Evaluation of Long-Term Pavement Performance and Noise Characteristics of Open-Graded Friction Courses Project 1: Final Report

WA-RD 683.2

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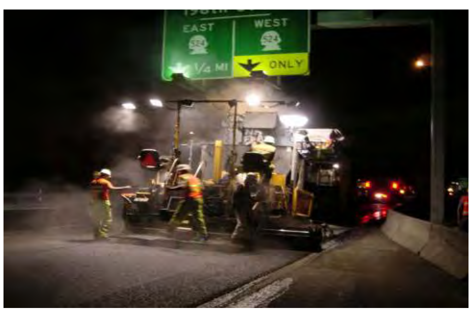
WSDOT Research Report

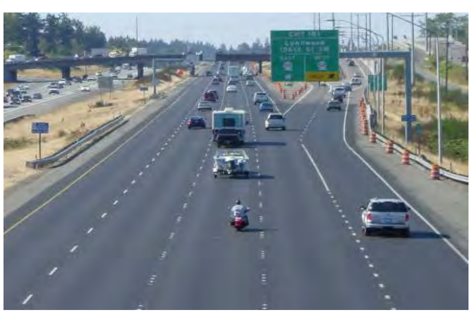
Experimental Feature Report

Report
Experimental Feature WA 05-06

Evaluation of Long-Term Pavement Performance and Noise Characteristics of Open-Graded Friction Courses

Contract 7134
I-5
52nd Avenue West to SR-526 – Southbound
MP 180.10 to MP 189.30

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16. ABSTRACT

This report describes the first of three experimental installations of open-graded friction course (OGFC) “quieter pavements” designed to reduce the noise generated at the tire/pavement interface. Experimental sections of OGFC were built using asphalt rubber (AR) and styrene-butadiene-styrene (SBS) polymer modified asphalt binders. A section of conventional hot mix asphalt (HMA) served as the control section for the two experimental sections.

The noise level of the OGFC-AR test section was audibly quieter than the HMA control section for only a period of four months after construction. The OGFC-SBS section was not initially audibly quieter than the HMA but attained that level of noise reduction for brief periods of time that extended to fourteen months after construction. The OGFC test sections were prone to excessive raveling and rutting, especially the OGFC-AR test section which in places wore through to the underlying pavement. The OGFC-AR test section was removed in the fall of 2010 after only four years of service due to safety concerns with vehicles having to cross the deep ruts during shifts of traffic necessary for the construction of new ramps for the Alderwood Mall interchange.

Open graded friction course quieter pavements are not recommended for use in Washington State due to the short duration of their noise mitigation properties and higher life cycle cost.

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Introduction

This experimental feature documents the construction and performance of two quieter pavements: (1) an open graded friction course (OGFC) modified with an asphalt rubber binder, hereafter referred to as OGFC-AR and (2) an OGFC with a styrene-butadiene-styrene (SBS) polymer asphalt binder, hereafter referred to as OGFC-SBS. OGFC pavement, with its higher volume of surface voids (a minimum of 15 percent air voids), absorb some of the noise generated at the tire/pavement interface and are thus "quieter" than dense graded pavements having fewer surface voids (around 4-8 percent). The performance of the OGFC sections are compared to the performance of a HMA control section constructed at the same time.

Open graded pavements are not new to the Washington State Department of Transportation (WSDOT). OGFC's were used extensively in the state in the early to middle 1980's. Their use was discontinued in 1995 due to problems with excessive rutting caused by studded tire wear. The renewed interest in open graded pavements is prompted by successful use of this type of pavement in other states, principally Arizona. The use of rubberized open graded pavements as one solution to making pavements quieter has been promoted in numerous road industry publications. News reports on rubberized open graded pavement as the answer to making pavements quieter has encouraged the public to ask that these types of pavement be used to lessen the noise in their neighborhoods.

Background

There are downsides with the use of open graded pavements. Open graded pavements are very susceptible to excessive wear from studded tires. This excessive wear produces ruts in the pavements that fill with water during rainy periods and pose the additional hazard of hydroplaning. The other downside is pavement life. The life of open-graded pavements is cut short by the studded tire wear mentioned previously. Pavement life of less than 10 years, and as short as three to four years were experienced with OGFC's in the 1980's in Washington. States where the use of OGFC has been successful (Florida, Texas, Arizona and California) do not experience extensive studded tire usage. Similarly, these states are southern, warm weather states; a clear advantage when placing a product like OGFC with asphalt-rubber. Arizona DOT, for example, requires the existing pavement to have an 85°F surface temperature at the time of placement. Paving in urban areas must, by necessity to lessen traffic impacts, be done at night when temperatures rarely approach 85°F even in summer, making successful placement of this type of pavement a challenge.

Open graded pavements are also popular with the drivers due to benefits beyond noise reduction. Drivers have improved visibility during rain storms on open-graded pavements due to the open void structure that drains away excess water. The quick drainage of water away from the surface of the pavement also improves the wet weather friction resistance of the roadway and decreases the potential for hydroplaning. At night the increased drainage capability helps to improve visibility by reducing the glare associated with standing water on the pavement. Painted traffic markings are also more visible at night because of less water standing on the roadway.

Project Description

The site selected for the first experimental use of the OGFC was located on I-5 near the town of Lynnwood, Washington (Figure 1). The project, Contract 7134, 52nd Ave. West to SR 526 SB Paving and Safety, consisted of paving the southbound lanes from Milepost (MP) 180.10 to MP 188.65. Wilder Construction Company* from Everett, Washington placed the OGFC on top of the existing pavement on all three general purpose lanes and the HOV lane with the exception of the second lane. The second lane, which was extensively cracked and rutted, was milled to a depth of 0.15 feet and inlaid with an equal depth of PG 64-22 HMA prior to placement of the ¾ inch OGFC overlay. The average daily traffic (ADT) on this section of I-5 is 79,800 with 7.3% trucks (2005 data).

*Note: Wilder Construction Company was sold to Granite Construction Company in early 2008.

Figure 1. Location of Contract 7134 near Lynnwood, Washington.

The OGFC-SBS test section extended from MP 180.75 to 181.82, a distance of 1.07 miles and the OGFC-AR test section ran from MP 181.82 to 182.58, a distance of 0.76 miles (Figure 2). The remainder of the project received a 0.15 foot mill and fill with Class ½ inch HMA. The section of HMA from the beginning of the project at MP 180.10 to the beginning of the OGFC-SBS section at MP 180.75 was designated as the control section.

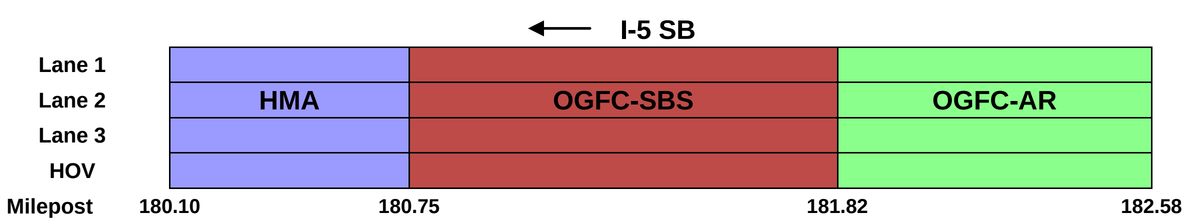


Figure 2. Map of section layout.

Mix Design Process

Special mix design processes were required for both of the open-graded pavements. The asphalt rubber mix design was the design developed and used by the Arizona Department of Transportation (ADOT). The polymer mix design was developed in-house and based on discussions with the National Center for Asphalt Technology. It used a slightly modified version of the ADOT aggregate gradation for the asphalt rubber mix design. The mix designs can be found in Appendix A.

OGFC-AR

ADOT was called upon for help in the development of the mix design for the OGFC-AR. Their expertise with the design and use of asphalt-rubber goes back many years. Granite Construction, the parent company of the project's Contractor, Wilder Construction, hired the firm of MACTEC Engineering and Consulting, Inc., located in Phoenix, AZ to develop the binder design which combined the binder with the crumb rubber. The binder design was provided to ADOT, along with the aggregate gradation, who then verified the mix design. The lab reports from both ADOT and MACTEC are found in Appendix A. The design called for a fine mix with 55-70 percent in the #4 to 3/8 inch size and 22-37 percent in the #8 to #4 size. The design binder content was 9.2 percent with 0.50 percent anti-strip additive and the amount of crumb rubber added was set at 22 percent by weight of the asphalt binder. Three stockpiles were used from pit site B-335, 3/8 inch chips, #4 to #8 sand, and #4 to 0 sand blended at the ratios of 72, 20 and 8 percent, respectively. Tesoro Corporation, Anacortes, Washington was the source of the PG64-22 asphalt binder and Crumb Rubber Manufacturers, Rancho Domingo, California, provided the crumb rubber for the project. The OGFC-AR mix design properties are summarized in Table 1.

Table 1. Mix design for the OGFC-AR.			
Sieve Size	**Gradation**	**Specifications**	**Source/Supplier**
3/8”	100	100	B-335
#4	34	30-45	B-335
#8	8	4-8	B-335
#200	1.5	0–2.5	B-335
Binder Grade	**Percent Asphalt**		**Source/Supplier**
PG64-22	9.2		Tesoro, Anacortes, WA
Anti-Strip	**Percent**		
ARR-MAZ 6500	0.50		Arr-Maz Custom Chemicals, Mulberry, FL
Crumb Rubber	**Percent by Wt. of AC**		**Source/Supplier**
CRM	22		Crumb Rubber Manufacturers, Rancho Domingo, CA

OGFC-SBS

The mix design for the OGFC-SBS was performed in the WSDOT Headquarters Materials Laboratory. The starting point for the percent of asphalt was determined by an asphalt retention process which suggested using 6.8 percent. Samples were mixed at 6.3, 6.8 and 7.3 percent asphalt and evaluated using the FHWA pie plate drain down test. The results of the drain down test suggested using more asphalt than the initial target of 6.8 percent. Additional samples were then mixed using 7.8, 8.3, and 8.8 percent asphalt and evaluated using the drain down test. All of the samples were in specification for volumetric properties, but the drain down test suggested that 8.3 percent was optimum; therefore, the asphalt content for the mainline paving was set at this level. The gyration level used for the mix design was 50 based on a recommendation from the ADOT.

Aggregate for the OGFC-SBS came from the same pit site, B-335, and the same three stockpiles were used, but the blend differed from the OGFC-AR having 66 percent from the 3/8 inch chips stockpile, 21 percent from the #4 to #8 stockpile, and 13 percent #4 to 0 stockpile. Fiber produced from recycled paper was used as a stabilizing additive to thicken the asphalt binder and prevent drain down. The SBS modified PG70-22 binder was provided by US Oil, Tacoma, Washington and the fibers were obtained from Hi-Tech Asphalt Solutions, Mechanicsville, Virginia. U.S. Oil reported that the amount of SBS in the PG70-22 was 3.6 percent by weight of the asphalt. The OGFC-SBS mix design properties are summarized in Table 2.

Table 2. Mix design for the OGFC-SBS.

Sieve Size	Gradation	Specifications	Source/Supplier
3/8"	100	100	B-335
#4	37	35-55	B-335
#8	10	9-14	B-335
#200	2.1	0–2.5	B-335
Binder Grade	**Percent Asphalt**		**Source/Supplier**
PG70-22	8.3		US Oil, Tacoma, WA
Anti-Strip	**Percent**		**Source/Supplier**
ARR-MAZ 6500	0.25		Arr-Maz Custom Chemicals, Mulberry, FL
Stabilizing Additive	**Percent**		**Source/Supplier**
Processed recycled paper	0.30		Hi-Tech Asphalt Solutions
Rubber	**Percent by Wt. of AC**		**Source/Supplier**
SBS	3.4±1		U.S. Oil, Tacoma, WA

Construction

The Special Provisions for the contract contains several items pertaining to the construction of the two special OGFC pavements. A brief description of these items is included in this section of the report as a guide to understanding the circumstances under which the sections were constructed. The unabridged Special Provisions are included as Appendix B.

OGFC-AR Special Provisions

The Special Provisions required that the asphalt binder for the OGFC-AR would be a PG58-22 or PG64-22. The crumb rubber must conform to the gradation requirements shown in Table 3. The crumb rubber will have a specific gravity of 1.15 ± 0.05 and will be free of wire or other contaminating materials, except that the rubber will not contain more than 0.5 percent fabric. Calcium carbonate could be added to prevent the particles from sticking together. The minimum amount of crumb rubber required in the mix was 20 percent by weight of the asphalt binder.

Table 3. Gradation requirement for crumb rubber.

Sieve Size	Percent Passing
No. 8	100
No. 10	100
No. 16	65 – 100
No. 30	20 – 100
No. 50	0 – 45
No. 200	0 – 5

The temperature of the asphalt binder at the time of the addition of the crumb rubber should be between 350 and 400°F. A one-hour reaction period was required after the mixing of the rubber with the binder. At the end of the reaction period the rubber particles must be thoroughly "wetted" without any rubber floating on the surface or agglomerations of rubber particles observable. The temperature of the asphalt-rubber immediately after mixing will be between 325 and 375°F.

The mixed asphalt-rubber must be kept thoroughly agitated during the period of use to prevent the settling of the rubber particles. In no case can the asphalt-rubber be held at a temperature of 325°F or above for more than 10 hours. Asphalt-rubber held for more than 10 hours must be allowed to cool and gradually reheated to the prescribed temperature. A batch of asphalt-rubber can only be cooled and reheated in this manner once.

OGFC-SBS Special Provisions

The asphalt binder for the OGFC-SBS will be a PG70-22 produced by adding SBS modifier to a non air blown or oxidized PG58-22 or PG64-22. The fibers required in the mixture can be cellulose fibers, cellulose pellets, or mineral fibers. If the mix was produced in a dryer-drum plant, fibers were required to be added to the aggregate and uniformly dispersed prior to the injection of the asphalt binder. Storage time for the OGFC-SBS was not to exceed four hours.

Asphalt Plant

The asphalt plant was a dryer-drum type plant located at Wilder Construction's Smith Island facility. Granite Construction provided the additional equipment for the production of the asphalt-rubber binder. The photos shown in Figures 3 through 6 depict the process used to load the crumb rubber into a shear mixer prior to its storage in heated and agitated tanks. Once the asphalt and rubber mixture was blended the process for producing the HMA was no different than conventional dryer-drum plant production.

Modifications to the plant were also necessary for the production of the OGFC-SBS. Figure 7 shows the fiber mixing and distribution machine which introduced a controlled amount of fiber into the dryer drum. Figure 8 shows the supply line hose and hose fitting on the dryer drum.

Figure 3. Lifting bags of crumb rubber.

Figure 4. Loading of crumb rubber into double bin supply hopper.

Figure 5. Double bin crumb rubber weigh hopper.

Figure 6. Shear mixer used to combine the crumb rubber with the asphalt binder.

Figure 7. Fiber supply machine.

Figure 8. Fiber supply hose and hose fitting in dryer drum plant.

Paving Operations

Paving began on August 17, 2006 with the construction of test sections of both OGFC-AR and OGFC-SBS. The test sections were located on Ross Avenue near the site of the Wilder Construction Company asphalt plant. Following successful completion of the test sections, the mainline paving began on August 19. The paving of the OGFC test sections was completed in two consecutive weekend closures, August 19 and 20 and August 25 and 26. A summary of the paving operation from the Inspector's Daily Reports is listed in Table 4. One additional paving day was needed to replace a section of the OGFC-SBS that had a rough ride and was showing excessive raveling immediately after construction. Additional information can be found on this problem under the section on Construction Problems.

Table 4. Paving history for OGFC-AR and OGFC-SBS.

Date	Lane	Milepost Limits	Pavement Type	Comments
August 19	Collector	181.83 – 182.27	OGFC-AR	Collector lane between I-405 and SR-524.
August 19	1	181.83 – 182.59	OGFC-AR	Mix temperatures from 285 to 317°F.
August 19	2	181.83 – 182.59	OGFC-AR	
August 20	3 & 4	181.83 – 182.59	OGFC-AR	
August 20	2	181.73 – 181.83	OGFC-SBS	
August 25	3 & 4	180.76 - 181.83	OGFC-SBS	Mix temperatures from 268 to 310°F. Air temperatures from 67-73°F.
August 26	1	180.76 – 181.83	OGFC-SBS	Mix temperatures from 260 to 305°F. Air temperatures from 68-74°F.
August 26	2	180.76 – 181.52	OGFC-SBS	Shuttle Buggy not used from Sta. 9584+00 to 9544+00.
September 23	2	180.76 – 181.52	OGFC-SBS	Sta. 9584+00 to 9544+00 replaced using Shuttle Buggy

The paving of the mainline was done at night; therefore, most of the photos of the paving operation shown below are from the paving of the test sections. The first two, Figures 9 and 10, show the streaky application of the CRS-2P tack coat at the beginning of each pass of the distributor truck on the test section paving.

Figure 9. Tack coat application on test section.

Figure 10. Globs of excess tack on the roadway.

The second set of photos, Figures 11 and 12, show the Ingersoll Rand PF 5510 Blaw-Knox paver laying down the OGFC-AR test section.

Figure 11. Paving the OGFC-AR test section.

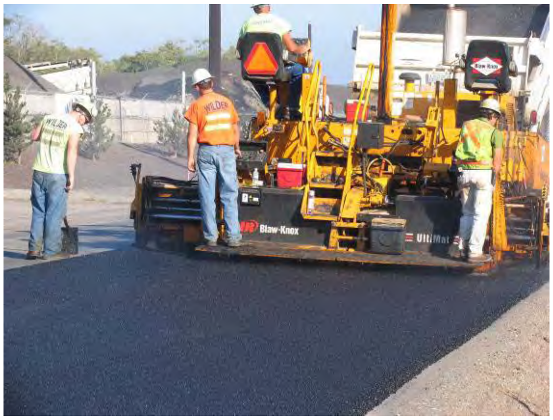


Figure 12. Another view of the OGFC-AR test section paving.

The third set of two photos, Figures 13 and 14, show the Ingersoll Rand DD-130 rollers that were used in the required static mode to compact the open-graded mix. Three rollers were used during the mainline paving with a fourth, an Ingersoll Rand DD-110, added on occasion.

Liquid soap was added to the water in the rollers (1 gallon of dish soap to 300 gallons of water) to prevent the rollers from sticking to the hot-mix.

Figure 13. Rolling of OGFC-AR test section with two rollers.

Figure 14. The soap bubbles on the pavement are from the soap added to the water to prevent sticking.

The fourth set of photos, Figures 15 and 16, show the OGFC-AR mix prior to the screed and a close-up of the mix during the compaction process.

Figure 15. OGFC-AR mix behind the material management kit prior to the screed.

Figure 16. Close-up of OGFC-AR mix during the compaction process.

The fifth set of photos, Figures 17 and 18, show the OGFC-SBS mix prior to rolling and after compaction.

Figure 17. OGFC-SBS prior to rolling. Blemishes are footprints.

Figure 18. OGFC-SBS after compaction.

The final two photos, Figures 19 and 20, show the Roadtec Shuttle Buggy material transfer vehicle used to remix the OGFC-AR and OGFC-SBS prior to passing it through the paving machine, and a long-distance view of the paving train. Both photos were taken from the actual mainline paving.

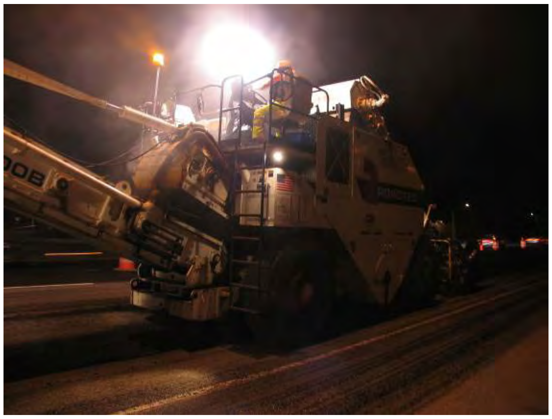


Figure 19. Roadtec Shuttle Buggy material transfer vehicle.

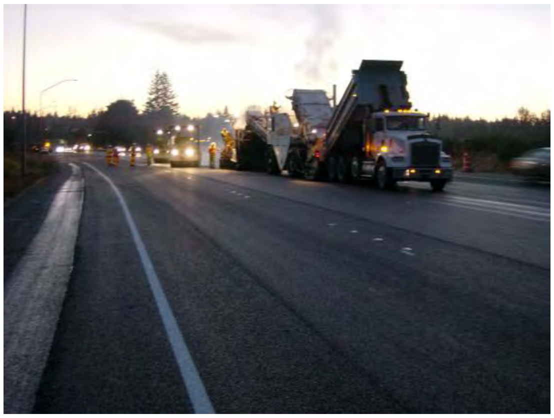


Figure 20. Paving train consisting of dump trucks, Shuttle Buggy, paver and rollers.

Cost

The bid prices for the three types of HMA used on Contract 7134 ranged from a low of $62.50 per ton for the Class 1/2 inch HMA to a high of $130.00 per ton for the OGFC-AR. The OGFC-SBS price was in the middle at $90.00 per ton. The estimated quantities and total cost for each type of pavement are shown in Table 5 along with the low bid price.

Table 5. Cost comparison information.

Bid Item	Estimated Quantity (tons)	Low Bid (per ton)	Total Cost
HMA	28,853	$62.50	$1,803,313
OGFC-AR	1,686	$130.00	$219,180
OGFC-SBS	2,441	$90.00	$219,690

Recycled Tire Use

One ton of OGFC-AR contains 33.2 lbs of crumb rubber. It takes approximately 300 tons of asphalt to pave one lane mile of pavement at a depth of 0.06 feet. Assuming that the weight of usable rubber in an average passenger tire is 18 pounds, there would be 550 tires consumed to produce one lane mile of OGFC-AR on this project. The total length of OGFC-AR paving including the four mainline lanes and ramps was 5.55 miles. This project, therefore, recycled approximately 3,050 tires.

Construction Problems

The only problem encountered during the placement of the open-graded sections was a breakdown of the Shuttle Buggy on the final day of paving of the OGFC-SBS section. The Contractor elected to complete the section without the use of a transfer device, which proved problematic, as the pavement had a rough riding surface and began to ravel almost immediately upon exposure to traffic. Wilder removed and replaced this section on September 23, 2006, a few days short of one month after the completion of the original OGFC-SBS test section.

Construction Testing

The construction process was monitored for specification compliance by the Project Engineers staff and for temperature differentials by personnel from the State Materials Laboratory Pavements Section.

Specification Compliance Testing

The gradation, percent asphalt and percent rubber for the paving of the test section and the two days of production paving are listed in Table 6. The Special Provisions called for acceptance to be based on meeting the gradation requirements because conventional asphalt content testing methods do not work with asphalt binders that contain rubber additives. The data for the percent of asphalt and rubber was from worksheets supplied by Granite Construction that listed the tons of binder, rubber and hot mix produced for each days paving. The production results show the gradations to be within specification limits, the asphalt content low on one day, high on the next day, on target for the third day, and the rubber percentage on target for all three days.

Table 6. Gradation, percent asphalt and percent rubber results for OGFC-AR.

Sieve	Target	Date					Specification
		8/17	8/19	8/19	8/20	8/20	
3/8	100	99	100	100	100	100	100
#4	34	36	33	33	32	36	30-38
#8	8	8	8	7	7	8	4-8
#200	1.5	1.6	1.6	1.6	1.5	1.8	0-2.5
% AC	9.2	9.01	9.64		9.20		9.2
% CRM	22.0	22.0	22.0		22.0		22.0

The information for the OGFC-SBS, Table 7, is similar in that data is only available for the gradations which all met the specification limits. The contract did not require that a percent of asphalt be measured for the production paving due to the addition of SBS which also, just as in the case of the rubber, negates normal asphalt content testing, however, percent asphalt values were calculated based on the tons of mix and tons of binder used on August 26 and it met the mix design recommendation.

Table 7. Gradation and percent asphalt results for OGFC-SBS.

Sieve	Target	Date				Specification
		8/17	8/25	8/26	9/22	
3/8	100	99	100	100	100	100
#4	37	35	37	40	41	35-55
#8	10	8	11	11	12	9-14
#200	2.1	0.8	2.3	2.5	2.0	0-2.5
%AC	8.3	8.2	-	8.3	-	8.3

In summary, both the OGFC-AR and OGFC-SBS pavements were constructed in compliance with the mix design recommendations for aggregate gradation, asphalt content, and in the case of the OGFC-AR, crumb rubber content.

Temperature Monitoring

An infrared camera was used throughout the paving operation to monitor the temperature of the mix as delivered in the dump trucks and as it passed through the paver and was placed on the roadway. Temperature differences of as much as 160°F were noted between the cooler crust of mix that forms on top of the delivery trucks (140°F) and the hot mix under the crust (300°F). Based on past experience, temperature differences of this magnitude would normally lead to significant portions of the mat having density problems because the significantly cooler material from the crust cannot be compacted when a transfer device is not used. However, the Special Provisions required the use of a Roadtec Shuttle Buggy to remix the asphalt from the dump trucks before it was placed in the paver. Figure 21 shows the typical temperature differences noted in the delivery vehicles with Spot 2 the crust at a temperature of 101° F and Spot 1 the hotter mix at 303° F.

