexture properties.

The Test Sections

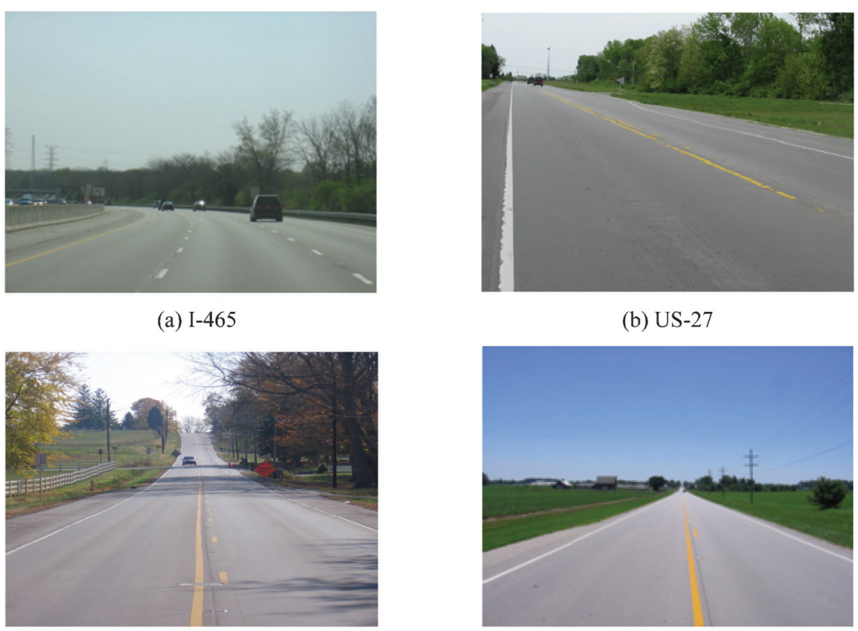
Site Description

Presented in Table 6.1 is the general information, such as traffic level and construction time, on the four pavement test sections (see Figure 6.1). The first thin overlay using 4.75-mm HMA mixes in Indiana is a 15.3 miles long, mill and fill project. The old pavement surface was milled and overlaid with ¾" 4.75-mm HMA mixes. This overlay was placed on I-465 in August, 2006. I-465 is a 52 miles long interstate ring-road that encircles Indianapolis, the capital city of Indiana. In order to provide first-hand information on 4.75-mm mixes as a pavement preservation treatment and identify possible performance issues, a 4-mile pavement test section without any major maintenance and repair activities was selected from this thin overlay project for field testing and evaluation. The selected pavement test section has 3 lanes in each direction and experiences heavy traffic. The AADT was over 100,000 in this section in 2007, including about 20,000 trucks. All testing, including pavement surface friction and texture, was conducted in the right driving lanes, i.e., the outer lanes in each direction.

Two test sections were constructed in accordance with the revised mix specifications in September, 2009, one on US-27 and the other on SR-227. Both test sections were profile milled and overlaid with ¾" 4.75-mm HMA mixes produced with the same job mix formula (JMF). US-27 is a north-south US highway. It runs from Southeast Indiana and proceeds all the way north to Fort Wayne. The pavement test section is located between mile mark (MM) 26 and MM 30. This section has a total of two lanes. The traffic volume was 7,735 AADT, including 741 trucks in 2007. SR-227 is a 37 miles long, north-south state route located in East Indiana. It begins at Indiana/Ohio border and proceeds all the way north to I-69. The pavement test section is located between MM 7 and MM 10 and consists of two lanes, one in each direction. The traffic volume was 1,964 AADT in 2007, including 77 trucks. The fourth test section was constructed between MM 0 and MM 9 on SR-29 with mixes produced according to a further modified JMF in August, 2010. The AADT was 5,552 in 2007 with a truck percentage of about 20%.

Materials and Mixes

Since the thin 4.75-mm HMA overlay on I-465 is the first 4.75-mm HMA project and INDOT had little experience with such mixes, the selection of materials relied mainly on the experiences with coarse aggregate



(a) I-465 (b) US-27

(c) SR-227 (d) SR-29

Figure 6.1 Photos of Four 4.75-mm Dense-Graded HMA Test Pavements

TABLE 6.2
Summary of Material Selection for 4.75-mm HMA Mixes

ESAL				Recycled Materials	
Category	Numbers (10^6)	Aggregate Type	Binder	RAP	ARS
1 (SR-227)	<0.3	BFS, SF, sandstone, dolomite, PRA, crushed stone,	PG 64-22	≤25%	≤5%
2	0.3 to <3	crushed gravel, natural sand	PG 64-22	≤25%	≤5%
3 (US-27, SR-29)	3 to <10	BFS, SF, sandstone, dolomite, PRA	PG 64-22, 70-22	≤15%	≤3%
4	10 to <30	BFS, SF, sandstone, dolomite[1], PRA\[1]	PG 70-22, 76-22	≤15%	≤3%
5 (I-465)	≥30		PG 76-22	≤15%	≤3%

[1]Dolomite and PRA may be used but cannot exceed 50% of the aggregate by weight when blended with BFS or sandstone, and 40% of the aggregate when blended with SF.

size mixes. The selection of aggregate type depends commonly on the equivalent single axle load (ESAL) category. INDOT uses both natural and synthetic materials as fine aggregates. The former include sandstone, dolomite, crushed stone, crushed gravel, and natural sand, the latter include blast furnace slag (BFS) and steel furnace slag (SF). There is another type of aggregate, i.e., polish resistant aggregates (PRA) that can be used as aggregates (48). The selection of asphalt binder is based on the performance grade binder system by taking into account the climatic condition and ESAL category. Generally, ESAL category 5 uses a PG 76-22 or higher grade asphalt binder, and ESAL categories 1 to 2 use a PG 64-22 asphalt binder. ESAL category 3 or 4 uses either a PG 70-22 or PG 76-22 asphalt binder, depending on the traffic conditions. Recycled materials, such as reclaimed asphalt pavement (RAP) and asphalt roofing shingles (ARS), may be used as a substitute for a portion of the new materials required to produce HMA mixtures according to the ESAL category. Table 6.2 shows a summary of requirements for material selections for 4.75-mm HMA mixes.

Presented in Table 6.3 are the summaries of mixes for these four test sections, including materials, aggregate properties such as gradation, fine aggregate angularity (FAA), and sand equivalency (SE), and mix volumetric properties such as voids in mineral aggregate (VMA), asphalt content (AC), air voids (V_a), voids filled with asphalt (VFA), and dust-to-binder ratio (DBR). Three main observations were made on the mixes used in these four test pavements. First, the

TABLE 6.3
Summaries of Materials, Gradations and Mixes for Experimental Pavements
(a) Materials

Test Section	Aggregate Components	Binder	RAP
I-465	BFS sand (39%), #24 dolomite sand (39%), #24 sand (20%), and Baghouse fines (2%)	PG 70-22	0%
US-27 and SR-227	#12 dolomite (34%), #24 dolomite sand (34%), dolomite sand (30%), and Baghouse fines (2%)	PG 64-22	0%
SR-29	#12 dolomite (27%), #24 QA sand (48%), QA fine sand (23%), and Baghouse fines (2%)	PG 64-22	0%

(b) Aggregate Gradations

	% Passing through Sieve Sizes (sieve size unit: mm)							
Test Section	9.5	4.75	2.36	1.18	0.60	0.30	0.15	0.075
I-465	100	96.1	78.6	52.3	34.8	17.5	9.6	6.3
US-27/SR-227	100	90.1	67.2	47.8	32.7	22.6	14.7	7.8
SR-29	100	90.0	68.4	42.4	25.4	15.8	11.0	6.7

(c) Aggregate and Mix Volumetric Properties

			AC					
Test Section	FAA	SE	Total	Eff.	V_a (%)	VMA (%)	VFA (%)	DBR
I-465	45.9	92.1	7.8	5.9	4	17.7	76.9	1.0
US-27/SR-227	47.0	80.0	6.9	5.2	4	16.0	75.0	1.5
SR-29	47.1	88.3	6.8	5.7	4	17.3	76.9	1.2

