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DEVELOPMENT OF AN ECONOMICAL, THIN, QUIET, LONG-LASTING, AND HIGH FRICTION SURFACE LAYER

VOLUME 1: MIX DESIGN AND LAB PERFORMANCE TESTING

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A Report of the findings of
ICT-R27-42
Development of a Thin, Quiet, Long-Lasting, High Friction Surface Layer for Economical Use in Illinois
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16. Abstract

This project developed and evaluated four new asphalt concrete (AC) mixtures that use locally available aggregates whenever possible with the ultimate goal of a cost-effective mixture that also improves pavement performance. Although numerous tactics have previously been introduced to improve the performance of asphalt pavement, these improvements often add expenses because they use unnecessarily large amounts of high quality aggregates and highly modified binder. The Illinois Department of Transportation initiated a program to develop sustainable asphalt pavements that use locally available aggregates as much as possible to reduce the material cost while also improving performance. These new mixtures were developed using the Bailey method to provide a promising aggregate structure that makes it possible to ensure compactability at thinner layers. The newly developed mixes use locally available natural aggregates such as dolomite, and including smaller amounts of imported materials such as quartzite, steel slag, and fibers to improve their performance in terms of durability, rut resistance, moisture susceptibility, fracture, and complex modulus. To evaluate the performance of each new mixture, five laboratory tests were conducted at the Advanced Transportation and Research Engineering Laboratory (ATREL), and the results suggest a preferred mixture.

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EXECUTIVE SUMMARY

As oil prices continue their instability in the United States, the Illinois Department of Transportation (IDOT) had an immediate need to develop innovative hot mix asphalt (HMA) and wearing surface alternatives to cost-effectively pave roads. Since the 1980s, significant improvements have been made to asphalt pavements regarding mix design, material selection, and construction technology, such as stone matrix asphalt (SMA) and ultrathin overlay with special construction equipment, which promise better performance for asphalt pavements. However, to achieve this improved performance, most of these mixtures require a highly modified asphalt binder, expensive or imported aggregates, or special equipment for construction.

This project developed four potentially cost-effective wearing surface mixtures and efficient cross-sections of wearing surfaces specifically through use of special additives and innovative surfacing technologies that incorporate locally available aggregates, whenever possible. These new mixtures include a quartzite mix, sprinkle mix, slag/fiber mix, and 4.75-mm SMA. Two conventional HMAs, a 9.5-mm coarse dense-graded mix and a 12.5-mm SMA, were selected for the controls. The Bailey design method was used to ensure proper aggregate structure of fine-dense gradation, thereby allowing a reduction in layer thickness. The ultimate goal is to improve pavement performance through optimized materials while controlling cost by using local materials and applying them efficiently. The study also considers the use of alternative aggregates such as steel slag to increase the quality of the asphalt mixture and therefore improve pavement performance. The new HMAs were developed to improve the functional condition of the asphalt pavement. Specifically, the desired improvements would ideally include one or more of the following characteristics: durability, high friction, low noise, and improved resistance to rutting and fracture. To evaluate the performance of each new HMA, five laboratory tests were conducted at the Illinois Center for Transportation (ICT). The test results and analysis are discussed in this report. The lab-mixed and lab-compacted (LMLC) specimens were prepared for the new HMA and the plant-mixed and lab-compacted specimens (PMLC) were also used.

To evaluate field performance of the considered HMA under actual traffic loading and environmental conditions, field construction and in situ testing over time were conducted in this project. Volume 1 presents the mix designs and laboratory tests, and Volume 2 will include field construction, field testing, and engineering cost analysis. The final decision about the candidate mixtures, based on cost effectiveness and efficiency, will be discussed in Volume 2.

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CHAPTER 1 INTRODUCTION

The cost of hot mix asphalt (HMA) has increased due to the high cost of asphalt binder in the United States; therefore, innovative HMA and pavement designs are urgently needed to help control construction costs. Over several decades, significant improvements have been made to HMA , including material selection, mixture design, and construction technology, with the ultimate goal of developing a perpetual pavement. Stone matric asphalt (SMA) and an ultrathin asphalt overlay are two such possibilities (Estakhri and Button 1994). However, most of the HMAs suggested thus far require higher aggregate quality, a highly modified binder, and additional construction cost for the equipment.

The Illinois Department of Transportation (IDOT) initiated development of cost-effective asphalt materials and alternative cross-sections of the wearing surface that use locally available aggregates along with special materials to improve the performance of asphalt mixtures. IDOT has also considered the Bailey method as an innovative technology that provides better aggregate structure for flexible pavements, making it possible to reduce layer thickness by using fine-dense gradation, which is defined by the Bailey method (Vavrik et al. 2002).

Project ICT R27-42, titled “Development of a Thin, Quiet, Long-Lasting, High Friction Surface Layer for Economical Use in Illinois,” was funded by IDOT in order to develop a cost-effective asphalt mixture for a new generation of wearing surface/overlay cross-sections used in Illinois. These asphalt surface mixtures are being designed for better performance and qualities such as good durability, high friction, and low noise. The new mixtures are expected to reduce material costs and be able to be placed as a thinner layer (Figure 1).

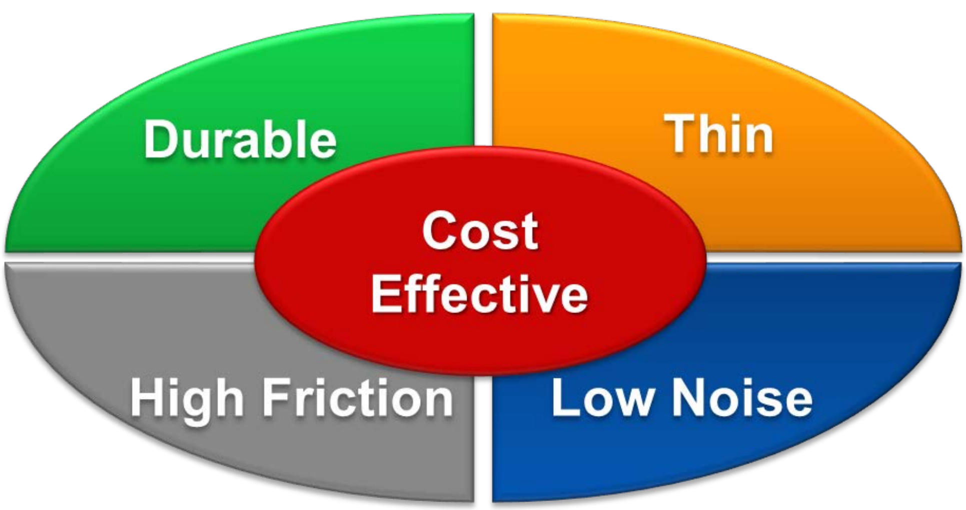


Figure 1. Five principles of new mixtures.

1.1 BACKGROUND

Due to the instability of the cost of asphalt binder in the United States, as shown in Figure 2, the development of a new HMA and a cross-sectional design for wearing courses has been emphasized to help ensure performance and cost effectiveness. The Illinois Department of Transportation has identified a need for research on asphalt mixtures for wearing courses for the state highway network, with a focus on friction, durability, noise, and cost. The cost effectiveness of new asphalt mixtures has been greatly emphasized during this project, and alternative materials to reduce material costs were widely investigated and selected to ensure pavement performance along with cost effectiveness.

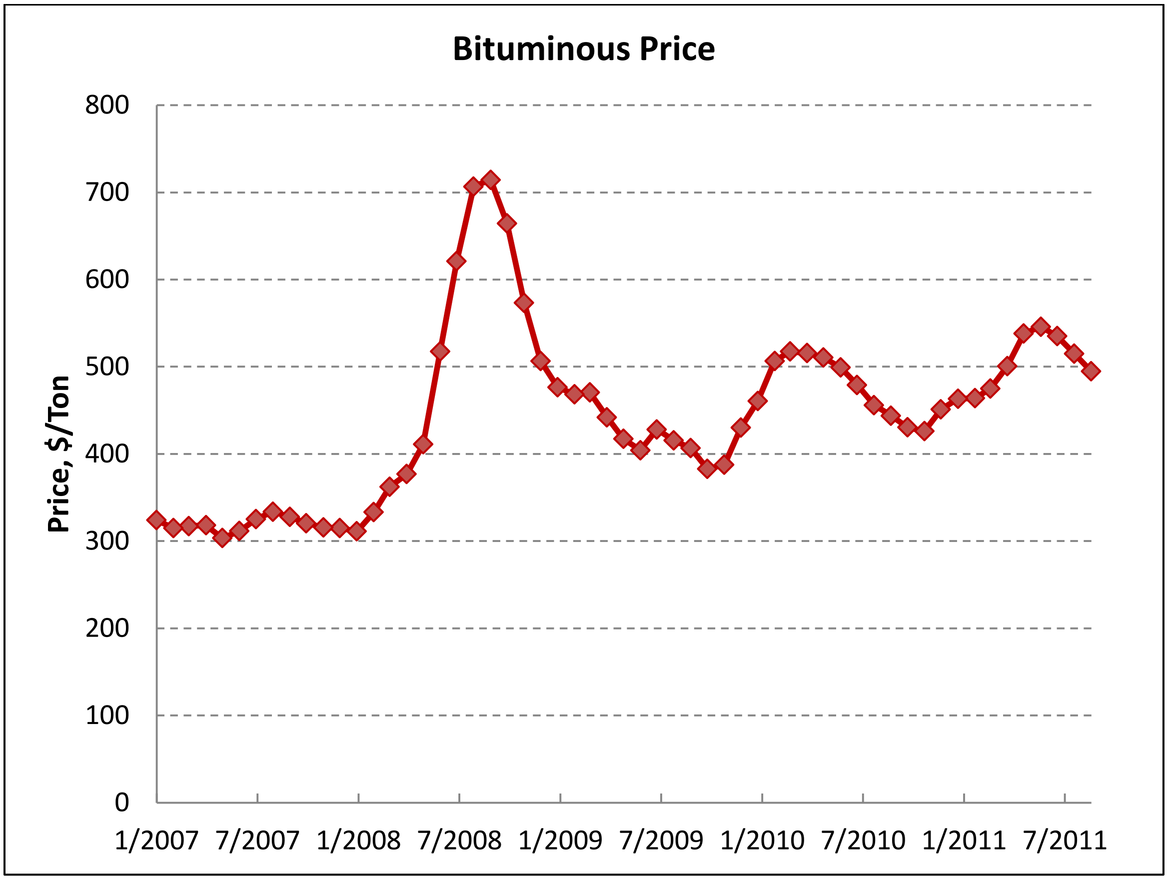


Figure 2. Bituminous price over the past four years (www.dot.state.il.us/desenv/asphaltpi.html).

1.2 OBJECTIVE

The primary purpose of the project is to develop new, cost-effective, and locally available HMAs for wearing courses and to evaluate their performance in the laboratory. The newly developed mixtures are being designed to improve one or more of the following characteristics of asphalt pavements: durability, moisture susceptibility, and resistance to rutting and fracture. This report details the development and testing of these mixtures and suggests a preferred mixture for use in Illinois. To provide best practice guidelines for selecting new asphalt mixes for wearing surface courses, several principal laboratory tests were conducted on newly developed mixtures.

CHAPTER 2 CURRENT STATE OF KNOWLEDGE

A comprehensive literature review was conducted to examine research on various HMAs and their feasibility for use in Illinois. The factors affecting durability, friction, and noise of HMA were also investigated, and various testing procedures were performed to evaluate the newly developed mixtures. Potential HMAs were selected based on a literature review. The list of potential mixtures was reviewed with IDOT for potential suitability in Illinois, and determination of the four final candidate HMAs was made by a technical review panel. Table 1 provides a summary of the potential HMAs and their general properties. Table 2 provides information on the general advantages and disadvantages of each potential mixture, as obtained from the literature. The details of the final candidate mixtures are described in the next section. The literature review and scope of study are included in Appendix A.

Table 1. Mixture Properties of Candidate Surface Mixtures

Mixture Type	Gradation	Binder	NMAS (mm)	Thickness (mm)	Air Voids (%)	Binder Content (%)	Durability	Friction	Life Cycle (years)
Current Surface Mixtures in Illinois (Noted Properties Are General and Not Specific to Illinois Mixtures)									
Stone Matrix Asphalt (SMA)	Gap-graded (nearly 75% with coarse agg. high quality and crushed agg.)	Modified binder stabilized with cellulose fiber	9.5–19	30–100	6 (4–8)	More than 5	Good	Good (good in wet conditions as well)	10–12
Dense-Graded HMA	Dense-graded	Asphalt binder or modified binder	9.5–19 (generally 9.5)	19–75	4 (5–9)	3–7	Good	Good	8–11
Better-Performing Mixtures (Higher Friction, Higher Durability, or Higher Resistance to Cracking and Rutting)									
HMA w/ Trap Rock (Diabase)	Dense-graded (possibly SMA and fine-graded)	Asphalt binder or modified binder	9.5–12.5	Variable	Depends on trap rock content	Depends on trap rock content	Very good	Superior friction	
Sprinkle Treatment	IDOT: chips (66% between ¾-in. and ½-in. sieves, 32.5% between ½-in. and 4.75-mm sieves)	Same as basic mixes (IDOT: coated with 1.3% of asphalt)	Coarser, one-sized agg. yields the best sprinkle applications (high quality agg.: trap rock, steel slag, air-cooled furnace slag)	Depends on basic mixes (IDOT: 1.5-in. surface, binder course)	Depends on basic mixes	Depends on basic mixes	Anti-stripping agent needed	Superior friction	
Cost-Efficient Mixtures									
HMA w/ Reclaimed Asphalt Pavement (RAP)	Dense-graded (10–35% RAP by IL spec., gap- or single-graded RAP is not allowed)	Asphalt binder (one grade down for mixes with more than 15% RAP)	9.5–12.5	Variable	4 (5–9)	3–7	Lower RAP % results in better performance	Same as dense-graded	
HMA w/ Steel Slag	Dense-graded (possibly SMA and fine-graded)	Asphalt binder or modified binder	9.5–12.5	19–37.5	4	4.5–6 (depends on steel slag content)	Very good	Superior friction	

Table 1. Mixture Properties of Candidate Surface Mixtures (Cont.)

Mixture Type	Gradation	Binder	NMAS (mm)	Thickness (mm)	Air Voids (%)	Binder Content (%)	Durability	Friction	Life Cycle (years)
HMA w/ Air-Cooled Blast Furnace Slag	Dense-graded (possibly SMA and fine-graded)	Asphalt binder or modified binder	9.5–12.5	Variable	Depends on slag content	Requires more binder due to high surface absorption	Very good	Very good	
HMA w/ Recycled Concrete Material (RCM)	Dense-graded				4–5 (depends on RCM content; Wong 2006)	5–7 (depends on RCM content; Wong 2006)	Not good for surface course (poor bond with asphalt)		
Rubberized Gap-Graded HMA (Wet Process)	Gap-graded (possibly SMA)	Rubberized binder (wet process) 15% rubber at Caltrans	12.5	30–60 at Caltrans (half of dense-graded HMA and 2~3 times MAS)	3–6 (4 at Caltrans)	7–9 at Caltrans	Good	Good	Longer than conven-tional mixes
Innovative Mixtures									
Fiber-Reinforced HMA (w/ Steel Slag)	Dense-graded (possibly SMA and fine-graded)	Asphalt binder or modified binder	Variable	Reduced up to 35% of dense-graded HMA	Variable (7 at Arizona State Univ.)	Variable	Very good	(Very high friction due to steel slag)	Longer than conven-tional mixes
Fiber-Reinforced WMA (w/ Steel Slag)	Dense-graded (possibly SMA and fine-graded)	Asphalt binder with WMA additive	Variable (9.5 at NCAT test)	Reduced up to 35% of dense-graded HMA	Variable (4.7 at Florida WMA)	Variable (5.6 at Florida WMA)	Very good	(Very high friction due to steel slag)	Longer than conven-tional mixes
Fine Dense-Graded HMA	Fine dense-graded	Asphalt binder or modified binder	High quality fine aggregate	Same as dense-graded HMA	Same as dense-graded HMA	More than conventional coarse-graded mixes due to fine aggregate	Good (needs more asphalt binder)	Good (requires good aggregate)	

Table 2. Advantages and Disadvantages of Candidate Surface Mixtures

Mixture Type	Advantages	Disadvantages
Current Surface Mixtures in Illinois		
Stone Matrix Asphalt (SMA)	• Excellent rut resistance and crack resistance due to stone-to-stone skeleton • Good wet weather friction due to coarser surface texture • Lower tire noise • Good durability • Used at intersections and other high traffic stress situations	• Increased material cost associated with higher binder and filler contents and fiber additive • Requires higher quality aggregates • Requires a significant compactive effort • Initial friction may be low until the thick binder film is worn off the surface by traffic
Dense-Graded HMA (F-mix)	• Lower initial cost • Most contractors and HMA producers are generally familiar with the production and placement of dense-graded mixtures	• Cannot accommodate high asphalt contents without becoming unstable and susceptible to rutting • Relatively low amounts of asphalt are typically used in dense-graded mixtures, which in turn makes them more susceptible to cracking and more permeable
Better-Performing Mixtures (higher friction, higher durability, or higher resistance to cracking and rutting)		
HMA w/ Trap Rock (Diabase)	• Very hard aggregate	• Relatively expensive (must be imported from other states)
Sprinkle Treatment	• Very high friction • Can be cost effective if the basic mixture allows use of a lower quality and less expensive aggregate	• Requires very high quality aggregates • Cost of sprinkle treatment is 16% over conventional treatments
Cost-Efficient Mixtures		
HMA w/ Reclaimed Asphalt Pavement (RAP)	• Cost efficient • Environmentally responsible • Performs well	• Need to control RAP variability to meet production tolerances • Difficult to identify the optimum RAP content
HMA w/ Steel Slag	• Superior friction due to its angularity • Generally low cost because it is a by-product of the steel-making process, however; this may not be true if the material is not locally available • Mixed with limestone (cost effective) • High shear resistance and rutting resistance • Available to pave during colder weather because steel slag retains heat longer than conventional aggregates	• Volume expansion due to the hydration of free lime or magnesia in the slag
HMA w/ Air-Cooled Blast Furnace Slag	• High resistance to polishing and weathering • Low cost because it is a by-product of the steel-making process • Can be more stable than steel slag	• Considerable variability in the physical properties depending on the iron production process • Lower thermal conductivities than conventional aggregates because of a more porous structure
HMA w/ Recycled Concrete Material (RCM)	• Cost efficient due to using recycled material	• Poor bond with asphalt binder • Potential stripping or wearing problems
Rubberized Gap-Graded HMA (Wet Process)	• Increased resistance to reflection cracking and rutting • Good surface friction • Can be used as a structural layer • Half the thickness of dense-graded HMA • Reduces maintenance costs • Decreases HMA stiffness at low temperatures which resists thermal cracking • Increases pavement life • Decreases noise levels (5 dB(A)) • Beneficially uses 500–2,000 scrap tires per lane mile	• Difficult to control rubber quality • High cost for wet process ($16/ton more than conventional mixes) • Needs more compaction than dense-graded HMA

Table 2. Advantages and Disadvantages of Candidate Surface Mixtures (Cont.)