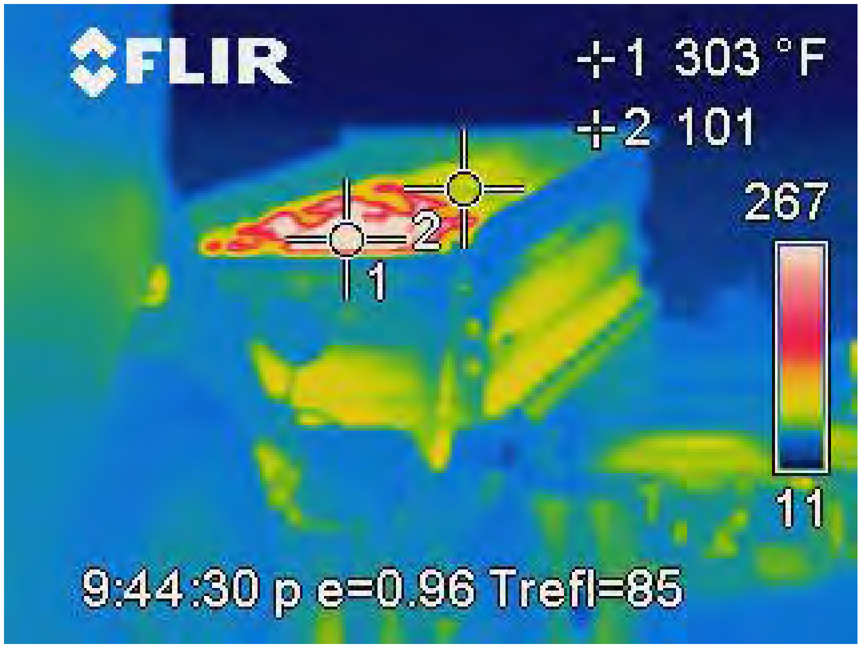


Figure 21. Temperature differentials in a delivery truck.

Figure 22 is an infrared photo of the mix as comes out of the paver after having been remixed by the Shuttle Buggy. The even red color across the width and length of the mat illustrates the positive effects that remixing has on the uniformity of the temperatures across the mat. The aforementioned was the typical image captured throughout the project when the Shuttle Buggy was in use. As noted previously, a portion of the project was completed without the use of the Shuttle Buggy and although this portion of the paving was not documented with infrared images, past experience would indicate that the mat would have significant temperature differentials, poor riding quality and the possibility of issues with raveling.

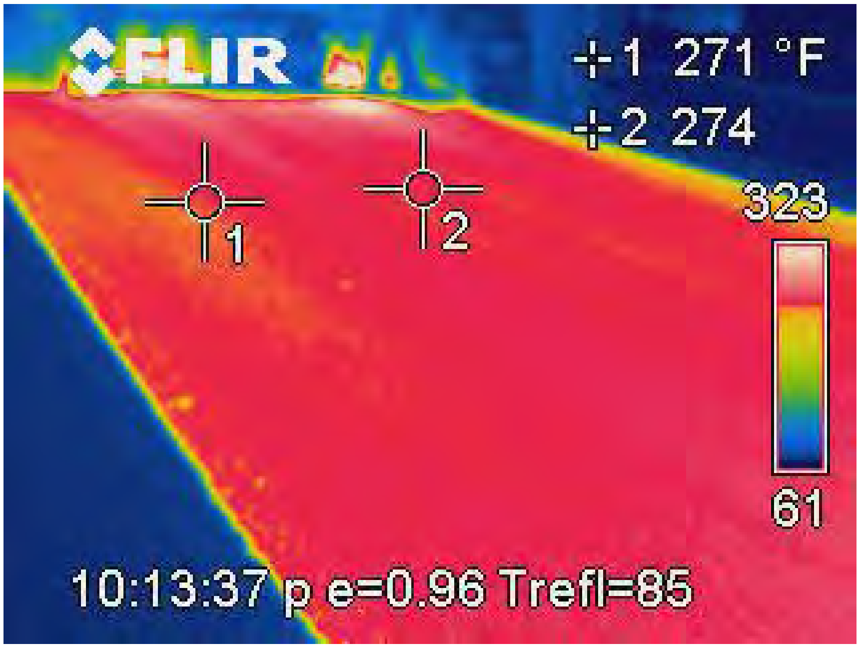


Figure 22. Infrared photo of the mat behind the paver showing uniform temperatures.

Temperature monitoring is important on all hot mix projects, but especially critical on overlays that are as thin as the OGFC used on this project. Documentation of the construction operation is included as Appendix C.

Performance Monitoring

Acoustic performance and pavement wear were the two main criteria used to judge the success of the OGFC quieter pavement test sections on I-5. For acoustic performance, the two questions to be answered were; (1) are the OGFC pavements audibly quieter than the HMA control section and, (2) how long do they remain audibly quieter? For pavement wear, the question was; how long will the OGFC pavements stand up to studded tire wear and the climatic conditions of Washington? Ride and friction resistance were also evaluated for all OGFC pavements and the conventional HMA.

Noise

Traffic noise is a concern for many residents living along state highways. This study was the first of three trial installations of OGFC pavements designed to reduce the noise generated from our highway facilities and its effects on nearby residents. Historically, noise barriers have been the most common method for reducing traffic noise. Noise barriers include noise walls and earthen berms that separate traffic noise from adjacent properties. Typical noise reduction is 5 to 10 decibels, with 10 decibels reducing the perceived noise level by 50 percent. While noise barriers can be effective, they can be expensive to install and are not constructible or effective in all locations.

Table 8 shows the relationship between the sound intensity level change, the relative loudness and the acoustic energy loss. Noise experts agree that sound intensity levels must differ by at least three decibels to be noticeable to the human ear (audibly quieter). For this study the OGFC sections will be considered “quieter” when their sound intensity levels are at least three decibels lower than the sound intensity level of the HMA control section. The use of OGFC quieter pavements would not be justified if they are not audibly quieter than the HMA for a reasonable period of time.

Table 8. Sound level changes, loudness and acoustic energy loss comparison.		
Sound Level Change	**Relative Loudness**	**Acoustic Energy Loss**
0 dBA	**Reference**	**0**
-3 dBA	**Barely Perceptible Change**	**50%**
-5 dBA	**Readily Perceptible Change**	**67%**
-10 dBA	**Half as Loud**	**90%**
-20 dBA	**1/4 as Loud**	**99%**
-30 dBA	**1/8 as Loud**	**99.9%**

Noise Measuring Equipment

The On-Board Sound Intensity (OBSI) method was used to measure acoustic performance, that is, "noise" on the quieter pavement sections. Measurement methods and equipment are in general conformance with the provisional AASHTO specification TP 76-13 *Measurement of Tire/Pavement Noise Using the On-Board Sound Intensity (OBSI) Method.* OBSI measures the noise at the tire/pavement interface using two microphones mounted vertically four inches from the outside tire sidewall on the rear passenger side tire (Figure 23). The microphones are suspended three inches above the pavement surface on either side of where the tire meets the roadway. This close proximity to the tire/pavement interface ensures that only the noise from this area is recorded and that traffic noise from other sources like drive train, engine, or exhaust are not captured.

Measured sound intensity data is analyzed using a PULSE multi-channel data analyzer. The PULSE system was set-up to report overall sound intensity values that are the sum of A-weighted sound intensity within the 1/3 octave frequencies of 500-5000Hz using a multi-channel analyzer (Figure 24). For each 1/3 octave band, the system also measures the coherence of sound pressure between the two microphones and the pressure-intensity index (PI index).

The Uniroyal Tiger Paw AWP (P225/60R16) mounted on the dedicated WSDOT Ford Taurus sedan used for all OBSI measurements is equivalent to the Standard Reference Test Tire (SRTT) (P225/60R16) defined in ASTM 2493.

Figure 23. Twin microphones mounted near the rear tire of a vehicle. Note Uniroyal Tiger Paw tire.

Figure 24. Computer used for data collection.

Noise Measurements

Initial measurements were made on the existing pavement prior to the overlay to serve as a base line. The pavements on this section of I-5 were eight to twenty years old with an average age of twelve years. Pre-construction sound intensity levels ranged from 103.8 to 105.3 dBA with an average of 104.6 dBA. After construction of the quieter pavement test sections, OBSI measurements were collected monthly, weather permitting (pavement must be dry), on all four lanes of the two OGFC test sections and the HMA control sections. Three measurements were collected for each lane in each section of pavement. The average sound intensity levels for the test sections (all four lanes) are listed in Table 9. Measurements of the sound intensity levels for individual lanes within the each test section are listed in Appendix D.

Table 9. Average monthly sound intensity level measurements for each test section.

Date	OGFC-AR	OGFC-SBS	HMA
August 2006	95.1		99.4
Early September 2006	95.0	96.0	98.8
Late September 2006	96.0	96.3	98.9
October 2006	98.4	98.0	100.3
Early December 2006	96.9	97.3	100.4
Late December 2006	98.5	99.7	101.8

Table 9. (Continued)			
Date	OGFC-AR	OGFC-SBS	HMA
January 2007	98.5	98.6	101.3
Early March 2007	99.5	98.2	101.5
Late March 2007	101.7	100.5	102.3
April 2007	99.8	99.4	101.5
May 2007	99.9	99.2	101.8
June 2007	99.5	98.7	100.6
July 2007	99.1	98.0	101.3
Early September 2007	99.4	97.6	100.7
Late September 2007	99.9	98.5	101.6
October 2007	101.5	99.2	102.7
December 2007	102.3	99.8	102.6
January 2008	102.1	99.5	102.2
February 2008	102.7	100.2	102.7
April 2008	103.0	100.5	103.3
May 2008	103.4	100.9	103.4
July 2008	101.4	99.9	101.9
August 2008	102.1	99.9	102.8
December 2008	104.4	102.3	103.7
January 2009	104.9	103.5	103.8
February 2009	104.1	102.0	103.6
March 2009	104.8	102.9	104.5
April 2009	103.8	102.6	103.2
May 2009	103.3	102.3	103.2
July 2009	104.8	104.8	105.5
September 2009	103.1	101.9	103.7
October 2009	104.9	103.2	104.0
December 2009	106.5	105.2	105.2
January 2010	106.6	104.6	105.4
February 2010	105.3	104.0	104.6
March 2010	103.9	103.0	103.5
April 2010	104.7	103.2	104.2
May 2010	104.4	103.2	104.1
June 2010	104.7	104.7	104.4
July 2010	104.7	103.6	104.3
August 2010	103.3	101.8	103.5

The sound intensity levels of each section increased over time with the OGFC-AR increasing the most with an 8.2 dBA gain after 48 months. The OGFC-SBS gained the next highest at 5.8 dBA and the HMA gained the least at 4.1 dBA. At the end of the four year monitoring period the sound intensity levels of the OGFC-AR (104.4 dBA), the OGFC-SBS (103.3 dBA), and the HMA (104.1 dBA) were approaching the same sound intensity level as the existing pavement prior to construction. The sound intensity levels quoted are the average of the last five measurements for the OGFC-AR and HMA.

Table 10 lists the difference between the average sound intensity level of the HMA control section and the average sound intensity level of each OGFC test section: HMA (dBA) – OGFC (dBA). Figure 25 shows these differences graphically. Data points above the black horizontal line are three decibels quieter than the HMA control section and considered to be audible to the human ear (≥3 dBA). Data points below the line are not audibly different. A red horizontal line at 0.0 marks the point below which the OGFC becomes noisier than the HMA control section.

Table 10. Difference in average sound intensity level between OGFC sections and HMA control section.

Pavement Age (months)	OGFC-AR Difference From HMA (dBA)	OGFC-SBS Difference From HMA (dBA)
0	4.3	no measurement
0.5	3.8	2.9
1	3.0	2.6
2	1.9	2.3
3	3.4	3.1
4	3.3	2.1
5	2.8	2.8
6	2.0	3.3
7	0.6	1.8
8	1.7	2.1
9	1.9	2.6
10	1.0	1.9
11	2.2	3.3
12	1.6	3.3
13	1.7	3.1
14	1.3	3.5
15	1.3	3.5

Table 10. (Continued)		
Pavement Age (months)	OGFC-AR Difference from HMA (dBA)	OGFC-SBS Difference from HMA (dBA)
16	0.3	2.8
17	0.1	2.7
18	0.0	2.5
19	0.0	2.5
20	0.2	2.8
21	0.0	2.5
22	0.0	2.5
23	0.4	2.0
24	0.7	2.9
25	0.7	2.9
26	0.7	2.9
27	0.7	2.9
28	-0.7	1.5
29	-1.1	0.3
30	-0.5	1.6
31	-0.3	1.6
32	-0.5	0.6
33	-0.1	0.9
34	-0.1	0.9
35	0.7	0.7
36	-0.7	0.4
37	0.0	1.1
38	-0.9	0.8
39	-0.9	0.8
40	-1.3	0.0
41	-1.2	0.8
42	-0.1	0.6
43	0.0	0.6
44	-0.1	1.0
45	-0.3	0.8
46	-0.3	-0.3
47	-0.2	0.7
48	0.2	1.8

Note: Readings in yellow are audible to the human ear; readings in green indicate the OGFC section is noisier than the HMA.

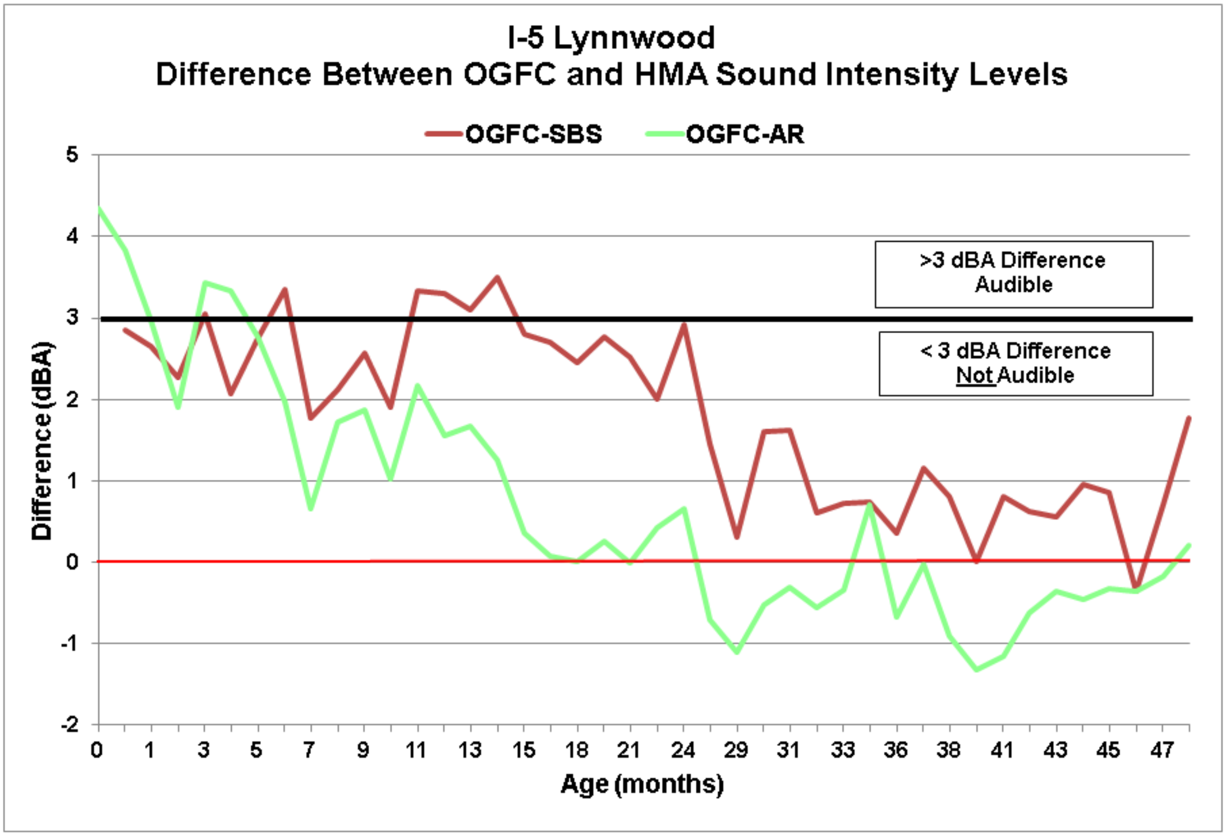


Figure 25. Difference in average sound intensity between the HMA control section and the OGFC-AR and OGFC-SBS pavements.

The OGFC-AR (green line) section was generally audibly quieter than the conventional HMA for four months after construction. After four months, the OGFC-AR was no longer audibly quieter than the conventional HMA. In fact, Figure 25 shows how the OGFC-AR became noisier (falls below the red line at zero difference) than the HMA section at 27 months.

Initially, the OGFC-SBS section (reddish-brown line) was not audibly quieter than the HMA. Between six months and 15 months after construction, the OGFC-SBS section was occasionally (six times) audibly quieter than the HMA section. At 27 months the OGFC-SBS began a downward trend and at one point became noisier than the HMA (46 months).

In summary, both of the OGFC sections were audibly quieter than the HMA control section for brief periods up to 15 months after construction. After that point neither was audibly quieter for the remainder of the four year monitoring period.

Seasonal Variations in Sound Intensity Level

The sound intensity levels of both OGFC sections increased much more over the August 2006 to August 2010 monitoring period than the HMA control section (Table 9).

- OGFC-AR increased 8.2 dBA
- OGFC-SBS increased 5.8 dBA
- HMA increased 4.1 dBA

The sound intensity level increases were also accompanied by higher rates of rutting and raveling of the OGFC section as compared to the HMA section (detailed later in this report). The data suggests a link between sound intensity level increases and rutting and raveling caused by studded tires. Table 11 shows the changes in average sound intensity levels for all three pavements during the winter when studded tires are legal (November 1 to March 31) and the summer when studded tires are banned (April 1 to October 31). Figure 26 is a line graph of the same information.

Table 11. Change in average sound intensity levels for winter and summer.

Studs or No Studs	Time Between Measurements (months)	OGFC-AR (dBA)	OGFC-SBS (dBA)	HMA (dBA)
Summer 2006	3	3.3	2.0	0.9
Winter 2006-07	5	3.3	2.5	2.0
Summer 2007	7	-0.3	-1.3	0.4
Winter 2007-08	5	1.9	1.3	0.6
Summer 2008	4	-1.2	-0.6	-0.5
Winter 2008-09*	7	2.7	3.0	1.7
Summer 2009	7	0.1	0.3	-0.5
Winter 2009-10	6	-0.2	0.0	0.2

* **Severe winter storm in December 2008 produced increases in noise levels and rutting.**

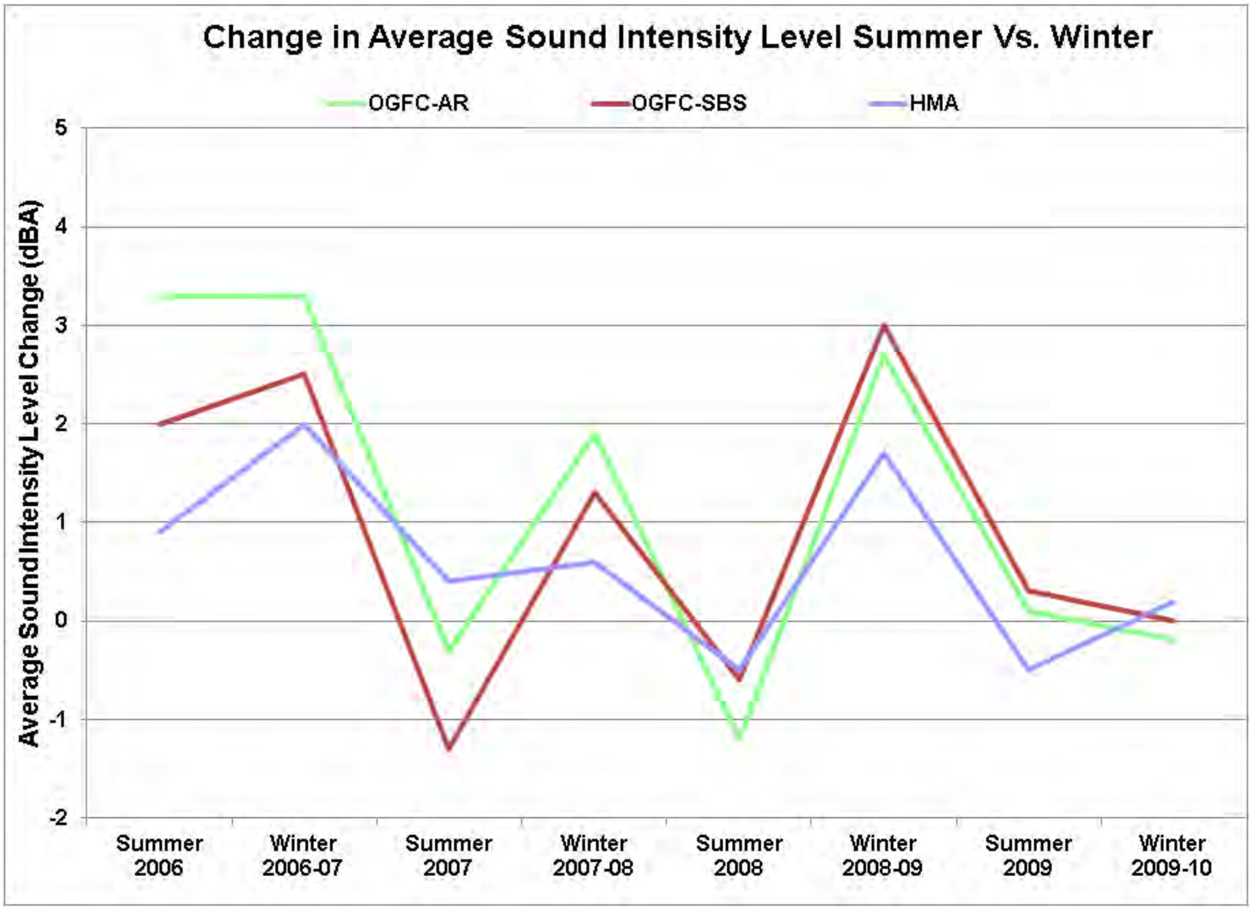


Figure 26. Change in average sound intensity level summer versus winter when studded tires are legal.

With the exception of the first summer following construction, the largest changes in sound intensity levels for all of the pavements occurs during the winter, as shown by the peaks each winter in Figure 26. The largest change in the sound intensity levels for all of the sections occurs at the Winter 2008-2009 readings correspond to a severe storm in the Seattle area in December of 2008. WSDOT Maintenance personnel reported increased use of studded tires and chains during this period. The larger increase in sound levels for both OGFC sections (green and reddish-brown lines) than the HMA pavement (blue line) suggests that winter conditions (studded tires, tires with chains, or colder temperatures and increased moisture) have a greater negative effect on the acoustic performance of OGFC pavement than HMA pavement. This is not surprising since open graded pavements are known to have less strength than dense graded pavements due to their higher void content. OGFC's lower strength makes them more susceptible to damage from studded tires or tires equipped with chains. Their higher content of voids also makes them more susceptible to damage from moisture and freezing and thawing.

Seattle Area 2008 Winter Storm

The Puget Sound region experienced multiple storms for a period of two weeks starting on December 13, 2008. Freezing temperatures, snow, sleet, freezing rain, heavy rain and high winds produced significant challenges to travel throughout the region between December 13 and the 27th. Seattle recorded 13.9 inches of snow when typical December accumulations average 2.2 inches. The storm resulted in more consecutive hours of snow on the ground than any other storm in the last 20 years. Statistics from the National Weather Service Forecast Office show that the average low temperature for the event was 25.4ºF with three days setting record lows of 22, 19, and 14 degrees. Snowfall was recorded on 11 of the 14 days with five of the days receiving record amounts for Seattle. "Metro, the Seattle areas transit system, put tire chains on 80 percent of its 1,329-bus fleet overnight. But after the chains kicked up sparks on bare pavement during the morning commute, forcing drivers to go 35 mph or less to avoid tearing up the roadways, the chains were removed" (see link below).
http://www.azcentral.com/offbeat/articles/2008/12/17/20081217seattlesnow.html

Center of Lane Noise Measurements

Additional reasons for the changes in the sound intensity levels of the OGFC sections can be found in the examination of special measurements taken between the wheel paths in the center of each lane in September of 2007, 13 months after construction. The between wheel paths sound intensity measurements are compared to the wheel path measurements to assess what affect environmental factors have on the sound intensity levels since the center of the lanes should have significantly less traffic than the wheel paths. The results are shown in Table 12 and Figure 27.