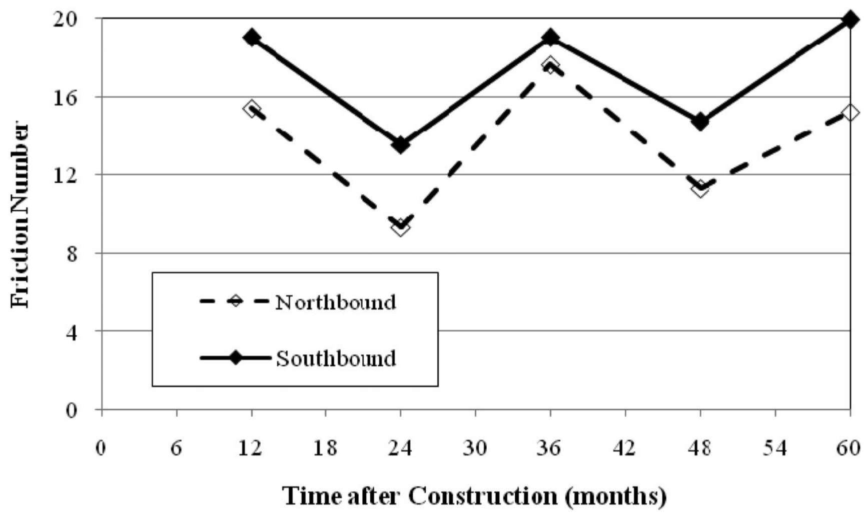


Figure 6.2 Friction Test Results in the Test Section on I-465

aggregate in the mixes on I-465 contained BFS sand that was not used in the mixes in the other three sections. The asphalt binder was a PG 70-22 binder on I-465 and a PG 64-22 binder in the other three sections. All binders were straight asphalt binders. Second, the revised mixes used on US-27, SR-227 and SR-29 were coarser than the mixes used on I-465, but also contained more fine particles passing through 0.60-mm sieve, particularly on SR-29. Third, the mixes on US-27, SR-227 and SR-29 contained more dust and less asphalt content than the mixes on I-465, leading to larger DBR values.

Friction Testing and Results

The Test Section on I-465

Since the construction of the thin 4.75-mm HMA overlay on I-465 was completed in 2006, one year earlier than the start of this study, the friction testing was conducted first in 2007, and then once every 12 months. Therefore, no information was available on the friction performance right after construction. Plotted in Figure 6.2 are the results of friction testing in terms of friction number conducted at 40 mph using the ASTM E 274 locked wheel trailer with the standard smooth tire over a period of 60 months in service. Three observations can be made through careful inspection of the variations of friction numbers presented in Figure 6.2. First, this test section produced friction numbers vary between 8 and 20, which indicated low friction performance. The detailed analysis will be provided later. Second, the variations of friction numbers in both directions followed a similar trend. Third, the friction numbers fluctuated significantly in both directions over time. This will be discussed later.

In order to verify the friction performance of this 4.75-mm HMA experimental pavement, a 1-mile segment between MM 5 and MM 6 eastbound was selected for the field testing in May 2009 to further assess the surface characteristics. Tabulated in Table 6.4 are the summaries of the test results from the locked wheel trailer, DF-Tester and CTM tests. The locked wheel friction testing was conducted to measure friction number (FN) using both the standard smooth tire and rib tire, respectively (see Figure 6.3). The DF-Tester was conducted to measure DF-Tester friction, i.e., DFT_{20}. The CTM was used to measure the mean profile depth, i.e., MPD. It is indicated that by both the FN (smooth tire) and MPD values, this test section experienced low friction performance. Notice that DFT_{20} is the friction coefficient at 20 km/h and should decrease when the test speed could increase. Also, the FN measured using the rib tire is much greater than that using the smooth tire. In reality, it has been reported that the smoother the pavement surface (49), the greater the difference between the friction numbers by the smooth and rib tires. Henry also pointed out that the friction number measured using the standard smooth tire is sensitive to both the macrotexture and microtexture (6).

TABLE 6.4
Summaries of Test Results on I-465

		Friction		
Test Section	MPD (mm)	DFT_{20}	FN (smooth tire)	FN (Rib tire)
4.75-mm HMA on I-465	0.24	0.43	16.7	44.4

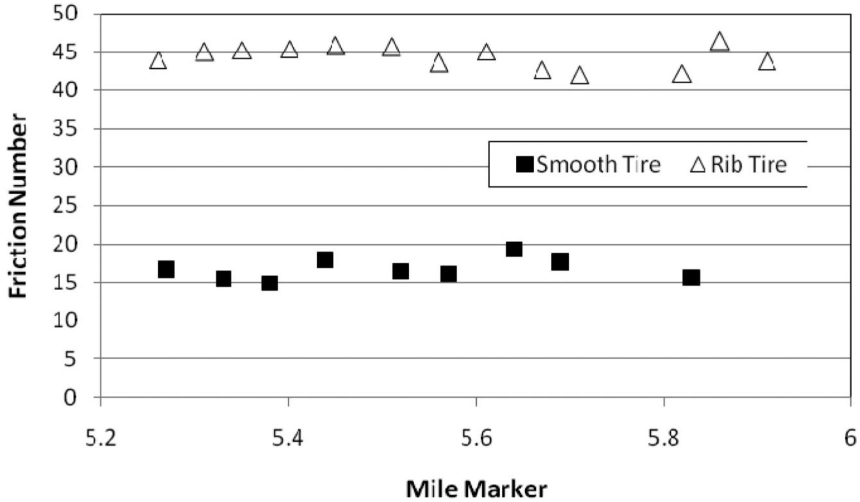


Figure 6.3 Friction Test Results Using Smooth and Rib Tires on I-465

The Test Sections on US-27 and SR-227

The locked wheel trailer friction testing was first conducted in the two test sections immediately after open to traffic to provide friction information on freshly laid 4.75-mm HMA surface. In the subsequent years, the locked wheel friction testing was conducted approximately every six months, i.e., one in spring season and one in fall season, respectively. Figure 6.4 shows the friction numbers measured on US-27 and SR-227, respectively, over a period of 18 months in service. It is shown that these two test pavements both demonstrated good friction performance right after opening to traffic. While the friction numbers on US-27 and SR-227 demonstrated noticeable differences on the freshly overlaid surfaces, they all decreased overt time, and started to converge after 6 months in service. After 12 months in service, the average friction number decreased by up to 48% on US-27 and by 36% on SR-227, respectively. One possible reason is that traffic volume on US-27 was much greater than that on SR-227. After 18 months in service, the friction numbers further decreased and converged to a friction level of 20 except for US-27 northbound.

CTM testing and DF-Tester friction testing were conducted after 18 months in service, i.e., in Spring, 2011 to verify the surface characteristics of these two experimental pavements. Three test locations were selected for both the CTM testing and DF-Tester testing in each direction. At each location, MPD and DFT_{20} were measured in both the right wheel path (RWP) and left wheel path (LWP). The average test results were tabulated in Table 6.5. Also presented in Table 6.5 are the FN values measured using the smooth tire right after the CTM and DF-Tester testing. On US-27, the MPD was 0.24 in southbound and 0.32 in northbound, and the FN was 19.7 in southbound and 28.6 in northbound. Apparently, FN, MPD and DFT_{20} follow a similar trend. While the correlation was not strong between the DFT_{20} and MPD values, the FN

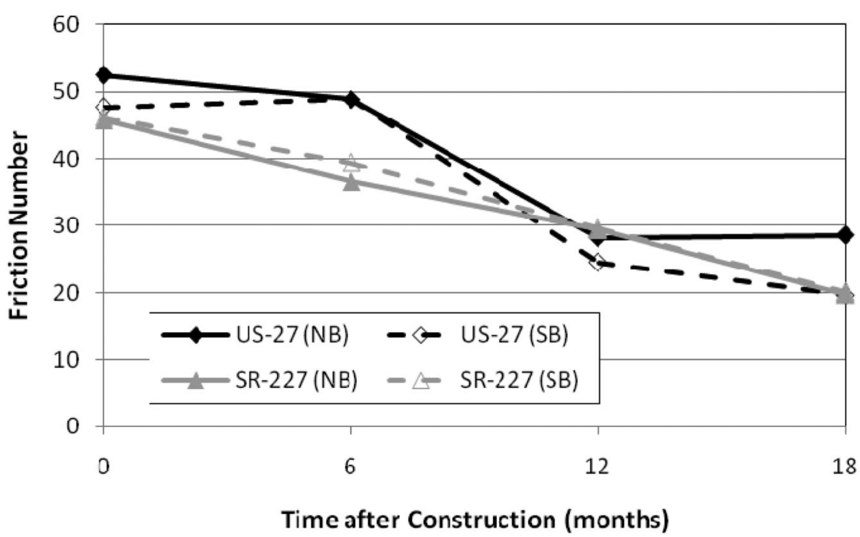


Figure 6.4 Friction Test Results in the Test Sections on US-27 and SR-227

TABLE 6.5
Summaries of Surface Characteristics Test Results on US-27 and SR-227

Test Results	US-27 SB	US-27 NB	SR-227 SB	SR-227 NB
MPD (mm), 18 months	0.24	0.30	0.18	0.20
DFT_{20}, 18 months	0.25	0.27	0.30	0.27
FN (smooth tire), 18 months	19.7	28.6	20.1	19.8

values agreed well with the MPD values. Similar observations can be made on SR-227.