Mixture Type	Advantages	Disadvantages
Innovative Mixtures		
Fiber-Reinforced HMA (w/ Steel Slag)	• High friction due to steel slag • High resistance to rutting and cracking due to both steel slag and fibers • Reduces thickness up to 35% compared to conventional mixes • Cost effective due to good performance with less asphalt thickness • Extended life • Improves tensile strength, resilient modulus, and stability	• Requires special fibers with high melting point • Fiber cost is $6/lb, and the mixture needs 1 lb of fiber per ton of HMA
Fiber-Reinforced WMA (w/ Steel Slag)	• There are many choices of fibers with a low melting temperature • High friction due to steel slag • High resistance to rutting and cracking due to both steel slag and fibers • Reduces thickness up to 35% compared to conventional mixes • Cost effective due to good performance with less asphalt thickness • Extended life • Reduced emissions, fuel/energy usage (25~30%) • Allows paving in colder temperatures • Able to incorporate higher percentages of RAP • Able to open to traffic in a short time	• Increases initial cost due to the WMA additive; however, the fuel cost is reduced • May reduce tensile strength due to more water remaining in WMA than in HMA (increased moisture susceptibility)
Fine Dense-Graded HMA	• Low initial cost—a higher natural/local sand content in this mixture results in a less expensive mix • Easy to construct—the fine texture and high sand content in this mixture makes it easy to place and easy to compact with a smooth finish; handwork is easy and blends in well without leaving surface blemishes • Smooth surface texture with small aggregate—less distortion of the tires around the aggregate particles reduces tire vibrations, resulting in lower noise	• Less rut resistant than other mixes—the high natural sand content creates a weak aggregate skeleton • Has a lower hydroplaning threshold because the surface texture is very fine; the macrotexture of this mix does not provide an escape route for the water • Requires good aggregates for rut resistance and frictional properties • Requires more binder

CHAPTER 3 TESTING MATERIAL CHARACTERIZATION

3.1 AGGREGATE

To develop the proposed HMAs, the research team used locally available aggregates as much as possible to ensure significant cost savings. Research was performed to determine the availability of aggregates in Illinois. Limestone, dolomite, gravel, and steel slag are the most common aggregates, as shown in Table 3. Although reclaimed asphalt pavements (RAP) are good alternative materials for HMA in terms of sustainability, they were not used in this study due to the difficulties inherent in the quality control of RAP materials and its optimal amount in HMA is being investigated in another study in Illinois (Aurangzeb et al. 2011).

Table 3. Aggregates Produced in Illinois

District	Limestone	Dolomite	Gravel	Sandstone	Steel Slag	Air-Cooled Blast Furnace Slag
1		X	X		X	
2		X	X		X	
3	X	X	X		X	
4	X	X	X		X	
5	X		X		X	
6	X		X			
7	X		X	X		
8	X		X			X
9	X		X	X		
Produced in/imported from	South of I-80	North of I-80, Wisconsin	Statewide (Peoria, Indiana)	District 9, Ohio River	Northwest Indiana, Peoria, St. Louis	Northwest Indiana, St. Louis

Limestone is not recommended as an aggregate for an HMA wearing course because of its tendency to polish resulting in lower friction properties. Relatively economical dolomite and natural sand were considered because they are commonly used aggregates in Illinois. Use of natural sand was limited because its round shape can result in poor rut resistance. Therefore, durable steel slag, quartzite, and polypropylene and aramid fibers were used to improve the properties of the candidate mixtures. Table 4 presents the aggregate type and additive blending percentage for each candidate mixture in this project.

Table 4. Aggregate Percentages and Additives for Each Mixture

Mixture Type		Aggregate Blending Percentage (%)					Additives
		Dolomite	Natural Sand	Quartzite	Steel Slag	RAP	
Control	F-Mix	46.5	7.8	—	35.7	10.0	
	SMA	16.0	—	—	84.0	—	Cellulous fibers
New	Quartzite Mix	64.3	17.9	17.8	—	—	
	Sprinkle Mix	80.2	19.8	—	—	—	Quartzite chips
	Slag/Fiber Mix	62.2	17.5	—	20.3	—	Polypropylene and aramid fibers
	4.75 SMA	60.3	—	39.7	—	—	Cellulous fibers

Aggregates were sampled from the same source as the aggregates used in the control HMAs to minimize the effect of the aggregate source. The sampled aggregates were stored in a tent, shown in Figure 3, to prevent them from being contaminated or change its gradation, such as from the loss of aggregate dust by rain. For the mix designs, each aggregate was fractionated after complete drying in an oven, and the sieved materials were stored in a sealed bucket to help ensure quality control. Table 5 shows the gradation and properties of each aggregate used in this study.

Figure 3. Aggregate storage at ATREL.

Table 5. Aggregates Used in the Mix Designs

	Material Code	**032CM16**	**038FM20**	**037FM02**	**004MF01**	**032CM13**	**032CM13**	**FM22**
Aggregate Source	**Material Type**	**Dolomite**	**Crushed Dolomite Sand**	**Natural Sand**	**Mineral Filler**	**Quartzite**	**Quartzite (9.5 mm & 4.75 mm Removed)**	**Crushed Dolomite (4.75 mm Removed)**
	Source Number	50312-78	50312-78	50970-02	50312-04	52402-25	52402-25	—
	Source Name	Vulcan	Vulcan	Thelan	Hanson	Michels	Michels	Bluff City
	Source Location	McCook	McCook	Antioch	Thornton	Wisconsin	Wisconsin	Bartlett
Gradation	1" (25.0 mm)	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	3/4" (19.0 mm)	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	1/2" (12.5 mm)	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	3/8" (9.5 mm)	97.0	100.0	100.0	100.0	79.2	100.0	100.0
	No. 4 (4.75 mm)	32.0	97.0	100.0	100.0	20.8	100.0	100.0
	No. 8 (2.36 mm)	9.0	68.0	94.5	100.0	4.2	17.7	13.4
	No. 16 (1.18 mm)	7.0	40.0	72.0	100.0	3.2	13.1	5.6
	No. 30 (600 μm)	6.0	24.0	49.0	100.0	2.8	11.4	4.6
	No. 50 (300 μm)	6.0	15.0	19.9	100.0	2.4	10.0	4.2
	No. 100 (150 μm)	5.0	9.0	4.1	95.0	2.1	8.5	3.9
	No. 200 (75 μm)	4.6	6.7	1.5	90.0	1.7	6.9	3.5
Gravity	Bulk Specific Gravity	2.644	2.691	2.619	2.900	2.675	2.637	2.681
	Apparent Specific Gravity	2.792	2.796	2.719	2.900	2.711	2.686	2.822
	Absorption (%)	2.0	1.4	1.4	1.0	0.5	0.7	1.9

The stockpile gradations for the CM13 quartzite and FM22 used in these mix designs were not available in typical stockpiles. Therefore, additional screening of these aggregates was performed to achieve the required gradations for the 4.75-mm SMA. The quartzite aggregate retained on the 4.75-mm (No. 4) sieve was used as sprinkle chips in the sprinkle mix.

3.2 ASPHALT BINDER

PG 70-22 SBS modified asphalt binder was used in all of the new mixtures and the control F-mix, except for the control SMA, for which PG 76-22 was used. To eliminate the effect of various sources, the PG 70-22 asphalt binder was sampled from the same source as the binder used in the control mixes. Because the control mixes were not blended in the laboratory, the PG 76-22 binder was not sampled for laboratory work. Table 6 presents the various properties of PG 70-22.

Table 6. Properties of Asphalt Binder (PG 70-22)

Producer Company Name	**Seneca Petroleum Company (Lemont, IL)**
Product No.	1757-05
Material Code	10129
Name	SBS PG 70-22
Specific Gravity @ 15.6°C	1.031
Flash Point, Cleveland Open Cup, °C	316
Rotational Viscosity @ 135°C, Pa·s	0.898
Change in Mass, RTFO, %	0.198
PAV Aging Temperature, °C	100
Bending Beam Rheometer, Temp, °C	–12
Bending Beam Rheometer, m-value	0.320
Bending Beam Rheometer, Creep Stiffness, MPa	183
DSR Original Binder, $G^*/\sin(\delta)$ @ 70°C, KPa	1.310
DSR RTFO, $G^*/\sin(\delta)$ @ 70°C, KPa	3.120
DSR PAV Residue, $G^*\sin(\delta)$ @ 28°C, KPa	2350
PEN @ 25°C, 100g, 5 Sec., 0.1 mm	61
Polymer Tests, Force Ratio	0.5
Polymer Tests, Elastic Recovery, %	75
Polymer Tests, Separation of Polymer, °C	0

To prevent stripping of HMA, a liquid anti-strip additive was mixed into the binder in the laboratory. The details are described in Section 3.3.2. The mixing and compaction temperatures were determined by the type of asphalt binder and temperature–viscosity curves. The mixing temperature was 325°F for PG 70-22 (SBS), and the compacting temperature was 305°F for PG 70-22 (SBS) and 330°F for PG 76-22 (SBS), respectively.

3.3 MIXTURE ADDITIVES

To improve the performances of HMA and ensure their stabilization when using the SMA mixtures, several additives were incorporated into the new mixtures. A special type of fiber (polypropylene and aramid blend) was used in the slag/fiber mix to provide better resistance to tension. Also, a cellulose fiber was used for the 4.75-mm SMA mixture to prevent the drainage of asphalt binder; the same type of cellulose fiber used in the control SMA was selected. For better bonding between aggregates and the asphalt binder, liquid anti-strip additives were used in all of the mixtures in this study.

3.3.1 Fiber

3.3.1.1 Polypropylene and Aramid Fiber

Several previous studies reported that fibers used in asphalt pavement improved resistance to shear deformation, permanent deformation, and fatigue cracking and also provided higher fracture energy (Bueno et al. 2003; Collins et al. 1994; Lee et al. 2005). A blend of polypropylene and aramid fibers was used to improve performance of the slag/fiber mix. The two fibers in the blend have different mixing and compacting temperatures as well as different melting points. The polyolefin fiber, with a melting point around 212°F, acts like glue between the HMA and aramid fiber, which has a melting point of 800°F, as shown in Figure 4(a). The aramid fiber disperses uniformly in the mixture and holds the mixture together to provide better tensile strength when loading is applied to the mixture. The physical characteristics of the fibers are shown in Table 7.

Table 7. Physical Characteristics of the Fibers (Kaloush et al. 2010)

	Polypropylene	**Aramid**
Property	**Twisted Fibrillated fiber**	**Multifilament Fiber**
Specific gravity	0.91	1.45
Tensile strength, MPa	483	3000
Length, mm	19	19
Color	Tan	Yellow
Acid/alkali resistance	Inert	Good
Decomposition temperature, °F	315	> 842

(a) (b)

Figure 4. (a) Polypropylene and aramid fibers at different temperatures (light yellow is aramid, and black, gray and dark yellow are polypropylene) and (b) cellulose fibers.

3.3.1.2 Cellulose Fiber

Cellulose fibers, such as shredded newspapers and magazines, as shown in Figure 4(b), are commonly used in SMA to prevent binder draindown of the mix. The optimum quantity of fibers is determined by a draindown test, in accordance with AASHTO T305. In this project, cellulose fibers were added to the 4.75-mm SMA mixtures, and 0.4% by weight was determined as the optimum fiber content.

3.3.2 Liquid Anti-Strip Additive

A moisture susceptibility test was performed as a part of the mix design. A liquid anti-strip additive was used in the new mixtures to control potential rutting, raveling, and cracking, which can be caused by stripping. Pavegrip 550, which was used in the control mixes, was selected for use in the new HMAs and added directly to the asphalt binder before the mixing process. The asphalt binder was heated in a 1-qt can, and then the liquid anti-strip (0.5% additive by weight of binder) was added to the asphalt binder and immediately mixed with a mechanical stirrer approximately 25 mm from the bottom of the container for 2 min, as shown in Figure 5. If the treated asphalt binder was not used the same day it was prepared, it was discarded.

(a) (b)

Figure 5. (a) Liquid anti-strip additive and (b) mechanical stirring.

CHAPTER 4 WEARING SURFACE MIX DESIGNS

4.1 SELECTION OF ASPHALT CONCRETE MIXTURE CANDIDATES

Various potential HMAs that could provide good friction, durability, and cost efficiency, as shown in Table 8, were identified. The final group of mixes for study was determined by combining some of the potential mixes and replacing the aggregate type with alternative aggregates available in Illinois or imported from a neighboring state. Two conventional mixtures commonly used in Illinois were selected as control mixes—12.5-mm SMA and 9.5-mm coarse dense-graded HMAs.

Table 8. Potential Asphalt Concrete Mixes Identified for This Study

Category	Potential Mixes
Control Mixes	• Stone Matrix Asphalt (SMA) • Dense-Graded HMA (F-mix)
Better-Performing Mixes	• HMA w/ Trap Rock (Diabase) • Sprinkle Treatment
Cost-Efficient Mixes	• HMA w/ Reclaimed Asphalt Pavement (RAP) • HMA w/ Steel Slag • HMA w/ Air-Cooled Blast Furnace Slag • HMA w/ Recycled Concrete Material (RCM) • Rubberized Gap-Graded HMA
Innovative Mixes	• Fiber-Reinforced HMA (w/ Steel Slag) • Fiber-Reinforced WMA (w/ Steel Slag) • Fine Dense-Graded HMA

4.2 NEWLY DEVELOPED ASPHLAT CONCRETE MIXTURES

Three fine dense-graded HMAs with 9.5-mm nominal maximum aggregate size (NMAS) and one SMA with 4.75-mm NMAS were developed using the Bailey method to allow a thinner wearing course. A fine dense-graded mixture is commonly defined as one with a gradation plotted above the maximum density line on a 0.45 power chart. However, the Bailey method defines a fine dense-graded mixture as a dense-graded mixture which has a volume of fine aggregate that exceeds the volume of voids in the coarse aggregate structure. Therefore, the fine fraction carries most of the load in this mixture because the coarse fraction is spread apart and floating in the fine fraction (Vavrik et al. 2002). All of the fine dense-graded mixtures in this project were designed according to the Bailey method's definition. A fine dense-graded mixture is generally easy to place and compact with a smooth finish, and it may reduce tire vibration and noise. Although a fine-graded mixture is expected to have less rut resistance than a conventional HMA, Kandhal and Cooley (2002) reported no significant difference in terms of resistance to rutting between coarse dense-graded and fine dense-graded HMAs. Because of the aggregate structure of fine dense-graded HMAs, fine-graded mixes are typically easier to compact than coarse-graded mixes when they are placed at an equal lift thickness, especially if that lift thickness is near the minimum allowable lift thickness for the corresponding coarse-graded mixes.

A 4.75-mm SMA is one of the proposed HMAs that can be used for ultrathin asphalt overlay. The minimum layer thickness required for SMA mixtures is four times that of the

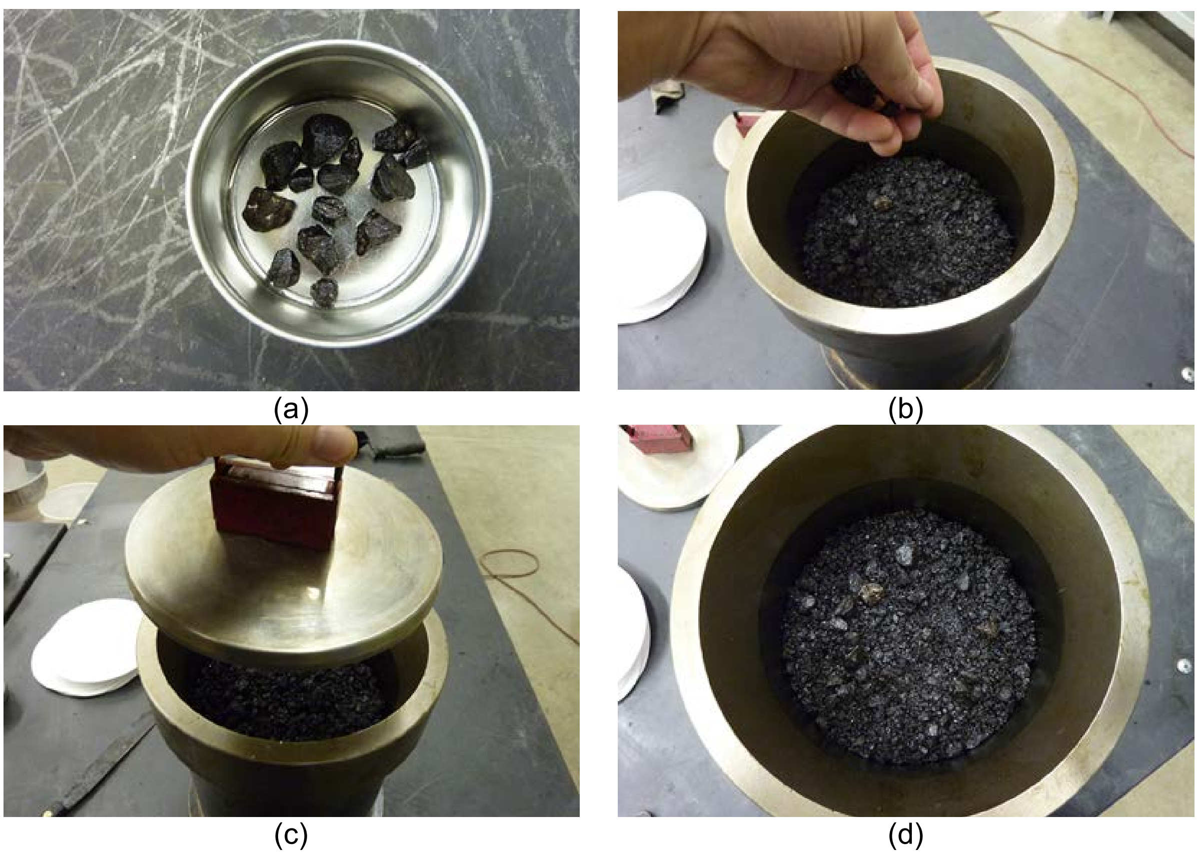
NMAS. Therefore, a 4.75 SMA can be placed in a layer as thin as 0.75 in. A PG 70-22 SBS modified asphalt binder was used for all of the considered mixtures, including the 4.75 SMA. To minimize the effect of voids in mineral aggregate (VMA) on the performance of the new mixtures, three fine dense-graded HMAs were designed to have similar VMAs.