Table 12. Sound intensity readings for Lane 2 in the wheel paths and in the center of the lane compared to initial results following paving.			
Date/Test Location/Date	OGFC-AR	OGFC-SBS	HMA
September 2007/Wheel Path	101.6	98.6	102.0
September 2007/Center of Lane	96.4	96.0	99.7
Wheel Path After Paving*	95.1	96.0	99.4

*** After paving values for each pavement section are from Table 8.**

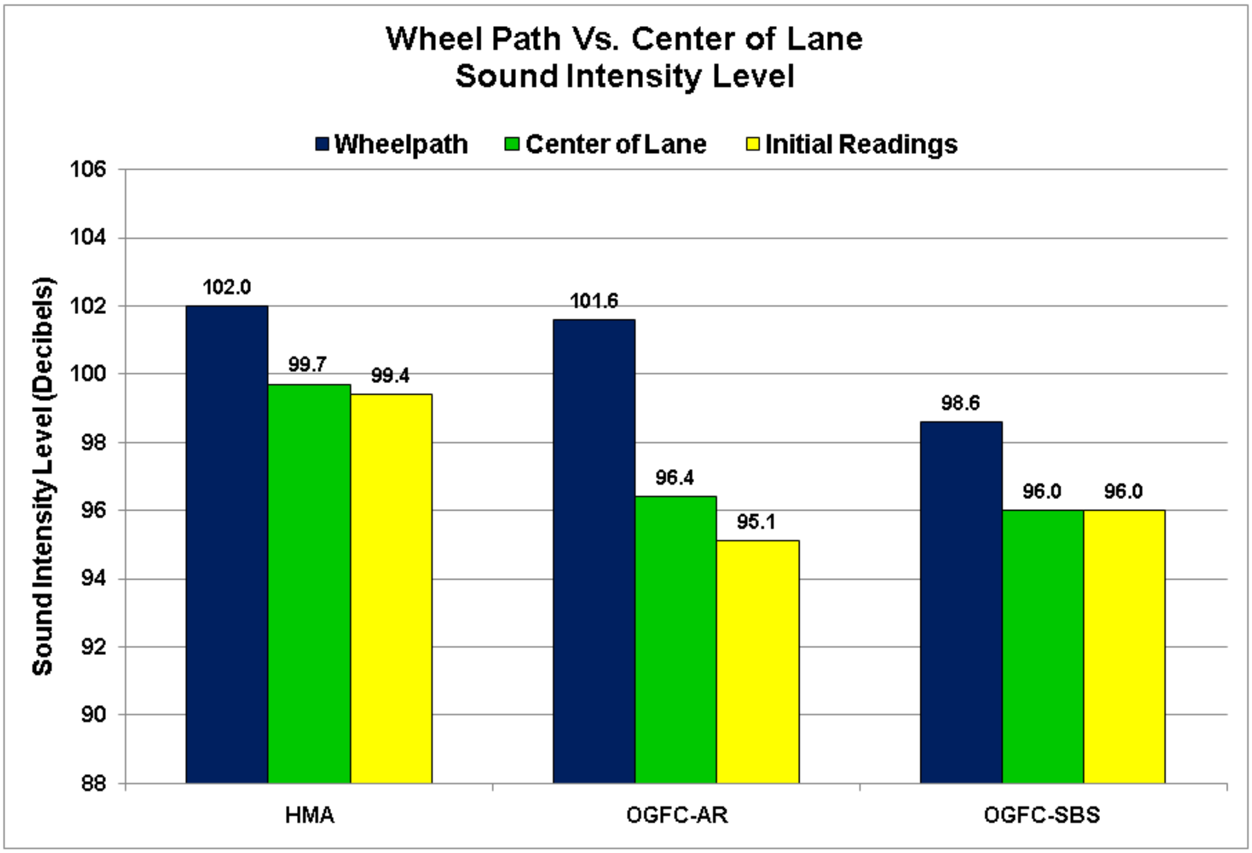


Figure 27. Sound intensity levels in the center of the lane and in the wheel path versus the initial readings after paving for Lane 2.

For all three pavements the center of lane sound intensity levels were very similar to the initial post-construction wheel path measurement with none of the pavements differing in sound intensity level by more than the 1.3 dBA (the OGFC-AR section). This absence of a difference in sound intensity levels is significant since the 13 month period of time between the two readings includes a winter studded tire season. This means that environmental factors do not have a big affect on the sound intensity levels of the sections and that traffic is the major contributor to changes in sound intensity level. In contrast, the comparison of wheel path measurements during the same time period showed a deterioration of acoustic performance of 6.5 dBA for the OGFC-AR, 2.6 dBA for the OGFC-SBS, and 2.6 dBA for the HMA. This data shows that all of the pavements are being negatively affected by traffic with respect to their sound intensity levels with the OGFC-AR pavement more affected than the OGFC-SBS or HMA.

Performance Difference between the OGFC-AR and OGFC-SBS

The rapid increase in the average sound intensity levels in the OGFC-AR section are not matched by equivalent increase in the OGFC-SBS section. Possible causes for the difference in noise mitigation characteristics are time, traffic, environment, and materials. Time, traffic and environment would seem to be identical for both the OGFC sections. Ruling out these as possible causes leaves only the materials as the possible reason for the unequal acoustical performance. Close examination of the material characteristics indicates a lot of similarity (Table 13). The aggregates, which make up almost 90 percent of the mix, are from the same pit site and had very similar gradations. The anti-strip additive for both mixes came from the same source and the amount used was within a quarter of a percent. The asphalt binder grades were also similar and the percent of asphalt binder used differed by less than one percent. The biggest difference between the OGFC mixes was the type and quantity of rubber used to modify the asphalt binder. The OGFC-AR used a very finely ground crumb rubber derived from recycled tires. The rubber content of the asphalt binder in the OGFC-AR mix was 22 percent. The rubber in the OGFC-SBS was a liquid synthetic rubber polymer that made up 3.4±1 percent of the asphalt binder. The OGFC-AR mix had more than six times the rubber content of the OGFC-SBS mix. The higher rubber content of the binder may be the cause of the accelerated raveling and rutting noted in the OGFC-AR section as well as the increase in sound intensity levels.

Table 13. Comparison of OGFC-AR and OGFC-SBS mix designs.			
Mix Characteristic		OGFC-AR	OGFC-SBS
Aggregate Source		B-335	B-335
Gradation	3/8” sieve	100	100
	#4 sieve	34	37
	#8 sieve	8	10
	#200 sieve	1.5	2.1
Asphalt Binder Grade		PG64-22	PG70-22
Asphalt Binder Percent		9.2	8.3
Anti-Strip		0.50 % ARR-MAZ 6500	0.25 % ARR-MAZ 6500
Rubber Type		Crumb	Synthetic Liquid
Percent Rubber in Binder		22 %	3.4±1 %

Noise Summary

The following facts have been determined concerning the noise mitigation performance of the OGFC and HMA sections.

- The OGFC-AR section was audibly quieter than the HMA control section for five months.
- The OGFC-SBS section was not initially audibly quieter than the HMA, but attained that noise reduction level at six months and between the eleventh and fifteenth month.
- Both of the OGFC sections became noisier than the HMA either briefly in the case of the OGFC-SBS at the 46th month, or for the remainder of the monitoring period at the 27th month in the case of the OGFC-AR.
- The changes in the noise reduction properties of all of the sections occurred during the winter season as a result of studded tire wear, chain usage, and snow plowing.
- Traffic is the primary cause of the changes in the noise properties of all three sections as attested to by the center of the lane testing at thirteen months showed essentially no change in the noise readings as compared to the post-construction values.
- The greater loss in the noise reduction properties of the OGFC-AR may be related to the type and quantity of rubber incorporated into the asphalt binder.

Wear/Rutting

It has already been shown that the changes in sound intensity levels during the winter studded tire season are greater than the change during the summer for all of the sections. The transverse profile measurements, which show the amount of wear or rutting in the wheel paths, are listed in Table 14 and shown graphically in Figures 28-30. Measurements were made in the Fall prior to the start of the legal studded tire season and in the Spring after studded tires must be removed.

Table 14. Wear/rutting measurements for each lane of the three test sections.

Section	Lane	Sep 2006	Apr 2007	Aug 2007	Oct 2007	Mar 2008	Oct 2008	Jan 2009	Apr 2009	Nov 2009	May 2010
AR	1	1.6	2.6	2.4	2.6	3.4	3.9	7.7	8.9	8.5	9.5
AR	2	1.9	2.5	2.2	2.3	3.2	3.5	5.1	6.0	5.9	7.4
AR	3	1.6	2.1	1.7	2.0	2.6	3.0	3.8	4.8	4.4	5.4
AR	HOV	1.3	1.9	1.4	1.5	2.5	2.4	3.5	4.5	4.1	4.8
Average		1.6	2.3	1.9	2.1	2.9	3.2	5.0	6.1	5.7	6.8
SBS	1	1.9	2.2	2.2	2.4	2.9	3.3	4.8	4.8	5.0	6.0
SBS	2	1.7	2.4	2.8	3.0	3.6	4.4	5.0	5.7	5.4	7.1
SBS	3	2.0	2.4	2.0	2.3	2.8	3.1	3.6	4.2	4.1	4.9
SBS	HOV	1.8	2.0	1.9	2.0	2.6	2.7	3.1	3.7	3.3	4.0
Average		1.9	2.3	2.2	2.4	3.0	3.4	4.1	4.6	4.5	5.5
HMA	1		1.9	1.7	2.2	3.1	2.8	3.6	4.1	4.2	4.8
HMA	2		2.4	2.1	2.8	3.5	3.5	3.9	4.7	4.7	5.8
HMA	3		2.2	1.9	2.6	3.1	3.0	3.5	4.0	4.0	4.8
HMA	HOV		1.8	1.4	1.9	2.7	2.3	2.5	3.0	2.8	3.4
Average		1.8*	2.1	1.8	2.4	3.1	2.9	3.4	4.0	3.9	4.7
Age in Months		1	7	12	14	19	26	29	32	39	45

*** No rutting measurement taken for the HMA so the average rut depth for both OGFC section was used.**
Note: Color of rows in the table is in the same green for AR, brown for SBS, and blue for HMA color palette as the bars in Figures 28-30.

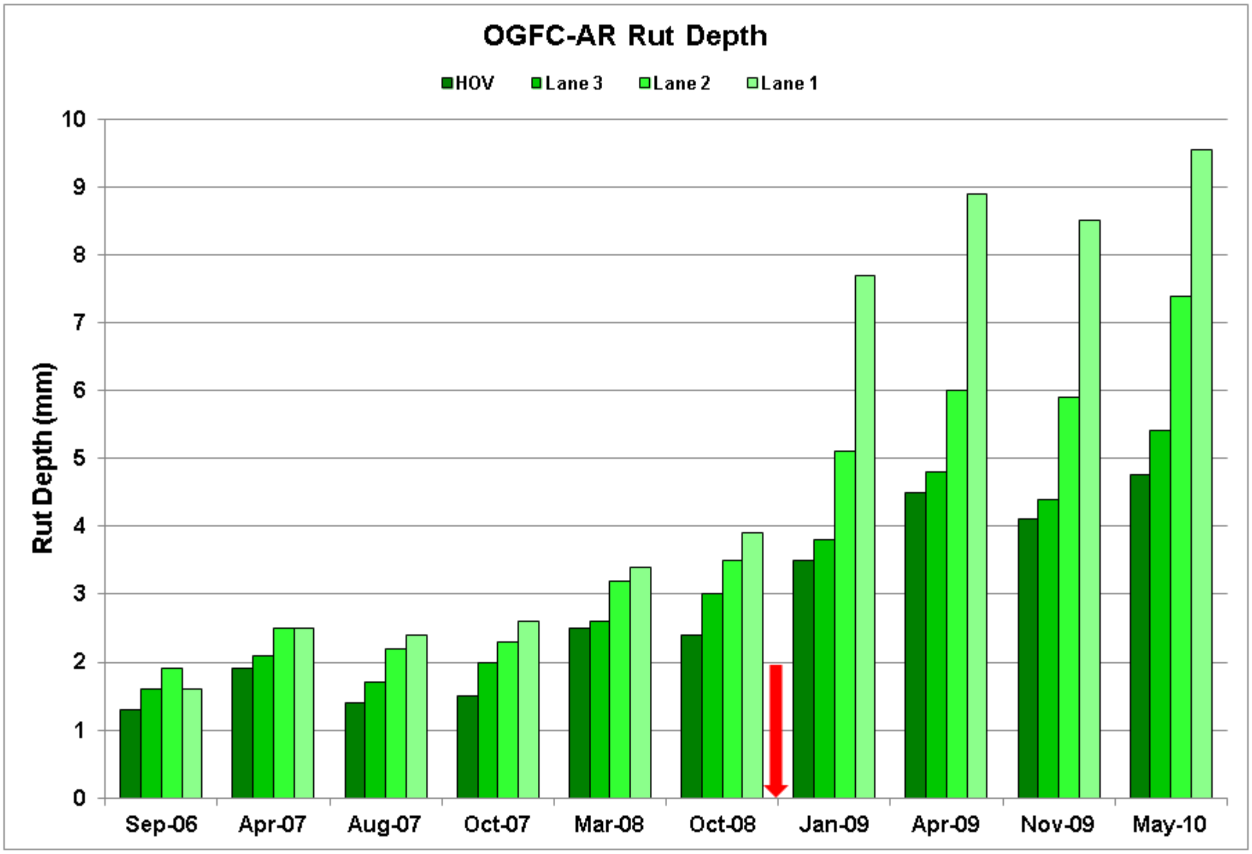


Figure 28. Rut depth for each lane of the OGFC-AR test section. Red arrow marks the severe winter of 2008/2009.

OGFC-AR Wear/Rutting

The OGFC-AR section showed the most amount of rutting/wear. A gradual increase is noted in the measurements through the October 2008 reading. A big jump is recorded in the rutting/wear in special readings taken in January 2009. These readings were taken following a severe storm in December of 2008 (marked with a red arrow in Figure 28). Maintenance personnel reported an increase in the use of chains and studded tires during this period. This increased rutting was very noticeable in Lane 1 (dark green bar) with almost a doubling of the rut depth (3.9 to 7.7 mm) between the October 2008 and January 2009 measurements. A maximum rut depth of 9.5 mm was recorded for Lane 1 in May 2010. The OGFC-AR was replaced in 2010 to facilitate the construction of a braided ramp at 196th Street SW. The rutting was too severe to allow a temporary lane shift because traffic would be driving across the ruts. Most of the OGFC-

AR was removed by grinding 0.06 to 0.08 feet for the lane shift and traffic drove on the ground pavement for a season. The remainder was removed at the end of the project when all lanes of I-5 were inlayed with HMA. The OGFC-AR would have been due for replacement in 2011 because of the rutting.

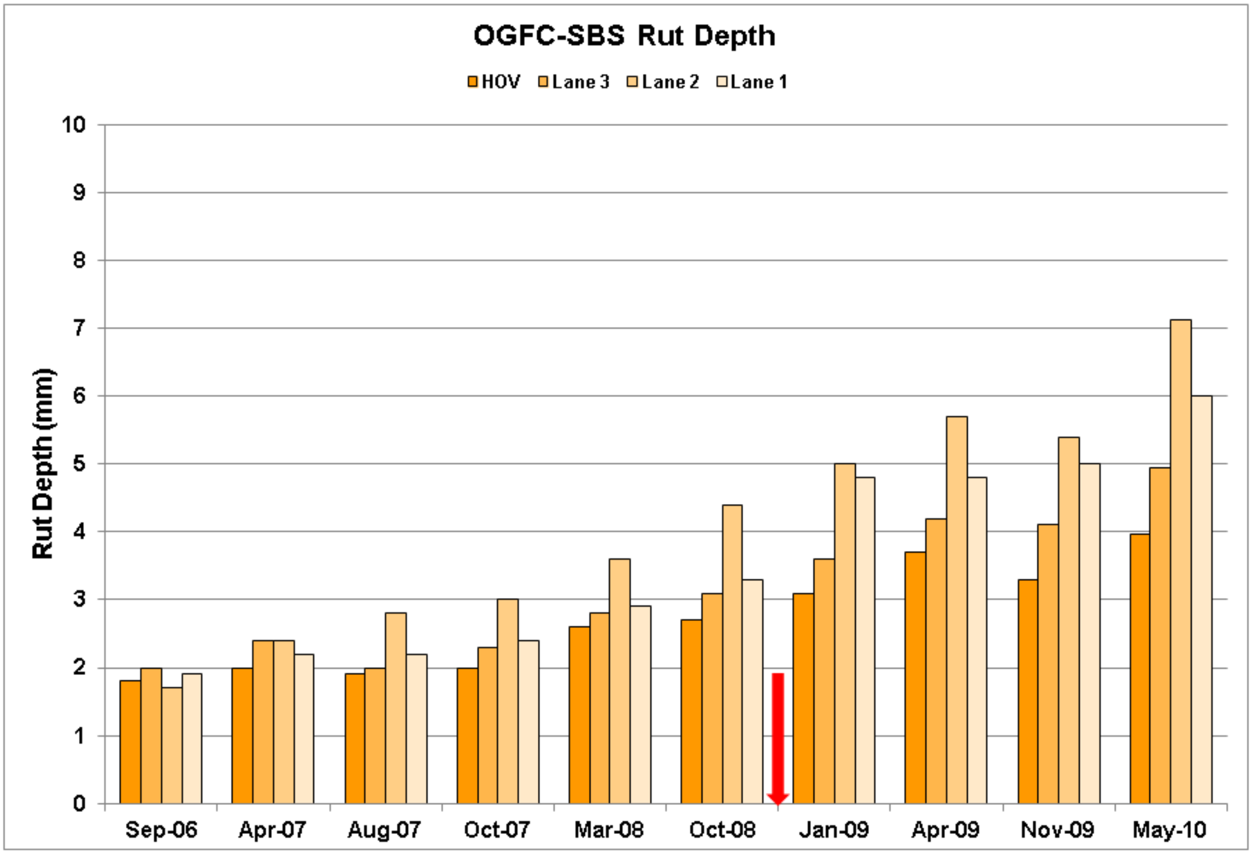


Figure 29. Rut depth for each lane of the OGFC-SBS test section. Red arrow marks the severe winter of 2008/2009.

OGFC-SBS Wear/Rutting

In contrast to the OGFC-AR, the increase in wear for the OGFC-SBS section is relatively linear with no large increases following the winter of 2008-2009 (Figure 29). The maximum rut depth for the OGFC-SBS was 7.1 mm recorded in Lane 2.

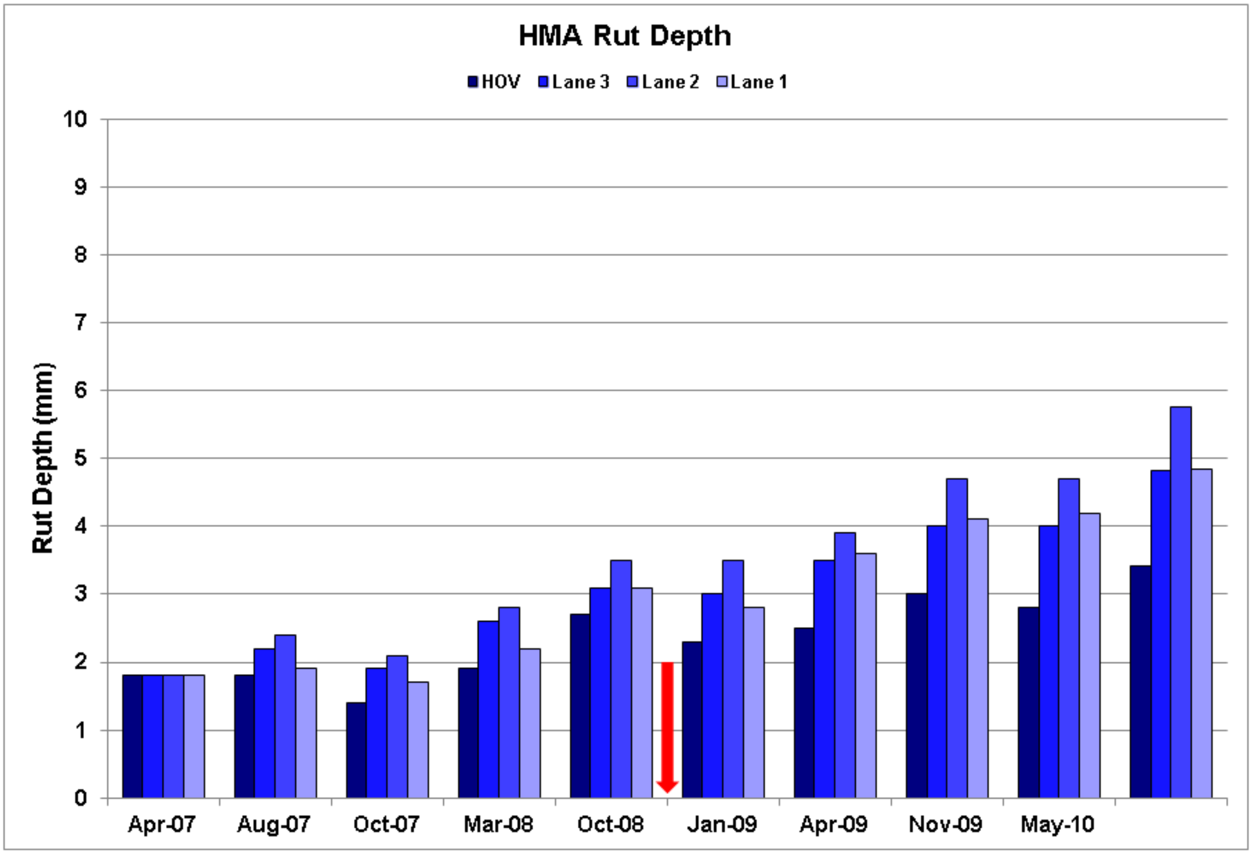


Figure 30. Rut depth for each lane of the HMA control section. Red arrow marks the severe winter of 2008/2009.

HMA Wear/Rutting

The rutting/wear in the HMA section is also linear with no big jump noted after the winter storm of December 2008 (Figure 30). The maximum wear/rutting recorded at the end of the monitoring period was 5.8 mm in Lane 2.

Historical Wear/Rutting On Open Graded Friction Courses

The rapid wear/rutting of the quieter pavement sections is consistent with our past experience with open graded friction courses (see Appendix E and F). The 56 projects listed in Appendix F had an average pavement life of 8.2 years with individual projects ranging from as little as 2.5 years of life to a maximum of 16 years. The end point for the life of the pavement was the time it took to reach a rut depth of 1/2 inch, the point at which a pavement is scheduled for rehabilitation.

The OGFC-AR test section was removed in the fall of 2010 after only four years of service due to safety concerns with vehicles having to cross the deep ruts during shifts of traffic necessary for the construction of new ramps for the Alderwood Mall interchange. The rutting in the OGFC-AR (Figure 31) was a result of raveling and not the result of deformation of the pavement due to compaction or shoving as evidenced by the accumulation of aggregate on the shoulder (Figure 32).

Figure 31. Rutting in OGFC-AR near Lynnwood. (2010)

Figure 32. Accumulation of aggregate on the shoulder in the OGFC-AR section. (2010)

Seasonal Variations in Wear/Rutting

The winter to summer change in the depth of the rut in the wheel paths shows a similar pattern to the sound intensity level readings with greater amounts of rutting/wear occurring during the winter season as compared to lesser amounts during the summer (Figure 33). The change in rut depth is calculated by subtracting the previous measurement from the current measurement. For example the OGFC-AR reading of 0.7 mm for the winter 2006-07 data point in Table 15 is calculated by subtracting the Fall 2006 reading from (1.6 mm) from the Spring 2007 reading (2.3 mm). Note that negative changes in rut depth are possible due to the transverse profile measurements which can vary a few millimeters due to inherent variations in the method used to measure rutting and the fact that wear can occur across the entire lane, not just the wheel path, thus reducing the depth of the rut.