The Test Section on SR-29

In the test section on SR-29, the locked wheel trailer friction testing was first conducted right after opening to traffic and again after 6 months in services. Also, extensive testing was conducted at every 0.25 miles using the CTM and DF-Tester after 6 months in service. However, both the CTM testing and DF-Tester testing were conducted only in northbound because of the site condition. Therefore, the surface texture depth in southbound was measured a few days later using a laser scanner. The detailed information on the laser scanner can be found elsewhere (50). Presented in Table 6.6 are the test results. It is shown that after the first 6 months in service, the surface friction dropped by approximately 25% from 36.6 to 27.6 in northbound and by 34% from 32.9 to 21.6 in southbound. The surface texture depths agreed well in both directions. The MPD was 0.21 mm and 0.22 mm in southbound and northbound, respectively. Noticeable differences arose between the friction numbers measured in both directions. After carefully examining all MPD, DFT_{20} and FN data and field inspections, it was concluded that the locked wheel friction testing might be conducted out of the wheel path between MM 4 and MM 7.

4.75-mm HMA Surface Friction Performance

Mix Type and Friction Characteristics

This study examined the friction properties of HMA mixes reported by other researchers nationwide, including the latest research report on 4.75-mm mixes by Randy et al. (51). As shown in Table 6.7 are the measurements of surface characteristics on the HMA pavements of different mix types, including regular HMA mixes, stone matrix asphalt (SMA), open-graded friction course (OGFC), and porous friction course (PFC). Several observations can be made by careful inspection of the data in Table 6.7, except for the FNs by Stacy et al. First, the 4.75-mm mixes demonstrated the smallest texture depth. The MPD for 4.75-mm mixes varied around 0.20 mm, and rarely exceeded 0.30 mm. Second, the surface friction of the 4.75-mm mixes was much less than that of the 9.5-mm or coarser mix. Third, the surface friction and macrotexture depth increased as the nominal maximum aggregate size (NMAS) increased. Also, as the surface macrotexture depth increased, the surface friction increased. Forth, the special mixes such as SMA, OGFC, and PFC provided much better friction performance than the dense-graded mixes. Finally, it was exhibited that the use of modified asphalt binders might have improved the surface friction properties.

TABLE 6.6
Summaries of Surface Characteristics Test Results on SR-29

Test	Location SB	Location NB
FN (smooth tire), fresh surface	32.9	36.6
FN (smooth tire), 6 months	21.6	27.6
MPD (mm), 6 months	0.21 (Scanner)	0.22 (CTM)
DFT_{20}, 6 months	–	0.23

The above observations indicate that poor surface characteristics may be the inherent nature of the 4.75-mm dense-graded mixes. While Stacy et al. showed that the 4.75-mm mix provided skid resistance that was equal or similar to that of the 12.5-mm mix when newly constructed, and better than 12.5-mm mix after 15 months of service, these results are worthy of further investigation. First, the test sections reported by Stacy et al. (47) were placed in a large residential community and could not address the effect of traffic on the long-term friction performance. Second, a rib tire was used in the friction testing by Stacy et al. According to the findings by Henry (6), the friction measured using the standard rib tire is more sensitive to the microtexture than the macrotexture. A smooth tire may provide friction measurements to yield more convincing evidences.

Typical Friction Characteristics of 4.75-mm Mixes

Apparently, the 4.75-mm HMA pavement test section on I-465 demonstrated poor surface friction. While the two test sections on US-27 and SR-227 demonstrated good surface friction right after opening to traffic, their friction numbers decreased dramatically after 12 months of service. Dramatic reduction in friction was also observed in the test section on SR-29 after 6 months in service. Notice that the three test sections on US-27, SR-227, and SR-29 were constructed with the mixes produced according to the revised specifications. Compared to the 4.75-mm mix on I-465, the mixes on US-27, SR-227 and SR-29 contained more coarse aggregates and used dolomite sand to replace natural sand. However, the improvement in surface friction was very limited. This further confirms that the 4.75-mm dense-graded mixes may not perform as well as coarse aggregate size mixes in terms of the friction performance. To provide a full picture of the friction performance of 4.75-mm HMA mixes, Figure 6.5 shows the variations of friction in terms of

TABLE 6.7
Frictional Characteristics of Various Pavement Surfaces

Surface Type	Friction	MPD (mm)
4.75-mm HMA on I-465, 36 months	16.7 (smooth tire)	0.24 (CTM)
4.75-mm HMA on US-27, 18 months	19.7/28.6 (smooth tire)	0.24/0.30 (CTM)
4.75-mm HMA on SR-227, 18 months	20.1/19.8 (smooth tire)	0.18/0.20 (CTM)
4.75-mm HMA on SR-29, 6 months	21.6/27.6 (smooth tire)	0.21/0.22 (CTM)
4.75-mm HMA, new (47)	62.5 (rib tire)	0.21 (CTM)
4.75-mm HMA, 15 months (47)	63.8 (rib tire)	0.26 (CTM)
12.5-mm HMA, new (47)	65.5 (rib tire)	0.39 (CTM)
12.5-mm HMA, 15 months (47)	45.9 (rib tire)	0.39 (CTM)
4.75-mm HMA, new, MODOT (51)	–	0.17–0.22 (CTM)
4.75-mm HMA, new, TNDOT (virgin mix) (51)	0.25–0.35 (DFT_{20})	0.16–0.33 (CTM)
4.75-mm HMA, new, TNDOT (15% RAP) (51)	0.28–0.33 (DFT_{20})	0.19–0.33 (CTM)
4.75-mm HMA, new, MNDOT (51)	0.34–0.49 (DFT_{20})	0.13–0.18 (CTM)
9.5-mm HMA (52)	42.5 (smooth tire)	0.59 (CTM)
9.5-mm Dense-Graded HMA (53)	–	0.53 (CTM)
12.5-mm SMA (53)	–	1.07 (CTM)
19.0-mm SMA (53)	–	1.11 (CTM)
12.5-mm OGFC (53)	–	2.31 (CTM)
Dense-graded HMA to OGFC (54)	32.5 (smooth tire)	1.63 (sand patch)
	27.4 (smooth tire)	0.53 (sand patch)
	41.4 (smooth tire)	1.57 (sand patch)
	33.4 (smooth tire)	2.19 (sand patch)
	37.8 (smooth tire)	2.97 (sand patch)
9.5-mm HMA (55)	61.9/64.3 (BPN)	0.62/0.78 (sand patch)
Polymer modified 9.5-mm HMA (55)	56.6/65.6 (BPN)	0.76/0.82 (sand patch)
Rubber modified 9.5-mm HMA (55)	60.6/64.5 (BPN)	0.78/0.73 (sand patch)
Polymer modified 12.5-mm SMA (55)	58.4/67.6 (BPN)	0.89/0.94 (sand patch)
12.5-mm SMA with fibers (55)	60.5/67.8 (BPN)	0.89/1.07 (sand patch)
19.0-mm HMA (55)	60.9/62.9 (BPN)	0.73/0.82 (sand patch)
9.5-mm PFC, new (56)	0.51 (DFT_{20})	1.37 (CTM)
9.5-mm PFC, 36 months (56)	0.52 (DFT_{20})	1.37 (CTM)
9.5-mm PFC, 60 months (56)	0.42 (DFT_{20})	1.48 (CTM)
9.5-mm SMA, newly constructed (56)	0.37 (DFT_{20})	1.17 (CTM)
9.5-mm SMA, 36 months (56)	0.61 (DFT_{20})	1.03 (CTM)
9.5-mm SMA, 60 months (56)	0.69 (DFT_{20})	0.93 (CTM)
9.5-mm HMA, newly constructed (56)	0.52 (DFT_{20})	0.30 (CTM)
9.5-mm HMA, 36 months (56)	0.39 (DFT_{20})	0.55 (CTM)
9.5-mm HMA, 60 months (56)	0.41 (DFT_{20})	0.63 (CTM)

*British pendulum number (57); and ** (58).

average number and standard deviation over time for all four test sections.