4.2.1 Quartzite Mix

Quartzite is a very hard rock that is originally sandstone. It is more expensive than other aggregates commonly used in HMAs. However, Selim (1986) reported that the total cost, including initial cost and maintenance during a reasonable lifespan of a pavement, was less for a quartzite surface course than for a natural aggregate surface course. Therefore, this aggregate is an option for a thin and durable wearing course. In this study, the proposed new mixture includes 17.8% of CM13 quartzite, with blends of dolomite and natural sand.

4.2.2 Sprinkle Mix

Sprinkle treatment is a surface application of pre-coated, coarse aggregate chips on top of a regular HMA to improve the friction features of asphalt pavement. Costly aggregate chips with high friction are required; however, the HMA can use less expensive aggregates. Sprinkle treatment was successfully used in several states in the late 1970s (Brown 1977; Burke 1981). Using information gained from previous studies, the research team modified the use of sprinkle chips and the application rate for this project.



(a) (b) (c) (d)

Figure 6. Application of sprinkle chips in the laboratory.

Regular dolomite and natural sand were used in the HMA, and coarse quartzite aggregate, which was retained on the 4.75-mm (No.4) sieve, was used for sprinkle chips and coated with 0.75% PG 64-22 binder. The sprinkle chips were applied at a rate of 1.56 lb/yd^2. Only durability and wheel tracking tests were conducted on the specimens with the sprinkle chips on top because the top surface of specimens for the other tests was cut off to prepare them according to the specification. At the laboratory, sprinkle chips were manually spread on top of the HMA after flattening the surface prior to gyratory compaction, as shown in Figure 6.

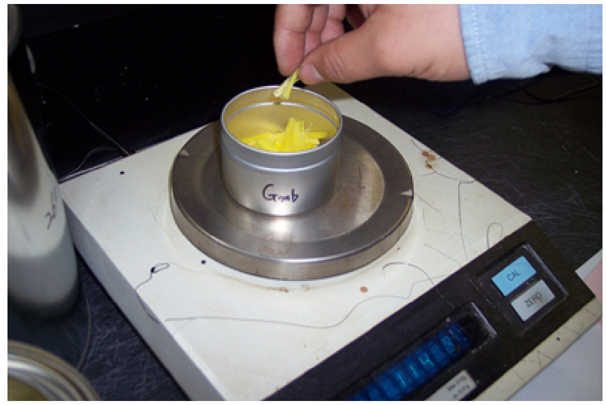
4.2.3 Slag/Fiber Mix

Steel slag is a by-product of steel production that is produced when impurities separate from the molten steel. The use of steel slag in HMA provides good friction and high resistance to stripping and permanent deformation (Collins et al. 1994). However, due to the potential expansion of free lime or magnesia in the slag, stringent quality control measures must be followed. Previous studies (Bueno et al. 2003; Lee et al. 2005) reported that using fibers in HMA pavement improved resistance to shear deformation, reduced permanent deformation and fatigue cracking, and provided higher fracture energy (Kaloush et al. 2010). For this project, a blend of polypropylene and aramid fibers was added into the slag/fiber mix to improve its performance. Before adding asphalt binder, 0.05% of fiber per ton of HMA was mixed with heated aggregates. During mixing, some fibers were melted to improve bonding between the fibers and the asphalt binder.

4.2.3.1 Adding Fibers

The fibers consisted of four different-colored fibers—black, gray, yellow, and light yellow—although only two different types of fibers were blended because of the producer's confidentiality about their blending percentages. To add the fibers representatively into the asphalt mixture in the lab, the fibers were sorted by color, as shown in Figure 7, and weighed in order to calculate blending percentage by weight. Based on the blending percentage of each colored fiber obtained in the laboratory, the fibers were re-blended for each gyratory sample to achieve 0.05% fiber by weight of HMA. These fibers were added into heated aggregates and blended with aggregates before an asphalt binder was added at the mixing temperature.

(a)



(b)

(c)

(d)

Figure 7. Adding fibers to asphalt mixture in the laboratory.

4.2.4 4.75 Stone Matrix Asphalt

Stone matrix asphalt was developed over 25 years ago in Germany and has been used successfully in the United States since 1991. A NMAS of 12.5 mm or 19.0 mm has been commonly used for SMA in the United States. Following the mix design criteria developed in a National Cooperative Highway Research Program (NCHRP) study in 1999 (Brown and Cooley 1999), a 4.75 SMA was developed for this project. Since the SMA mixtures require durable aggregates for the stone-to-stone skeleton, quartzite aggregate was used in this project. The 4.75 SMA is beneficial because it allows for a thin overlay starting at only 0.75 in, which makes it more cost effective, even though durable and expensive aggregates are required for SMA mixtures. A PG 70-22 modified binder was used for this mixture, and cellulous fiber was added to prevent drainage of asphalt binder.

4.2.4.1 Adding Cellulose Fibers

The same type of cellulose fibers and blending percentage as the control SMA (0.4% by weight) were used for the 4.75 SMA. The fibers were added into the heated aggregates and pre-blended with them before the asphalt binder was added at the mixing temperature. The asphalt content was calculated based on the combined weight of aggregates and fibers.

4.2.4.2 Draindown Test

Because SMA mixes have a high asphalt binder content, the asphalt binder tends to drain off the aggregate and down to the bottom, which is known as “mix draindown.” This is usually controlled by adding cellulose or mineral fibers. To evaluate the draindown potential of this mixture, the draindown test was conducted in accordance with AASHTO T305. The amount of draindown was measured at 0.15% at the laboratory mixing temperature and 0.17% at 15°C higher than the mixing temperature. Both drainage values passed the 0.3% maximum draindown limit.

4.3 MIX DESIGNS FOR CONSIDERED HMAs

Once the locally available aggregates were selected, the mix designs of candidate mixtures were developed using the Bailey method. The volumetric properties of each mixture were determined as presented in Table 9 and Figure 8.

Table 9. Summary of Mix Designs

Aggregate Type	Control		New			
	F-Mix	**SMA**	**Quartzite Mix**	**Sprinkle Mix**	**Slag/Fiber Mix**	**4.75 SMA**
CM11 Dolomite		27.0	—	—	—	—
CM13 Steel Slag	35.7	57.0	—	—	20.3	—
CM13 Quartzite		—	17.7	—	—	—
CM16 Dolomite	26.0	—	17.7	35.6	16.3	—
FM20 Crushed Dolomite	19.5	9.0	45.0	43.7	44.0	12.4
FM02 Natural Sand	7.8	—	17.8	19.7	17.5	—
MF01 Mineral Filler	1.0	7.0	1.8	1.0	1.9	8.2
Scalped CM13 Quartzite	—	—	—	—	-—	39.7
Scalped FM22 Dolomite	—	—	—	—	—	39.7
RAP	10.0					
Additives	Liquid anti-strip additive, RAP	Liquid anti-strip additive, fiber (cellulous)	Liquid anti-strip additive	Liquid anti-strip additive	Liquid anti-strip additive, fiber (polypropy-lene and aramid)	Liquid anti-strip additive, fiber (cellulous)
PG Grade	PG 70-22	PG 76-22	PG 70-22	PG 70-22	PG 70-22	PG 70-22
Gradation	Coarse-Dense	SMA	Fine-Dense	Fine-Dense	Fine-Dense	SMA
NMAS (mm)	9.5	12.5	9.5	9.5	9.5	4.75
N_{des}	90	80	90	90	90	80
Design Air Void (%)	4.0	3.5	4.0	4.0	4.0	4.0
AC (%)	5.1	6.0	5.8	6.0	5.7	7.3
VMA (%)	14.5	17.6	15.2	15.4	15.4	18.5
VFA (%)	72.4	80.1	73.7	73.9	74.0	78.4
G_{mm}	2.700	2.949	2.504	2.500	2.606	2.454

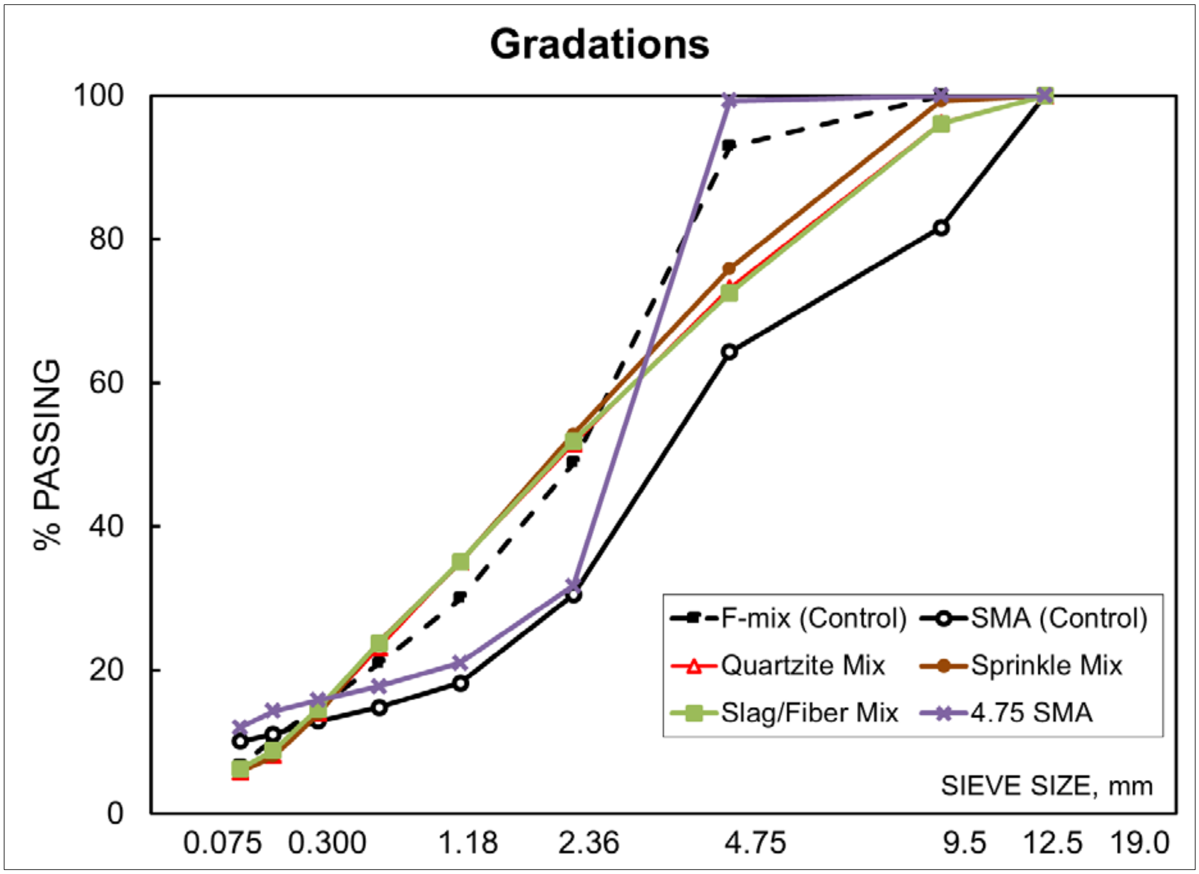


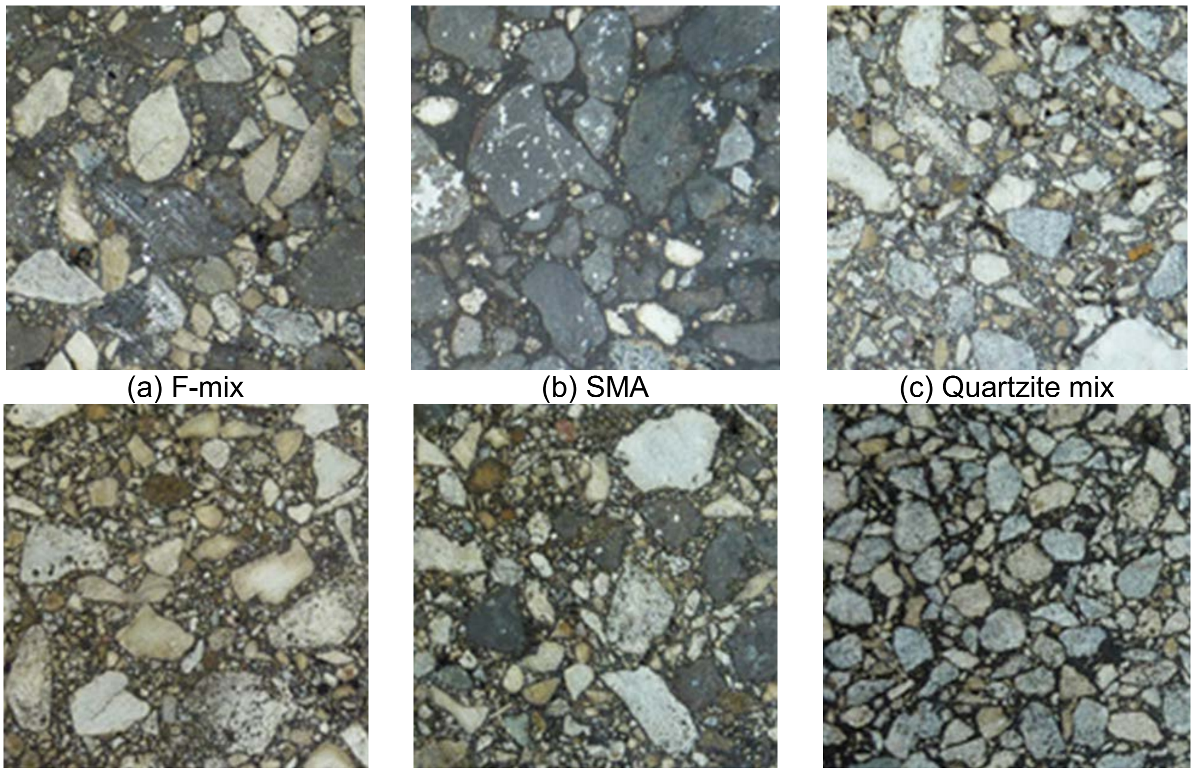
Figure 8. Gradation of each mixture.

The Bailey method is a practical tool that has been successfully utilized to develop and analyze blend gradations and to provide a better understanding of aggregate packing and its influence on volumetric and compactability of asphalt mixtures. Accordingly, all of the mix designs in this project were developed using the Bailey method. The layer thickness is determined by the gradation and NMAS. The minimum thickness should be four times higher than the NMAS for the coarse dense-graded mix and SMA and three times greater than that of the fine dense-graded mix. Loose unit weight (LUW) and rodded unit weight (RUW) tests were performed for each aggregate. The LUW represents the minimum density required to provide particle-to-particle contact of the coarse aggregate, and the RUW is used to determine any increase in the mass and volume of coarse aggregate within a unit volume as a result of compactive effort. Both tests were performed three times on separate samples from the aggregate stockpiles, in accordance with AASHTO T19. To allow placement of the newly designed mixture as a thinner layer, three new dense-graded mixtures were designed as a fine dense gradation.

Three candidate HMAs were designed to have a fine dense gradation with a 9.5-mm NMAS and similar gradations, as shown in Figure 8. The VMA were designed to be similar in order to satisfy the 15% of minimum VMA requirement so that the effect of those parameters could be minimized. Therefore, aggregate type, asphalt binder type, asphalt binder content, and additives were, for the most part, expected to affect the performance of the HMAs. To obtain proper gradations of the aggregates for the 4.75 SMA, additional screening was conducted for CM13 quartzite and FM22 dolomite aggregates; the aggregates passing through the 4.75-mm (No. 4) sieve were used for this mixture. The 4.75

SMA met the 18% VMA requirements, and it required more asphalt binder content. The aggregates and binders for all mixtures were sampled from the same stockpiles to minimize variations in materials between the new and the control mixtures. The summary of the mix designs is presented in Table 9.

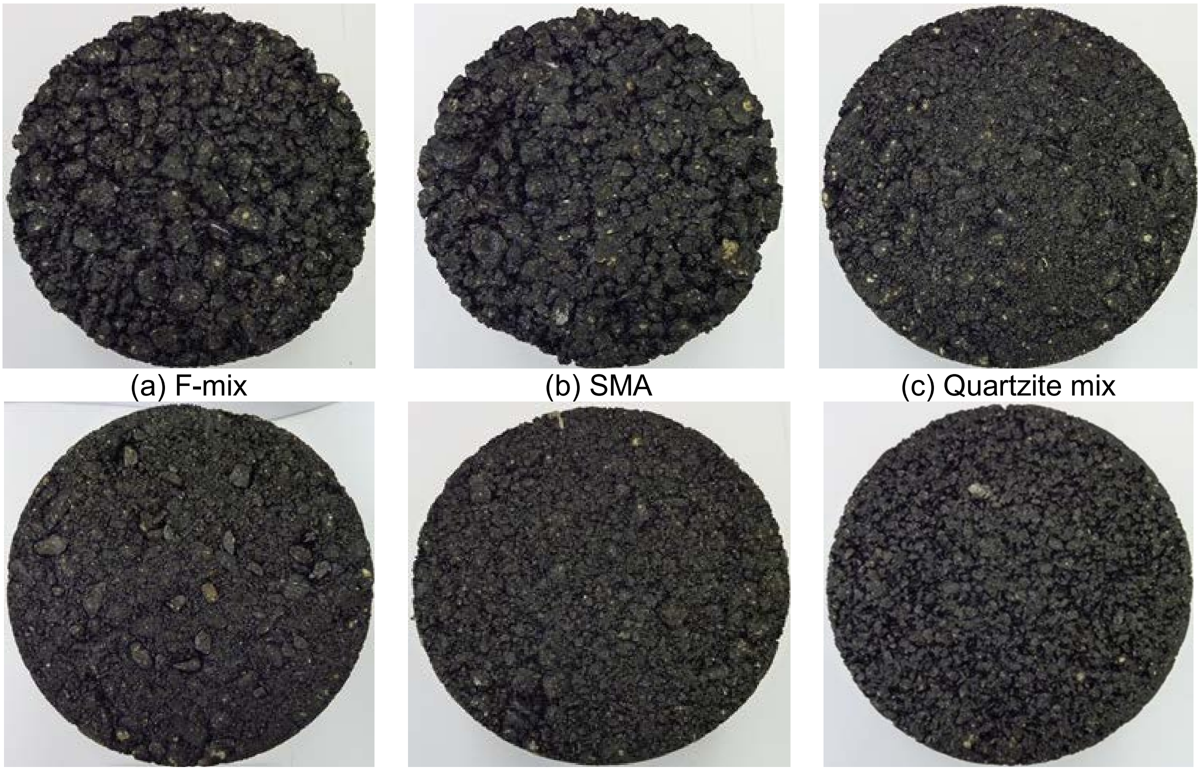
Figure 9 shows the cross-section of each mix. Dark aggregates in the F-mix, SMA, and slag/fiber are steel slag. The 4.75 SMA contains relatively uniform aggregate sizes. As shown in Figure 10, the two control mixes provided rougher surface than did the new mixtures.