Table 15. Change in rut depth winter versus summer.								
Section	Fall 2006	Spring 2007	Fall 2007	Spring 2008	Fall 2008	Spring 2009	Fall 2009	Spring 2010
OGFC-AR	1.6	2.3	2.1	2.9	3.2	6.0	5.7	6.8
Difference		0.7	-0.2	0.8	0.3	2.9	-0.3	1.1
OGFC-SBS	1.9	2.3	2.4	3.0	3.4	4.6	4.5	5.5
Difference		0.4	0.2	0.6	0.4	1.2	-0.1	1.1
HMA	1.8	2.1	2.4	3.1	2.9	3.9	3.9	4.7
Difference		0.3	0.3	0.7	-0.2	1.1	0.0	0.8

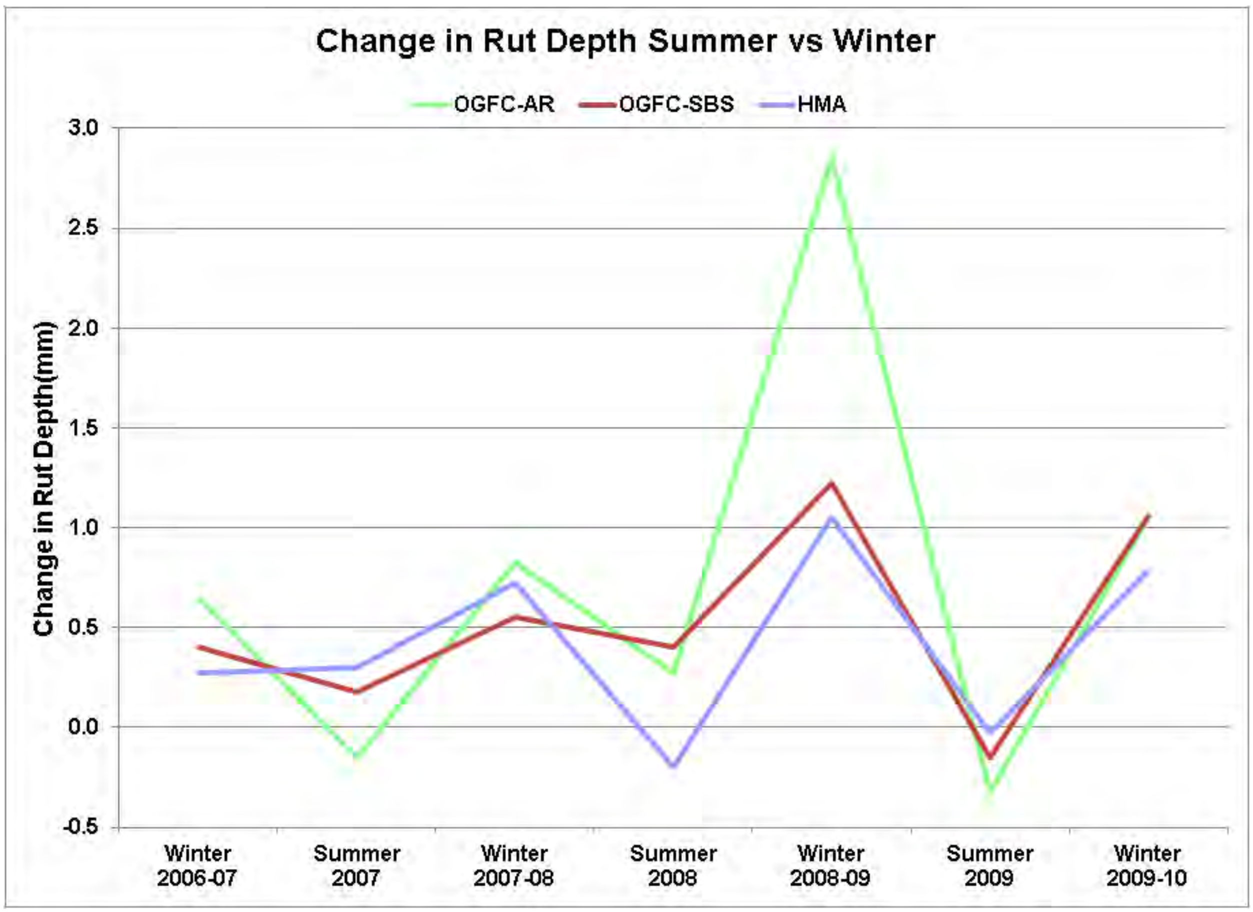


Figure 33. Change in rut depth during summer and winter for all three pavement types.

The average change in rut depth for each season is shown in Table 16. Figure 34 is a bar chart of the average change in the rut depth summer versus the winter. It shows that the OGFC-AR increases 1.3 mm each winter as compared to a -0.1 mm or essentially no increase during each summer. The OGFC-SBS increased 0.8 mm each winter as compared to a 0.1 mm increase each summer. The HMA increased 0.7 mm each winter and 0.0 each summer. This shows that for all of the sections the raveling and rutting is occurring only during the winter months which points to studded tires as the cause of this rutting and raveling. This may explain why OGFC-AR quieter pavements are more successful in states like Arizona, California, Texas and Florida which do not have high volumes of vehicles with studded tires.

Table 16. Average change in rut depth for each section.

Season	OGFC-AR	OGFC-SBS	HMA
Winter 2006-07	0.7	0.4	0.3
Summer 2007	-0.2	0.2	0.3
Winter 2007-08	0.8	0.6	0.7
Summer 2008	0.3	0.4	-0.2
Winter 2008-09	2.9	1.2	101
Summer 2009	-0.3	-0.2	0.0
Winter 2009-10	1.1	1.1	0.8
Summer Average	-0.1	0.1	0.0
Winter Average	1.3	0.8	0.7

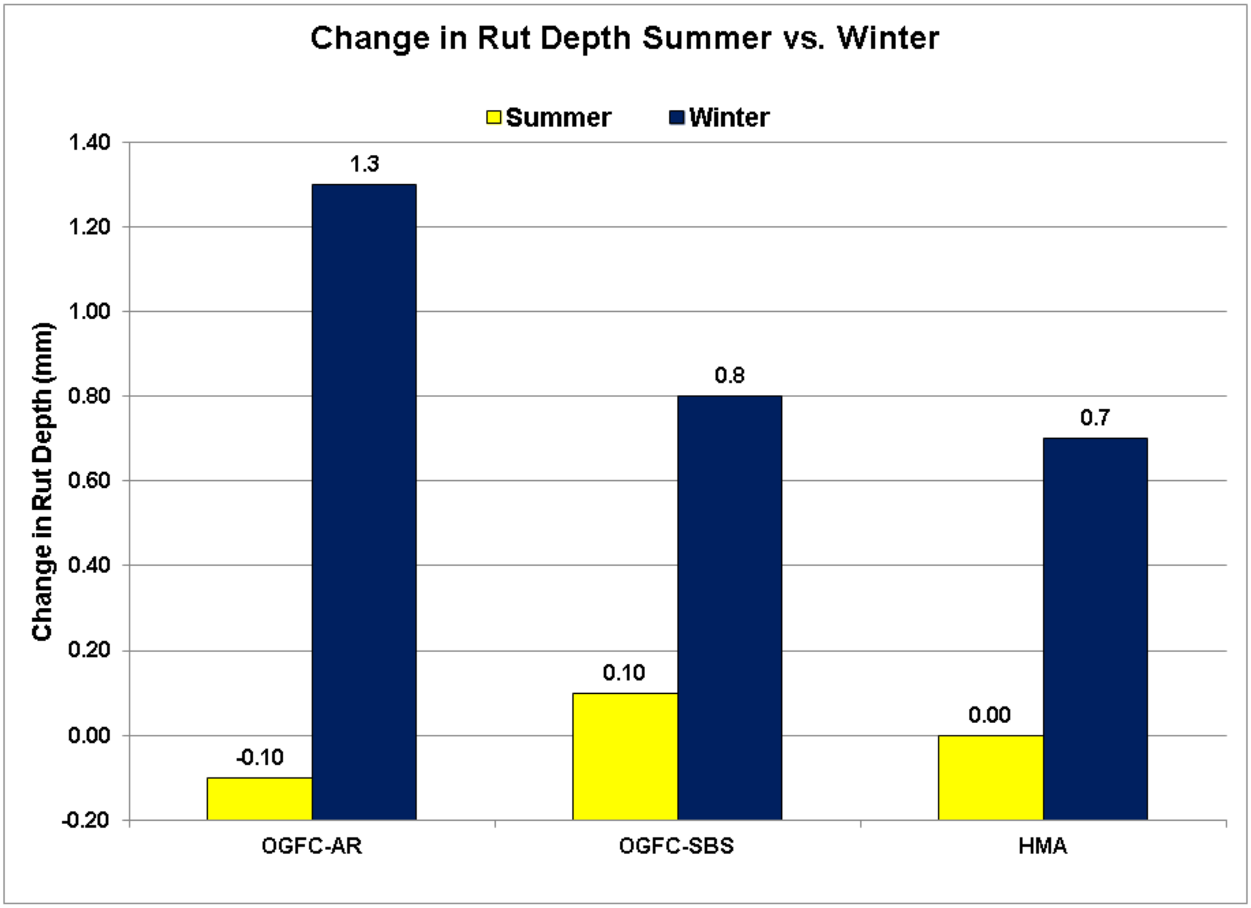


Figure 34. Average change in the rut depth during winters and summers for each section.

Wear/Rutting Summary

The following facts have been determined concerning the wear/rutting performance of the OGFC and HMA sections.

- The OGFC-AR section experienced the greatest amount of wear/rutting followed by the OGFC-SBS and then the HMA.
- The wear/rutting of the OGFC-AR increased dramatically following a severe winter storm that resulted in a higher use of studded tires and chains.
- The poor wear/rutting performance of the OGFC sections is consistent with previous experience with these types of pavements in Washington.
- The wear/rutting on all three sections is occurring primarily during the winter studded tire season.

Friction

Friction resistance measurements (Table 17) were made in the spring and fall of each year bracketing the studded tire season. The friction numbers were excellent for all three pavement types. The readings were lowest in the fall due to the accumulation of oils and other contaminants on the pavements and highest in the spring when the pavements had been cleaned by winter rains. The OGFC-AR generally had the highest readings and the HMA control section the lowest with the OGFC-SBS in the middle (Figure 35). The higher readings for the OGFC-AR may be the result of the rubber in the pavement or a more coarse surface texture due to raveling.

Table 17. Friction resistance (FN) readings.

Section	Lane	Apr-07	Oct-07	Apr-08	Oct-08	Apr-09	Nov-09	Apr-10
AR	1	53.5	48.1	50.6	44.4	57.0	50.9	51.6
AR	2	53.8	45.3	50.3	43.3	54.8	50.1	51.1
AR	3	56.1	44.5	51.4	44.5	56.6	50.8	53.3
AR	HOV	58.7	45.5	51.9	44.9	58.5	51.9	53.6
Average		55.5	45.9	51.1	44.3	56.7	50.9	52.4
SBS	1	52.4	46.8	49.4	42.6	55.8	49.4	49.6
SBS	2	52.7	45.2	49.2	41.4	53.8	46.6	48.4
SBS	3	54.7	44.0	50.9	42.1	57.7	47.2	50.6
SBS	HOV	55.7	45.1	51.5	44.1	51.8	49.0	50.9
Average		53.9	45.3	50.3	42.6	55.7	48.1	49.9
HMA	1	-	50.2	45.3	39.9	51.8	45.4	46.1
HMA	2	-	48.2	47.0	40.4	51.1	45.6	45.6
HMA	3	-	47.2	48.9	42.9	55.5	46.4	49.2
HMA	HOV	-	46.9	51.4	45.8	57.4	50.1	51.1
Average		-	48.1	48.2	42.3	54.0	46.9	48.0

Note: Row colors in the table are the same as the bars in Figures 35.

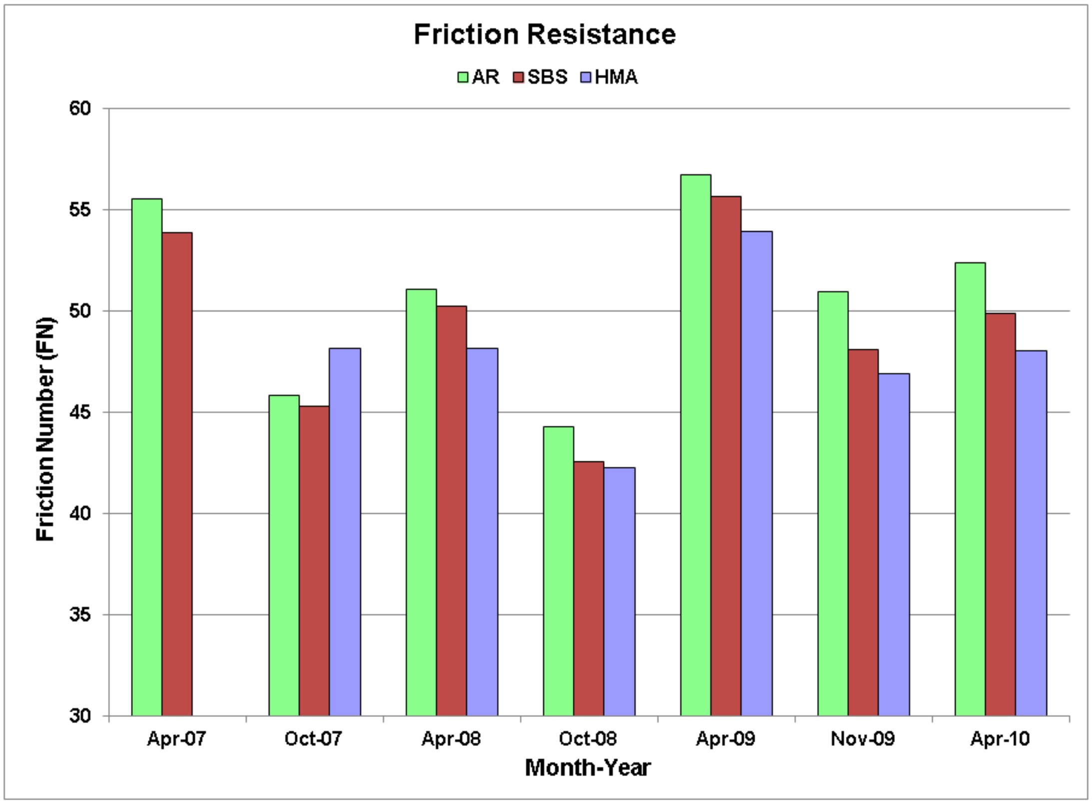


Figure 35. Average friction resistance for each pavement type.

Ride

Ride measurements made on all sections in the spring and fall of each year, bracketing the studded tire season, are listed in Table 18 and shown graphically in Figure 36. The measurements are in International Roughness Index (IRI) units of inches per mile. A ride score of 60 or lower is typical of good pavement construction for HMA. The OGFC sections had better ride readings than the HMA control section with the OGFC-AR (green) section being the smoothest until the winter of 2008-2009. At this point the OGFC-AR became rougher than the OGFC-SBS (brown) and this trend continued through to the end of the evaluation period. The ride readings on the OGFC-AR changed more than the OGFC-SBS or HMA as a result of the severe winter weather of 2008-2009.

Table 18. Ride measurements for each lane of the three test sections.											
Section	Lane	Sep 2006	Apr 2007	Aug 2007	Oct 2007	Mar 2008	Oct 2008	Jan 2009*	Apr 2009	Nov 2009	May 2010
AR	1	48	51	53	58	57	58	75	82	74	84
AR	2	36	40	40	44	45	44	54	57	52	58
AR	3	40	42	42	47	46	47	50	55	51	56
AR	HOV	39	40	43	45	47	47	55	60	57	63
Average		41	43	45	49	49	49	59	64	59	65
SBS	1	45	45	52	55	52	52	59	61	57	61
SBS	2	48	49	57	59	57	56	58	62	58	63
SBS	3	43	44	48	51	49	48	49	50	50	54
SBS	HOV	49	50	57	62	62	61	63	69	61	66
Average		46	47	54	57	55	54	57	61	57	61
HMA	1	-	64	53	71	69	64	68	76	73	75
HMA	2	-	64	46	66	62	60	59	64	64	71
HMA	3	-	58	51	65	66	60	62	65	65	73
HMA	HOV	-	71	60	72	73	70	69	77	74	82
Average		-	64	53	69	68	64	65	71	69	75

***Reading after the severe winter of 2008-2009.**
Note: Row colors in the table are the same as the bars in Figures 36.

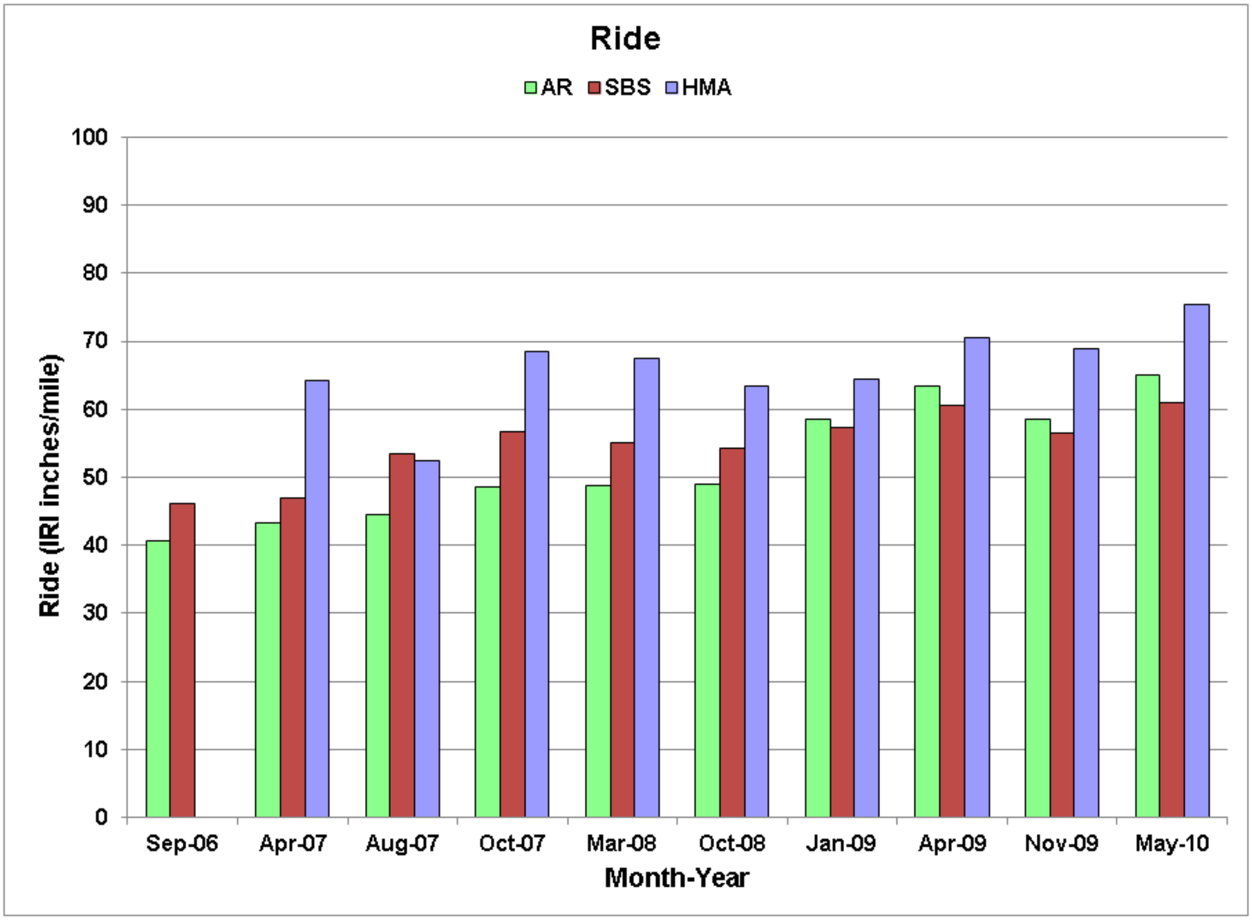


Figure 36. Average ride for each pavement.

Life Cycle Cost Analysis

WSDOT uses life cycle cost analysis (LCCA) to compare the cost of different pavement options. LCCA is a method of economic analysis that takes into account the initial as well as the future costs as noted in the following excerpt:

> "…an analysis technique that builds on the well-founded principles of economic analysis to evaluate the over-all-long-term economic efficiency between competing alternative investment options. It does not address equity issues. It incorporates initial and discounted future agency, user, and other relevant costs over the life of alternative investments. It attempts to identify the best value (the lowest long-term cost that satisfies the performance objective being sought) for investment expenditures."

In the case of the OGFC and the HMA control section, the future cost is the cost of repaving the roadway at the end of the pavements life. The life cycle cost is a function of how much it cost to pave the road and the time between each cycle of repaving.

The bar chart below compares the OGFC-AR and OGFC-SBS if they were replaced as soon as they were no longer audibly quieter than the HMA control section. Cost has been converted to uniform annual cost in order to directly compare the different pavement types. Although the audible noise reduction capability of the OGFC's were less than six months, for simplicity, one year was used in the calculation as the OGFC's pavement life with respect to audible noise reduction. The LCCA for the HMA control section based on performance data is also included for comparison.

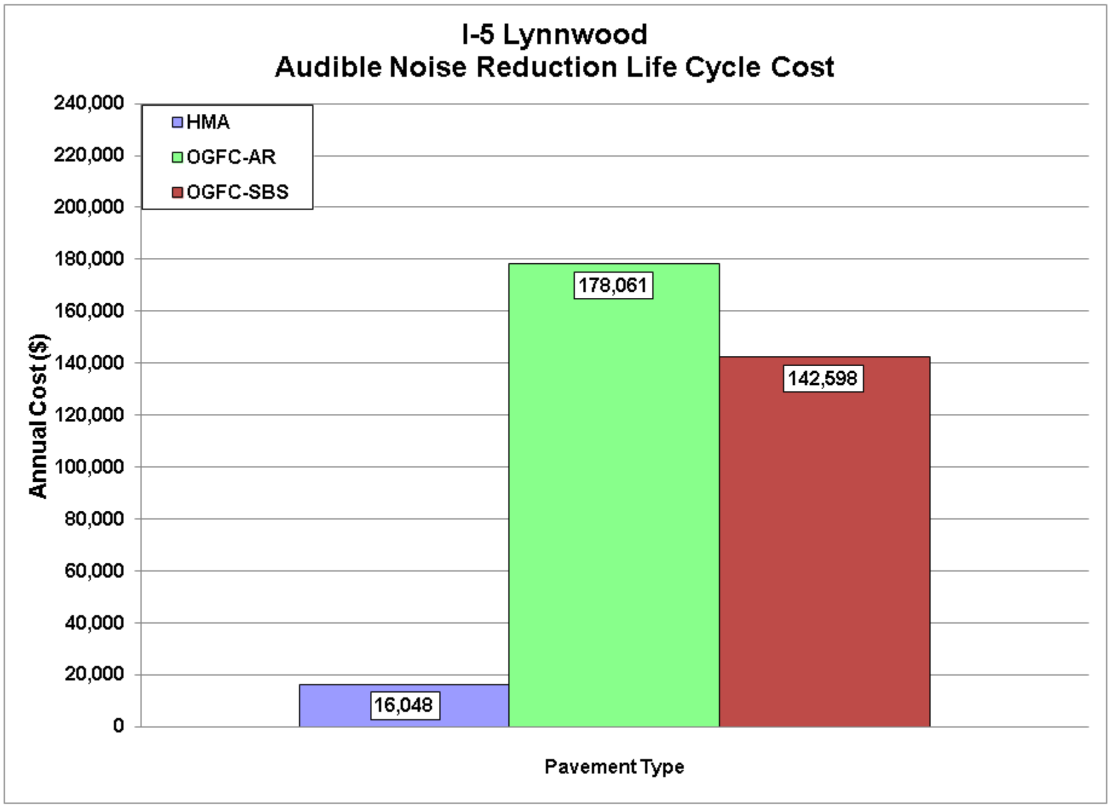


Figure 37. Life cycle cost based on audible noise reduction.

The short duration of audible noise reduction for the OGFC's leads to a high life cycle cost. Current performance data for the HMA control section indicates that it will need to be replaced at an age of about 13 years. The OGFC-AR life cycle cost is ten times and the OGFC-SBS eight times the life cycle cost of the longer lasting HMA.

An LCCA analysis based on strictly pavement performance was also performed. Data from the Washington State Pavement Management System (WSPMS) indicates that the OGFC-AR would have needed replacement at five years due to rutting from studded tires. The OGFC-SBS would have lasted nine years with rutting also dragging down its performance. The bar chart below shows the LCCA comparison of the OGFC sections and HMA control section.

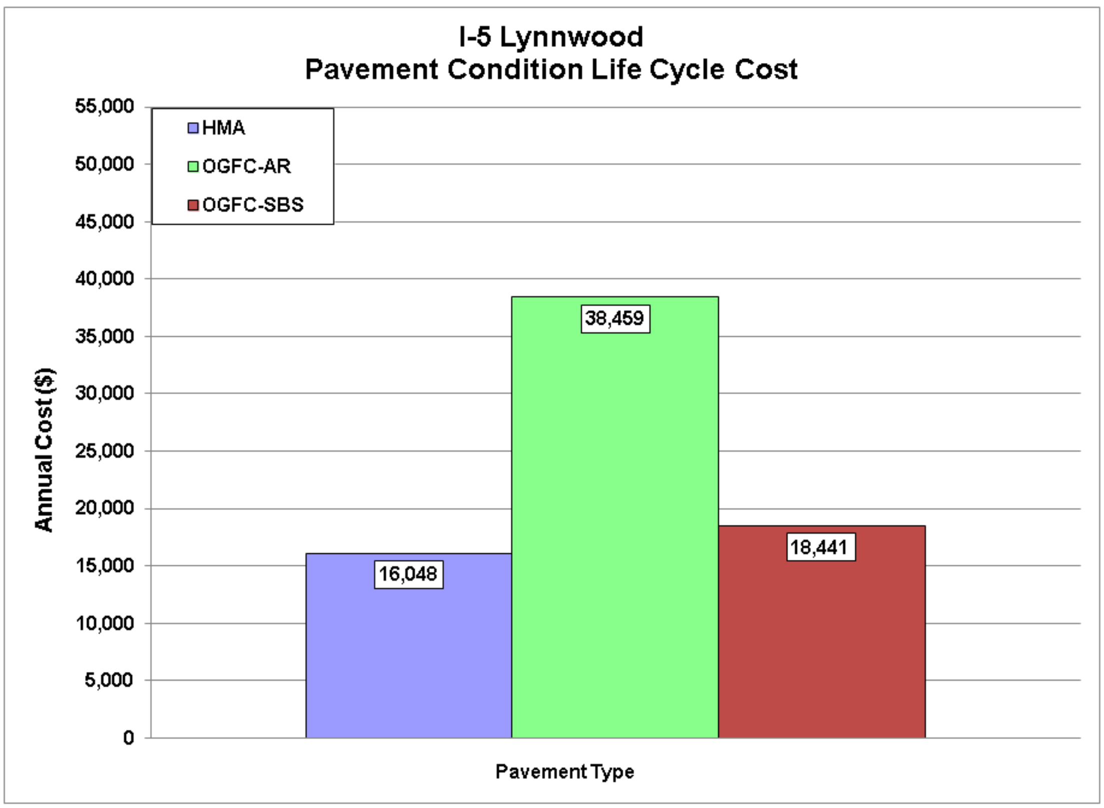


Figure 38. Life cycle cost passed on WSPMS performance data.