Based on the test results presented in Figure 6.5, three findings can be made on the surface friction characteristics of 4.75-mm dense-graded mixes. First, a 4.75-mm dense-graded mix may produce a good initial friction number that is commonly greater than 30. However, a dramatic reduction in surface friction will occur after 12 months in service and varies between 20% and 50%, depending on the initial friction and traffic volume. Second, it is demonstrated that by both the average friction number and standard deviation (except for I-465 northbound), it may take 12–18 months for a newly constructed 4.75-mm HMA pavement to produce stable surface characteristics. Third, the typical surface friction number is around 20 and the typical surface MPD is around 0.20 mm but commonly less than 0.25 mm for a 4.75-mm HMA pavement after 12–18 months in service.

Aggregate Gradation and Surface Friction Performance

Gradation Limits

Superpave specifies the aggregate gradation by applying the gradation limits, such as control points and restricted zones along the maximum density line. The control points are placed on the maximum aggregate size sieve, NMAS, 2.36-mm sieve (intermediate sieve), and 0.075-mm sieve, and are widely used to establish the gradation master ranges, taking into account the durability, stability, permeability, workability, and stiffness. It is well known that a gradation lying above the maximum density line is fine gradation and a gradation below the maximum density line is coarse gradation. As a general rule of thumb, an aggregate gradation is classified as fine-graded or coarse-graded according to the percent passing through the primary control sieve

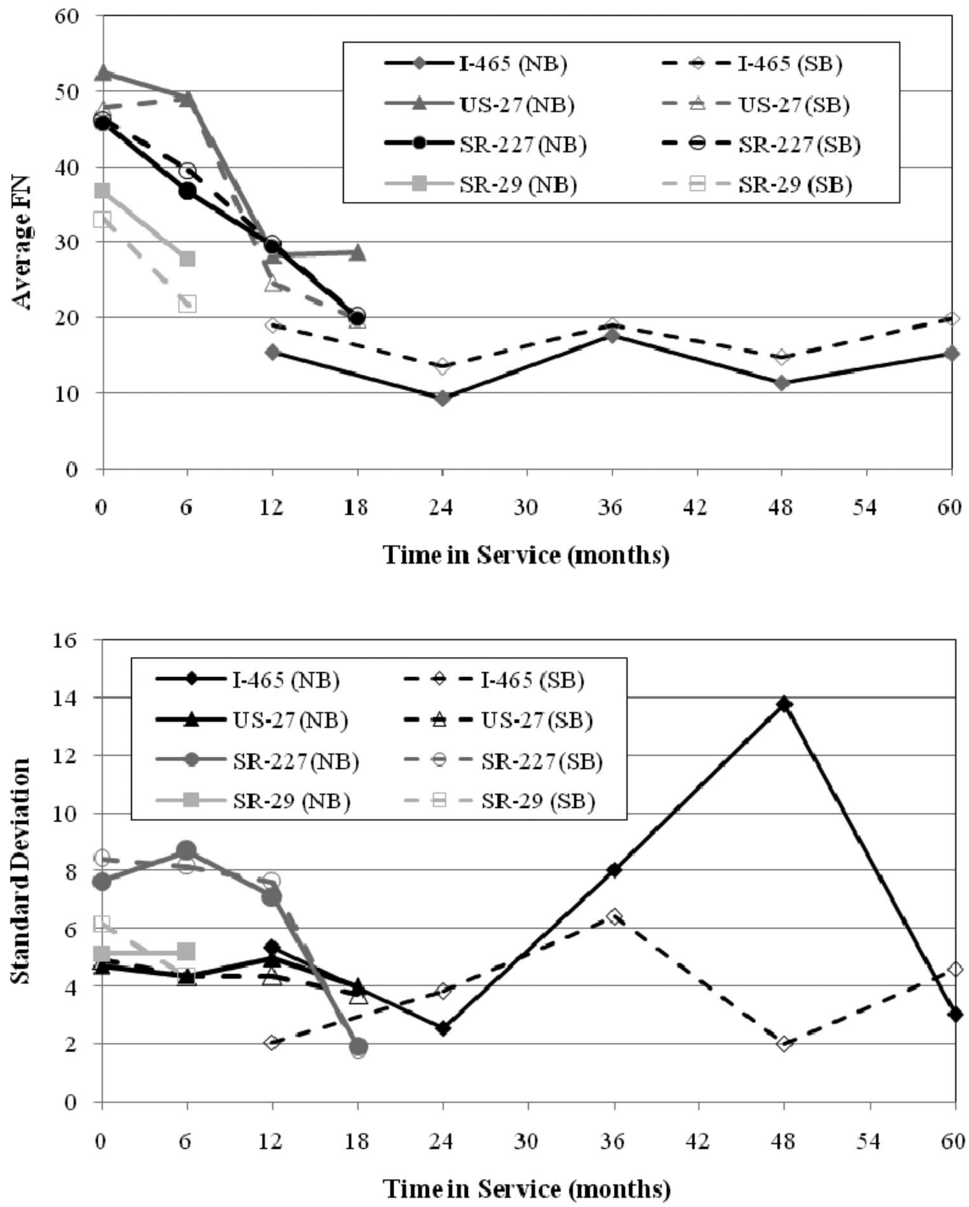


Figure 6.5 Friction Variations in All Four Test Sections

(PCS). Fundamentally, the control point placed on the PCS is actually the percent passing through the PCS on the maximum density line. The aggregate gradation is classified as fine-graded when it passes above the PCS control point, and classified as coarse-graded when it passes below the PCS control point. The restricted zone is placed between the 2.36-mm sieve and 0.30-mm sieve to avoid over-sanded mixes, too much fine sand or rounded natural sand, and was recommended to be removed from the Superpave specifications (59).

In order to examine the possible effect of aggregate gradation on the friction performance, Figure 6.6 shows the aggregate gradations for the three mixes and the maximum density line for 4.75-mm mixes. The aggregate gradations for the three mixes are all convex curves with the most part lying far above the maximum density line. This indicates that the three mixes are basically fine-graded. Applying the concept of Superpave gradation limits to the 4.75-mm dense-graded mixes yields the PCS sieve size and restricted zone shown in Figure 6.7. The PCS is the 1.18-mm sieve with an associated percent passing of 39%. The restricted zone boundary is 53.4% for the 2.36-mm sieve, 36.1%–42.1% for the 1.18-mm sieve, 26.8%–30.8% for the 0.60-mm sieve, and 21.1% for the 0.30-mm sieve. It is shown that the aggregate gradations for all three mixes pass above the PCS control point. The gradation for the mixes on US-27 and SR-227 is a typical ARZ (above the restricted zone) without violating the Superpave restricted zone. The gradations for the mixes on I-465 and SR-29 are CRZ (crossover through the restricted zone) gradations. The gradation for the mix on SR-29 crosses through the restricted zone between 0.6-mm and 1.18-mm sieves. However,

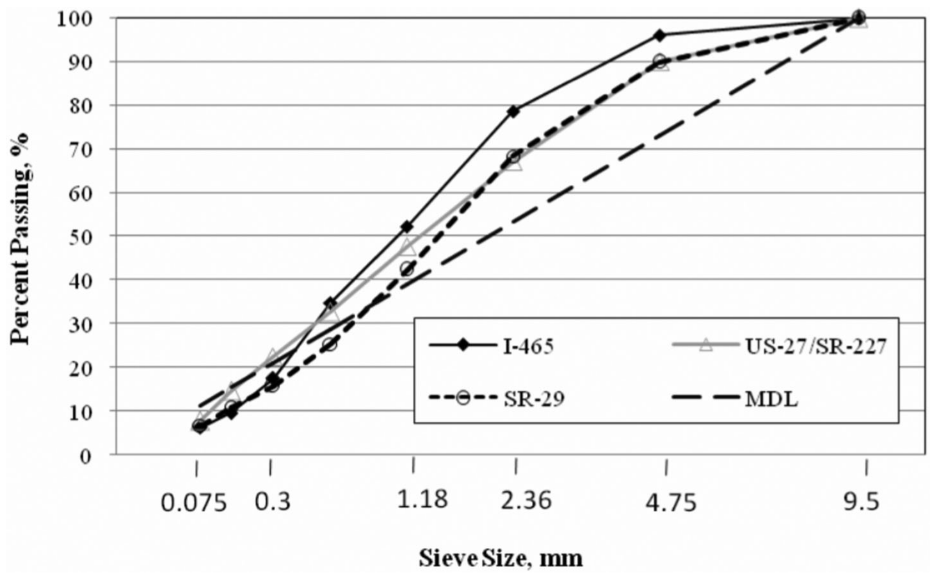


Figure 6.6 Aggregate Gradations for the Three 4.75-mm Dense-Graded Mixes

the gradation on I-465 lies above the restricted zone for the most part and crosses through the restricted zone between 0.30-mm and 0.60-mm sieves.