(a) F-mix (b) SMA (c) Quartzite mix

(d) Sprinkle mix (e) Slag/fiber mix (f) 4.75 SMA

Figure 9. Cross-section of each mix.



(a) F-mix (b) SMA (c) Quartzite mix

(d) Sprinkle mix (e) Slag/fiber mix (f) 4.75 SMA

Figure 10. Surface texture of each mix.

CHAPTER 5 LABORATORY DURABILITY AND PERFORMANCE TEST RESULTS

5.1 INTRODUCTION

Five laboratory performance tests were conducted to evaluate the newly developed mixes and compare them to the control mixes. Table 10 presents the test methods and specifications referred to in this study. Although newly developed HMAs must be placed in thinner layers, the test specimens were prepared in accordance with the standard for each test. For example, the specimens for the wheel tracking test were fabricated at 62 mm in height for all of the mixes, even though the layer thickness of the wearing surface will be less than 62 mm. The laboratory test does not consider the effect of various thicknesses in the field. The behavior of each mixture at various thicknesses is discussed with field performance data in Volume 2.

Table 10. Test Methods and Specifications

Performance	Test Method	Specification
Durability	Cantabro Loss Test	TxDOT-245
Rutting	Wheel Tracking Test	TxDOT-242
Moisture Susceptibility	Illinois Modified Lottman Test	AASHTO T283
Simple Performance	Complex Modulus Test	AASHTO TP62
Fracture	Semi-Circular Bending (SCB) Test	AASHTO Draft

5.2 SAMPLE PREPARATION

Each developed HMA was prepared by two groups of test specimens as follows: lab mixed and lab compacted (LMLC) and plant mixed and lab compacted (PMLC).

5.2.1 Lab Mixed and Lab Compacted (LMLC)

After the selected aggregates were blended according to the mix designs as shown in Table 9, they were mixed with asphalt binder using a laboratory mixer, and then the loose mixtures were aged in an oven for 2 hr, as shown in Figure 11(a). The test specimens were prepared using a Superpave gyratory compactor (SGC) at target air void content for each test. Material quality of the lab-mixed HMA was achieved by controlling aggregate blending percentages and asphalt binder content.

5.2.2 Plant Mixed and Lab Compacted (PMLC)

During construction in the field, the loose HMA materials were sampled at the plant, as shown in Figure 11(b). They were reheated in the laboratory until they reached the compaction temperature and then compacted immediately using the SGC without additional oven aging. Each mixture was designed to be produced in accordance with the mix design; however, material quality is not as controllable as that in the laboratory. Quality control for each mixture was overseen during production by a QC manager in the plant and by IDOT engineers. Usually, mixtures were produced over 2 days; the HMA produced on the second day was adjusted from the QC data obtained on the first day. This process ensured that the loose HMA created on the second day was as close to the mix design as possible. The gradations and volumetric properties of the plant-mixed HMA are shown in Table 11.

Because the tolerance of the design air void content is ±0.5%, the quartzite mix and sprinkle mix met the design requirement, but the slag/fiber mix and the 4.75 SMA did not.


(a) (b)

Figure 11. Mixing methods: (a) lab mixing and (b) plant mixing.

Table 11. Gradations and Volumetric Properties of Plant-Mixed HMAs

Mixture		F-mix (control)		SMA (control)		Quartzite		Sprinkle		Slag/Fiber		4.75 SMA	
Day		1	2	1	2	1	2	1	2	1	2	1	2
AC, %		5.3	5.1	5.8	5.7	5.6	5.8	5.9	6.1	6.3	5.7	7.5	6.7
N_{des}		90	90	80	80	90	90	90	90	90	90	80	80
G_{mb}		2.616	2.601	2.846	2.850	2.387	2.388	2.392	2.376	2.585	2.522	2.296	2.310
G_{mm}		2.718	2.740	2.962	2.981	2.507	2.487	2.486	2.486	2.612	2.605	2.432	2.464
AV, %		3.8	5.1	3.9	4.4	4.8	4.0	3.8	4.4	1.0	3.2	5.7	6.3
VMA, %		14.8	15.0	18.6	18.3	15.6	15.8	15.5	16.2	13.1	14.6	20.8	19.6
Dust/AC		0.00	0.80	—	1.66	0.70	0.90	0.91	0.83	1.23	1.04	1.22	1.68
G_{sb}		2.910	2.910	3.290	3.290	2.670	2.670	2.662	2.662	2.787	2.787	2.681	2.681
Gradation (% Passing)	3/4"	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	1/2"	99.0	99.0	81.8	85.0	100.0	100.0	100.0	100.0	99.5	99.5	100.0	100.0
	3/8"	92.0	89.5	65.3	67.8	92.5	96.5	97.5	98.0	95.3	96.5	100.0	100.0
	#4	52.0	47.0	29.5	30.5	68.0	72.5	74.5	77.5	69.0	73.5	96.7	98.0
	#8	28.0	26.0	16.5	17.5	49.5	50.5	53.5	57.5	47.3	52.0	34.0	36.7
	#16	19.0	18.0	13.5	13.8	32.5	32.5	34.0	36.5	31.3	33.0	19.0	21.3
	#30	13.0	13.0	11.5	12.0	21.0	21.0	22.0	23.0	20.3	20.5	14.7	16.3
	#50	8.0	8.5	11.0	11.5	11.0	11.0	12.0	12.0	12.0	11.0	12.7	13.7
	#100	5.0	5.5	10.0	10.8	6.0	7.0	7.0	7.0	7.8	7.0	11.3	12.0
	#200	3.8	3.7	8.5	9.0	4.0	5.0	5.2	5.3	6.2	5.5	9.7	9.7

The maximum theoretical specific gravity (G_{mm}) for each mixture was obtained in the laboratory (U of I) from the sampled loose HMAs and is shown in Table 12. The G_{mm} was also measured in the plant by a QC manager (Plant) and IDOT (IDOT) engineers (Table 12). The G_{mm} obtained in the laboratory was used to calculate volumetric properties for test specimens because those specimens had been prepared after reheating the sampled materials, while the G_{mm} calculated by the QC manager and IDOT engineers was obtained from plant-fresh mixtures. Relatively higher variations of G_{mm} for the F-mix and the control SMA were caused by the very high content of steel slag in those mixtures, a factor that cannot be easily controlled.

Table 12. Maximum Theoretical Specific Gravity of Plant-Mixed HMA Materials

Mix Type		Mix Design	U of I		Plant		IDOT	
F-Mix (Control)	Rep. 1	2.729	2.754	2.757 (0.47%)*	2.728	2.728	2.752	2.752
	Rep. 2		2.743		—		—	
	Rep. 3		2.774		—		—	
	Rep. 4		2.755		—		—	
SMA (Control)	Rep. 1	2.987	2.944	2.972 (0.75%)*	2.977	2.966	3.014	2.997
	Rep. 2		2.979		2.954		2.979	
	Rep. 3		3.004		—		—	
	Rep. 4		2.961		—		—	
	Rep. 5		2.972		—		—	
Quartzite Mix	Rep. 1	2.504	2.496	2.499 (0.17%)*	2.493	2.493	2.480	2.480
	Rep. 2		2.504		—		—	
	Rep. 3		2.497		—		—	
Sprinkle Mix	Rep. 1	2.500	2.511	2.507 (0.14%)*	2.485	2.485	2.486	2.486
	Rep. 2		2.504		—		—	
	Rep. 3		2.506		—		—	
Slag/Fiber Mix	Rep. 1	2.606	2.617	2.624 (0.23%)*	2.601	2.601	2.608	2.608
	Rep. 2		2.628		—		—	
	Rep. 3		2.627		—		—	
4.75 SMA	Rep. 1	2.454	2.457	2.454 (0.18%)*	2.462	2.453	2.488	2.494
	Rep. 2		2.455		2.443		2.499	
	Rep. 3		2.449		—		—	

* indicates a coefficient of variation.

5.3 AIR VOID CONTENT

The procedure to calculate the percentage of air void contents in test specimens was performed in accordance with ASTM D3208-94:

$$AV(\%) = \left(1 - \frac{G_{mb}}{G_{mm}}\right) \times 100$$

where
G_{mb} = bulk specific gravity
G_{mm} = maximum theoretical specific gravity

Two methods were used to obtain the bulk specific gravities of HMA specimens — the saturated surface dry (SSD) method and the CoreLok vacuum sealing method, as shown in Figure 12. Several researchers (Chehab et al. 2000; Harvey, Eriksen et al. 1994; Harvey, Mills et al. 1994) have discussed the effect of each technique on air void content measurement. The SSD method is commonly used for specimens that do not have porous structure or interconnecting air voids, that absorb more than 2% water by volume, or both. For the mix design, this method is commonly used because it has a lower air void content; however, the SSD method is not recommended for use with porous HMA or specimens with higher air void contents. The CoreLok method uses a vacuum chamber with specially designed plastic bags to seal the specimen's surface. After the specimen is completely sealed, the submerged weight was measured. This method is fairly repeatable, but the plastic bags are expensive. Consequently, the majority of test specimens were prepared using the CoreLok method except for specimens with lower air void contents, which were measured with the SSD method, as shown in Table 13.

(a) (b)

Figure 12. (a) CoreLok method and (b) SSD method.

Table 13. Target Air Void Contents for Each Test

Test	**Air Void Content (%)**		**G_{mb} Method**
	Dense-Graded	**SMA**	
Mix Design	4	4 (3.5 for the control SMA)	SSD
Cantabro Loss Test	4	4 (3.5 for the control SMA)	SSD
Wheel Tracking Test	7	6	CoreLok
Illinois Modified Lottman Test	7	6	SSD
Complex Modulus Test	7	6	CoreLok
Semi-Circular Bending Test	7	6	CoreLok

In accordance with AASHTO R46-08, the target air void content for SMA was determined to be 6 ± 0.5%. While for other HMAs the target air void content was 7 ± 0.5%. The design air void content was used for specimens tested for the Cantabro loss, in accordance with the AASHTO specifications.

5.4 DURABILITY

The Cantabro loss test was used to determine the abrasion loss of compacted specimens of asphalt mixtures, in accordance with TX DOT 245-F. After being compacted to the design air void content of 4% (except for the control SMA, at 3.5%), the test specimen was cooled down and placed in the Los Angeles (LA) abrasion machine with the temperature controlled at 25°C. The LA abrasion machine was rotated at a speed of 30 to 33 revolutions per min for 300 revolutions without steel balls, and then the percentage of weight loss was calculated according to the following equation. This value, known as the Cantabro loss, is an indication of durability for the HMA:

$$Cantabro\ \ Loss\,(\%) = \frac{A-B}{A} * 100$$

where
A = initial weight of test specimen
B = final weight of test specimen

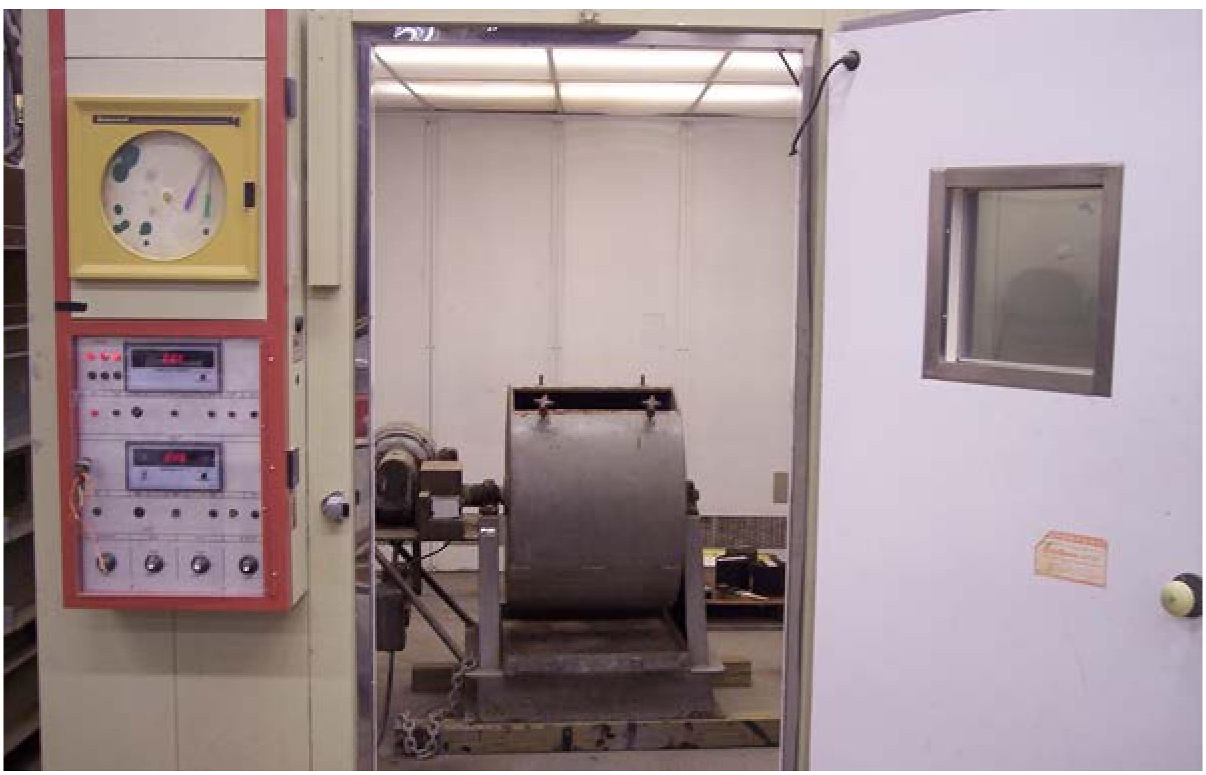


Figure 13. LA abrasion machine under temperature control at 25°C.

To maintain the test temperature at 25°C, the LA abrasion machine was placed in an environmental chamber, as shown in Figure 13. Test specimens were conditioned in the chamber for 2 hr before testing. Figure 14 shows post-test specimens whose edges were broken off. The loose materials were discarded, and the dust and broken materials stuck to the surface were gently removed with a soft brush to prevent the loose materials from being included in the tested specimen. Three replicates were made for each mixture.

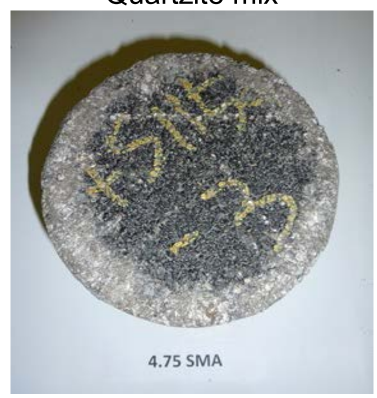
F-mix (control)

12.5 SMA (control)

Quartzite mix

Sprinkle mix

Slag/fiber mix



4.75 SMA

Figure 14. Specimens after Cantabro loss test.