The annual cost of the OGFC-AR is over twice that of the HMA and the OGFC-SBS is about 15 percent higher. Even if audible noise reduction were not an issue the OGFC-AR would be very expensive to maintain and the OGFC-SBS moderately more expensive.

Discussion of Results

The special test sections of OGFC-AR and OGFC-SBS were constructed, from all indications, according to the contract specifications. The use of an MTV ensured that the mix going into the paving machine was uniform in temperature and as a result no significant temperature differentials were observed in the mat behind the paver. Post-construction testing also confirmed that the pavements placed were up to standards and suitable for the long-term evaluation of the benefits of open-graded pavements with respect to friction resistance, ride, rutting, and tire/pavement noise mitigation (Appendix G, Experimental Feature Work Plan).

The sound intensity level data, the wear/rutting data, and the ride data indicated that studded tires had a major affect on the performance of the OGFC-AR section and to a somewhat lesser extent on the performance of the OGFC-SBS. The severe winter storm of December 2008 caused an acceleration of the wear in the OGFC-AR section which was accompanied by increases in the sound intensity level and a decrease in the smoothness of the ride over the pavement.

Conclusions

The following conclusions were derived concerning the performance of the OGFC quieter pavements.

- The OGFC-AR and OGFC-SBS mixes produced a pavement that was audibly quieter than a conventional dense graded asphalt pavement for only a short duration of time, four months for the OGFC-AR and fifteen months for the OGFC-SBS.
- The OGFC-AR and OGFC-SBS pavements are susceptible to raveling and rutting by studded tires and those with recycled tire rubber are especially vulnerable to this type of damage.
- The life cycle cost of the OGFC pavements is eight to ten times higher than the HMA with respect to audible noise reduction.
- The life cycle cost of the OGFC-AR is twice that of the HMA and the OGFC-SBS fifteen percent higher than the HMA based strictly on their performance as pavements.

Recommendations

WSDOT's use of open graded friction course mixes as quieter pavements is not recommended based on the short duration of their noise mitigation properties and susceptibility to excessive raveling and rutting leading to a high life cycle cost.

References

Scofield, Larry (2009) "Transportation Noise and Concrete Pavements – Using Concrete Pavements as the Noise Solution", American Concrete Paving Association, May 2009

Appendix A

Mix Designs

ARIZONA DEPARTMENT OF TRANSPORTATION
MATERIALS GROUP

414 AR ACFC Mix Design | Lab # 2006-09050 | Date: 07/17/2006

This design meets ADOT Specifications

Tracs # XM12201X | Project # XM12201X | Project Name: WSDOT C-7134
Resident Engineer: | Contractor: WILDER CONSTRUCTION COMPANY

General Design Information

lab# 2006-16133 CRA Grade CRA-2	CRA Source	CRA Specific Gravity 1.041
lab# 2006-16132 PG Grade PG64-22	PG source	PG Specific Gravity 1.017
lab# 2006-00364 Rubber Type B	Rubber Source CRM	% by wt of asphalt cement 22.0
Admixture Type NONE Design% 0.0	Admix Source	Admix Specific Gravity
		Total number of stockpiles: 3

Aggregate

	lab#	Description	Pit#	Design %
1.	2006-00353	3/8" Chips	B335	72
2.	2006-00352	#4-8 Sand	B335	20
3.	2006-00351	#4-0	B335	8
4.				
5.				
6.				
7.				
			Total	100%

Composite Gradation

Sieve #	Specification band % passing min	Specification band % passing max	Gradation w/o admix % passing	Gradation w/admix % passing	Field Target Band % passing min	Field Target Band % passing max
1 ½"			100			
1"			100			
¾"			100			
½"			100			
3/8"	100		100			
¼"			64			
#4	30	45	34			
#8	4	8	8			
#10			7			
#16			5			
#30			4			
#40			3			
#50			2			
#100			1			
#200	0.0	2.5	1.5			

Composite Aggregate Properties

Property	Test Value	min	max
L.A. Abrasion % at 100 revolutions (AASHTO T96)	3		9
L.A. Abrasion % at 500 revolutions (AASHTO T96)	13		40
Sand Equivalent (AZ 242)	58	45	
Two Fractured Faces, % (AZ 212)	95	85	
Flakiness Index, % (AZ 238)	14		25
Carbonates, % (AZ 238)	0.9		30
Combined O.D. Specific Gravity (AZ 210)	2.679	2.35	2.85
Corrected Combined O.D. Specific gravity (with admix)	2.679		
Combined Water Absorption, % (AZ 814)	1.31		2.50

Calculated Mix Properties Results		
Description	Design Values	Specification Limits
Design Binder Content	9.2	
Bulk Density pcf	119.9	
Asphalt Absorption	0.26	< 1.0

Stockpile Gradations			
	2006-00354 3/8” Chips	2006-00352 #4-8 Sand	2006-00351 #4-0
Sieve #			
1 ½”	100	100	100
1”	100	100	100
¾”	100	100	100
½”	100	100	100
3/8”	100	100	100
¼”	51	98	99
#4	15	77	95
#8	2	4	70
#10	2	2	64
#16	1	1	50
#30	1	0	36
#40	1	0	30
#50	0	0	25
#100	0	0	18
#200	0.4	0.1	13.7

Laboratory Aggregate Specific Gravity Test Results				
Type	O.D. Sp. Gr.	SSD Sp. Gr.	Water Absorption %	Tested On
Fine (AZ 211)	2.503	2.585	3.26	-#8
Coarse (AZ 210)	2.695	2.726	1.11	-#8

Laboratory Rice Data (AZ 806)			
% Asphalt	Maximum Specific Gravity	Maximum Density – pcg	Effective Specific Gravity
4.0	2.536	158.0	2.697

This design has been prepared and submitted under the direction of:

Lab Supervisor: Hu, Changming   Bituminous Engineer: Simpson, Don

Remarks: The design is acceptable on the condition that the 3/8” chips are scalped to remove material retained on the 3/8” sieve.

Design approved by: ______________________________

MACTEC
engineering and constructing a better tomorrow

July 7, 2006

Mr. Nathan Huschka
Granite Construction
38000 Monroe Street
Indio, California 92203

Subject: **Asphalt-Rubber Binder Testing**
Washington Department of Transportation
WaDOT Project Name: I-5, 52nd Avenue to SR 526, SB Paving and Safety
WaDOT Project Number: 05A045
MACTEC Lab No. 68943
MACTEC Project No. 4975-06-0060

Dear Mr. Huschka:

As authorized by Granite Construction, MACTEC Engineering and Consulting, Inc. (MACTEC) has completed a series of tests on asphalt cement and crumb rubber for the subject asphalt-rubber (A-R) binder. The materials used for this A-R binder design are presented below and were submitted to our Phoenix laboratory by supplier representatives. A summary of the tests performed and MACTEC's results are presented in this report.

Materials

Material	Source/Supplier	Phoenix, Arizona
PG 64-22 Asphalt Cement	Tesoro Corporation	Anacortes, Washington
AD-here LOF 65-00 Liquid Anti-Strip	Arr Maz Custom Chemicals	Mulberry, Florida
Scrap Tire (WaDOT) Crumb Rubber Modifier	Crumb Rubber Manufacturers	Rancho Domingo, California

Asphalt Cement Grade Confirmation

Test	Result	Specified Limits
Dynamic Shear Rheometer, 64°C, G*/sinδ (T315)	1.20	1.00 minimum

CRM Physical Analysis

Test	Result	Specified Limits
Metal Content, %	None	None
Fiber Content, %	Trace	0.5 Maximum
Specific Gravity (D1817)	1.157	1.1-1.2

CRM Gradation, Percent Passing (AASHTO T 11/27)

Sieve Size	Result	Specified Limits
2.00 mm/No. 10	100.0	100
1.18 mm/No. 16	70.6	65 - 100
600 µm/No. 30	30.3	20 - 100
300 µm/No. 50	9.0	0 - 45
150 µm/No. 100	1.8	
75 µm/No. 200	0.2	0 - 5

MACTEC Engineering and Consulting, Inc.
3630 East Wier Avenue • Phoenix, AZ 85040 • Phone: 602.437.0250 • Fax: 602.437.3675
www.mactec.com

Washington DOT, I-5, 52nd Ave to SR 526
Granite Construction
MACTEC Project No. 4975-06-0060 (68943)

July 7, 2006
Asphalt-Rubber Binder Testing

As specified by Wilder Construction and directed by Granite Construction, 0.5% liquid anti-strip by weight of asphalt cement was added to the PG 64-22 asphalt cement. It should be noted that use of liquid anti-strip in A-R binders is not typical ADOT practice and MACTEC does not have any documentation as to its effectiveness in open-graded asphalt-rubber concrete mixes. Mineral admixture, usually hydrated lime, is typically included to enhance resistance to moisture damage and there is a considerable body of experience with its effectiveness in such mixes. Hydrated lime provides some additional benefits in its interaction with the A-R binder that enhance binder stiffness and curing of the resulting hot mix when it is newly placed and most vulnerable to damage by traffic. MACTEC does not know if substitution of liquid anti-strip for mineral admixture will have any adverse impacts on long term pavement performance.

The asphalt-rubber binder testing was performed in our laboratory by heating a known quantity of the blended asphalt cement and liquid anti-strip to 204°C (400°F). The CRM, (proportioned by total binder weight) was slowly added to the hot asphalt cement and liquid anti-strip. The asphalt rubber blend was tested for viscosity at 177°C (350°F) using a Haake style Viscotester Model VT-04 with Rotor 1, resilience (ASTM D5329), softening point (ASTM D36), and needle penetration (ASTM D5). The gradation of the CRM was determined in accordance with AASHTO T 11/27. Results of testing are presented in this report.

A variety of interaction periods were conducted to evaluate stability and retention of properties of the asphalt-rubber binder. The interaction periods cover a time range to identify properties after completion of field mixing (60 minutes after addition of the CRM), and possible job delay (4 to 6 hours). Tests at 24 hours (with exposure from 6 to 22 hours at a lower temperature to simulate overnight-unheated storage) were also performed to evaluate the stability of the asphalt rubber blend. Results of this testing indicate properties remain satisfactory throughout the 24 hour reaction period.

The proportions of Crumb Rubber Modifier (CRM) and PG 64-22 Asphalt Cement with 0.5% AD-here LOF 65-00 presented in this A-R binder design are as follows:

PHYSICAL PROPERTIES OF ASPHALT-RUBBER BINDER

18.0% Crumb Rubber (Crumb Rubber Manufacturers)
82.0% Asphalt Cement with 0.5% Anti-Strip (Tesoro PG 64-22/Arr Maz AD-here LOF 65-00)

Test Performed	Minutes of Reaction					Specified Limits
	60	90	240	360	1,440	
Rotational Viscosity at 350°F, Pa's, (10^{-3}), or cP	2100	2400	2700	2700	2200	1500 - 4000
Resilience at 77°F, % Rebound (D5329)	40		39		38	25 Minimum
Ring & Ball Softening Point, °F (D36)	148	150	149	149	147	130 Minimum
Penetration at 39.2°F, 200g, 60 sec., 1/10 mm (D5)	22		21		24	15 Minimum

Washington DOT, I-5, 52nd Ave to SR 526
Granite Construction
MACTEC Project No. 4975-06-0060 (68943)

July 7, 2006
Asphalt-Rubber Binder Testing

If you have any questions regarding this information or if we may be of further assistance in any way, please do not hesitate to contact us.

Sincerely,

MACTEC ENGINEERING AND CONSULTING, INC.

Sam W. Huddleston
Principal Scientist

Anne Stonex
Senior Engineer

SWH:AS:adm

(projects\4975\4975-06-0060\deliverables\68943arb)

Washington State Department of Transportation - Materials Laboratory
PO Box 47365 Olympia / 1655 S 2nd Ave. Tumwater / WA 98504
BITUMINOUS MATERIALS SECTION - TEST REPORT

TEST OF: OPEN GRADED FRICTION COURSE (OGFC)
DATE SAMPLED: 6/28/2006
DATE RECVD HQS: 7/6/2006
SR NO: 5
SECTION: 52nd AVENUE WEST TO SR 526, SB PAVING AND SAFETY

WORK ORDER NO: 007134
LAB ID NO 0000230062
TRANSMITTAL NO: 230062
MIX ID NO: G61681

CONTRACTOR'S PROPOSAL

Mat'l:	3/8" CHIPS	#4-0	#4 - #8 SAND	COMBINED	SPECIFICATIONS
Source:	B-335	B-335	B-335		
Ratio:	66%	13%	21%		
3/8"	99.8	100.0	100.0	100	100
No. 4	12.3	93.9	78.4	37	35 – 55
No. 8	1.2	63.2	4.3	10	9 – 14
No. 200	0.7	12.0	0.5	2.1	0 – 2.5

LABORATORY ANALYSIS

				SPECIFICATIONS
ASPH% BY TOTAL WT OF MIX:	**7.8**	**8.3**	**8.8**	
% VOIDS @ Ndes: 50	18.8	18.3	19.4	15.0 Min.
% VMA @ Ndes: 50	31.1	31.4	33.3	24.0 Min.
% Gmm @ Ndes: 50	81.3	81.7	80.7	82.0 Max.
Draindown @ 339°F	0.0	0.0	0.1	0.3 Max.
Stabilizing Additive	0.3	0.3	0.3	0.2 – 0.5
Gmm – MAX S. G. FROM RICE	2.427	2.417	2.392	
Gmb – BULK S. G. OF MIX	1.972	1.976	1.929	
Gsb – OF AGGREGATE BLEND	2.638	2.638	2.638	
Gsb – OF FINE AGGREGATE	2.551	2.551	2.551	
Gb – SPECIFIC GRAVITY OF BINDER	1.025	1.025	1.025	

LOTTMAN STRIPPING EVALUATION

% ANTI-STRIP	0.0%	1/4 %	1/2 %	3/4 %	1.0%
Visual Appearance:	SLIGHT	NONE	NONE	NONE	NONE
% Retained Strength:	84	107	115	113	139

RECOMMENDATIONS

SUPPLIER	U.S. OIL
GRADE	PG70-22
% ASPHALT (BY TOTAL MIX)	8.3
% ANTI-STRIP (BY WT. ASPHALT)	0.25
TYPE OF ANTI-STRIP	ARR-MAZ 6500
MIX ID NUMBER	G61681
MIXING TEMPERATURE	346°F
COMPACTION TEMPERATURE	316°F

Headquarters:
Construction Engineer----------------X
Materials File--------------------------X
General File----------------------------X
Bituminous Section------------------ X
Region: Northwest
Construction Office- 41 ----------X
Materials Eng------- 41 ----------X
P.E.: M. LENSSEN --X(2)

T152 - 3
T153 -
T166 - 3
T172 -
T175 -
T178 - 1

REMARKS: **Revised report to reflect adjusted asphalt content and completed mix design test data, 8/24/06.**

THOMAS E. BAKER, P.E.
Materials Engineer
By: Dennis M. Duffy P.E. ____________________
(360)709-5420
Date: 8 / 24 / 2006

Appendix B

Special Provisions

Description

The first paragraph of Section 5-04.1 is supplemented with the following:

(******)
This work shall consist of providing and placing Quieter Pavement overlays consisting of Open Graded Friction Course (OGFC) and Open Graded Friction Course Asphalt-Rubber (OGFC-AR) on the existing roadway in accordance with these Specifications and lines, grades, thicknesses, and typical cross-sections shown in the Plans and shall meet the requirements for hot-mix asphalt as modified herein.

OGFC shall consist of a mixture of asphalt, mineral aggregate, mineral filler, and other additives properly proportioned, mixed and applied on a paved surface.

OGFC-AR shall consist of a mixture of rubberized asphalt, mineral aggregate, mineral filler and other additives properly proportioned, mixed and applied on a paved surface.

Materials

The first paragraph of Section 5-04.2 is supplemented with the following:

(******)
Portland Cement, Type II9-02.1(1)
Hydrated Lime ASTM C 1097

The third paragraph of Section 5-04.2 is supplemented with the following:

(******)
The use of RAP shall not be used in the OGFC or OGFC-AR.

The fourth paragraph of Section 5-04.2 is supplemented with the following:

(******)
Asphalt binder material for the OGFC shall be PG 70-22. SBS modifier shall be added to the neat asphalt to produce a binder that complies with the requirements for PG 70-22.

Asphalt binder material for the OGFC-AR shall be asphalt-rubber conforming to the requirements of Asphalt Rubber (A). The crumb rubber gradation shall conform to the requirements of Asphalt-Rubber (B).

In no case shall the asphalt-rubber be diluted with extender oil, kerosene, or other solvents. Any asphalt-rubber so contaminated shall be rejected.

Asphalt-Rubber

(A) Asphalt Binder

Asphalt binder shall be PG 58-22 or PG 64-22 conforming to the requirements of 9-02, Bituminous Materials.

(B) Crumb Rubber

Rubber shall meet the following gradation requirements when tested in accordance with AASHTO T 11/27.

Sieve Size	Percent Passing
No. 8	100
No. 10	100
No. 16	65 – 100
No. 30	20 – 100
No. 50	0 – 45
No. 200	0 – 5

The rubber shall have a specific gravity of 1.15 ± 0.05 and shall be free of wire or other contaminating materials, except that the rubber shall contain not more than 0.5 percent fabric. Calcium carbonate, up to four percent by weight of the granulated rubber, may be added to prevent the particles from sticking together.

Certificates of Compliance conforming to 1-06.3 shall be submitted. In addition, the certificates shall confirm that the rubber is a crumb rubber, derived from processing whole scrap tires or shredded tire materials; and the tires from which the crumb rubber is produced are taken from automobiles, trucks, or other equipment owned and operated in the United States. The certificates shall also verify that the processing does not produce, as a waste product, casings or other round tire material that can hold water when stored or disposed of above ground.

Asphalt-Rubber Proportions

The asphalt-rubber shall contain a minimum of 20 percent ground rubber by the weight of the asphalt binder.

Asphalt-Rubber Properties

Certificate of Compliance conforming to 1-06.3 shall be submitted to the Engineer showing that the asphalt-rubber conforms to the following:

Property	Requirement
Rotational Viscosity*: 350 ^{o}F; pascal seconds	1.5 - 4.0
Penetration: 39.2 ^{o}F, 200 g, 60 sec. (ASTM D 5); minimum	15
Softening Point: (ASTM D 36); ^{o}F, minimum	130
Resilience: 77 ^{o}F (ASTM D 5329); %, minimum	25

* The viscotester used must be correlated to a Rion (formerly Haake) Model VT-04 viscotester using the No. 1 Rotor. The Rion viscotester rotor, while in the off position, shall be completely immersed in the binder at a temperature from 350 to 355 F for a minimum heat equilibrium period of 60 seconds, and the average viscosity determined from three separate constant readings (± 0.5 pascal seconds) taken within a 30 second time frame with the viscotester level during testing and turned off between readings. Continuous rotation of the rotor may cause thinning of the material immediately in contact with the rotor, resulting in erroneous results.

Asphalt-Rubber Binder Design

At least two weeks prior to the use of asphalt-rubber, the Contractor shall submit an asphalt-rubber binder design prepared by one of the following laboratories who have experience in asphalt-rubber binder design:

MACTEC Engineering and Consulting, Inc.
Contact: Anne Stonex
Address: 3630 East Wier Avenue
Phoenix, Arizona 85040
Phone: (602) 437-0250

Western Technologies, Inc.
Contact: John Hahle
Address: 2400 East Huntington Drive
Flagstaff, Arizona 86004
Phone: (928) 774-8700

Such design shall meet the requirements specified herein. The design shall show the values obtained from the required tests, along with the following information: percent, grade and source of the asphalt binder used; and percent, gradation and source(s) of rubber used.

Construction Requirements

Section 5-04.3 shall be supplemented with the following:

(******)

During production of asphalt-rubber, the Contractor shall combine materials in conformance with the asphalt-rubber design unless otherwise approved by the Engineer.

Direct transfer of the OGFC and OGFC-AR from the hauling equipment to the paving machine will not be allowed. A Shuttle Buggy will be required to deliver the OGFC and OGFC-AR from the hauling equipment to the paving machine.

Mixing of Asphalt-Rubber

The temperature of the asphalt binder shall be between 350 and 400°F at the time of addition of the ground rubber. No agglomerations of rubber particles in excess of two inches in the least dimension shall be allowed in the mixing chamber. The ground rubber and asphalt binder shall be accurately proportioned in accordance with the design and thoroughly mixed prior to the beginning of the one-hour reaction period. The Contractor shall document that the proportions are accurate and that the rubber has been uniformly incorporated into the mixture. Additionally, the Contractor shall demonstrate that the rubber particles have been thoroughly mixed such that they have been "wetted." The occurrence of rubber floating on the surface or agglomerations of rubber particles shall be evidence of insufficient mixing. The temperature of the asphalt-rubber immediately after mixing shall be between 325 and 375°F. The asphalt-rubber shall be maintained at such temperature for one hour before being used.

Prior to use, the viscosity of the asphalt-rubber shall be tested and conform to the asphalt-rubber properties, which is to be furnished by the Contractor or supplier.

Handling of Asphalt-Rubber

Once the asphalt-rubber has been mixed, it shall be kept thoroughly agitated during periods of use to prevent settling of the rubber particles. During the production of asphaltic concrete the temperature of the asphalt-rubber shall be maintained between 325 and 375°F. However, in no case shall the asphalt-rubber be held at a temperature of 325°F or above for more than 10 hours. Asphalt-rubber held for more than 10 hours shall be allowed to cool and gradually reheated to a temperature between 325 and 375°F before use. The cooling and reheating shall not be allowed more than one time. Asphalt-rubber shall not be held at temperatures above 250°F for more than four days.

For each load or batch of asphalt-rubber, the contractor shall provide the Engineer with the following documentation:

1. The source, grade, amount and temperature of the asphalt binder prior to the addition of rubber.
2. The source and amount of rubber and the rubber content expressed as percent by the weight of the asphalt binder.
3. Times and dates of the rubber additions and resultant viscosity test.
4. A record of the temperature, with time and date reference for each load or batch. The record shall begin at the time of the addition of rubber and continue until the load or batch is completely used. Readings and recordings shall be made

at every temperature change in excess of 20°F, and as needed to document other events which are significant to batch use and quality.

HMA Mixing Plant

Section 5-04.3(1) ~~shall be~~ is supplemented with the following:

(******)
Fiber Supply System
When fiber stabilizing additives are required for OGFC, a separate feed system that meets the following will be required:

1) Accurately proportions by weight the required quantity into the mixture in such a manner that uniform distribution will be obtained.
2) Provides interlock with the aggregate feed or weigh systems so as to maintain the correct proportions for all rates of production and batch sizes.
    a) Controls dosage rate accurately to within plus or minus 10 percent of the amount of fibers required.
    b) Automatically adjusts the feed rate to maintain the material within the 10 percent tolerance at all times.
    c) Provides flow indicators or sensing devices for the fiber system that are interlocked with plant controls so that mixture production will be interrupted if introduction of the fiber fails or if the output rate is not within the tolerances given above.
3) Provides in-process monitoring, consisting of either a digital display of output or a printout of feed rate, in pounds per minute to verify the feed rate.

When a batch type plant is used, the fiber shall be added to the aggregate in the weigh hopper or as approved by the Engineer. The batch dry mixing time shall be increased by 8 to 12 seconds, or as directed by the Engineer, from the time the aggregate is completely emptied into the mixer. The fibers are to be uniformly distributed prior to the injection of the asphalt binder into the mixer.

When a continuous or drier-drum type plant is used, the fiber shall be added to the aggregate and uniformly dispersed prior to the injection of asphalt binder. The fiber shall be added in such a manner that it will not become entrained in the exhaust system of the dryer or plant.

Surge and Storage Systems
The storage time for OGFC mixtures not hauled immediately to the project shall be no more than 4 hours.

Hot Mix Asphalt Pavers

Section 5-04.3(3) is supplemented with the following:

(******)
For OGFC and OGFC-AR the direct transfer of these materials from the hauling equipment to the paving machine will not be allowed. A Shuttle Buggy shall be used to deliver the OGFC and OGFC-AR from the hauling equipment to the paving machine.

The Shuttle Buggy shall mix the OGFC and OGFC-AR after delivery by the hauling equipment but prior to laydown by the paving machine. Mixing of the OGFC and OGFC-AR shall be sufficient to obtain a uniform temperature throughout the mixture.

Rollers

Section 5-04.3(4) is supplemented with the following:

(******)
The wheels of the rollers used for Quieter Pavement shall be wetted with water, or if necessary soapy water, or a product approved by the Engineer to prevent the OGFC or OGFC-AR from sticking to the steel wheels during rolling.