Therefore, the 4.75-mm mixes in these four test sections, particularly the two on I-465 and SR-29, may contain too much fine sand, leading to poor macrotexture properties. Mixes with too much fine sand usually tend to possess weak aggregate structure and poor stabilities. This might be the reason causing the significant fluctuation of surface friction over time under high traffic as demonstrated in the test section on I-465. Presented in Table 6.8 are the aggregate gradations of 4.75-mm mixes used by INDOT (14), Maryland Department of Transportation (MDDOT) (60), Georgia Department of Transportation (GADOT) (61), Ohio Department of Transportation (OHDOT) (44), and AASHTO (62), respectively. The 4.75-mm mixes used by INDOT are slightly finer than those used by other DOTs. Since the restricted zone requirement has been removed, it becomes critical to ensure the aggregate gradation to pass below the PCS control point so as to enhance the friction properties of 4.75-mm dense-graded mixes. In reality, it was observed that a 9.5-mm mix with the aggregate gradation passing through the restricted zone and above the PCS control point exhibited poor friction performance (63).

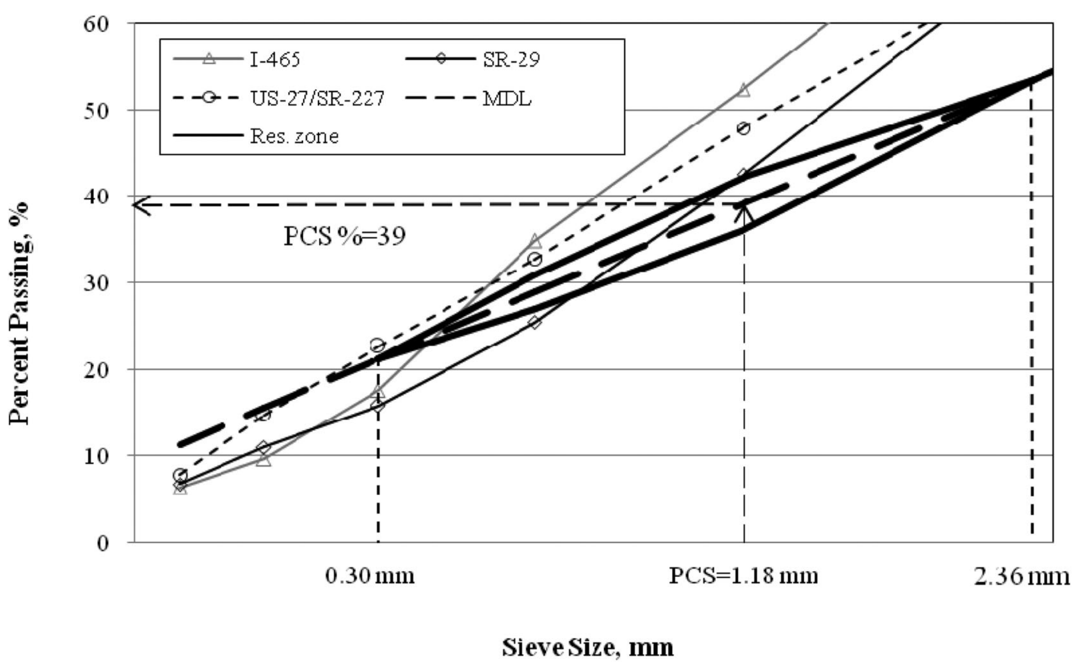


Figure 6.7 Gradation Limits for the Three 4.75-mm Dense-Graded Mixes

TABLE 6.8
Aggregate Gradations for 4.75-mm Dense-Graded Mixes by Different Agencies

Sieve Size	INDOT	MDDOT	GADOT	OHDOT	AASHTO
12.5 mm	100	100	100	100	100
9.5 mm	100	100	90–100	95–100	95–100
4.75 mm	90–99	80–100	75–95	85–95	90–100
2.36 mm	–	36–76	60–65	53–63	–
1.18 mm	30–60	–	–	–	30–60
0.60 mm	–	–	–	–	–
0.30 mm	–	–	20–50	4–19	–
0.075 mm	6–12	2–12	4–12	3–8	6–12
PCS Control Point	NA	NA	NA	NA	NA

Packing Properties

The aggregate packing properties are used in the Bailey Method to predict volumetric and compactability characteristics of HMA mixes (64). The Bailey Method defines three control sieves, including the PCS, secondary control sieve (SCS), and tertiary control sieve (TCS) that are used to break the aggregate into three portions, such as coarse aggregate, coarse sand and fine sand. The entire analysis is undertaken according to the three ratios, including coarse aggregate (CA) ratio, fine aggregate coarse (FA_c) ratio, and fine aggregate fine (FA_f) ratio. For 4.75-mm dense-graded mixes, the desired ranges are 0.30–0.45 for CA ratio and 0.35–0.50 for both FA_c and FA_f ratios. In general, VMA increases as CA ratio increases. Also, an increase in FA_c ratio or FA_f ratio tends to result in an decrease in VMA. For a blend of coarse and fine aggregates, CA ratio has the greatest effect on VMA. For the fine aggregate, the effect of FA_c ratio becomes dominant. Presented in Table 6.9 are the three ratios and the corresponding MPD values for the 4.75-mm mixes in the three test sections.

It is shown that for the 4.75-mm mix on I-465, the CA ratio is 1.23 and is greater than 1.0. This indicates that the coarse aggregates did not control the aggregate skeleton and it might be difficult to compact the mix. In other words, the 4.75-mm mix on I-465 might experience a tendency to move under the application of vehicle wheels. This further confirmed that the significant fluctuation of surface friction over time on this test section. The CA ratios are 0.59 on US-27/SR-227 and 0.82 on SR-29, respectively and also exceed the desired CA ratio range. When computed in terms of fine-graded mixes, the CA ratios are 0.27, 0.15 and 0.13 for the mixes on I-465, US-27/SR-227, and SR-29, respectively. This may imply that 4.75-mm dense-graded mixes tend to have a CA ratio falling outside the desired range. For all three mixes, the FA_c and FA_f ratios fell in the desired range. It appears that the CA ratio has the greatest effect on the macrotexture of a 4.75-mm dense-graded mix. A greater CA ratio may produce larger macrotexture and can be achieved by reducing the percent passing the PCS or increasing the percent passing the HS. However, a large CA ratio may result in unbalanced coarse aggregate structure. Therefore, the selection of CA ratio comprises a trade-off between macrotexture and compaction. Likely, a CA ratio approaching but less than 1.0 will have positive effect on the macrotextrue or friction of a 4.75-mm mix.

Aggregate Type, FAA and SE

The current Superpave specifications have no explicitly defined criteria for ensuring friction properties of HMA mixes. Due to the fact that pavement surface friction depends on both macrotexture and microtexture, it is natural to conclude that aggregate properties beside gradation, such as aggregate type (or source), particle shape, aggregate surface feature, FAA, and SE that affect macrotexture or microtexture will affect pavement surface friction. However, the effect of aggregate properties on pavement surface friction is a very complicated phenomenon that is the result of the interaction among the aggregate properties. In addition, some aggregate properties, such as particle shape

TABLE 6.9
Fractions of Coarse and Fine Aggregates

Experimental Section	I-465	US-27/SR-227	SR-29
% Half Sieve (2.36-mm)	78.6	67.2	68.4
% Primary control sieve (PCS) (1.18-mm)	52.3	47.8	42.4
% Secondary Control Sieve (SCS) (0.3-mm)	17.5	22.6	15.8
% Tertiary Control Sieve (TCS) (0.075-mm)	6.3	7.8	6.7
Coarse aggregate (CA) ratio	1.23	0.59	0.82
Fine aggregate coarse (FA_c) ratio	0.33	0.47	0.37
Fine aggregate fine (FA_f) ratio	0.36	0.35	0.42
MPD (mm)	0.24	0.21–0.30/0.18–0.20	0.21–0.22

and Los Angles Abrasion, may not apply to 4.75-mm dense-graded mixes very well, in particular when designed to use existing aggregate screenings. Therefore, the only controllable aggregate properties other than gradation are aggregate type, FAA and SE.