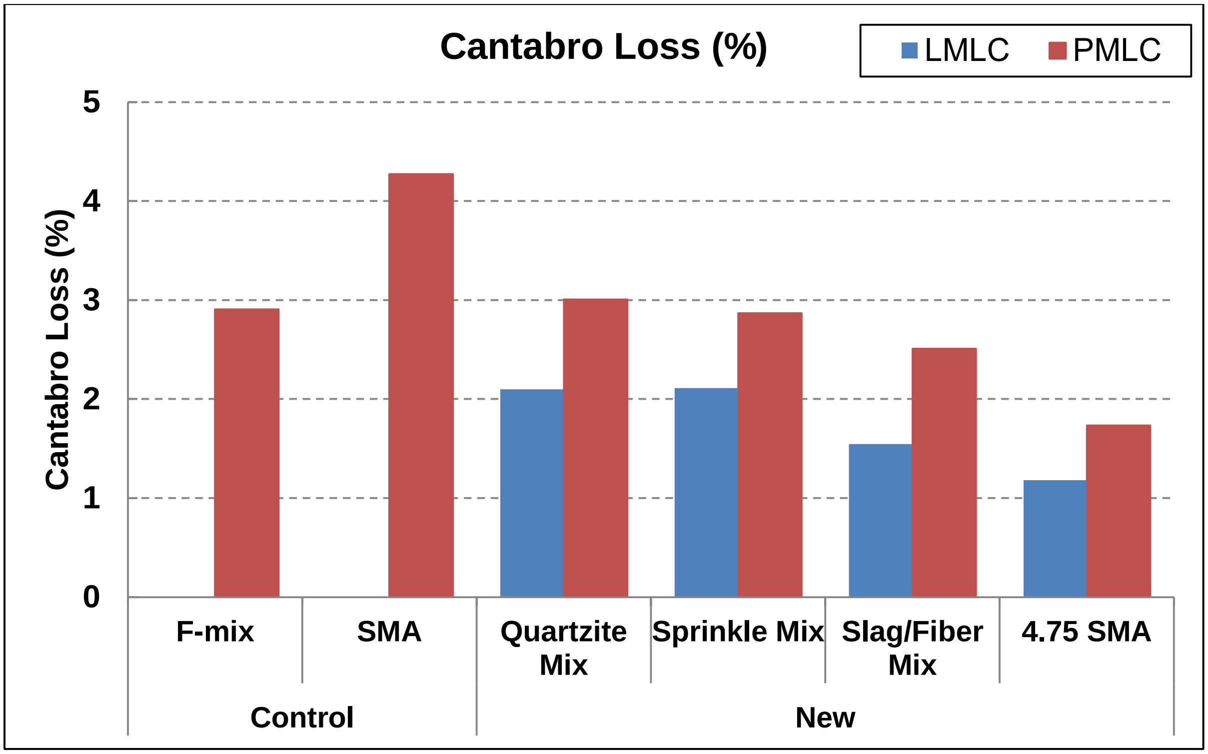
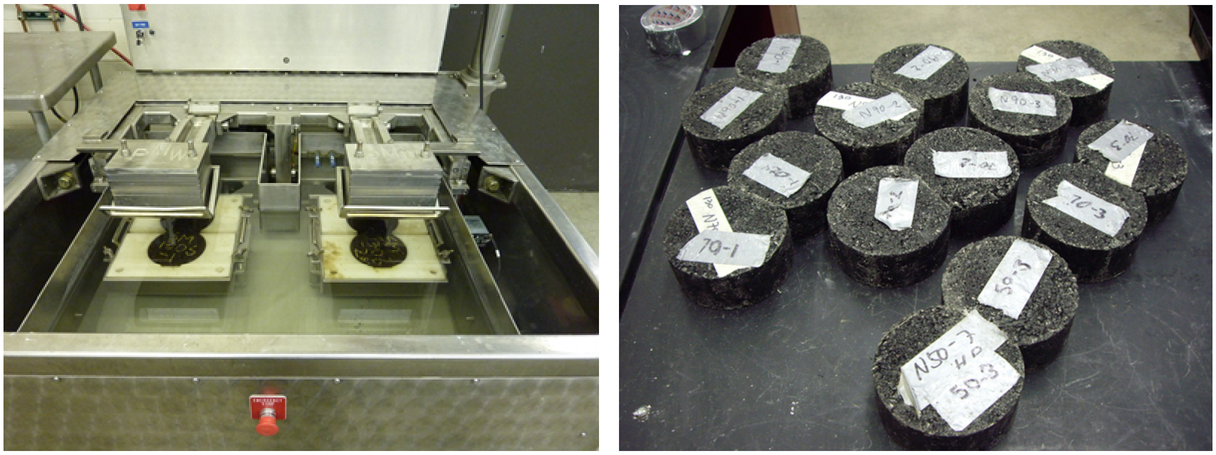


Figure 15. Cantabro loss test results.

Figure 15 shows the test results for the LMLC and PMLC specimens. Control mixes for the LMLC specimens were not produced in the laboratory; therefore, the comparison between developed mixes was made for LMLC specimens only. The quartzite mix and sprinkle mix showed the highest loss, indicating the least durable mixes. The abrasion loss for the 4.75 SMA was almost half that of the quartzite and sprinkle mixes. The smaller NMAS of the 4.75 SMA might be one of the reasons it had less broken-off material than the others. The fibers in the slag/fiber mix play an important role in improving the durability of that mix. For the PMLC specimens, the Cantabro loss values for the new mixes were generally higher than the losses for the LMLC specimens. Longer aging and reheating processes seemed to make the PMLC specimens more brittle. The control SMA had the highest Cantabro loss, indicating that it was the least durable, while the 4.75 SMA is expected to be the most durable mixture among the candidate mixtures. As shown by the results for the LMLC specimens in Figure 15, the quartzite and sprinkle mixes had somewhat higher losses than the slag/fiber mix and the 4.75 SMA. Mixtures with smaller NMAS had less material loss, indicating potential higher durability.

5.5 RUTTING POTENTIAL

ATREL's wheel tracking device measures rutting potential by rolling a steel wheel on the surface of test specimens molded in a tray and submerged in water, as shown in Figure 16. Once the water temperature reaches 50°C, the test specimen is conditioned for an additional 30 min before the test starts. The load applied by the steel wheel is 158 lb ± 1 lb, and the wheel loading rate is 50 ± 2 passes per min across the test specimen. The rut depth is taken every 20 passes, up to 20,000 passes.



(a) (b)

Figure 16. Rutting test: (a) wheel tracking device and (b) fabrication of test specimen.

Table 14 shows the rutting test results for the LMLC and PMLC specimens. For the LMLC specimens, four HMAs were compared for their potential rut resistance because the control mixes had not been laboratory prepared. The quartzite mix showed the best rut resistance: 4.0 mm of displacement after 20,000 passes. The maximum allowable rut depth is less than 12.5 mm (0.5 in.), and the minimum number of wheel passes at the 12.5-mm (0.5-in.) criteria is based on the high-temperature binder graded used in the mix, which is 10,000 passes for PG 64- or lower, 15,000 passes for PG 70- , and 20,000 passes for PG 76- or higher. All the mixtures met the requirements of wheel tracking test criteria. The 4.75 SMA had a significantly weaker rut resistance among the LMLC HMAs. Due to the viscoelastic behavior of HMAs, the asphalt content of the HMA plays an important role in rut resistance. It was found that the 4.75 SMA with higher asphalt content, around 7.3%, showed greater rut depth. Among the dense-graded mixes containing 9.5-mm NMAS, the sprinkle mix showed relatively greater rut depth because the sprinkle mix is a standard mix with local aggregates and is not designed for strength. Although the SMA mixture, owing to its aggregate structure, is generally known to provide better rut resistance, the NMAS and asphalt content also had significant effect on rutting behavior. Previous studies (Romero and Mogawer 1998; Brown and Bassett 1990) reported that HMAs with larger maximum aggregate sizes exhibited better rutting performance than those with smaller maximum aggregate sizes.

F-mix (control)

12.5 SMA (control)

Sprinkle mix

Quartzite mix

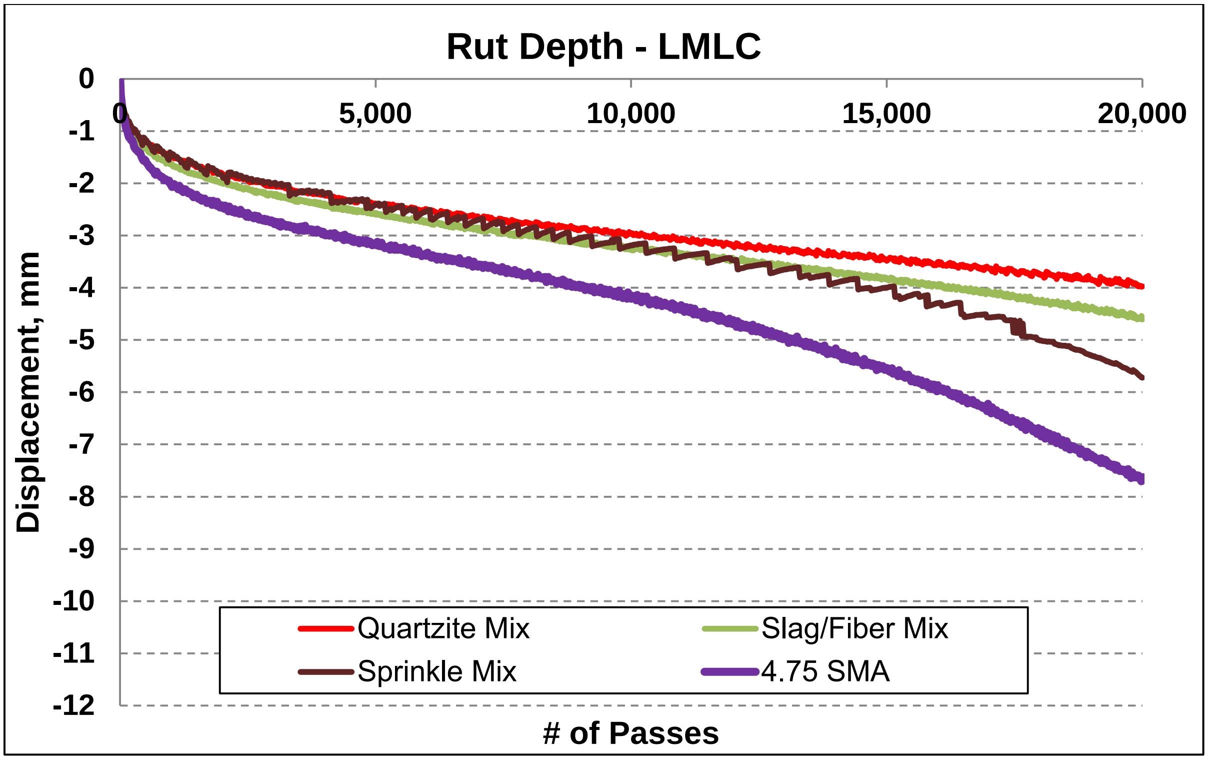
Slag/fiber mix

4.75 SMA

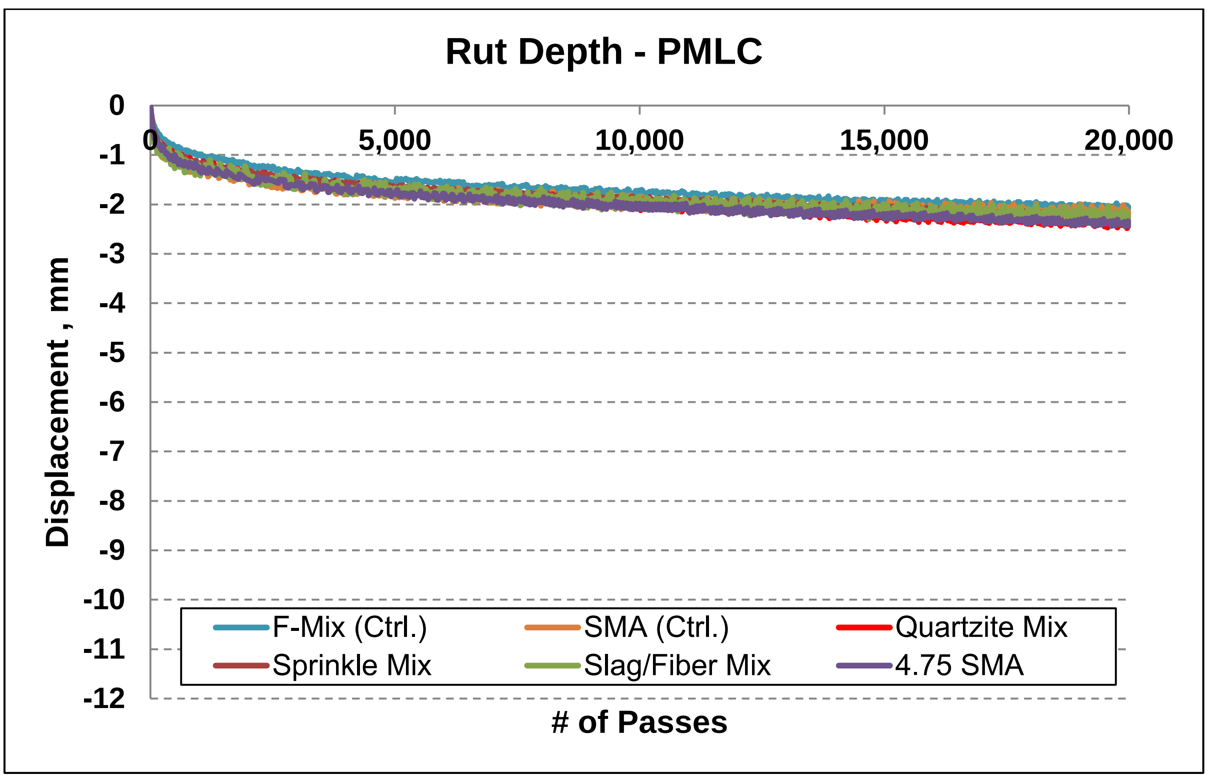
Figure 17. Rutting on the tested specimens.

Table 14. Rutting Test Results

Mixture Type		LMLC			PMLC		
		Max. Displacement (mm)	COV (%)	AV (%)	Max. Displacement (mm)	COV (%)	AV (%)
Control	SMA	-	-	-	2.3	8.9	5.8
	F-Mix	-	-	-	2.1	13.3	6.8
New	Quartzite Mix	4.0	3.3	6.8	2.5	7.0	7.1
	Sprinkle Mix	5.7	3.6	7.1	2.3	15.1	7.0
	Slag/Fiber Mix	4.6	12.8	7.0	2.4	6.5	7.0
	4.75 SMA	7.7	12.6	6.1	2.5	5.5	6.0



(a)



(b)

Figure 18. Displacement curves: (a) LMLC and (b) PMLC.

The rutting performance of the PMLC specimens differed from those for the LMLC specimens, as shown in Figure 18. All HMAs, including control mixes, showed almost identical rut depth after 20,000 passes. In spite of various aggregate types, asphalt types, and asphalt contents used in each mixture, there was no significant difference found in rut resistance among the candidate mixes. After the loose HMA materials were sampled in the plant, the plant mixes were reheated in the laboratory until reaching the target compaction temperature without any additional aging time. However, those materials were aged in the silo before sampling, and additional aging might have occurred during reheating. This could explain why the PMLC mixes were stiffer than the LMLC mixes. It appears that aging significantly affects rutting potential test results. Therefore, the rutting test results for PMLC specimens might not be sufficiently accurate to compare the rutting performance of HMAs.

The sprinkle chips were applied at various pre-coating and application rates, and the effects were evaluated at the surface of the HMA. The results are shown in Figure 19. Various asphalt contents were pre-coated on the chips, and the chips were spread on the surface of the HMA in the following variations: 1.5% asphalt binder content with 3.75 lb/yd^2 of spreading rate, 0.75% with 1.56 lb/yd^2, and no chips at the surface. The pre-coated chips with higher asphalt content enhance the sprinkle HMA rut resistance. The coarse sprinkle chips provide higher shear strength at the surface of the HMA.

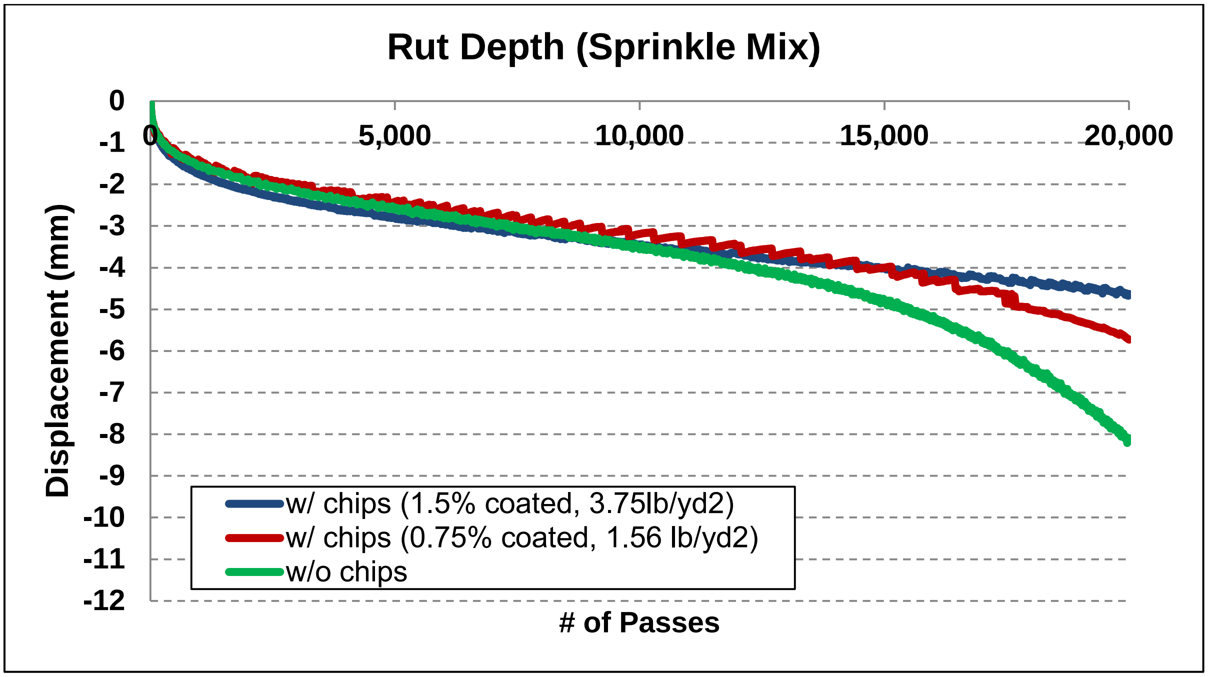


Figure 19. Effect of sprinkle chips on rutting potential.

In addition, the effect of air void contents on rutting potential was investigated in the laboratory using the control SMA loose mixture. The loose HMA materials were compacted to three different air void content levels: 5%, 6%, and 7%. As Figure 20 shows, the specimen with higher air void contents had weaker rut resistance. Therefore, special attention should be given to preparation of test specimens. Likewise, proper density is important during construction in order to improve rut resistance.