A minimum of three static steel wheel rollers, weighing no less than eight tons, shall be provided. The drums shall be of sufficient width that when staggered, two rollers can cover the entire lane width.

Vibratory rollers must be used in the static mode only.

A pass shall be defined as one movement of a roller in either direction. Coverage shall be the number of passes as are necessary to cover the entire width being paved.

Two rollers shall be used for initial breakdown and be maintained no more than 300 feet behind the paving machine. The roller(s) for final compaction shall follow as closely behind the initial breakdown as possible. As many passes as is possible shall be made with the rollers before the temperature of the OGFC or OGFC-AR falls below 220 °F.

Preparation Of Existing Surfaces

Section 5-04.3(5)A is supplemented with the following:

(******)
For OGFC and OGFC-AR, a tack coat of CRS-2 or CRS-2P shall be applied to the existing surface at a rate of 0.12 to 0.20 (0.08 to 0.12 residual) gallons per square yard or as otherwise directed by the Engineer.

(NWR February 9, 2004)
The Contractor shall limit the amount of tack coat placed to that amount that will be fully covered by the asphalt overlay at the end of each work shift.

In accordance with Section 1-07.15(1) **Spill Prevention, Control and Countermeasures Plan** (SPCC), as part of the SPCC the Contractor shall address the mitigating measures to be taken in the event that the paving operation is suspended or terminated prior to the asphalt for tack coat being fully covered.

Mix Design

Section 5-04.3(7)A is supplemented with the following:

(******)

4. **Mix Design (OGFC-AR)** Approximately 500 pounds of produced mineral aggregate, in proportion to the anticipated percent usage, shall be obtained that are representative of the mineral aggregate to be utilized in the OGFC-AR production.

   The Contractor shall also furnish representative samples of the following materials: a five-pound sample of the crumb rubber proposed for use, one gallon of asphalt binder from the intended supplier, five gallons of the proposed mixture of binder and rubber, and a one-gallon can of the mineral admixture to be used in the OGFC-AR.

   Along with the samples furnished for mix design testing, the contractor shall submit a letter explaining in detail its methods of producing mineral aggregate including wasting, washing, blending, proportioning, etc., and any special or limiting conditions it may propose. The Contractor's letter shall also state the source(s) of mineral aggregate, the source of asphalt binder and crumb rubber, the asphalt-rubber supplier, and the source and type of mineral admixture.

   The above materials and letter shall be shipped to the Arizona DOT Central Laboratory at 1221 North 21st Avenue, Phoenix, AZ 85009 (Attention – Julie Nodes), with companion materials and letter sent to the WSDOT State Materials Laboratory in Tumwater. Within 10 working days of receipt of all samples and the Contractor's letter in the Arizona DOT Central Laboratory, the Arizona DOT will provide the Contractor with the percentage of asphalt-rubber to be used in the mix, the percentage to be used from each of the stockpiles of mineral aggregate, the composite mineral aggregate gradation, the composite mineral aggregate and mineral admixture gradation, and any special or limiting conditions for the use of the mix.

   Mix Design (OGFC) Mixtures shall be compacted with 50 gyrations of a Superpave Gyratory Compactor and the draindown at the mix production temperature (AASHTO T 305) shall be 0.3 max.

5. **Mix Design Revisions.** The Contractor shall not change its methods of crushing, screening, washing, or stockpiling from those used during production of material used for mix design purposes without approval of the Engineer, or without requesting a new mix design.

   During production of OGFC and OGFC-AR, the Contractor, on the basis of field test results, may request a change to the approved mix design. The Engineer will evaluate the proposed changes and notify the contractor of the Engineer's decision within two working days of the receipt of the request.

   If, at any time, unapproved changes are made in the source of bituminous material, source(s) of mineral aggregate, production methods, or proportional changes in violation of approved mix design stipulations, production shall cease until a new mix design is developed, or the Contractor complies with the approved mix design.

At any time after the mix design has been approved, the Contractor may request a new mix design.

The costs associated with the testing of materials in the developing of mix designs after a mix design acceptable to the Department has been developed shall be borne by the Contractor.

If, during production, the Engineer on the basis of testing determines that a change in the mix design is necessary, the Engineer will issue a revised mix design. Should these changes require revisions to the Contractor's operations which result in additional cost to the Contractor, it will be reimbursed for these costs.

6. **Fiber Stabilizing Additives.** If needed, fiber stabilizing additives shall consist of either cellulose fibers, cellulose pellets or mineral fibers and meet the properties described below. Dosage rates given are typical ranges but the actual dosage rate used shall be approved by the Engineer.

A. Cellulose Fibers: Cellulose fibers shall be added at a dosage rate between 0.2% and 0.5% by weight of the total mix as approved by the Engineer. Fiber properties shall be as follows:

1.	Fiber length:	0.25 inch (6 mm) max.
2.	Sieve Analysis	
	a. Alpine Sieve Method Passing No. 100 sieve:	60-80%
	b. Ro-Tap Sieve Method Passing No. 20 sieve: Passing No. 40 sieve: Passing No. 100 sieve:	80-95% 45-85% 5-40%
3.	Ash Content:	18% non-volatiles (±5%)
4.	pH:	7.5 (±1.0)
5.	Oil Absorption: (times fiber weight)	5.0 (±1.0)
6.	Moisture Content:	5.0% max.

B. Cellulose Pellets: Cellulose pellets shall consist of cellulose fiber and may be blended with up to 20% asphalt cement. If no asphalt cement is used, the fiber pellet shall be added at a dosage rate between 0.2% and 0.5% by weight of the total mix. If asphalt cement is blended with the fiber, the pellets shall be added at a dosage rate between 0.4% and 0.8% by weight of the total mix.

1.	Pellet size:	1/4 in^3 (6 mm^3) max.
2.	Asphalt:	25 - 80 pen.

C. Mineral Fibers: Mineral fibers shall be made from virgin basalt, diabase, or slag and shall be treated with a cationic sizing agent to enhance disbursement of the fiber as well as increase adhesion of the fiber surface to the bitumen. The fiber shall be added at a dosage rate between 0.2% and 0.5% by weight of the total mix.

1.	Size Analysis:	
	Average Fiber length:	0.25 in. (6 mm) max.
	Average Fiber thickness:	0.0002 in. (0.005mm) max.
2.	Shot content (ASTM C1335)	
	Passing No. 60 sieve (250 µm):	90 - 100%
	Passing No. 230 sieve (63 µm):	65 - 100%

Acceptance Sampling and Testing – HMA Mixture

Section 5-04.3(8)A is revised as follows:

Item 3 is supplemented with the following:

(******)
Sampling - OGFC and OGFC-AR
OGFC and OGFC-AR will be evaluated for quality of gradation based on samples taken from the cold feed bin.

Item 5 is supplemented with the following:

(******)
Test Results - OGFC and OGFC-AR
Mineral Aggregate Gradation - OGFC
For the OGFC, a sample shall be taken in accordance with WSDOT T-2 on a random basis just prior to the addition of mineral admixture and bituminous materials. At least one sample shall be taken during the production of the OGFC. Samples will be tested for conformance with the mix design gradation. The gradation of the mineral aggregate shall be considered to be acceptable, unless average of any three consecutive tests or the result of any single test varies from the mix design gradation percentages as follows:

Passing Sieve	Mixture Control Tolerance
⅜ Inch	± 5.7
No. 4	± 5.5
No. 8	± 4.5
No. 200	± 2.0

(******)

Mineral Aggregate Gradation - OGFC-AR

For each approximate 300 tons of OGFC-AR, at least one sample of mineral aggregate shall be taken. Samples shall be taken in accordance with WSDOT T-2 on a random basis just prior to the addition of mineral admixture and bituminous materials. Samples will be tested for conformance with the mix design gradation. The gradation of the mineral aggregate shall be considered acceptable, unless the average of any three consecutive tests or the result of any single test varies from the mix design gradation percentages as follows:

Passing Sieve	Number of Tests	
	3 Consecutive	One
No. 4	± 4	± 4
No. 8	± 3	± 4
No. 200	± 1.0	± 1.5

(January 3, 2006)

The first paragraph of item 5 is revised to read:

The Engineer will furnish the Contractor with a copy of the results of all acceptance testing performed in the field within either 24 hours of sampling or four hours after the beginning of the next paving shift, whichever is later. The Engineer will also provide the Composite Pay Factor (CPF) of the completed sublots after three sublots have been produced. The CPF will be provided by the midpoint of the next paving shift after sampling results are completed.

The first sentence in the second paragraph of item 5 is revised to read:

Sublot sample test results (gradation, asphalt binder content, VMA and Va) may be challenged by the Contractor.

The third paragraph of item 5 is revised to read:

The results of the challenge sample will be compared to the original results of the acceptance sample test and evaluated according to the following criteria:

	Deviation
U.S. No. 4 sieve and larger	Percent passing ±4.0
U.S. No. 8 sieve	Percent passing ±2.0
U.S. No. 200 sieve	Percent passing ±0.4
Asphalt binder %	Percent binder content ±0.3
VMA %	Percent VMA ±1.5
Va %	Percent Va ±0.7

The last sentence of item ~~7~~5 is revised to read:

> The calculation of the CPF in a test section with a mix design that did not verify will include gradation, asphalt binder content, VMA and Va.

Item 7 is supplemented with the following:

(******)
Test Section - OGFC
A mixture test section shall be constructed off-site prior to production paving of the OGFC. The test section shall be used to determine if the mix meets the requirements of mineral aggregate gradation and recommended asphalt binder content.

For the test section to be acceptable the mineral aggregate gradation shall be within the limits as shown in 5-04.3(8)A as supplemented and the asphalt content varies by no more than ±0.5 percent.

Test Section - OGFC-AR
A mixture test section shall be constructed off-site prior to production paving of the OGFC-AR. The test section shall be used to determine if the mix meets the requirements of mineral aggregate gradation and recommended asphalt-rubber binder content.

For the test section to be acceptable the mineral aggregate gradation shall be within the limits as shown in 5-04.3(8)A as supplemented and the asphalt-rubber content varies by no more than ±0.5 percent.

Compaction

(NWR March 1, 2004)
Control
The first sentence of item 1 in Section 5-04.3(10)B is revised to read:

> HMA used in traffic lanes, including lanes for ramps, truck climbing, weaving, speed change, and shoulders, and having a specified compacted course thickness greater than 0.10 foot, shall be compacted to a specified level of relative density.

Joints

Section 5-04.3(12) is supplemented with the following:

(NWR May 9, 2005)
Transverse Joint Seal
The Contractor shall construct contraction joints at the bridge ends/bents as shown in the Plans. The joints shall be sawed to the dimensions shown in the Plans and filled with joint sealant filler meeting the requirement of Section 9-04.2(1).

Joints shall be thoroughly clean and dry at the time of sealing. Care shall be taken to avoid air pockets. The compound shall be applied in two or more layers, if necessary.

Planing Bituminous Pavement

Section 5-04.3(14) is supplemented with the following:

(January 5, 2004)
The Contractor shall perform the planing operations no more than *** five *** calendar days ahead of the time the planed area is to be paved with HMA, unless otherwise allowed by the Engineer in writing.

(January 5, 2004)
At the start of the planing operation the Contractor shall plane a 500 foot test section to be evaluated by the Engineer for compliance with the surface tolerance requirements. The test section shall have a minimum width of 10 feet. If the planing is in accordance with the surface tolerance requirements, the Contractor may begin production planing. If the planing is not in conformance with the surface tolerance requirements, the Contractor shall make adjustments to the planing operation and then plane another test section.

If at any time during the planing operation the Engineer determines the required surface tolerance is not being achieved, the Contractor shall stop planing. Planing shall not resume until the Engineer is satisfied that specification planing can be produced or until successful completion of another test section. The forward speed during production planing shall not exceed the speed used for the test section.

The completed surface after planing and prior to paving shall not vary more than 1/4 inch from the lower edge of a 10-foot straightedge placed on the surface parallel or transverse to the centerline. The planed surface shall have a matted texture and the difference between the high and low of the matted surface shall not exceed 1/8 inch.

Pavement repair operations, when required, shall be accomplished prior to planing.

(January 3, 2006)
Transverse Joints
The full depth end of each lane of planing shall be squared off to form a uniform transverse joint. The Contractor shall construct and maintain a temporary HMA wedge in accordance with Section 5-04.3(11) across the entire width of the transverse edge when traffic is allowed on the planed surface prior to paving. The wedge shall be

constructed before opening the lane to traffic. The Contractor shall remove the wedge immediately prior to paving.

(NWR May 9, 2005)
Transverse Joint Seal
The Contractor shall construct contraction joints at the bridge ends/bents as shown in the Plans. The joints shall be sawed to the dimensions shown in the Plans and filled with joint sealant filler meeting the requirement of Section 9-04.2(1).

Joints shall be thoroughly clean and dry at the time of sealing. Care shall be taken to avoid air pockets. The compound shall be applied in two or more layers, if necessary.

Weather Limitations
Section 5-04.3(16) is supplemented with the following:

(******)
The mixing and placing of OGFC and OGFC-AR shall not be performed when the existing pavement is wet or frozen. OGFC and OGFC-AR shall not be placed when the air temperature is less than 55°F.

Measurement
Section 5-04.4 is supplemented with the following:

(******)
Open-Graded Friction Course and Open-Graded Friction Course Asphalt Rubber will be measured by the ton in accordance with Section 1-09.2, with no deduction being made for the weight of asphalt binder, blending sand, mineral filler or any other component of the mixture.

(NWR May 9, 2005)
Transverse joint seal will be measured by the linear foot of joint sealed.

Payment
Section 5-04.5 is supplemented with the following:

(******)
“Open Graded Friction Course”, per ton.
"Open Graded Friction Course" - Asphalt Rubber”, per ton.

The unit contract price per ton for “Open-Graded Friction Course” and “Open-Graded Friction Course Asphalt Rubber” shall be full compensation for all costs incurred to carry out the requirements of Section 5-04 except for those costs included in other items which are included in this sub-section and which are included in the proposal.

(NWR May 9, 2005)
“Transverse Joint Seal”, per linear foot.
The unit contract price for “Transverse Joint Seal” shall be full pay to complete the work as specified.

Price Adjustment for Quality of HMA

The first paragraph of Section 5-04.5(1)A is revised to read:

> Statistical analysis of quality of gradation, asphalt content and volumetric properties will be performed based on Section 1-06.2 using the following price adjustment factors:

Table of Price Adjustment Factors

Constituent	Factor “f”
VMA (Voids in mineral aggregate)	30
Va (Air Voids)	30
All aggregate passing 1/2"	2
All aggregate passing 3/8"	2
All aggregate passing U.S. No. 4	2
All aggregate passing U.S. No. 8	15
All aggregate passing U.S. No. 200	15
Asphalt Binder Content	30

The first two sentences of the second paragraph are revised to read:

> A pay factor will be calculated for sieves listed as a control point for the class of HMA, for the asphalt binder and volumetric properties (VMA and Va).

Appendix C

Infrared Images and Construction Comments

Lynnwood Quite Pavements
I-5, 52nd Avenue to SR-526
Construction Comments

The comments within this document are those of Jim Weston, Pavement Implementation Engineer, and are not necessarily the views of the WSDOT.

TACK APPLICATION

The tack coat for both the OGFC test sections was applied by an Etnyre tack truck. The application of the CRS-2P tack was sporadic at the start of each application but generally very uniform after 500 feet of application (Figure 39). However, there were areas where high application rate caused problems. In these locations, the excess tack coat was picked up by the Shuttle Buggy tires and then deposited on the existing pavement surface as a mound of material. These mounds of cool tack coat would show up in the OGFC-SBS overlay as a cold spot or globule. This did not happen with the OGFC-AR. Tracking of the tack coat by the Shuttle Buggy and delivery trucks was observed in the wheel paths. The amount of tracking was minimal in areas of good coverage but was somewhat substantial in areas that received light coverage (startup locations).

Figure 39. Image of typical tack application with some pickup visible in the wheelpaths.

DELIVERY VEHICLES

The use of tarps on the HMA delivery trucks and trailers was very sporadic throughout the paving operations. Thermal camera readings of the hot mix in the trucks showed that a cool crust of material had developed with temperatures as low as 101ºF whereas the internal temperatures of the mix were at, or above, 300ºF (Figure 40). This project was fortunate to have warmer paving temperatures than are typically seen on night pavers in Western Washington.

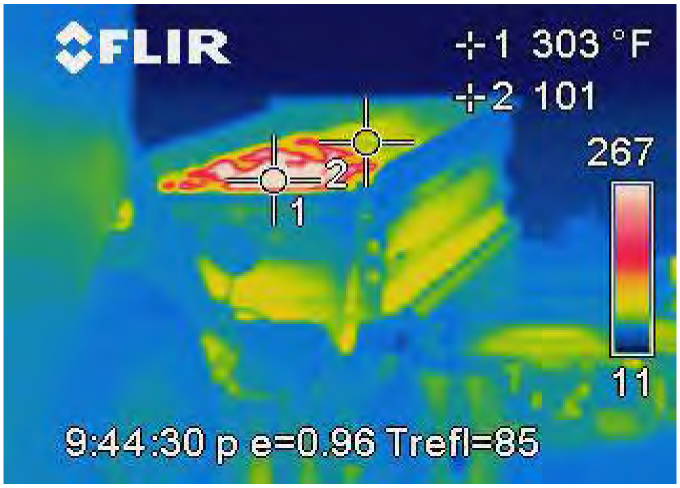


Figure 40. Thermal image of trailer showing cool crust on the HMA at 101°F and internal temperature of 303°F.

MATERIAL TRANSFER VEHICLE

A ROADTEC Shuttle Buggy (SB) material transfer vehicle was used throughout the project. The remixing and storage capabilities of this vehicle made it a smart choice because of the thinness (1/2 inch) of the overlay and the fact that all paving was done at night when temperatures are generally cooler. Temperatures from the SB into the paver hopper were typically around 300ºF (Figure 41). The insulating and remixing capability of this device allowed for the pavement to have consistent temperatures across the mat and behind the screed (Figure 42). On the last night of paving and last lane paved, the SB encountered mechanical problems. A decision was made to continue without the SB in order to complete the project that night. This proved to be a poor decision as this section of pavement began to ravel shortly after placement due to the inconsistencies in the density of the mat caused by thermal differentials. The raveling problems resulted in the Contractor having to remove and replace the defective pavement.

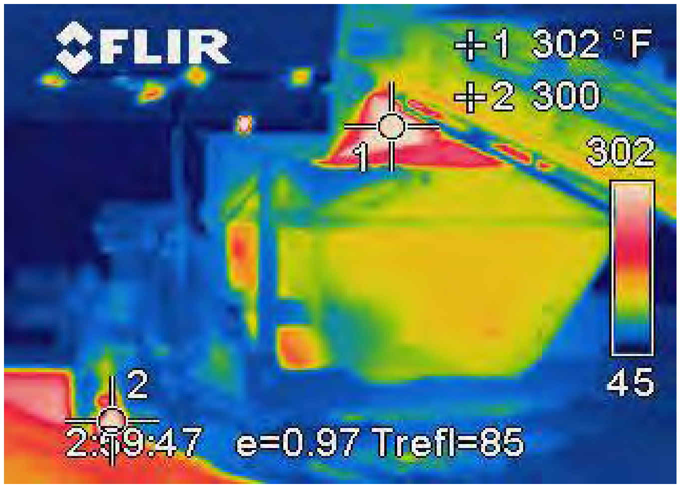


Figure 41. Thermal image of the mix as it leaves the Shuttle Buggy and enters the paver hopper at 302°F and exits the screed at 300°F.

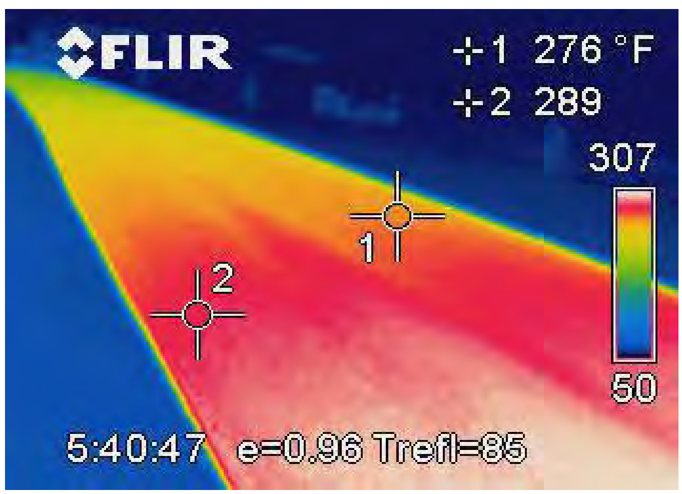


Figure 42. Image from back of screed looking towards the rollers (not shown).

PAVER

The paver was a Blaw-Knox PF-5510 paver equipped with a 12-ton paver hopper box and a UltiMat screed. It was also outfitted with a retrofit kit that kept the screed from being starved at the gearbox (Figure 43). The only paver related problems had to do with the screed. When the screed was extended a cool area of mix would form on the outside edge of the extension. This cool mix would work its way to the front of the screed and then show up in the mat behind the screed (Figure 44).

Figure 43. Thermal image looking at augers where spot 1 is typical temperature and spot 2 is where the mix would cool and slowly work to the front of the extended screed.

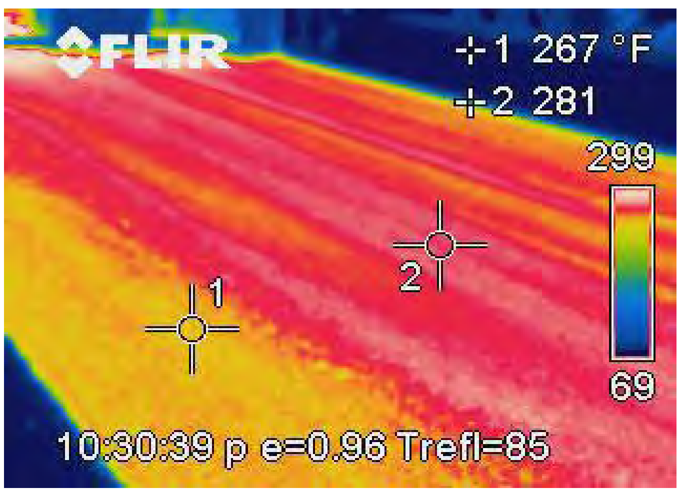


Figure 44. Lower temperature mix behind the screed extension (spot 1) and the higher temperature mix at the middle of the paver (spot 2).

A remedy for the problem of cool mix from the extended screed is probably an auger extension which was not practical for this project that only used the extension for shoulder paving.

The other item, also discussed in the next section, was the longitudinal joint. It is important to know that the screed should only allow for the material to be approximately 1/2 inch above the existing joint. Of course this may change slightly depending on the mix design but an OGFC will generally compact in the same manner.

ROLLERS
Three to four rollers were used for compaction of the OGFC-AR and OGFC-SBS (Figure 45). The first three rollers were Ingersoll-Rand DD-130's and the fourth (used on one occasion as a finish roller) was an Ingersoll-Rand DD-110. All rollers operated solely in static mode as specified in the contract Special Provisions. The first two rollers worked in tandem so that complete coverage of the lane width could be achieved in one pass. Generally, the only time that the rollers did not meet the requirement to be within 300 feet of the paver was at the beginning of a new lane. This was because while the rollers were addressing the construction at the end of the previous lane the paver would move down the mat, leaving the rollers behind. The only other time that roller operations did not meet specifications were on the first night of production paving (Ramp, Lane 1 and Lane 2) where roller operations began slowly but became more aggressive as the night progressed.

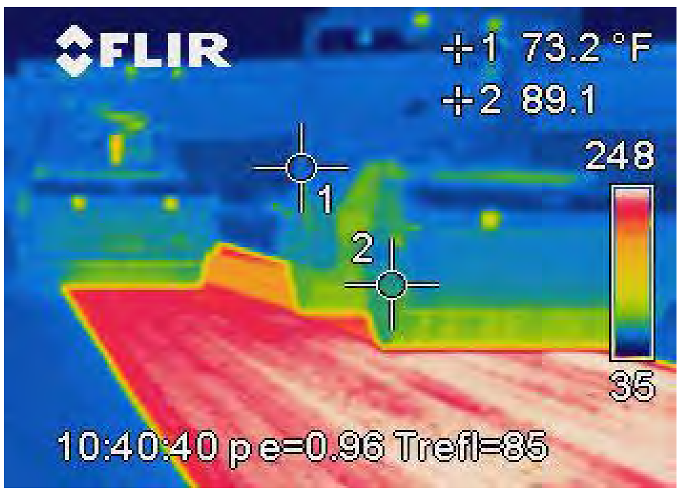


Figure 45. Image of breakdown rollers working in tandem with each other.

The longitudinal joints constructed on the first night of paving were visible, due to the thinness of the pavement and an aggregate structure that resisted compaction (Figure 46). The joints were constructed with nearly 1/4 inch height difference between lanes after compaction. This issue appeared to have had two contributing factors; (1) a screed being too high resulting in an excess of mix, and (2) improper roller operation. The longitudinal joint was generally rolled from the cold-side with a four to six inch overlap onto the hot-side. It has been found that rolling from the hot-side with four to six inch overlap to the cold-side results in a better joint[1].