Aggregate type or source has major effect on pavement surface friction. Different types of aggregates may not only have different polishing resistances, but also have different aggregate surface features that affect pavement surface microtexture. In reality, 4.75-mm dense-graded mixes tend to produce macrotextures of small dimensions, including depth and wavelength, the microtexture characteristics may become more dominant in pavement surface friction for 4.75-mm dense-graded mixes, particularly in the long term. As pointed out earlier, the 4.75-mm mixes on US-27 and SR-227 utilized mainly dolomite, but the macrotexture depths did change too much. Nevertheless, aggregate type may affect pavement microtexture more significantly. It was pointed out that by Shuo et al., the use of steel slag will produce larger microtexture (52).

The other two variables are SE and FAA. Summarized in Table 6.10 are the values of FAA and SE and the corresponding macrotexture depths. It is well known that FAA is one of the important factors affecting rutting, and rutting decreases as FAA increases. Shuo et al. reported that HMA surfaces with rutting issues also experienced low friction performance (28). It is likely that any measures for enhancing FAA will improve pavement surface friction. FAA varies with aggregate shape and aggregate surface feature. Rougher aggregate surface and cubic aggregate tend to yield larger FAA. It is also shown that there was no problem for the 4.75-mm dense-graded mixes to meet the current SE criteria. The SE values for the mixes used in both this study and the study by Randy et al. were much greater than the minimum SE, i.e., 40% for ESALs$<3\times10^6$ and 45% for ESALs$\geq3\times10^6$. A greater SE simply indicates more coarse particles and may result in better macrotexture properties.

Volumetric Properties and Friction Performance

AC, V_a, VMW, and VFA

For Superpave dense-graded mixes, AC, V_a, VMA, and VFA are probably most important volumetric properties. Currently, most DOTs use the volumetric design method for the design of 4.75-mm dense-graded HMA mixes. The targeted air voids is 4%, and the volume in mineral aggregate (VMA) is 16% or more. Because 4.75-mm dense-graded mixes are usually fine-graded and may contain too much fine sand, they require greater binder content than 9.5-mm or coarser mixes. Too much fine sand combined with greater amount of binder tends to produce clay-like mixes, leading to smooth surface and poor surface macro-texture or friction properties in the long term. Davis investigated the relationship between HMA mix properties and pavement surface characteristics (65). Based on the test results on those 9.5-mm and 12.5-mm mixes, he concluded that the surface macrotexture depth increased as VMA increased. A greater binder grade also resulted in better macrotexture properties. This can also been seen from the friction and macrotexture results in the test section on I-465.

Based on the test results presented in the preceding sections, there is no strong evidence showing a relationship between the volumetric properties and surface characteristics. This is due in part to that the surface characteristics of a 4.75-mm dense-graded HMA mix may not be sensitive enough to the volumetric properties. As shown in Table 6.3(c), the volumetric properties were very close for these three mixes. There is not much room left in the 4.75-mm mixes to modify the volumetric properties so as to improve the surface properties. In reality, the surface properties of a 4.75-mm dense-graded HMA mix may depend mainly on the aggregate gradation that also significantly affects the volumetric properties of the mix. It is well known that the depth of macrotexture varies over a range of 0.1 and 20 mm. However, the MPD values for these four test sections are all in the lower end of this range. It is possible that the surface characteristics of individual aggregate particles may also play a role in generating macrotexture in the lower end of the range. Therefore, the aggregate angularity and abrasion resistance play an important role in producing and maintain sound surface characteristics for a 4.75-mm dense-graded HMA mix.

TABLE 6.10
Summaries of FAA, SE and MPD Results

Experimental Section	FAA (%)	SE (%)	MPD (mm)
I-465	45.9	92.1	0.24
US-27/SR-227	47.0	80.0	0.21–0.30/0.18–0.20
SR-29	47.1	88.3	0.21–0.22
MODOT project (24)	45	Not reported	0.17–0.22
MN project (24)	47	83	0.13–0.18

CHAPTER 7 PROFILE MILLING

General Description

Profile milling, commonly called diamond grinding when used on concrete pavements, is a process that utilizes a special cutting head that consists of a series of closely spaced diamond-tipped saw blades mounted on a horizontal shaft to remove bumps and rectify surface defects. Diamond grinding was first used in 1965 on an old concrete pavement to eliminate excessive faulting in California (66) and has been effectively used as part of concrete pavement restoration (CPR) (67). A study surveyed a total of 76 diamond-ground concrete pavements from 9 states (68). It was found that the average life of a typical diamond-ground concrete pavement is about 14 years. It was also found that after diamond grinding, the pavement surface texture and skid resistance improved considerably and the ground surfaces lasted 8–15 years. The identified main advantages of diamond grinding include better ride quality, enhanced safety, quieter travel surface, extended service life, and reduced rehabilitation costs.

To date, profile milling has been used as part of pavement preservation for both concrete and asphalt pavements to provide smooth, safe, and quiet pavement surface (69). For pavement preservation, continuous diamond grinding is mainly utilized. The International Grooving and Grinding Association (IGGA) has developed specifications for the use of profile milling in pavement preservation. For concrete pavement preservation, a so-called Next Generation Concrete Surface (NGCS) has been created by IGGA to guide the concrete pavement grinding (70). Recently, profile milling has been considered by INDOT as a surface preservation treatment for asphalt pavement to correct the pavement profile or to roughen the existing surface. In other DOTs (71), however, profile milling has also been used to restore pavement cross-sections when rutting is greater than 1/2". When profile milling is used as a surface preservation treatment for asphalt pavement, the immediate result is the improved surface texture, particularly macrotexture. This will not only enhance pavement surface friction dramatically, but also reduce the possibility of hydroplaning considerably.

The Test Sections

The Test Sites

Due to limited availability, only three profile milling test sections, including SR-162, US-50, and I-469, were included in this study. In reality, the profile milling in the two test sections on SR-162 and I-469 is spot grinding to remove bumps. Only the profile milling on US-50 is continuous grinding as shown in Figure 7.1. Table 7.1 is the general information, such as road classification, traffic volume in 2007 and pavement type. SR-162 is a 2-lane conventional highway and the test section is located in a rural area. The test section on US-50 is an urban road consisting of several signalized intersections. The test section on I-469 consists of two lanes in each direction. Apparently, all three test sections have experienced high traffic volumes. The truck traffic volume carried by the test section on I-469 was 7,779, accounting for approximately 30% of total traffic volume. Figure 7.2 shows the close-up views of the ground surfaces in these three test sections. All the three surfaces were longitudinally ground.

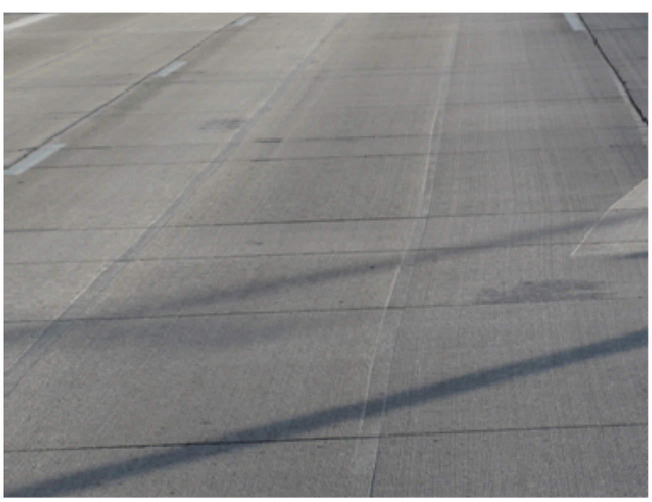


Figure 7.1 Continuous Grinding on US-50

TABLE 7.1
General Information on Three Profile Milling Sections

Road	No. of Lanes	Length	AADT	Trucks
SR-162	2-Lane	3.0 miles	5,095	488
US-50	4-Lane	1.2 miles	33,143	1,905
I-469	4-Lane	8.5 miles	24,597	7,779

Typical Grinding Texture

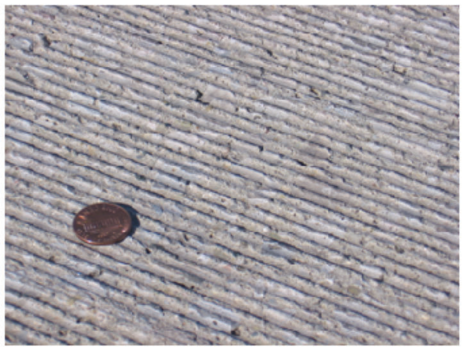
When applying profile milling, the pavement surface is commonly ground to provide longitudinal line-type texture. For a typical ground surface, its friction performance, particularly in the long-term friction performance, depends mainly on the dimension of grinding texture and the hardness of aggregate. The dimension of grinding texture is defined with respect to three parameters, such as depth, land area and groove as shown in Figure 7.3 (72). In general, the depth and groove do not vary from project to project. However, the land area should be adjusted with respect to the hardness of aggregate. A larger land area (wider blade spacing) is commonly utilized in pavements with softer aggregate such as limestone. It was reported that when a larger land area is used, light vehicles and motorcycles may experience vehicle tracking (73). Presented in Table 7.2 are the dimensions of typical grinding texture for asphalt and concrete pavements, respectively. It appears that the ground texture for asphalt pavement is finer and deeper than that for concrete pavement.