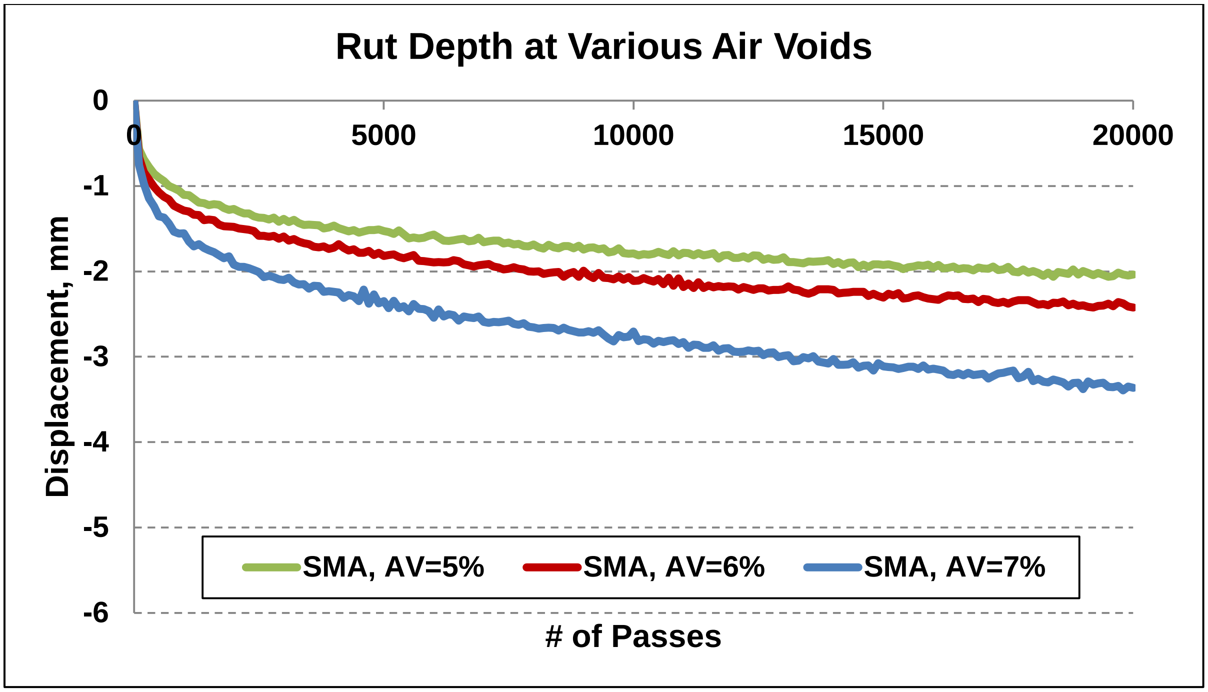


Figure 20. Effect of air voids on rutting potential.

5.6 THERMAL CRACKING POTENTIAL

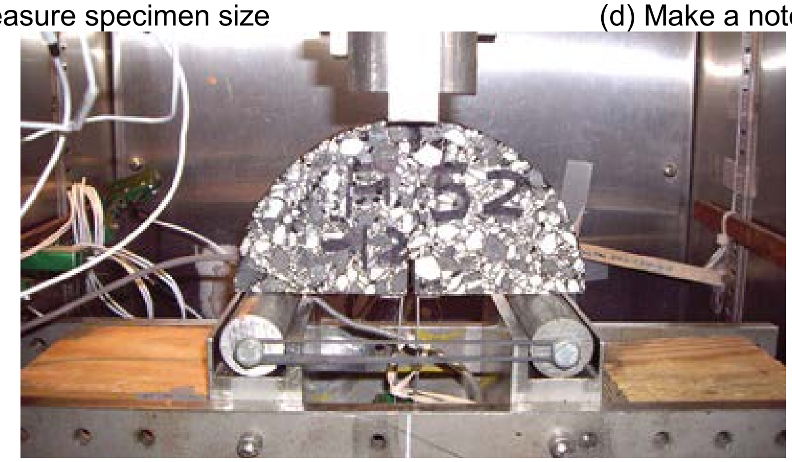
The fracture potential of HMA at low temperature is an important property related to thermal cracking. A semi-circular bending (SCB) test was conducted to evaluate low temperature fracture potential. For each mixture, a gyratory specimen with a diameter of 150 mm and a height of 115 mm was prepared in the laboratory and was then sliced to obtain one 25-mm-thick sample from the middle. The SCB slice taken from the gyratory specimen was cut into two semi-circular bend samples with a notch 15 mm long and 1.5 mm wide, as shown in Figure 21. Three replicates were tested at –12°C and –24°C; those temperatures were determined based on the lower performance grade (PG) limit of the asphalt binder, which is 10°C above and 2°C below the lower PG limit of the asphalt binder. The test was executed under load, which was kept constant at 0.7 mm/min of crack mouth open displacement (CMOD) for the entire duration of the test. The fracture energy was calculated by dividing the area under load displacement by the ligament area of the SCB specimen.

(a) Gyratory sample

(b) Slice and cut

(c) Measure specimen size

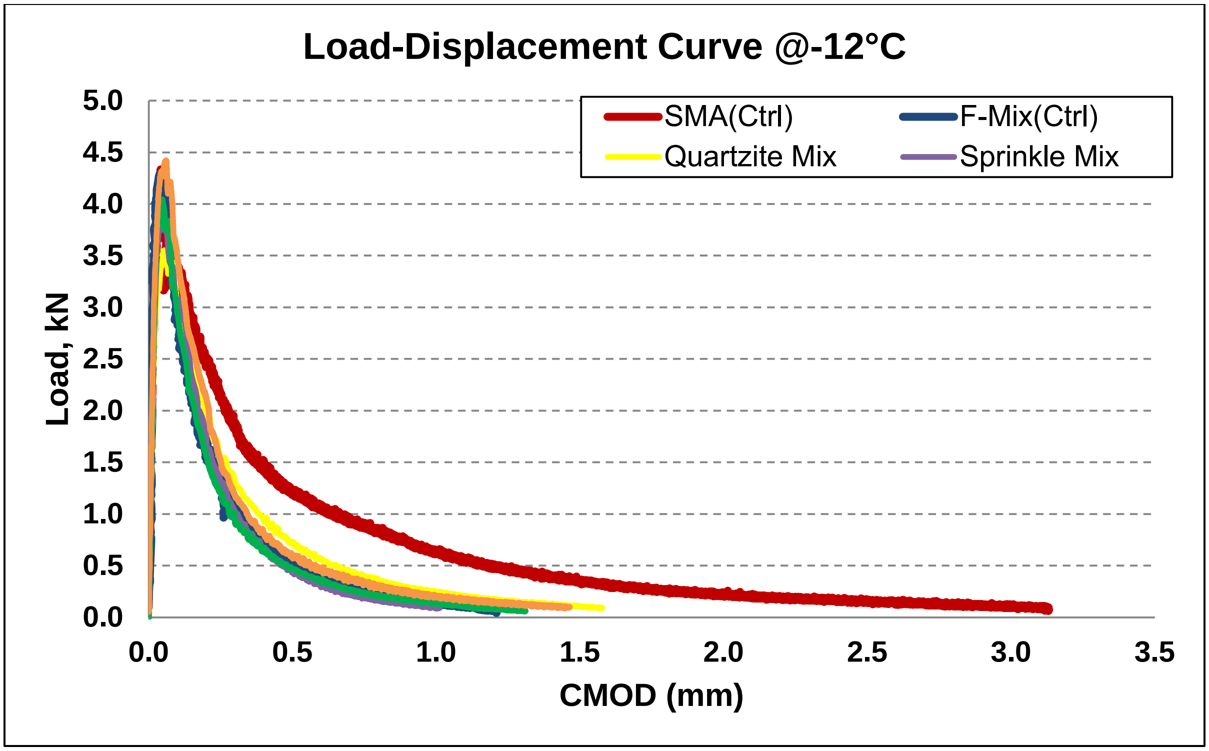
(d) Make a notch



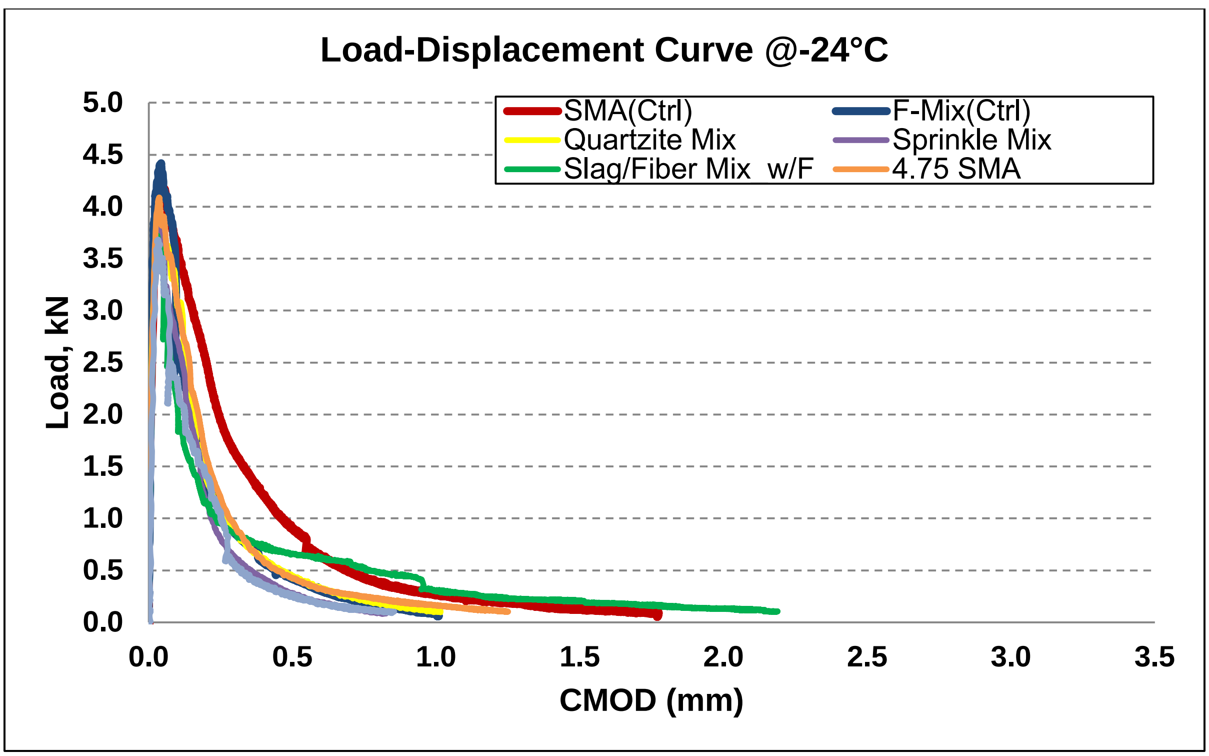
(e) Fracture testing at –12°C and –24°C

Figure 21. SCB test procedure.

For all the mixtures, the fracture energy decreased at lower temperatures because the HMA becomes more brittle. The control SMA was observed to have the highest fracture energy at both test temperatures for both LMLC and PMLC specimens. Rather than the gradation and asphalt binder contents, the asphalt binder type (a PG 76-22 highly modified binder) was the most responsible for increased fracture energy compared to the specimens that contained PG 70-22. The PMLC specimens showed lower fracture energy at both test temperatures because they became stiffer due to longer aging time in the plant and the reheating process in the laboratory. That was evident also through the wheel track test. Overall, the introduced HMAs had lower fracture energy at both temperatures than did the control HMAs. However, the slag/fiber mix was observed to have higher fracture energy than the F-mix only when the fibers were located at the fractured face. This finding will be discussed in more detail later in this report. Among the introduced HMAs, the 4.75 SMA with higher asphalt content had higher fracture energy in all cases (Figures 22-24).



(a)



(b)

Figure 22. Load-displacement curves: (a) –12°C and (b) –24°C.

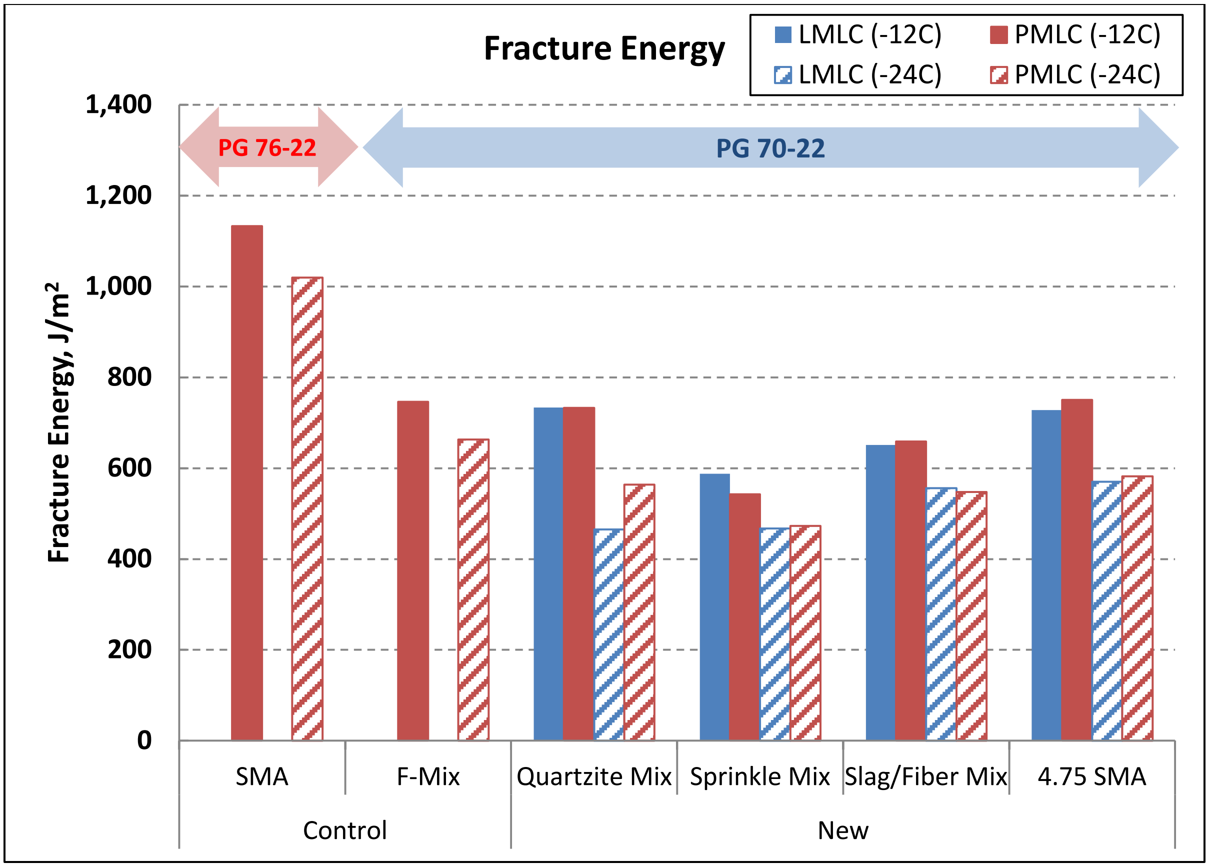
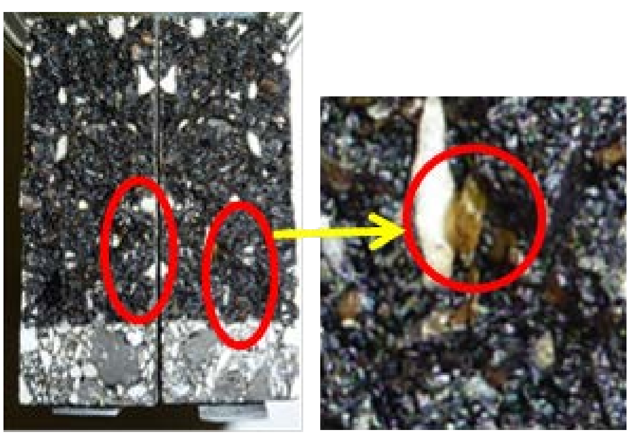


Figure 23. Fracture energy.



(a)

(b)

Figure 24. Existence of fibers at fractured face: (a) fibers and (b) no fibers.

For the slag/fiber mixture, the test results showed a high coefficient of variation of the fracture energy, around 30% at –24°C; therefore, a visual inspection of the fractured area was conducted. The fibers were observed at the fractured face for only two specimens of six replicates at –24°C, as shown in Figure 24. Figure 25 shows that fibers significantly increased the fracture energy by 57%, resulting in a longer tail for the load-displacement curve when the fibers were located at the fractured face. No fibers were found in the six test specimens at –12°C. Hence, the distribution of fibers plays an important role in fracture test results. As long as fibers are well distributed, they will significantly improve the fracture resistance of HMA.

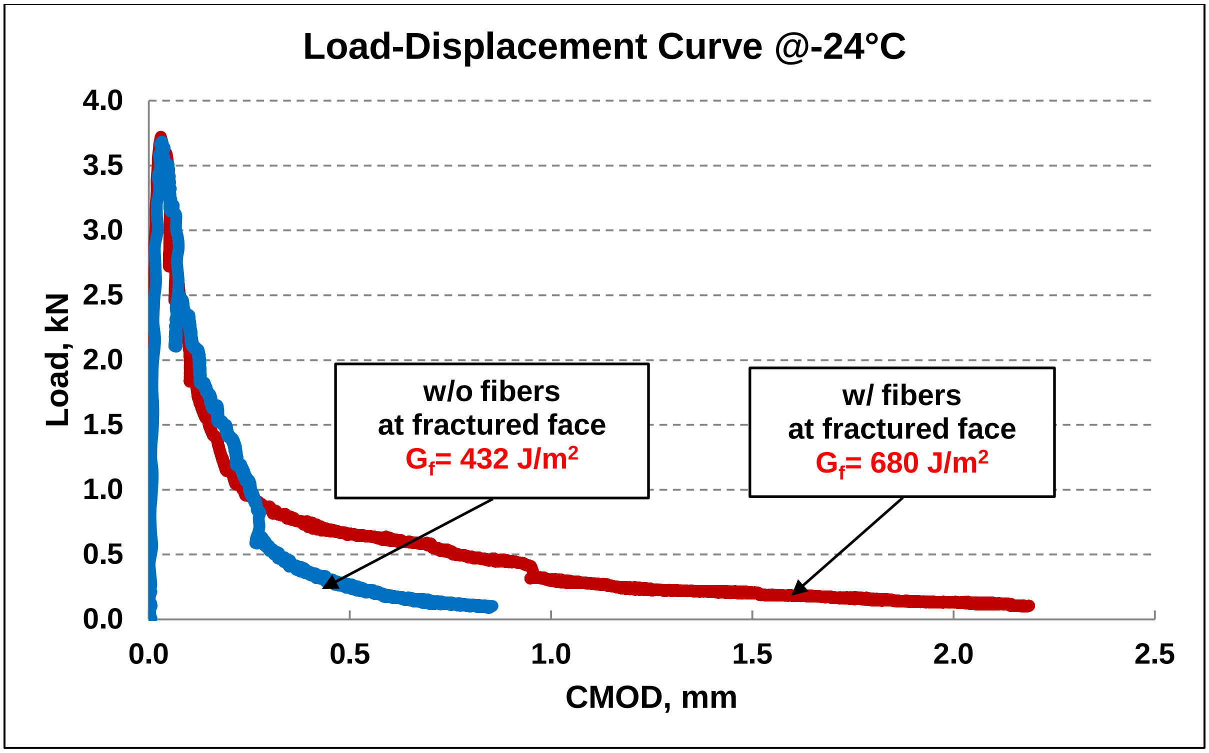


Figure 25. Effect of fibers on fracture energy.