[1] *Longitudinal Joint Construction Technote*, WSDOT, February 2003 - http://edit.wsdot.wa.gov/NR/rdonlyres/D0E12249-96F8-4EB3-ABA8-5E8FCE979A37/0/LJTN2007FINAL2.pdf

Figure 46. Image of the mismatch at the longitudinal joint.

The only way that roller operations could have adhered to the 300-foot specification would be if the paver slowed down at the beginning of the new lane. This would require coordination with the plant and the timing of the delivery trucks so that all of the operations slowed down at the same time. As it was on this project trucks were lined up waiting to transfer their loads to the SB. The result was that the mix cooled in the trucks and thus the start of the next lane had some cooler pavement temperatures (Figure 47).

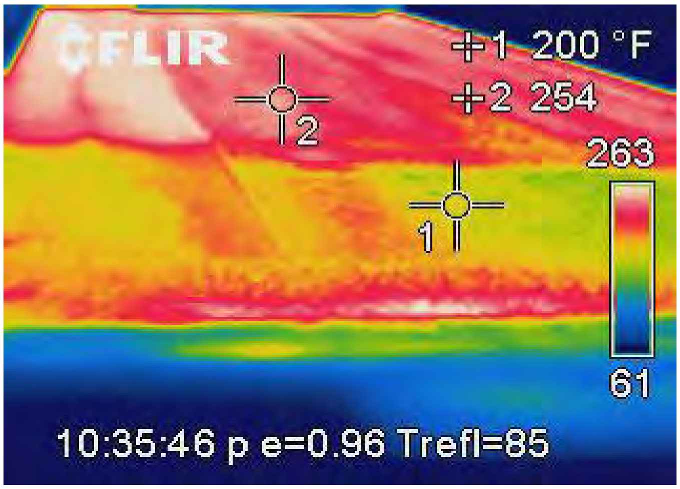


Figure 47. Thermal image of construction joint at startup.

OGFC-AR

OGFC-AR was sticky and adhered to the paving equipment (i.e.: rakes, truck beds, SB tires, etc.). It was difficult to work with this material around utilities, catch basins, and other objects and create a good appearing mat. The sticky nature of the mix resulted in the liberal use of release agents on all of the equipment including the tires of the Shuttle Buggy because it had begun to lift the CRS-2P that was now sticking to the tires. It was thought that this action might affect the long-term performance of the OGFC-AR because it would not have proper adhesion to the existing pavement surface.

Temperatures recorded direction behind the paver where generally between 280 ºF and 290ºF. These temperatures were consistent with those measured in Arizona when paving with the same mix.

OGFC-SBS

The OGFC-SBS was even stickier then the OGFC-AR. It also appeared to be more influenced by the CRS-2P tack coat. As the paver passed over a globule of CRS-2P, it would appear as a cold spot in the mat which, if not removed, became a fat spot in the pavement (Figure 48). It is not known to what degree this will affect pavement performance, if at all. This also occurred during the paving of the test section.

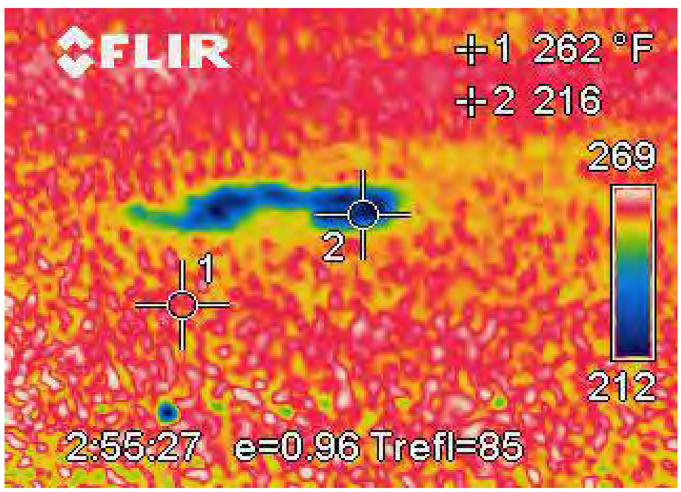


Figure 48. Thermal image of a cool glob of CRS-2P in the newly place OGFC-SBS surface.

OTHER ISSUES

Other issues that were present but not necessarily related to either OGFC-AR or OGFC-SBS was that of mix on the roadway. On two different instances, one OGFC-AR and the other OGFC-SBS, a substantial amount of material was accidentally dumped on the roadway prior to the SB.

For the OGFC-AR, when this happened the mix was picked up using the end-loader of a backhoe. The residual material was approximately 1/4 – 1/2 inch thick and the paver paved over the top of this. This left a cooler spot in the mat where this was paved. In addition, the tires of the SB also tracked through the material that was loaded into the paver hopper. Results of this caused globs of cooler mix to reflect through the new pavement surface which ended up causing fat spots.

One other problem that occurred during the paving was spillage of mix from the paver hopper onto the existing roadway prior to the paving operation. When this happed the clean-up of this spilled mix was not as good as it should have been with the result that some residual material remained (Figure 49). The Shuttle Buggy would track through this material, pick it up on the tires and deposit it as globs of cooler mix to be paved over (Figure 50). The result for the OGFC-AR mix was fat spots in the final mat.

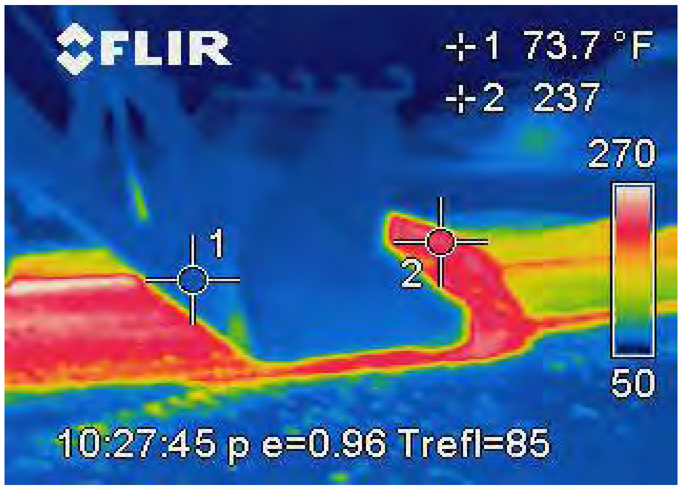


Figure 49. Thermal image of backhoe scooping dumped OGFC-AR from roadway.

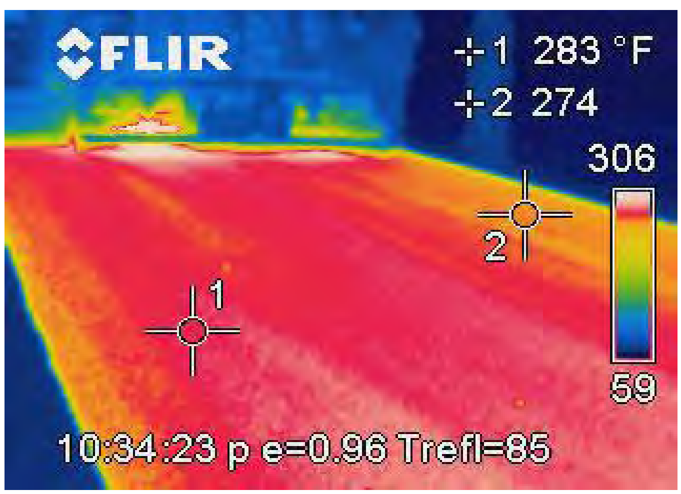


Figure 50. Thermal image where paver paved over remaining material that was picked up.

For the OGFC-SBS, the material lost was not as substantial as that of the OGFC-AR. The spilled mix was picked up using shovels (but more mix was left on the roadway), and the paver continued to pave over the mix (Figure 51). Because there was more mix on the roadway prior to the paver, a slight hump was created where the spilled mix remained. In addition, a significant amount of globs of mix were dropped on the roadway prior to the paver that reflected through (Figure 52 and 53). This ended up causing a considerable amount of remedial work to be done. Most of this work was involved in either removing a glob from the pavement, or placing material in a location that had material missing (this was typically caused from a glob that would drag under the paver until coming loose).

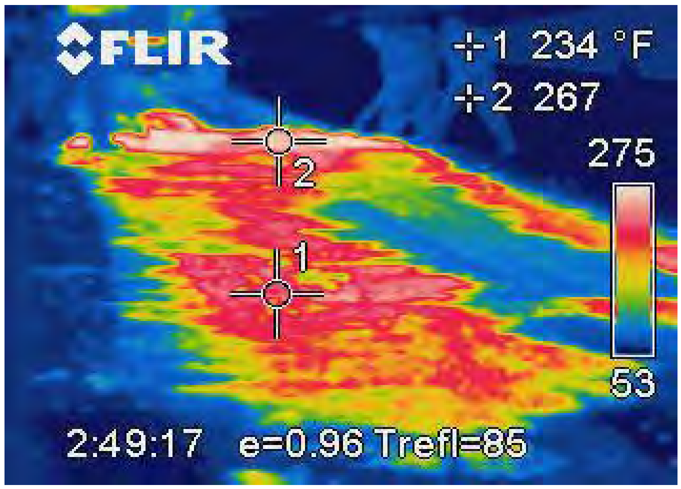


Figure 51. Thermal image of dumped material being shoveled.

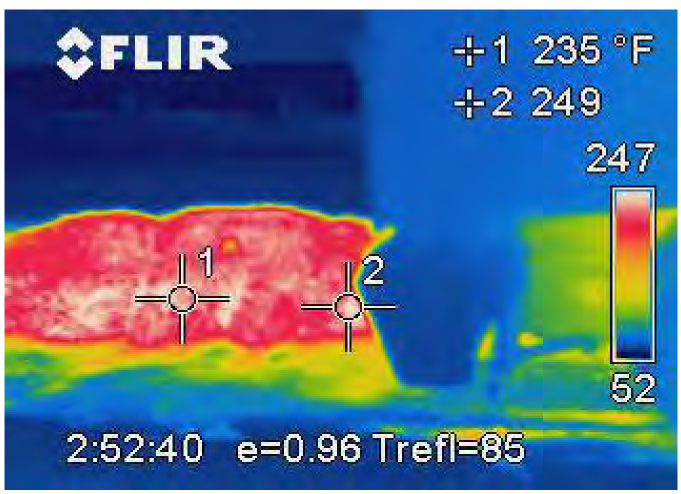


Figure 52. Thermal image of paver paving over remaining material on roadway.

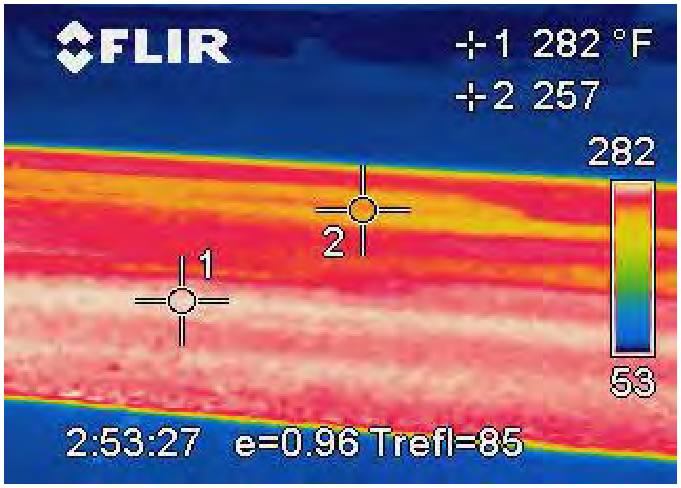


Figure 53. Image of cool spot where paver paved over remaining mix on roadway.

RECOMMENDATIONS

- It may be of benefit on future projects that use CRS-2P as a tack coat to modify the tack specifications to ensure an even application of the material. This might include a test of the tack coat truck prior to beginning paving to ensure that all of the snivies are clean and operating properly.
- Specify that tarps be used on all trucks and trailers to ensure maximum heat retention in the mix between the plant and the paving operation.
- Specify a material transfer vehicle be used on all thin lift open-graded friction course projects.
- Specify that the retrofit kit be used on all applicable paver models.
- Specify that auger extensions be used when the screed is extended a certain specified distance.

COMMENTS

- Proper procedures need to be adhered to when matching the pavement thicknesses at the longitudinal joint and proper rolling techniques need to be employed in order to ensure a tight, flush joint.
- The temperature of the screed should be at the same temperature as the mix prior to starting the paving operation.
- All of the paving operations need to be coordinated to keep the rollers within 300 feet of paver.
    - Slow down production at the plant at the end of the completion of one lane so that the material does not build up when the paver is being moved.
    - Don’t load too many trucks prior to working at a construction joint.
    - Allow the rollers time to work effectively at the construction joint.
    - Keep the pavers moving consistently at a slow speed.

  - Ensure the paver doesn't begin paving out until rollers have completed the work at a construction joint.
- Minimize handwork as much as possible.
- Keep delivery trucks and MTV tires as clean as possible to avoid bringing debris into work area.
- Keep work area as clean as possible at all times. If material gets dumped onto the roadway, or the build-up on tires becomes excessive, clean thoroughly. Remember that this is a thin surface and defects will reflect through.

Appendix D

Sound Intensity Measurements

Table 19. OGFC-AR sound intensity readings (dBA).					
Date	Lane 1	Lane 2	Lane 3	HOV	Average
8/23/2006	94.7	95.0	94.9	95.7	95.1
9/9/2006	94.3	95.6	94.6	95.5	95.0
9/28/2006	96.0	95.6	96.0	96.2	96.0
10/17/2006	97.9	97.7	101.1	96.9	98.4
12/4/2006	98.1	97.8	96.6	95.2	96.9
12/28/2006	101.8	98.2	97.9	96.0	98.5
1/25/2007	100.4	98.7	98.0	97.0	98.5
3/6/2007	98.9	101.1	99.3	98.8	99.5
3/21/2007	103.3	103.4	100.9	99.0	101.7
4/23/2007	100.5	101.4	99.2	97.9	99.8
5/29/2007	101.2	101.7	98.4	98.3	99.9
6/26/2007	100.5	101.5	99.0	97.1	99.5
7/26/2007	99.6	101.3	98.6	96.9	99.1
9/6/2007	100.4	101.6	98.7	96.8	99.4
9/26/2007	100.6	101.8	99.5	97.6	99.9
10/31/2007	102.4	103.6	101.1	98.8	101.5
12/6/2007	103.2	104.1	101.7	100.0	102.3
1/24/2008	103.4	103.4	102.2	99.5	102.1
2/8/2008	103.1	104.3	102.8	100.4	102.7
4/8/2008	103.5	104.2	102.8	101.6	103.0
5/8/2008	103.9	101.4	103.3	102.0	103.4
7/8/2008	102.6	102.9	101.2	99.2	101.4
8/8/2008	103.0	103.0	102.5	100.0	102.1
12/2/2008	105.1	105.7	103.9	103.0	104.4
1/20/2009	104.7	105.6	104.4	104.9	104.9
2/9/2009	104.7	104.8	104.0	103.0	104.1
3/12/2009	105.4	105.6	103.9	104.4	104.8
4/20/2009	103.9	104.2	103.5	103.4	103.8
5/27/2009	103.5	103.9	103.1	102.7	103.3
/13/2009	104.2	104.6	104.5	104.1	104.4
9/2/2009	103.4	103.8	102.7	102.4	103.1
10/6/2009	104.8	105.5	104.5	104.8	104.9
12/22/2009	106.6	106.4	106.4	106.4	106.5
1/7/2010	106.7	105.8	106.0	106.8	106.6
2/10/2010	105.3	105.6	105.2	104.9	105.3
3/10/2010	104.2	103.9	103.7	103.7	103.9
4/10/2010	104.7	105.1	104.5	104.3	104.7
5/10/2010	104.2	105.1	104.0	104.3	104.4
6/10/2010	105.0	104.7	104.7	104.9	104.8
7/13/2010	104.8	105.0	104.4	103.6	104.5
8/16/2010	103.3	103.7	103.2	103.1	103.3

Table 20. OGFC-SBS sound intensity readings (dBA).					
Date	Lane 1	Lane 2	Lane 3	HOV	Average
8/23/2006	-	-	-	-	-
9/9/2006	96.3	95.8	96.1	95.7	96.0
9/28/2006	96.6	96.4	96.3	95.8	96.3
10/17/2006	101.1	96.9	97.0	98.1	98.0
12/4/2006	98.1	97.1	97.3	96.7	97.3
12/28/2006	101.8	101.3	98.9	96.9	99.7
1/25/2007	98.1	100.1	98.8	97.2	98.6
3/6/2007	97.2	98.6	98.8	98.0	98.2
3/21/2007	102.0	101.7	99.5	98.9	100.5
4/23/2007	100.0	100.2	98.4	98.8	99.4
5/29/2007	100.2	100.1	98.2	98.3	99.2
6/26/2007	99.8	99.8	97.4	97.6	98.7
7/26/2007	99.2	98.8	96.7	97.1	98.0
9/6/2007	97.9	98.6	96.6	97.4	97.6
9/26/2007	100.3	99.0	97.0	97.5	98.5
10/31/2007	100.8	99.6	98.1	98.4	99.2
12/6/2007	101.6	100.7	97.9	99.0	99.8
1/24/2008	101.3	100.3	98.0	98.4	99.5
2/8/2008	101.3	100.6	99.6	99.3	100.2
4/8/2008	101.1	100.9	100.0	100.0	100.5
5/8/2008	101.6	101.6	99.4	100.8	100.9
7/8/2008	99.4	102.9	98.0	99.2	99.9
8/8/2008	100.5	100.6	98.5	99.9	99.9
12/2/2008	102.5	102.6	101.0	103.0	102.3
1/20/2009	102.8	103.4	103.2	102.8	103.1
2/9/2009	103.9	102.4	100.5	101.2	102.0
3/12/2009	104.7	103.2	102.0	101.7	102.9
4/20/2009	103.4	103.7	102.2	101.1	102.6
5/27/2009	103.4	102.6	101.5	101.5	102.3
/13/2009	103.8	103.2	102.7	103.6	103.3
9/2/2009	102.2	102.4	101.5	101.5	101.9
10/6/2009	104.2	103.7	102.2	102.7	103.2
12/22/2009	105.4	105.2	106.4	105.5	105.6
1/7/2010	105.4	105.5	104.2	103.4	104.6
2/10/2010	104.6	103.6	103.8	104.0	104.0
3/10/2010	103.3	103.0	102.6	103.7	103.2
4/10/2010	103.9	103.8	102.9	102.3	103.2
5/10/2010	103.6	103.5	103.0	102.8	103.2
6/10/2010	105.0	104.7	104.7	104.9	104.8
7/13/2010	103.9	104.4	103.6	102.4	103.6
8/16/2010	101.8	102.6	102.4	100.2	101.8

Table 21. HMA sound intensity readings (dBA).					
Date	Lane 1	Lane 2	Lane 3	HOV	Average
8/23/2006	99.5	100.0	99.0	99.2	99.4
9/9/2006	97.8	99.5	99.2	98.8	98.8
9/28/2006	97.7	99.1	99.1	99.8	98.9
10/17/2006	100.4	100.0	100.2	100.6	100.3
12/4/2006	100.1	100.7	99.8	100.8	100.4
12/28/2006	102.1	102.0	101.5	101.6	101.8
1/25/2007	101.3	101.7	101.3	100.9	101.3
3/6/2007	100.6	101.5	101.4	102.5	101.5
3/21/2007	102.1	102.9	102.4	101.8	102.3
4/23/2007	101.7	102.2	101.5	100.5	101.5
5/29/2007	102.5	102.3	101.8	100.5	101.8
6/26/2007	100.8	101.2	100.4	99.8	100.6
7/26/2007	101.6	101.5	101.6	100.4	101.3
9/6/2007	101.7	102.0	100.2	99.8	100.7
9/26/2007	101.8	102.2	101.6	100.6	101.6
10/31/2007	103.2	103.4	102.6	101.7	102.7
12/6/2007	103.4	103.3	102.1	101.6	102.6
1/24/2008	102.7	102.9	101.8	101.4	102.2
2/8/2008	103.0	102.1	103.4	102.1	102.7
4/8/2008	103.6	103.3	103.5	102.7	103.3
5/8/2008	102.8	104.5	104.0	102.2	103.4
7/8/2008	102.0	102.5	102.0	101.0	101.9
8/8/2008	103.5	103.3	102.5	101.9	102.8
12/2/2008	104.4	104.1	103.6	102.8	103.7
1/20/2009	105.0	103.8	104.0	102.4	103.8
2/9/2009	104.8	104.1	103.4	102.1	103.6
3/12/2009	105.4	105.0	104.7	103.0	104.5
4/20/2009	104.2	103.3	103.0	102.3	103.2
5/27/2009	103.8	103.8	103.2	101.9	103.2
/13/2009	99.7	101.7	104.0	102.8	102.1
9/2/2009	103.3	103.7	103.2	102.0	103.1
10/6/2009	104.8	104.4	104.2	102.6	104.0
12/22/2009	106.1	105.2	104.6	104.9	105.2
1/7/2010	106.3	105.2	105.7	104.5	105.4
2/10/2010	105.2	105.0	104.8	103.5	104.6
3/10/2010	104.2	104.2	103.4	102.3	103.5
4/10/2010	104.5	105.0	104.4	102.9	104.2
5/10/2010	104.8	104.6	104.1	102.8	104.1
6/10/2010	105.1	105.1	104.7	102.5	104.4
7/13/2010	104.5	104.7	104.6	103.3	104.3
8/16/2010	103.2	104.1	104.1	102.7	103.5

Appendix E

In-House Report on Class D HMA Performance

ASSESSMENT OF ASPHALT CONCRETE PAVEMENT - CLASS D

September 1995

OVERVIEW

Rehabilitation of Washington State highways does not always require additional pavement structure but merely some type of surface treatment. For roadways with small average daily traffic (ADT), low speeds and adequate structure, a bituminous surface treatment (BST) will provide the desired surface. On higher ADT routes a BST (or chip seal) is not a practical option, as the roadway may have to be restricted or closed during construction. In addition, flying rock caused by higher vehicle speeds causes headlight and windshield damage.

To allow a surface treatment, similar to a BST, on heavily trafficked routes Class D asphalt is used. Class D asphalt overlays are used primarily to seal and maintain aged, but otherwise structurally sound pavements. Class D asphalt pavement (known previously as "open-graded asphalt seal coat") is commonly referred to as "open-graded asphalt pavement", and is basically a chip seal aggregate mixed hot in a plant with a relatively high percentage of asphalt cement. Class D overlays are placed with an asphalt paver at a compacted depth of 0.06 ft.

Class D asphalt differs from WSDOT's "standard" paving mix such as Class A or B in that the compacted mix appears as a honeycombed matrix. This matrix is caused by the gradation of aggregate where there is a higher percentage of course than fine aggregate. Essentially, there are not enough fine particles to fill the voids between the larger rocks. The result of this open-graded mixture is that water can drain laterally through the pavement.

BENEFITS

One of the benefits of this material for WSDOT is its use as a finish overlay over both lanes and shoulders when only the lanes are milled out and replaced. As Class D asphalt is designed for wearing surfaces only, the total pavement structure of the roadway must be in adequate condition prior to placement. This material does not add structural capacity, but acts to seal and restore skid resistance to certain roadways. WSDOT's targeted service life for a Class D overlay is eight years.

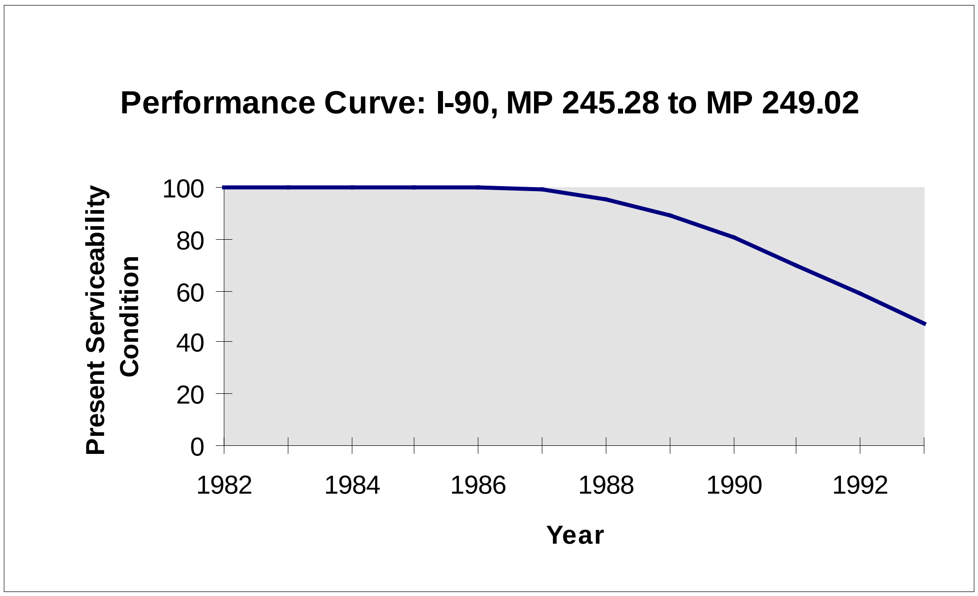
Performance benefits as experienced by WSDOT and outlined in the 1992 NCHRP Synthesis 180 report include the following:

- Improved skid resistance during wet weather
- Reduced hydroplaning potential
- Improved smoothness
- Reduced splash and spray
- Improved visibility of painted traffic markings

- Reduced wet pavement glare at night
- Reduced noise levels

CONCERNS

Initially, the performance of Class D pavements is steady as indicated by a flat performance curve. An example of a performance curve for a section on I-90, MP 245.28 to MP 249.02 follows:



The surface condition as indicated by the present serviceability condition changes little or none for several years. Beyond a certain point, the performance curve drops off rapidly as the pavement becomes brittle, micro-cracks, and then fails quickly due to pavement deterioration (such as raveling or delamination). Typically, WSDOT attempts to resurface roads at a present serviceability condition rating of 50.