Friction Performance of Profile Milling

For a newly ground surface, thin fins remained in the ground surface commonly produces great surface friction. However, those thin fins may be knocked off quickly by one or two passes of traffic, which results in land area with a uniform height. Consequently, a smooth grinding texture will be created and a steady-state friction can be generated. Presented in Figure 7.4 are the friction numbers measured over time in the three profile milling test sections shown in Table 7.1. Due to limited availability of profile milling projects, no newly ground pavement project was available during the study period and the most recent profile milling project was the one on SR-1621 that was completed in July 2007. It is shown that the friction number was

(a) SR-162

(2) US-50



(c) I-469

Figure 7.2 Close-Up Views of Three Ground Surfaces

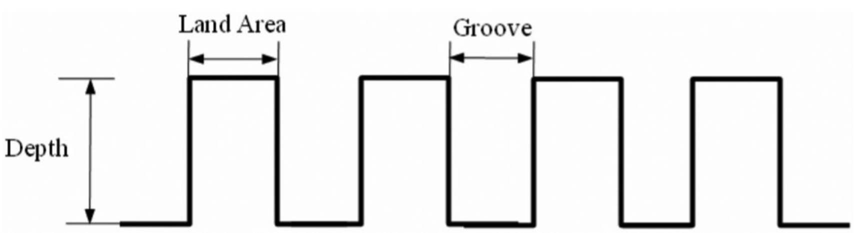


Figure 7.3 Dimension Parameters for Grinding Texture

around 36 in both directions on the ground surface in fall 2008. It should be pointed out that the test section on SR-162 consists of sporadic spot grinding. Because each spot grinding covers a small portion of pavement surface, the measured friction numbers might involve the friction forces on unground pavement surface. Therefore, there is no doubt that the friction numbers of newly ground surface may be greater than 36.

As evident in Figure 7.4, the friction of ground surface varied over time. However, it is shown that in the same test section, the friction variations in both directions demonstrated significant differences, particularly in the test sections on SR-162 and I-469, respectively. One possible reason is that the profile milling in the test sections on SR-162 and I-469 was produced using spot grinding. As explained earlier, spot grinding is sporadic and covers only a small portion of pavements. Also, the friction number measured with the ASTM E-274 locked wheel trailer at 40 mph is actually the average friction approximately over a 25-meter pavement segment. Therefore, it is possible that the field testing might cover unground pavement surface and random errors might be involved in the results.

In pavement preservation, continuous grinding is commonly utilized to rectify rutting and restore surface friction. As demonstrated by the friction numbers measured over time in the test section on US-50, it

TABLE 7.2
Typical Grinding Texture Dimensions

Design Parameter	Asphalt Pavement (74)	Concrete Pavement (72)
Groove	Decided by Contractor	2.0–4.0 mm
Land Area	1.5–2.3 mm	1.5–3.5 mm
Depth	3.2 ± 1.6 mm	1.5 mm
Number of Grooves	Decided by Contractor	164–194 per meter

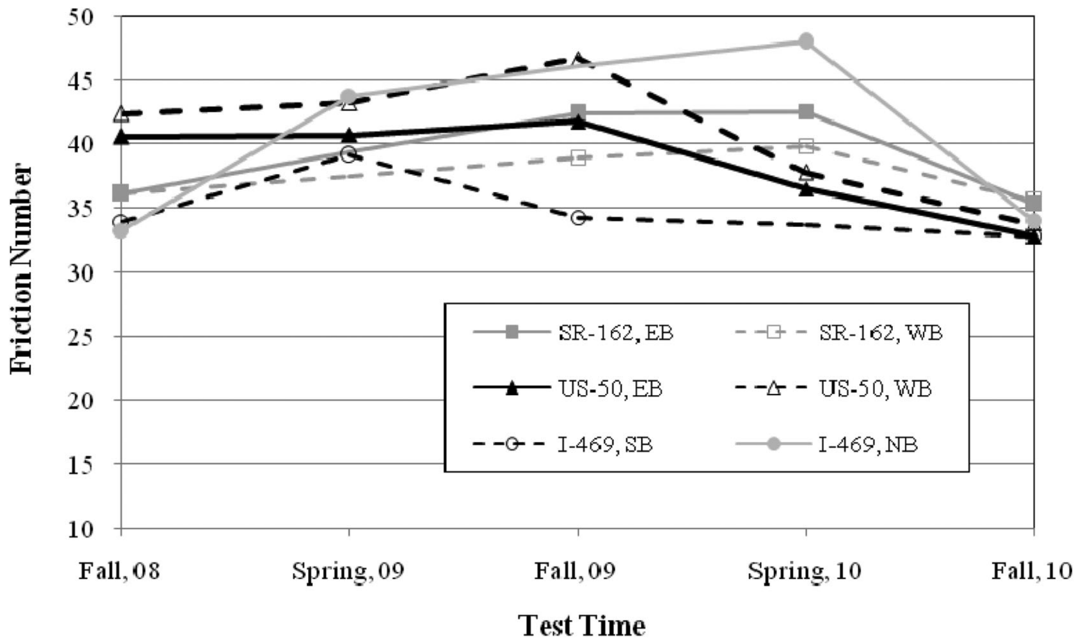


Figure 7.4 Friction Measurements over Time on Ground Surfaces

can be concluded that the friction number on a newly ground surface should not be less than 40. After opening to traffic, the surface friction will remain at a certain level for at least 12 months. Once all thin fins have been knocked down by vehicles and uniform land areas have been formed, the surface friction will decrease due to the pounding and polishing from vehicle tires. The long term friction performance of a ground surface depends on the aggregate properties and grinding texture configuration. Because tire pounding and polishing exert mainly on the land area of grinding texture, it is the aggregate and land area that play an important role in providing durable texture, particularly microtexture. As demonstrated in Figure 7.4, the friction decrease is less than 4% on both SR-162 and I-469 over a two-year period. While the surface friction on US-50 decreased by up to 20%, the friction number is still greater than 32 and is very good considering the high traffic volume and the presence of signalized intersections. Apparently, longitudinal grinding is capable of providing immediate and long term improvement to surface friction.

CHAPTER 8 CONCLUSIONS

Chip Seal and Fog-Chip Seal

A total of 10 chip seal test sections and 8 fog-chip seal test sections were included in this study. The age of chip seals in these test sections varied from 12 months to 42 months. The traffic level in terms of AADT varied from approximately 730 to 6300 in these test sections. The main findings are summarized below:

1) On the newly chip sealed pavements, surface friction numbers varied approximately between 50 and 70. The greater the friction number on the old pavement, the greater the friction number on the new chip sealed surface.
2) The friction variations over time in all test sections roughly followed a similar trend. The surface friction decreased after opening to traffic. The greatest friction decrease occurred after 12 months in service. When the chip seals reached the age of approximately 30 months, the surface friction started to decrease continuously over time.
3) It was confirmed that it commonly takes 12 months for a typical chip seal to form a stable mosaic. A successful chip seal can produces a friction number in a range of 44 to 52 at a confidence level of 95% after 12 months in service during the 3-year study period.
4) Failure was also observed in two chip seal test sections. A failed chip seal tended to experience dramatic decrease in friction at the age of 12 months and the resulting friction number was less than 30 or commonly around 20.
5) Applying a fog seal on top of a chip seal resulted in an immediate decrease in friction by 20%–33%. The greater the surface friction before fog seal, the greater the decrease in surface friction after fog seal.
6) The friction decrease after applying fog seal was temporary. The surface friction increased due to the material wearing off the tops of the chips tended to reach the maximum value after 6 months or more in service. This indicates that a fog-chip seal may form a stable mosaic faster than a standard chip seal.
7) The surface friction in the fog-chip seal test sections demonstrated a tendency to decrease over time after 12 months in service. The average friction number in the fog-chip seals at the age of 12 months was almost the same as that in chip seals at the age of 12 months.
8) Failure was observed in one fog-chip seal section that demonstrated a friction number of about 20 after 12 months of service. It appears that a fog-chip seal may not necessarily perform as well as a standard chip seal in terms of surface friction.