5.7 MOISTURE SUSCEPTIBILITY OF HMA

A moisture susceptibility test determines how susceptible to weakening an HMA's internal bonding between asphalt binder and aggregate is, due to the presence of water. In accordance with Illinois-modified AASHTO T283, six gyratory specimens were prepared in the lab, and three specimens were conditioned at 25°C in a water bath for 2 hr before testing and were referred to as dry. As shown in Figure 26, the other specimens were saturated at 70% to 80%, conditioned at 60°C in a water bath for 24 hr, and then conditioned at 25°C for 2 hr before testing and were referred to as wet. A load was applied to the specimens at a rate of 2 in. per min, and the maximum load—known as the indirect tensile load—was recorded for each specimen. The average wet and dry strengths were used to determine the tensile-strength ratio (TSR).

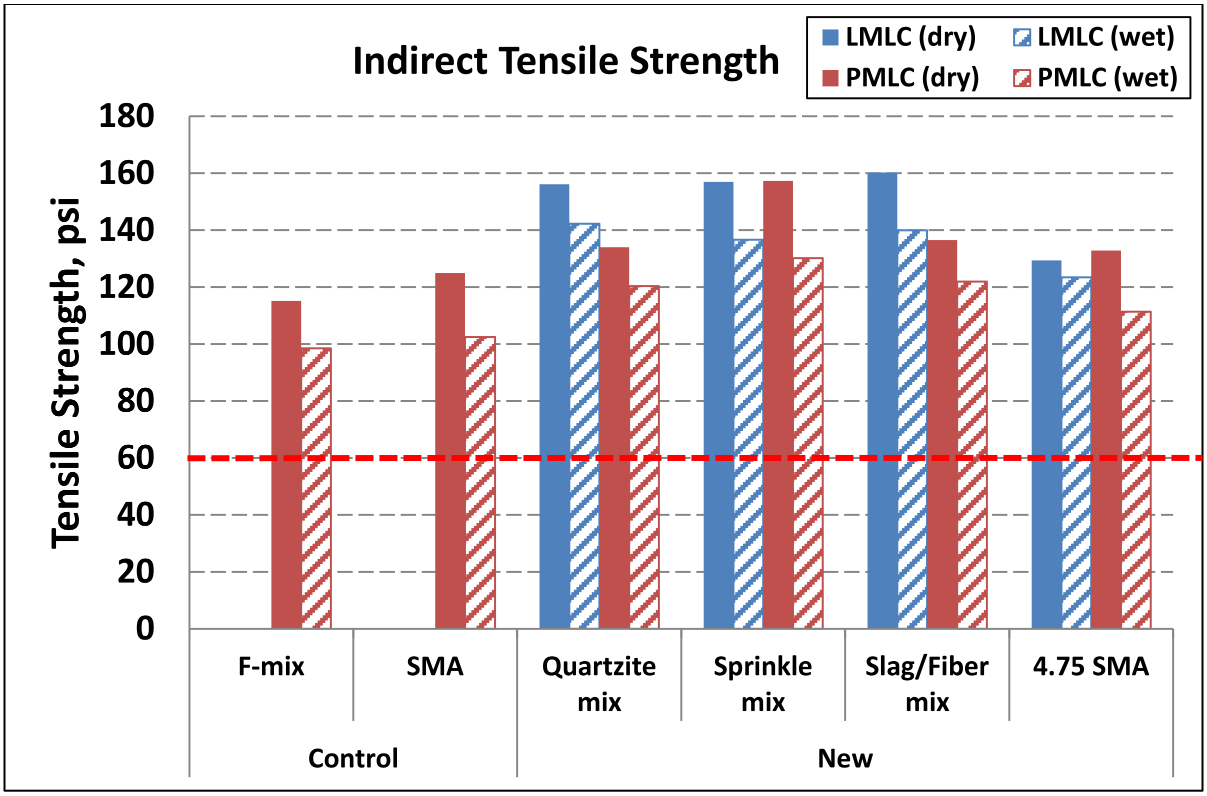
Figure 26. TSR test procedures.

As shown in Table 15, for the LMLC, the slag/fiber mix was observed to have the highest tensile strength when dry, while the quartzite mix had the highest tensile strength when wet. The 4.75 SMA had the lowest tensile strength for both conditions; however, the TSR values were the highest. TSR values primarily indicate how the HMA is affected by moisture in terms of the tensile strength ratios during dry and wet conditions. This does not mean that a mixture with a higher TSR value is always better with respect to moisture susceptibility. As shown for the other HMAs, if the tensile strength when wet is high, a mixture still has good resistance to moisture. It is also possible to have a very high TSR value from very low strengths. Therefore, the Illinois-modified AASHTO T283 requires a minimum acceptable tensile strength of 60 psi in addition to a minimum 0.85 TSR value. All of the new mixtures satisfy both requirements, and the fine dense-graded mixture provides an excellent tensile strength under both dry and wet conditions.

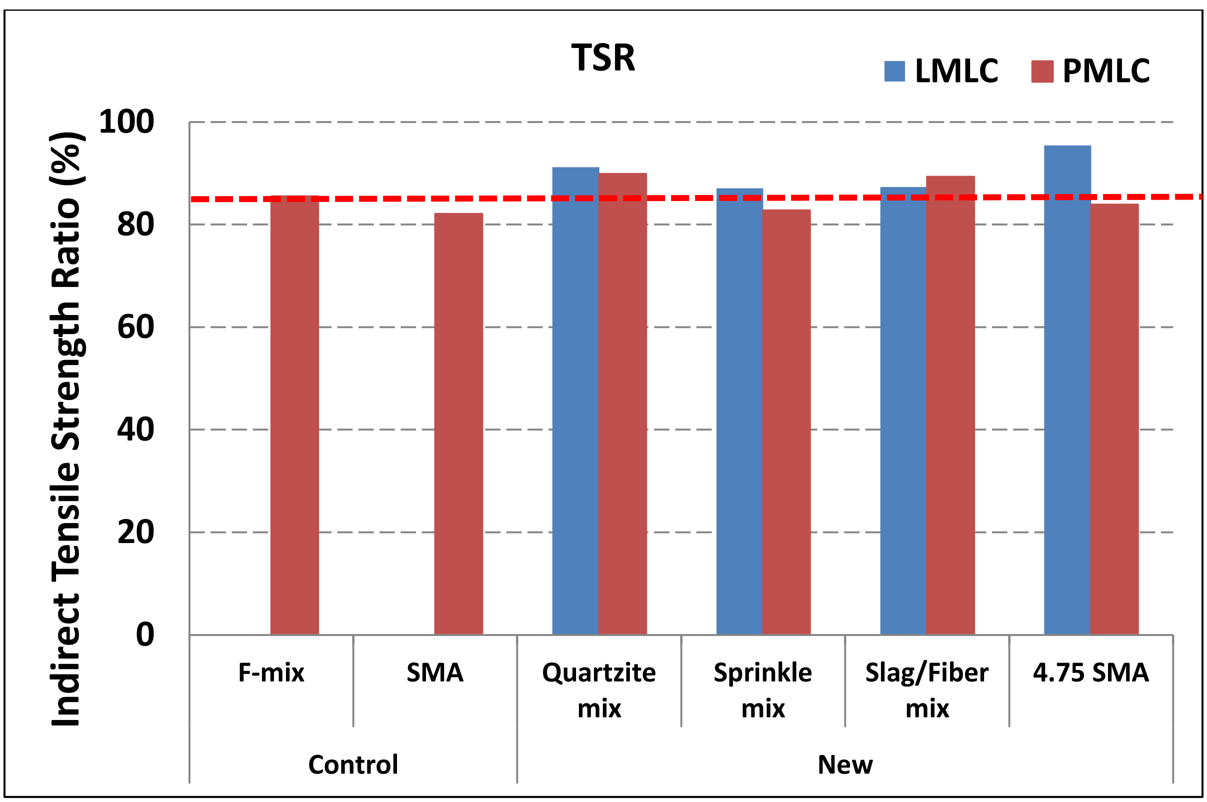
Table 15. Indirect Tensile Strength and TSR Values

Mixture Type		LMLC					PMLC				
		Dry		Wet		TSR (%)	Dry		Wet		TSR (%)
		Tensile Strength (psi)	Stripping Rate	Tensile Strength (psi)	Stripping Rate		Tensile Strength (psi)	Stripping Rate	Tensile Strength (psi)	Stripping Rate	
Control	F-mix	—	—	—	—	—	115.2	1	98.5	1	85.5
	SMA	—	—	—	—	—	124.9	1	102.5	1	82.1
New	Quartzite	156.0	1	142.2	1	91.2	133.9	1	120.3	1	89.9
	Sprinkle	156.9	1	136.6	1	87.1	157.3	1	130.2	1	82.8
	Slag/Fiber	160.2	1	139.9	1	87.3	136.5	1	121.9	1	89.3
	4.75 SMA	129.3	1	123.4	1	95.4	132.8	1	111.4	1	83.9

For the PMLC, the F-mix showed the lowest tensile strength under both wet and dry conditions, and the sprinkle mix was observed to have the highest tensile strength for both conditions (Figure 27). Among the introduced HMAs, the fine dense-graded mixtures had relatively higher tensile strengths than the SMA mixture did, and the introduced HMAs had higher tensile strengths than the control mixes did. However, the TSR values of the control SMA, sprinkle mix, and 4.75 SMA were lower than the minimum acceptable threshold of 0.85. Because the test specimens were prepared from reheated loose materials, test results for the PMLC, especially TSR values, were limited to use as an indicator of the mix design. Variations of the HMAs with respect to the mix design possibly affected the test results. The PMLC specimens had lower tensile strengths than the LMLC because HMA materials become stiffer during the reheating process in the laboratory. Testing the PMLC is meaningful in that the performance of the introduced HMA can be compared to that of the control mixtures.



(a)



(b)

Figure 27. (a) Tensile strength and (b) TSR.

5.8 COMPLEX MODULUS OF HMA

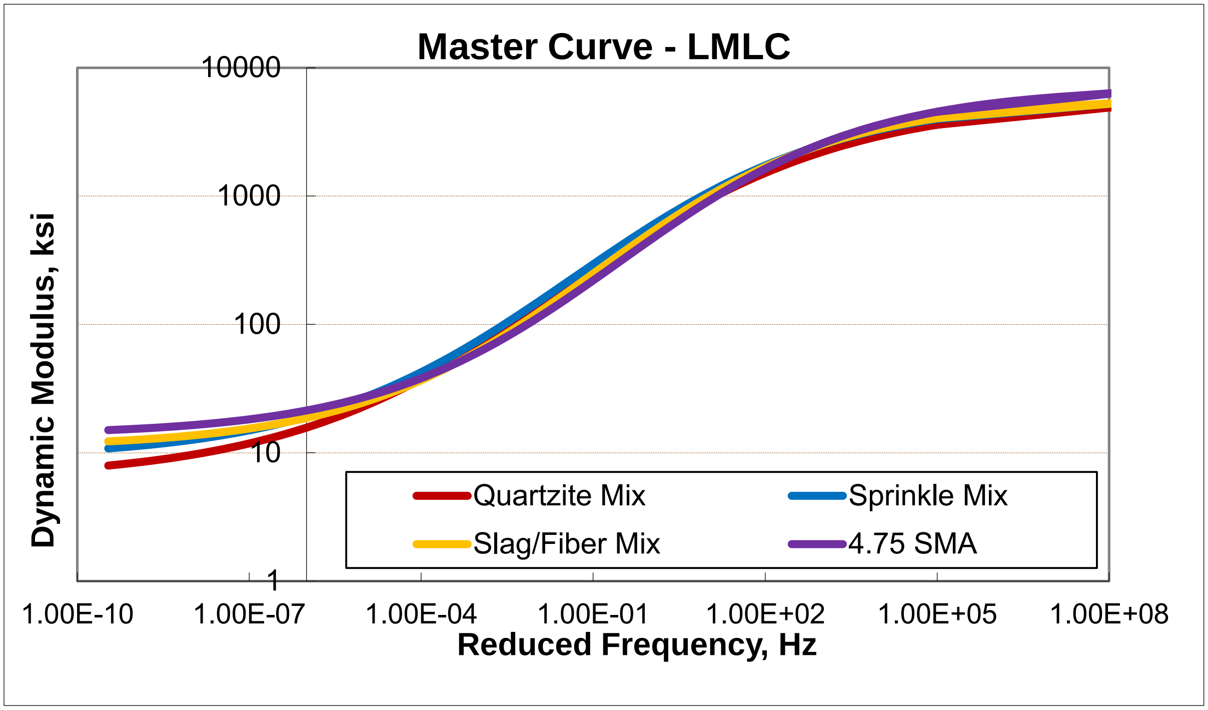
Following AASHTO TP62-03, the stress-to-strain relationship for each mix was investigated under continuous sinusoidal loading. The main purpose of this test is to determine the viscoelastic behavior of HMAs under repeated loading. Three test specimens were cored from 170-mm gyratory compacted mixtures to an average diameter of 100 mm and height of 150 mm. After sample fabrication, test specimens were conditioned for the required time at each temperature and tested, as shown in Figure 28. This test was performed at –10°C, 4°C, 21°C, 38°C, and 54°C under loading frequencies of 0.1, 0.5, 1, 5, 10 and 25 Hz. A master curve was constructed at a reference temperature of 21°C, as shown in Figure 29.

(a)

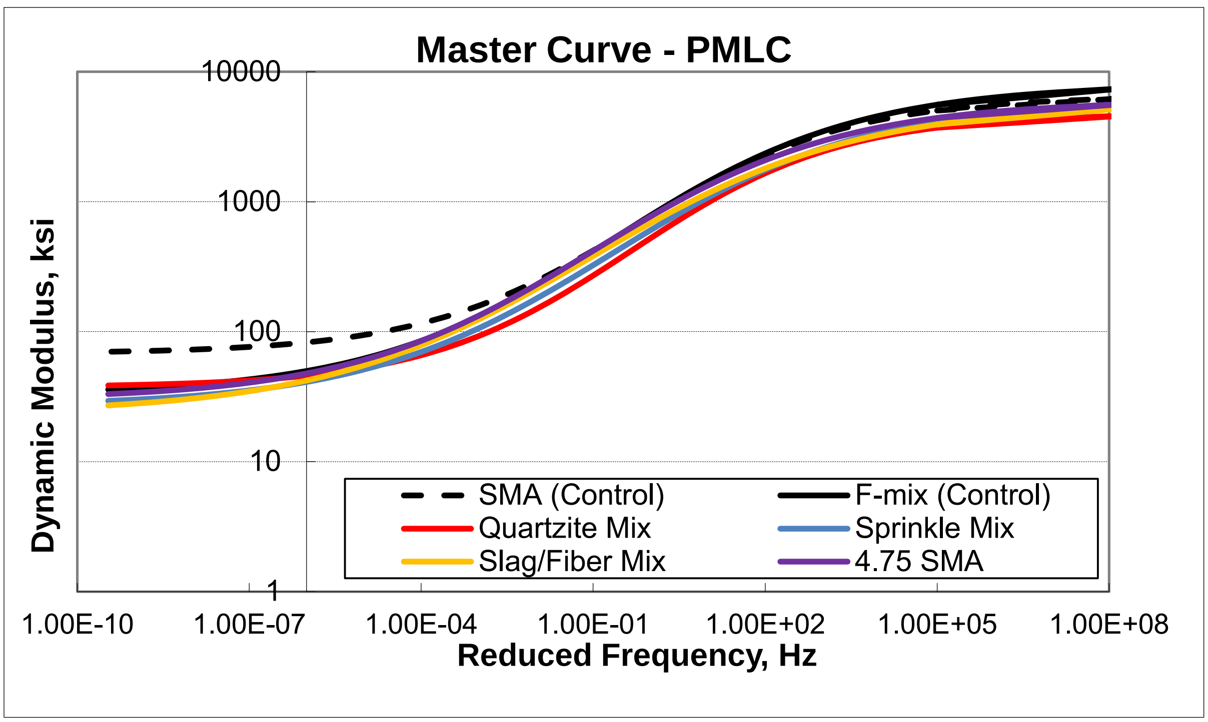
(b)

Figure 28. (a) Specimen conditioning and (b) complex modulus test.

The introduced mixtures were generally observed to have similar moduli except at lower frequencies for the LMLC and higher frequencies for the PMLC. To identify more details about the behaviors of the mixtures, two extreme cases were chosen and the complex moduli were compared, as shown in Figure 30. A high temperature and low frequency indicates that the HMA is more viscous (hence, higher values are better), while a low temperature and high frequency indicates that it is more elastic (hence lower values are better). In general, the moduli of the LMLC specimens were higher than those of the PMLC specimens because the reheating process makes HMA material stiffer and possible able to absorb more asphalt binder, which affects the performance of HMA. For the LMLC, the sprinkle mix had a higher modulus than the others, and the slag/fiber mix had a lower modulus. At lower temperatures and higher frequencies, the 4.75 SMA resulted in the highest moduli in this test. The control mixes had higher moduli than the introduced HMAs for both LMLC and PMLC, which means that the potential rutting resistance of the introduced HMAs is expected to be weaker than that of the controls; however, the control mixes are expected to be more brittle than the introduced HMAs at low temperatures and high frequencies. Because the control SMA uses PG 76-22, it resulted in the highest modulus with respect to other mixes, which use PG 70-22.