The failure modes of Class D experienced by WSDOT included raveling (aggregate particles that are dislodged from the pavement) and delamination (loss of bond between pavement layers). Raveling can be caused by improper mix design, placement during cold or wet weather, and oxidation. Raveling can also be accelerated by the use of studded tires coupled with high ADT. Delamination can be caused by placement during cold or wet weather or the improper application of the tack coat.

The FHWA recommends placing a fog seal at five-year intervals or at the first sign of raveling. The placement of a fog seal may delay raveling, but consequentially some of the internal drainage within the Class D matrix will be lost.

Some other concerns that impact the use of this material are:

- Urban areas with high volume ADT should not be considered due to the higher use of studded tires. Studded tires tend to break apart the aggregate structure and cause premature raveling.
- The beneficial effects of reduced splash and spray seems to last only two to three years depending on the amount of sanding and other fine debris that is deposited on the surface. Thus, the benefits obtained from the open-graded nature may be short lived.
- In order to obtain compaction, Class D asphalt must be placed at temperatures above 60º Fahrenheit. Where nighttime paving is required, this material may not be used. Class D overlays are placed in thin lifts and are susceptible to rapid cooling.
- On higher volume roadways consideration must be given to the effects of traffic disruptions since the life of the Class D is shorter than typical dense graded mixes.
- There can be problems with snow and ice removal. Due to the permeability of the material the retention of deicing solutions is reduced. In addition, ice tends to “stick” to the surface and causes snowplows to ride up over it and pull out aggregate from the surface matrix.
- Due to the lower service life occurring in some regions, life cycle cost analysis does not satisfy the expenditure for using the material.

WSDOT’S EXPERIENCE

WSDOT has been placing Class D overlays with varied success since the early 1980’s. There are sections on Interstates 82 and 90 that were placed in the early 1980’s that are still in service. However, there are sections that were placed more recently on I-90 and I-5 that are not providing the targeted eight-year service life. During 1995, WSDOT estimated there were about 800 lane miles of Class D asphalt placed in the state. This represents less than eight percent of the statewide 10,500 lane miles. The majority of miles already in place are on the primary highway system where chip seals are not a viable option due to heavy traffic volumes.

Eastern Region

The Eastern Region has used Class D asphalt concrete overlays since 1980. Projects that have been selected are sections that are structurally sound but are experiencing some type of surface distress such as rutting or raveling. Five projects along I-90 totaling 36.8 miles or 147.2 lane miles have been constructed. Only 35.3 lane miles of Class D pavement remain in the Eastern Region. These remaining miles will be milled and resurfaced during 1995 leaving no Class D pavements in service.

Construction of the overlays on I-90 included milling the distressed lanes 0.15 ft and inlaying the lanes with a dense graded asphalt such as ACP Class B. The Class D asphalt concrete was overlaid shoulder-to-shoulder 0.06 ft thick followed by a CSS-1 fog seal.

The Eastern Region has obtained varied success with Class D overlays. A summary of the construction history and the years obtained from each overlay in the Eastern Region is as follows:

Eastern Region - Class D Projects

SR Route	Year Constructed	Contract No.	Location	Year Repaved	Pavement Life (years)	Average Daily Traffic
90	1982	2279	MP 191.89 to MP 200.36	1993	11[1]	7,800
90	1982	2293	MP 244 90 to MP 254.31	1993	11[1]	11,600
90	1982	2058	MP 254.32 to MP 257.35	1993	11[1]	12,100
90	1980	1869	MP 270.36 to MP 277.51	1988	8	31,000
90	1991	3958	MP 290.36 to MP 299.19	1995	4	40,000

[1]Noticeable rutting occurred during years seven or eight.

The table indicates some sections have lasted 11 years, well beyond the WSDOT goal of eight years. Actually, the time when rutting becomes noticeable has typically been seven or eight years. Therefore, the service life or the time when rehabilitation should have been performed was indeed closer to eight years. The service life for the project placed during 1980 was closer to six years rather than eight years. The difference in times is the time that rehabilitation was needed and actually programmed.

As was indicated earlier WSDOT does not recommend Class D placement in high volume, urban areas. For the 1980 project (Contract 1869) the current ADT is 31,000. For the 1991 project (Contract 3958) the current ADT is 40,000. For the projects outside the Spokane urban area that obtained 11 years between overlays the current ADT ranges from 7,800 to 12,100.

Review of the construction history shows the Eastern Region has had two Class D projects that did not perform as expected. One project, paved during 1980, was repaved eight years later and another project, paved during 1991, is being repaved during 1995. The failure mode for these projects was raveling accelerated by the use of studded tires. This raveling typically appears as ruts isolated to car wheel paths and proceeds through the full thickness of the Class D overlay.

Other factors may contribute to early raveling, such as late season paving. Late season projects (depending on the weather) do not always allow adequate compaction of the asphalt prior to cooling. Late season projects also do not allow curing of the asphalt before the onset of winter.

Another factor that may contribute to early raveling is the mixing of Class D asphalt higher than the recommended 235° Fahrenheit (asphalt temperatures are sometimes elevated during mixing to compensate for cooler air temperatures). When open-graded mixtures such as Class D are mixed too hot, drain down occurs. The higher mixing temperatures cause the asphalt cement to flow off the rock and settle to the bottom of the mat. The film thickness on the rock at the top of the mat becomes reduced. Heavy asphalt films on the aggregates are essential to resist stripping, oxidation, and ultimately raveling.

Since environmental conditions during paving can adversely affect the performance of any pavement, the 1994 WSDOT Standard Specification was revised to prohibit paving past October 15. The October 15 deadline will improve pavement performance and reduce likelihood of problems such as early raveling.

Eastern Region Class D Construction Costs

Construction costs for the Eastern Regions Class D overlays were researched and are presented in the table that follows:

Eastern Region - Class D Construction Costs

Year	Contract No.	Location	Cost - Class B		Cost - Class D	
			(ton)	(sy)	(ton)	(sy)
1982	2279	MP 244.90 to MP 200.36	$15.90	$1.63	$23.50	$0.97
1982	2293	MP 244.90 to MP 254.31	$13.30	$1.37	$23.10	$0.95
1982	2058	MP 254.31 to MP 257.35	$22.10	$2.27	$23.29	$0.96
1992	3958	MP 290.36 to MP 299.10	$22.25	$2.29	$23.00	$0.95

The square yard prices are based on thickness of 0.15 ft for Class B and 0.06 ft for Class D overlays. The Class B and Class D prices were obtained from the same contract with the exception of the ACP Class B on Contract 3958, which was not placed on this project. A price for Class B asphalt concrete was obtained from similar projects placed during that year.

It should be noted that Class B is usually placed in thicker lifts providing structural support to the roadway. The Class D overlay is merely a wearing surface that adds little structural benefit.

Southwest Region

The Southwest Region's experience has been similar to the Eastern Region's. The typical age of Class D pavements in the Southwest Region is approximately 11 years. Many Class D pavements are still in service although ruts have developed through the 0.06 ft overlays. Discussions with the region revealed noticeable rutting actually occurred during the seventh or eighth year, sometimes earlier. The use of studded tires is not as extensive in the Southwest Region as in the Eastern Region; however, the Southwest Region does feel that the use of tire chains adds considerably to their Class D pavement distress.

Four projects are summarized:

Southwest Region - Class D Projects

SR Route	Year Constructed	Contract No.	Location	Year Repaved	Pavement Life (years)	Average Daily Traffic
5	1986	3044	MP 0.27 to MP 2.42	Future	10+[1]	80,200
5	1984	2591	MP 4.32 to MP 7.53	1996	11[1]	56,000
5	1984	2608	MP 20.11 to MP 22.12	Future	12+[1]	48,400
5	1990	3522	MP 72.29 to MP 73.28 & MP 78.44 to 79.21	Future	6+	48,000

[1]**Noticeable rutting occurred during years seven or eight.**

South Central Region

The South Central Region has experienced good success with Class D pavements and have nearly 200 miles of Class D pavements that are still in service. Some pavements are currently 12 to 15 years old and have only recently raveled the full 0.06 ft thickness. Discussion with the South Central Materials Lab revealed that they have never had a Class D pavement fail in less then 10 years. Fog seals on class D pavements have not been applied.

A summary of several Class D project follows:

South Central Region - Class D Projects

SR Route	Year Constructed	Contract No.	Location	Year Repaved	Pavement Life (years)[1]	Average Daily Traffic
12	1983	2339	MP 277.09 to MP 281.08	Future	12+	26,000
12	1980	1850	MP 288.95 to MP 290.24	Future	15+	--------
12	1982	2270	MP 295.41 to MP 303.36	Programmed. 1994	13+	9,100
12	1982	3721	MP 304.97 to MP 305.36	1990	8	6,200
12	1984	2680	MP 314.20 to MP 318.01	1994	10	4,800

SR Route	Year Constructed	Contract No.	Location	Year Repaved	Pavement Life (years)[1]	Average Daily Traffic
12	1983	2339	MP 277.09 to MP 281.08	Future	12+	26,000
24	1985	2957	MP 0.05 to MP 4.11	Programmed 1997	10+	9,600
82	1982	2196	MP 0.62 to MP 3.23	1994	1	11,600
82	1984	2692	MP 23.88 to MP 29.02	Programmed 1996	11+	12,800
82	1983	2310	MP 97.46 to MP 100.66	Programmed 1996	12+	12,400
90	1981	1904	MP 106.34 to MP 110.00	1994	13	20,000
90	1984	2663	MP 110.00 to MP 122.23	Programmed 1998	11+	10,400
90	1982	2231	MP 126.14 to MP 137.19	1994	13	10,400
90	1978	1012	MP 102.61 to MP 103.19	Programmed 1998	17+	18,800
224	1982	2339	MP 7.42 to MP 9.78	1994	12	11,800

[1]Noticeable rutting occurred beyond 10 years

The South Central Region noted that Class D pavements that go beyond 12 years display the full 0.06 ft depth rutting such as experienced in both the Southwest and Eastern Regions.

The South Central Region will continue to consider Class D pavements. Class D overlays have been placed in lieu of chip seals on select routes.

SUMMARY

WSDOT should continue to consider Class D asphalt in the analysis of our pavement designs. Class D overlays have performed well for roadways outside of urban areas and may be cost effective. With recommended overlay cycles of eight years, a life cycle cost analysis is necessary to determine the cost effectiveness. To place a statewide moratorium (as some states have done) on Class D overlays may be premature. However, the use of Class D asphalt in areas of high ADT or high studded tire use should be avoided.

Appendix F

Performance Data for Open-Graded Pavements

The following table was put together by the Pavement Section of the Headquarters Materials Laboratory from historical data derived mainly from the Washington State Pavement Management System, but also in part from other sources within the Pavements Section. The average pavement life for the 55 projects listed is 8.2 years.

Table 22. Open-graded friction course pavement performance data.

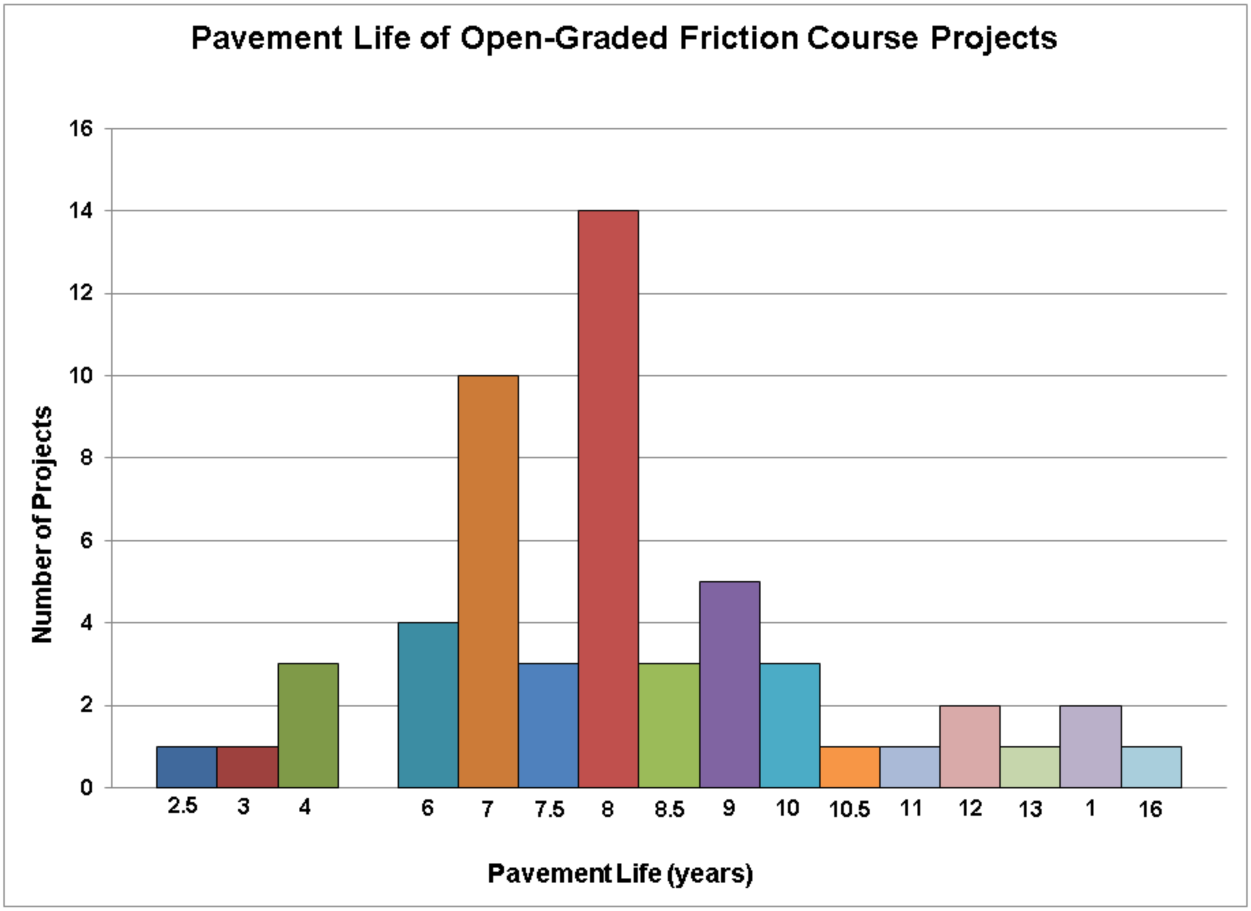
Route	Dir.*	Beg. MP	End MP	Region**	Contract Number	Date Constructed	Age At 1/2 Inch Rutting	ADT Per Lane
23	B	42.64	51.86	E	2293	1982	16	500
90	I	191.89	200.36	E	2279	1982	7	3,000
90	D	191.89	200.36	E	2279	1982	7	3,000
97	D	69.32	74.74	SC	4673	1995	9	3,000
970	B	0.77	5.85	SC	4627	1995	9	3,000
82	D	0.61	3.23	SC	2196	1982	11	4,000
82	I	0.63	3.22	SC	2196	1982	10	4,000
90	I	239.15	244.90	E	2058	1982	8	4,000
90	D	239.15	244.90	E	2058	1982	9	4,000
90	I	244.90	254.31	E	2293	1982	8	4,000
90	D	244.90	254.33	E	2293	1982	7	4,000
90	I	254.13	257.35	E	2058	1982	9	4,000
90	D	254.33	257.35	E	2058	1982	8.5	4,000
12	B	304.51	307.66	SC	3721	1990	14	4,000
22	B	35.98	36.45	SC	4819	1997	7	4,000
90	I	110.00	121.96	SC	2663	1984	12	4,500
90	D	110.00	121.96	SC	2663	1984	10.5	4,500
82	D	82.14	84.35	SC	4819	1997	7	5,000
12	B	302.21	305.36	SC	2270	1982	13	5,000
82	I	97.64	100.66	SC	2310	1983	9	6,000
82	D	97.64	100.66	SC	2310	1983	10	6,000
90	I	102.61	106.34	SC	1012	1978	12.3	7,000
90	D	102.61	106.34	SC	1012	1978	14	7,000
90	I	270.36	275.40	E	1869	1980	8	10,000
90	D	270.36	275.55	E	1869	1980	8	10,000
82	I	33.84	36.29	SC	4102	1993	6	12,000
82	D	33.84	36.29	SC	4102	1993	6	12,000
82	I	30.96	33.84	SC	4346	1994	4	12,500
82	D	30.96	33.84	SC	4346	1994	4	12,500
90	I	290.36	299.10	E	3958	1992	2.5	20,000
90	D	290.36	299.10	E	3958	1992	3	20,000
12	I	2.08	4.94	O	XE2906	1992	7	6,000
12	D	2.08	4.94	O	XE2906	1992	8	6,000
5	I	20.07	22.12	SW	2608	1984	8	8,000
5	D	20.78	22.01	SW	2608	1984	8	8,000
5	I	88.02	98.88	SC	2571	1984	8	14,000
5	D	88.02	98.88	SC	2571	1984	7.5	14,000
5	I	98.88	102.70	O	2571	1984	10	14,000
5	D	98.88	102.70	O	2571	1984	6	14,000
5	I	101.23	102.69	O	3939	1993	7	14,000
5	D	101.23	102.69	O	3939	1993	7	14,000
5	D	70.68	72.24	SW	3934	1991	8.5	15,000
5	I	72.29	73.28	SW	3522	1990	7	15,000

Route	Dir.*	Beg. MP	End MP	Region**	Contract Number	Date Constructed	Age At 1/2 Inch Rutting	ADT Per Lane
5	D	72.29	73.28	SW	3522	1990	8.5	15,000
5	D	77.55	78.44	SW	3934	1991	7	15,000
5	I	4.32	7.53	SW	2591	1984	7.5	16,000
5	D	4.32	7.53	SW	2591	1984	7.5	16,000
5	D	68.96	70.61	SW	2720	1985	8	16,000
5	I	0.27	2.42	SW	3044	1986	8	18,000
5	D	0.27	2.42	SW	3044	1986	8	18,000
5	I	78.44	79.21	O	3522	1990	8	18,000
5	D	78.44	79.21	O	3522	1990	8	18,000
5	I	82.38	83.35	O	3522	1990	8	22,000
5	D	82.38	83.35	O	3522	1990	8	22,000
5	I	135.54	139.50	O	2554	1985	4	22,000
5	D	135.54	139.50	O	2554	1985	6	22,000

Table 22. (continued). Open-graded friction course pavement performance data.

*I = Increasing, D = Decreasing, B = Both (for 2 lane undivided highways)
**E = Eastern, O = Olympic, SC = South Central, and SW = Southwest

The average pavement life of all projects is 8.2 years with the distribution of ages shown below.



Appendix G

Experimental Feature Work Plan

Washington State Department of Transportation

WORK PLAN

EVALUATION OF LONG-TERM PAVEMENT PERFORMANCE AND NOISE CHARACTERISTICS FOR OPEN-GRADED FRICTION COURSES

Interstate 5

52^{nd} Avenue West to SR-526 – Southbound

Milepost 180.10 to Milepost 189.30

Linda M. Pierce, PE
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Washington State Department of Transportation

Introduction

Hot-mix asphalt (HMA) open-graded friction courses (OGFC) can reduce traffic noise and splash and spray from rainfall. These performance benefits come at a cost in durability, greatly reducing pavement life compared to traditional asphalt and concrete pavements. The benefit of noise reduction, and splash and spray reduction degrades over relatively short periods of time, reducing the effectiveness of the OGFC pavement. Pavement lives of less than ten years, and as short as three to four years, have occurred with the use of OGFC pavements in Washington's high traffic corridors. The life of asphalt based quieter pavement in the USA and around the world tends to average between 8 and 12 years. Compare this to an average pavement life of 16 years in western Washington and the loss of durability is clear. Under RCW47.05, WSDOT is instructed to follow lowest life cycle cost methods in pavement management. Less durable pavements do not meet this legislative direction.

Studded tire usage in Washington State is another complicating factor. Studded tires rapidly damage OGFC pavements, resulting in raveling and wear. When OGFC was used on I-5 in Fife, the pavement had significant wear in as little as four years. States where the use of OGFC has been successful (Florida, Texas, Arizona and California) do not experience extensive studded tire usage. Similarly, these states are southern, warm weather states; a clear advantage when placing a product like OGFC with asphalt-rubber. Arizona DOT, for example, requires the existing pavement to have an 85°F surface temperature at the time of placement. Washington State urban pavements, placed at night to avoid traffic impacts, rarely reach this temperature during the available nighttime hours for paving (10:00 p.m. to 5:00 a.m.), even in summer. Other pavements and bridge decks reach such temperatures at night only on rare occasions, making successful placement of rubberized OGFC difficult or impossible at night.

Plan of Study

The objective of this research study will be to determine the long-term pavement performance characteristics of OGFC pavements in Washington State. It will focus primarily on the OGFC's resistance to studded tire wear, its durability and its splash/spray characteristics. In addition, noise reduction characteristics will also be measured. WSDOT, at a minimum, will be evaluating noise levels using sound intensity measurement equipment (additional evaluations to be determined in the next couple of months). The pavement performance and noise intensity measurements will be conducted on an annual basis.

In addition, this study will also document any challenges with the construction of the OGFC during nighttime paving operations.

Scope

This project will construct two OGFC test sections, each ½-mile in length, one with asphalt-rubber and the other with PG70-22. This section of southbound interstate consists of three 12-foot lanes, a 10-foot right and 10-foot left shoulder.

Both sections of the OGFC will be placed full roadway, including shoulders, to a depth of 0.06 feet.

WSDOT will be designing the mixes in accordance with the Arizona DOT specifications for OGFC with asphalt-rubber (AR) and OGFC with a styrene-butadiene-styrene (SBS) modified asphalt binder.

Layout

The first test section will begin at MP 188.65 and end at MP 188.15 and the second will begin at MP 188.15 and end at MP 187.65.

Figure 1. Interstate 5 at MP 188.65

Figure 2. Interstate 5 at MP 188.15

This location was selected for ease of construction (occurs at the beginning of the pavement project), relatively similar terrain and the same level of traffic over both test sections.

Control Section

A ½ mile length of the project will serve as the control for the evaluation of the OGFC mixes. The project calls for a ½ inch Superpave mix using a PG 64-22 binder. The limits of the control section will be determined after construction is completed. The location will be chosen so that it duplicates, as closely as possible, the same environment and traffic conditions as the two test sections.

Staffing

This research project will be constructed as part of a larger rehabilitation project. Therefore the Region Project office will coordinate and manage all construction aspects. Representatives from the WSDOT Materials Laboratory (1 – 3 persons) will also be involved with the process.

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Testing

The following annual testing procedures will be conducted on the test sections and control section.

- Pavement condition
  - Surface condition (cracking, patching, flushing, etc)
  - Rutting/wear (using the INO laser which provides true transverse profile)
  - Roughness
- Some measure of splash and spray characteristics
  - WSDOT is currently in the process of determining if a procedure exists for measuring splash and spray.
  - At a minimum, splash and spray may be documented through photographs during a rainstorm
- Sound intensity noise measurements

Reporting

An “End of Construction” report will be written following completion of the test sections. This report will include construction details of the test sections and control section, construction test results, and other details concerning the overall process. Annual summary reports will also be issued over the next 5 years that document any changes in the performance of the test sections. At this time a final report will be written which summarizes performance characteristics and future recommendations for use of this process.

Cost Estimate

Construction Costs

Description	Quantity	Unit Cost	Unit	Total Cost
OGFC – AR	300	$62.00	Ton	$86,800
OGFC – SBS	300	$55.00	Ton	$77,000
Total				**$163,800**

Testing Costs

The pavement condition survey will be conducted as part of the statewide annual survey (all lanes will be tested).

WSDOT is in the process of purchasing sound intensity measurement equipment and will be installed on the appropriate testing vehicle.

Report Writing Costs

Initial Report – 60 hours = $4,800
Annual Report – 20 hours (4 hours each) = $1,600
Final Report – 100 hours = $8,000

TOTAL COST = $178,200

Schedule

Project Ad Date – January 2006
Estimated Construction – August 2006

Date	Pavement Condition Survey	Sound Intensity Measurement	End of Construction Report	Annual Report	Final Report
July 2006	X	X			
January 2007			X		
July 2007	X	X			
October 2007				X	
July 2008	X	X			
October 2008				X	
July 2009	X	X			
October 2009				X	
July 2010	X	X			
October 2010				X	
July 2011	X	X			
October 2011				X	
June 2012					X