9) For new chip seal, the crushed stone produced a friction number of 59 and the crushed gravel produced a friction number of 49. After 12 months in services, the friction number was 50 for the crushed stone and 46 for the crushed gravel. The difference in friction between the crushed stone and crushed gravel dropped from 16.6% to 7.0% over a 12-month period. If this trend remains in the long term, it is very promising for future use of crushed gravels in chip seals.
10) The use of uncrushed aggregate could result in a friction reduction.
11) Chip seals can be successful even on high traffic volume roads. It seems that truck traffic affected the performance of a chip seal more significantly than AADT.
12) To facilitate pavement engineers to select surface treatment and materials, the non-interstate roads can be grouped in terms of three traffic levels as follows:

Traffic Level	AADT
Light	0–2000
Medium	2000–5000
High	>5000

13) The failed chip seals commonly demonstrated poor surface friction and could easily be identified from the visual appearance, such as insufficient aggregate chips, binder-rich surface or both.
14) The contributing factors to the failures in the test sections were not readily apparent. However, it appears that care should be exercised when applying chip seal to a pavement when its overall condition level or surface roughness is rated fair or worse.

Fog Seal and Rejuvenating Seal

A total of three test sections, two fog seals and one rejuvenating seal, were included in this study. The traffic volume varied between 2362 AADT and 15635 AADT. Friction testing was conducted over a 36-month period. The main findings are as follows:

1) After applying a fog seal, the surface friction experienced a dramatic decrease by more than 50%. However, the variability of friction also deceased by more than 60%. This implies that a fog seal may improve the pavement surface uniformity, and provide more consistent surface characteristics.
2) It took about 18 months for the surface friction to return to the original level. The maximum life of a typical fog seal is around 24 months without the effect of traffic application.
3) The performance and effectiveness of a fog seal varied with the surface characteristics of existing pavement to a large extent.
4) After applying a rejuvenating seal, the surface friction experienced a significant reduction by more than 40% and then increased and peaked in 30 days.
5) Sand blotters resulted in a friction increase for rejuvenating seal by 8 points and became ineffective after 1 or 2 days.
6) The rejuvenating agent on the pavement surface could dry out in about 1 month. The pavement surface friction was unable to return to its original level after the application of a rejuvenating seal. The friction dropped by more than 18% in the passing lane and 35% in the driving lane after 24 months of application.

Microsurfacing

A total of 6 microsurfacing test sections were monitored in this study. The AADT in these test sections varied from 1,740 to 15,600. The age of microsurfacing in these test sections varied between 18 months and 42 months. The main conclusions are drawn as follows:

1) The freshly placed microsurfacing could produce sufficient surface friction when opening to traffic. The lowest friction number was 28 and the greatest friction number was 57 in the test sections.
2) The surface friction on the freshly placed microsurfacing varied significantly from test section to test section due probably to the effect of curing process. However, a freshly placed microsurfacing pavement could produce consistent surface properties and early opening to traffic.
3) The surface friction of microsurfscing increased significantly in the first six months and reached the maximum number approximately after 12 months of service. It appears that the microsurfacing surfaces became stable and produced true friction numbers between 40 and 60 after 12 months of service.
4) After 12 months of service, the surface friction in the test sections tended to decrease continuously over time. However, no friction numbers less than 30 were observed after a service period up to 42 months.
5) The MPD values in the test sections varied between 0.66 mm and 0.94 mm.
6) The smoothness improvement from microsurfacing depended to some extent on the smoothness of existing pavement. The rougher the existing pavement surface the greater the smoothness improvement after placing microsurfacing. However, the improvement of smoothness was limited.
7) The two main distress modes observed in the test sections are delamination and reflective cracking. It appears that special care should be exercised when microsurfacing is applied to high traffic volume roads, particularly urban roads.
8) Delamination commonly occurred at the interface between the old pavement and microsurfacing. Also, the microsurfacing is prone to reflective cracking.
9) The noise differences between microsurfacing and 4.75-mm HMA overlay were not perceptible on the roadside and in the vehicle. After about 22 months, the L_{Aeq} was 69.7 dB(A) on the roadside and 66.1 dB(A) in the vehicle at 55 mph in the microsurfacing section. In the 4.75-mm HMA section, the L_{Aeq} was 69.2 dB(A) on the roadside and 64.8 dB(A) in the vehicle at 55 mph.

Ultrathin Bonded Wearing Course

Four UBWC test sections were included in this study. The traffic volume varied from light to high and the age varied between 12 and 48 months. The main findings are as follows:

1) UBWC is capable of providing sufficient and consistent skidding resistance to allow quick opening to traffic. The friction numbers on the fresh UBWC pavements varied between 48 and 59 in the test sections.
2) The friction numbers in the test sections tended to peak after 6 months of service or less, approximately 6 months earlier compared to conventional HMA mixes.
3) UBWC has the potential to provide durable friction performance. The friction number in one test section decreased by 8.3% after 48 months in service.
4) UBWC can provide coarse pavement surface. The measured MPD varied between 0.95 mm and 0.99 mm, which is much greater than that for conventional 9.5-mm HMA mixes.
5) Significant friction decrease over time was also observed. The surface friction could decrease by more than 34% after 33 months in service. Noticeable polishing had occurred to limestone aggregate.
6) The use of steel slag could enhance the long-term friction performance. In order to provide satisfied long-term friction performance, it requires highly durable, highly polish-resistant aggregates.

Thin 4.75-mm Dense-graded HMA Overlay

Four 4.75-mm HMA pavements were included in this study, covering a wide range of traffic volume from less than 2,000 AAD (77 trucks) on a state road to over 100, 000 AAD (19,475 trucks) on an interstate highway. The service life varied from 6 months to 54 months. The following conclusions are drawn from this study:

1) A fresh 4.75-mm HMA overlay is capable of providing a friction number between 35 and 52. However, the surface friction may decrease quickly and dramatically over time after opening to traffic. It was observed that in the test sections, the surface friction decreased by 25% after 6 months of service and by 36% to 48% after 12 months in service.
2) The rate of friction decrease depends on traffic volume. After approximately 12 to 18 months, the 4.75-mm HMA overlays in the test sections tended to produce steady surface characteristics. It appears that on a steady 4.75-mm HMA surface, the friction number commonly varies between 20 and 30 and was commonly around 20 on high traffic volume roads.
3) The 4.75-mm HMA overlays produced very smooth surface. The majority of MPD measurements varied between 0.20 mm and 0.25 mm.
4) The 4.75-mm HMA mixes in the test sections were basically fine-graded mixes with too much fine sand and tended to produce significant fluctuation in surface friction over time, particularly on high traffic volume roads. It is critical to employ an aggregate gradation to pass below the PCS control point so as to enhance the friction properties of 4.75-mm dense-graded mixes.
5) The CA ratio has effect on the macrotexture of a 4.75-mm dense-graded mix. A greater CA ratio may produce larger macrotexture and can be achieved by reducing the percent passing the PCS or increasing the percent passing the HS. However, a large CA ratio may result in unbalanced coarse aggregate structure. Likely, a CA ratio approaching but less than 1.0 will have positive effect on the macrotextrue of a 4.75-mm mix.
6) Aggregate type has great effect on surface friction. In reality, 4.75-mm dense-graded mixes tend to produce macrotextures of small dimensions and the microtexture may become more dominant in surface friction for 4.75-mm dense-graded mixes, particularly in the long term. The use of steel slag will produce good texture properties.
7) Any measures for enhancing FAA may improve surface friction. A greater SE indicates more coarse particles and may result in better macrotexture properties. The aggregate angularity and abrasion resistance also play an important role in producing and maintaining sound surface characteristics. In addition, it was observed that a greater binder grade could result in better surface texture.

Profile Milling

Three profile milling test sections were included in this study. The traffic volume varied between 5,000 AADT and 25,000 AADT. The main findings are provided below:

1) While no new profile milling project was included in this study, it can be concluded that the steady-state friction number should not be less than 36 on a freshly profile milled HMA surface and should not be less than 40 on a freshly profile milled concrete surface.
2) The surface friction on the profile milled surface varied over time. The long term friction performance of a ground surface depends on the aggregate properties and grinding texture configuration. It is the aggregate and land area that play an important role in providing durable texture, particularly microtexture.
3) It appears that longitudinal grinding is capable of providing immediate and long term improvement to surface friction on both light and high traffic volume roads.

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