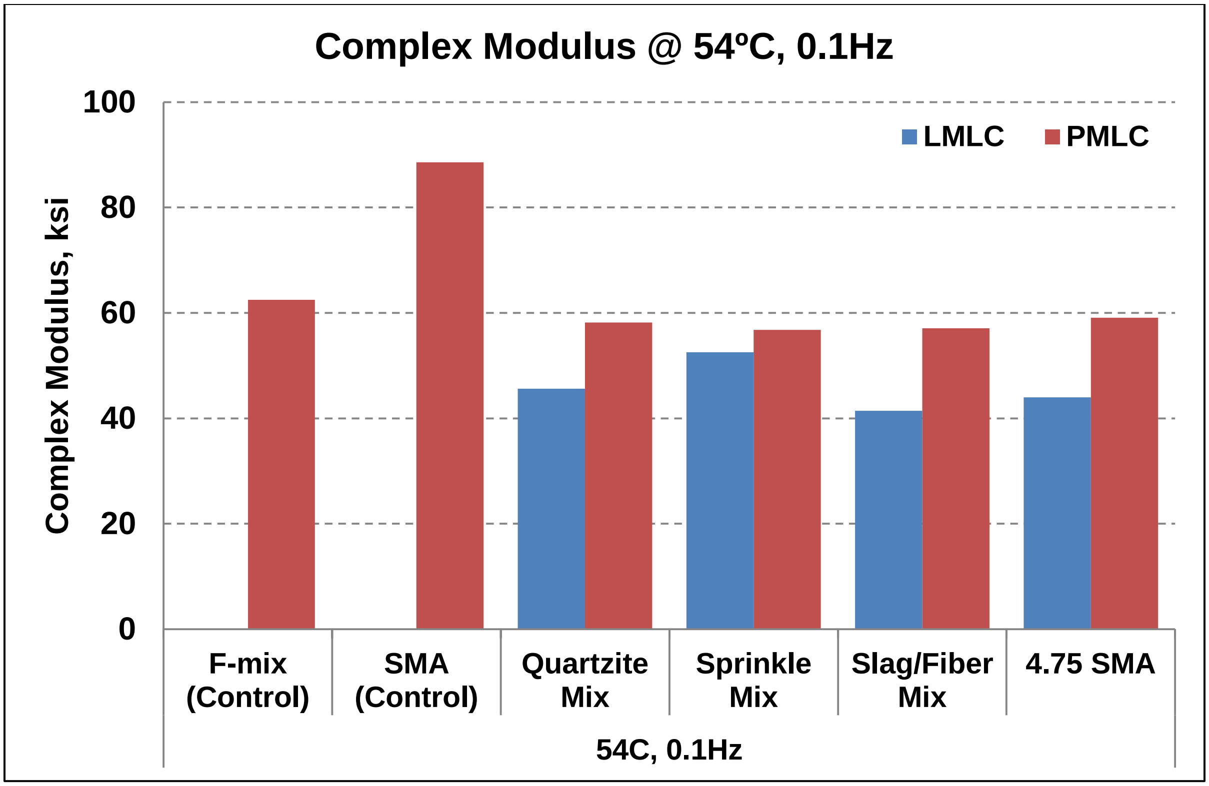


(a)

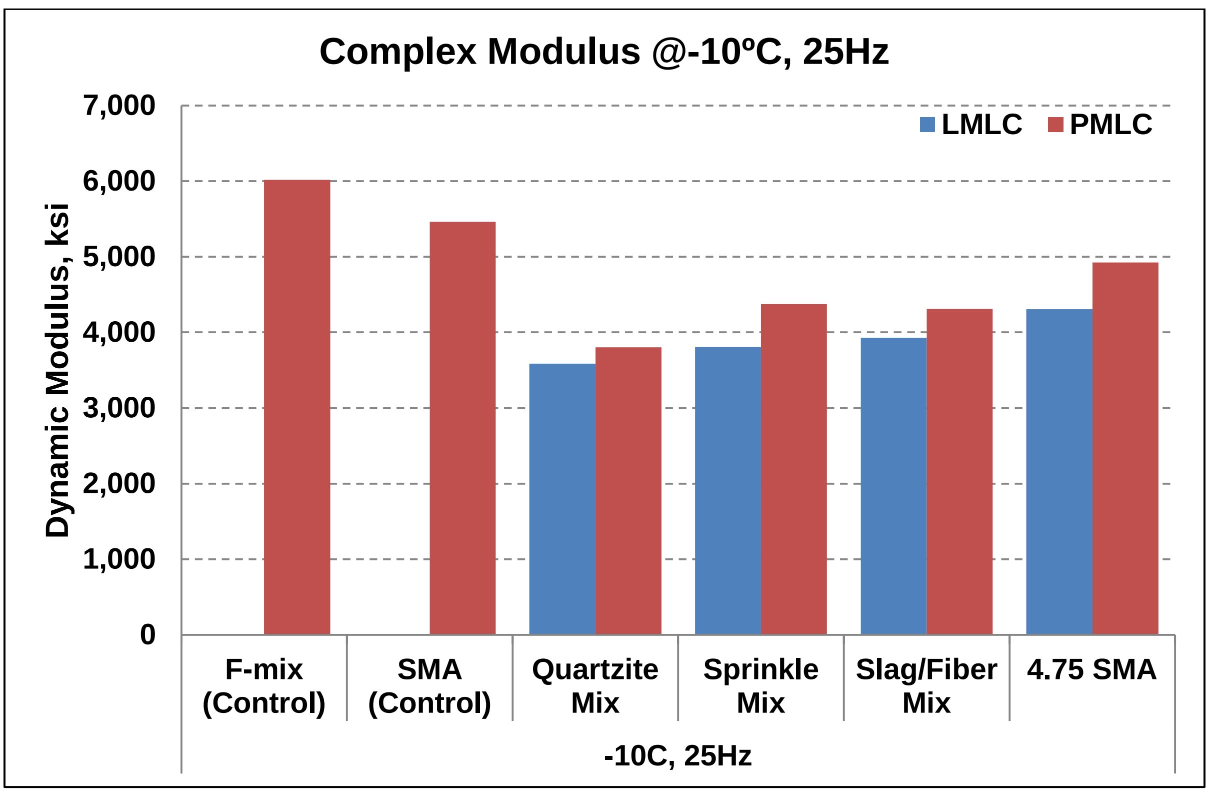


(b)

Figure 29. Master curves at T_{Ref} = 21°C: (a) LMLC and (b) PMLC.



(a)



(b)

Figure 30. Complex modulus: (a) 54°C, 0.1 Hz and (b) –10°C, 25 Hz.

CHAPTER 6 SUMMARY AND FINDINGS

The introduced HMAs were designed to ensure better aggregate structure, using the Bailey method. The new mixtures are intended to reduce the wearing surface thickness; hence, the aggregate structure of 9.5-mm fine dense-graded HMA and a 4.75-mm SMA were utilized. A thinner wearing course can reduce material and construction costs as well as reducing disturbance to traffic due to reduced construction time. The use of mixes that include a higher content of local aggregates such as dolomite and natural sand can also decrease material costs (including transportation and environmental costs); but any trade-off in HMA's performance needs to be considered. The laboratory test results showed relatively good performance of the introduced HMAs. The findings of the laboratory study are summarized below.

- The plant-mixed lab-compacted (PMLC) specimens showed better performances in rutting, fracture, and complex modulus tests, while the lab-mixed lab-compacted (LMLC) specimens showed improved performance in durability and moisture susceptibility tests. Longer aging times and the effects of reheating of PMLC specimens compared to LMLC specimens resulted in stiffer PMLC HMA.
- The introduced HMAs had smaller Cantabro losses than the control mixtures did, which indicates that they are more durable than the control HMAs. The 4.75 SMA with higher asphalt binder content was observed to be the most durable mixture.
- The slag/fiber mixture showed the highest fracture resistance among the tested HMAs when using the same asphalt binder grade (PG 70-22) when fibers are located at the fractured face.
- The higher the PG binder grade (i.e. SMA with PG 76-22), the better the fracture resistance. The 4.75 SMA provided the highest fracture energy among introduced HMAs, probably because of its higher asphalt binder content due to higher VMA. From an economical point of view, reducing the layer thickness when using 4.75 SMA can be more cost effective in spite of the higher asphalt binder content.
- Among the introduced LMLC HMAs, the quartzite mixture showed the best resistance to rutting potential when using the wheel track test; while 4.75 SMA showed the greatest potential for rutting. For PMLC specimens, all HMAs showed almost identical rutting potential. The aging during the storage time in the silo at the plant and the reheating process in the lab to prepare test specimens resulted in less induced rutting potential of PMLC specimens than the LMLC specimens. For the sprinkle mix, pre-coating the chips with 0.75% asphalt binder improved the rutting resistance. The SMA mixture has better durability; however, the higher asphalt content can result in an increase in rutting potential.
- The results of moisture susceptibility tests showed that the introduced AC mixtures had higher tensile strengths than the two control mixes under both wet and dry conditions. The TSR from the PMLC specimens should be used only for reference because they were reheated.
- The lower complex modulus of the introduced HMAs compared to the control mixtures at high temperatures and low frequencies indicated that the introduced HMAs are expected to have higher rutting potential; however, wheel track test results no significant difference. The higher complex modulus of the control mixtures at low temperatures and high frequencies indicated that they are expected to be more brittle at low temperatures. However, the fracture test results showed that the control mixtures, specially the control SMA, have less fracture potential. It has to be noted

that rutting failure in the field is associated with shear; while thermal cracking is associated with tensile strength and fracture energy. Complex modulus test is conducted in compression.

Based on the findings of the lab performance tests, the 4.75 SMA showed the best relative performance when considering durability, moisture susceptibility, and fracture of the LMLC specimens. However, relative tensile strength and potential for rutting were marginal. These results are related to the relatively high asphalt binder content and aggregate gradation and low NMAS. For the PMLC specimens, the sprinkle mixture provided the highest TSR value and good performance at low temperature. The two control PMLC HMAs, in general, performed the worst, which include durability, moisture susceptibility, complex modulus at low temperature and high frequency range. Upon the completion of the field testing, a recommendation will be prepared as to appropriateness of implementing these mixes in the field.

The pavement performance for each HMA under actual traffic loading and environmental conditions will be discussed in Volume 2. The field test results, to be provided in Volume 2, will provide data on seasonal effects on HMA with various additives and determine whether seasonal effects have an impact on HMA performance in the field. In addition, any difficulties of field application will be discussed for each HMA. Although the introduced HMAs are chosen to remedy typical problems with current HMA overlays, final decisions should be made based on cost effectiveness and efficiency, even though the thickness of the wearing course can be reduced when using fine-dense gradation and smaller NMAS. The engineering cost analysis will be included in Volume 2. Hence, the study recommendations will be included in Volume 2.

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APPENDIX A: LITERATURE REVIEW AND SCOPE OF STUDY

1 INTRODUCTION AND OVERVIEW

As the cost of hot-mix asphalt (HMA) that provides high surface friction and good durability increases, cost-effective HMA surfacing alternatives are needed. Therefore, the primary objective of this project is to develop a cost-effective mix profile for a new generation of HMA surface layers.

A systematic literature review has been performed to find references that provide relevant information that will address the research objectives. The literature databases used for the search include ASCE Civil Engineering, Compendex (via Engineering Village), Web of Knowledge (Web of Science), Conference Papers Index, TRIS, and ScienceDirect, along with Google Scholar.

The results of the literature review were used to

- Review the causes of pavement raveling and to determine what tests can be used to check for raveling potential
- Review the factors that affect the friction properties of pavements
- Review the tests that are available to measure friction in the laboratory as well as in the field
- Review the mechanisms of pavement noise generation
- Review the available tests to measure pavement noise
- Review the types of surface layers that are currently used in Illinois and in other countries
- Review the specifications for surface mixtures in Illinois
- Determine which materials are available in Illinois
- Determine which mixes are potential candidates for this study

2 HMA DURABILITY AND CHARACTERISTICS

Durability is the capacity of a pavement to keep its functionality over time. It can be evaluated in terms of distress development. Surface distresses can be in the form of cracks, deformation such as rutting, corrugation, bleeding, or shoving; or disintegration such as raveling, stripping, and spalling. When cracks are present on the pavement surface, water may enter the pavement structure. Since the base, subbase, and subgrade lose their load-carrying capacities when they are wet, the water entering through the cracks may lead to more severe pavement failures. Presence of distresses on the pavement surface may lead to rougher pavements and hence poor ride quality. Additionally, the presence of bleeding on the pavement surface may reduce friction. Considering that the goal of this study is to develop a new thin wearing surface, two of the major concerns for the thin surface layer are material disintegration (raveling) and moisture susceptibility (stripping). Therefore, the mechanisms that cause pavement raveling and stripping are presented along with the laboratory and field test methods for evaluating HMA susceptibility to these distresses. The complex modulus (E*) is used as an additional property to compare the characteristics of the various mixtures.

2.1 HMA RAVELING

Raveling is the roughening of the asphalt surface texture due to the wearing away of aggregates from the pavement surface. The bond between the aggregate particles and asphalt is lost, and the asphalt wearing course disintegrates from the top downward (Wolters 2003).

Raveling occurs as aggregate particles and asphalt binders are stripped and lost due to wearing of the pavement surface. The severity of raveling can be evaluated by the roughness of the surface and by the size of aggregate that is being lost. Low severity indicates that the asphalt binder has begun to wear and some of the fine material is lost. Moderate severity is an indication that the texture is becoming rough, as some of the coarse aggregate breaks loose. High severity raveling is evident when significant loss of coarse aggregate results in a rough, pitted surface and eventually potholes (Hawks and Teng 1993). Raveling may be the most obvious form of aggregate failure, but it is not the only one. For example, stripping results when bonding between the binder and the aggregate breaks down, which although usually attributed to the detrimental effects of moisture, may also be exacerbated by use of unsuitable aggregate.

The severity of raveling was categorized as coarse when the wearing away of the pavement surface resulted in a very rough surface texture due to dislodged coarse aggregates and loss of binder, and it was categorized as fine when the surface texture was moderately roughened due to the wearing away of fine aggregate and asphalt binder. Improvements in the technology of mixture design, including the use of modified binders to reduce the tendency to ravel, have improved the performance of porous friction courses (Henry 2000).

2.1.1 Possible Mechanism of Raveling

Possible causes of raveling are oxidation and hardening of the binder, thin asphalt binder film, low asphalt content, or the reduction of effective asphalt content as a result of the presence of absorptive aggregates, segregation, and high air void content. Excess fine dust on the aggregates and contaminants spilled on the HMA may also be possible contributing factors to raveling. Additionally, the presence of water in asphalt under traffic may result in hydrostatic pressure, which may cause debonding of aggregates from the binder (Wolters 2003).

2.1.2 Additional Factors Contributing to Raveling

2.1.2.1 Mixture Type

When porous surfaces are used to provide good friction and splash and spray qualities, there is often a sacrifice in durability. Raveling occurs because of the aging of the binder, and the layer may be worn away in a very short period of time (Huber 2000). For rubberized asphalt mixes, Aybike Ongel et al. (2008) showed that fewer rubberized mixes have raveling after the first year; however, raveling increased both for rubberized and non-rubberized mixtures in the second year.

2.2.2.2 Environmental and Loading Conditions

As the pavement ages, the asphalt binder becomes stiffer, increasing the likelihood that aggregate will separate from the asphalt as traffic repetitions accumulate. The effects of rainfall are cumulative over time. As the amount of rainfall experienced by the pavement increases, the probability of raveling increases. The pavement is also more likely to have raveling when it experiences high annual equivalent single-axe loads (ESALs) above 800,000. The interaction of age, average annual rainfall, and traffic provides a combined effect, meaning that the probability of raveling is very high for older sections under heavy traffic and high rainfall. Ongel (2008) conducted a regression analysis for the factors that affect pavement raveling. Mixture type, rubber inclusion, annual freeze-thaw cycles, annual average daily truck traffic (AADTT), and ESALs were used as categorical variables. The amount of rainfall and number of wet days over the pavement life, ESALs, age, and the interaction of age, rainfall, and traffic were found to be the significant variables affecting the presence of raveling.

2.1.3 Prevention of Raveling

Placement in wet weather greatly increases the raveling potential of a mix. During paving, if moisture is present due to rain, fog, or high humidity, a film of moisture forms on the coated aggregate particles of HMA and may prevent the development of a strong bond between the binder film and aggregates. The potential for raveling under these conditions can be accentuated by a cool mix that is unable to rapidly evaporate any moisture it is exposed to(i.e., sitting too long in a truck or paver), and by lower compaction, which is more likely under the cooler ambient temperatures associated with wet weather.

Layer thickness plays a critical role in preventing raveling, as it relates to compaction. A minimum layer thickness of two times the maximum aggregate size should always be maintained. This allows enough room for particle reorientation and proper compaction to occur. If layer thicknesses thinner than twice the maximum aggregate size, there is insufficient room for the aggregate to reorient itself in a dense configuration that is impermeable to water. In addition, if the layer thickness/maximum aggregate size ratio is less than 2:1, aggregate fracture often occurs in the rolling process, leaving uncoated aggregate surfaces in the mix that will not bond.

Mix design and binder content are critical to mix performance. A binder content that is too low for a given mix does not provide enough "glue" to bind the aggregate together. This can become a problem with a good mix design if the characteristics of the aggregate change, particularly if the aggregate becomes more absorbent. Asphalt binder that is absorbed into the aggregate is not effective. The quantity of binder in the mix must exceed the absorbed amount (saturate the aggregate) in sufficient quantity to provide free asphalt cement to bind the aggregate particles together. It should be noted that mix quality problems arise with too high of a binder content as well. Care should be taken to stay within the recommended limits provided in the mix design.

The cleanliness of the aggregate is very critical in mitigating raveling. Dirty aggregates coated with dust or fines create a barrier to direct contact (bonding) between the asphalt binder and the aggregate. The weakest portion of the resulting aggregate-dust-asphalt bond is the aggregate-dust adhesion. After several repetitions of the physical forces of traffic loading, the forces of adhesion holding the dust to the aggregate will break, and dislodging of the aggregate from the pavement mass will occur. An excess amount of dust or fines also has the potential to absorb a high percentage of asphalt cement, lowering the effective asphalt content of the mix, leading to the binder content problems outlined earlier.

Finally, the plant mixing temperature is critical. Asphalt binders that are heated too high, typically over 330°F, will become "age-hardened" and lose their maximum effectiveness as a binder. The active bonding ingredients, or "volatiles," in the asphalt cement are burned off, resulting in a much weaker binder. Extreme care should be taken by plant operators to maintain consistent mix temperatures that do not exceed the recommended temperature for the grade of asphalt binder being used (Mansell 2008).

2.2 TEST METHODS FOR PAVEMENT RAVELING

2.2.1 Laboratory Tests for Raveling

ASTM provides the Standard Test Method for Raveling Test of Cold Mixed Bituminous Emulsion Samples (ASTM D7196). Several states in U.S. apply this method on different asphalt mixes. Kansas follows the procedure of ASTM D7196 to perform the raveling test on recycled asphalt (Kansas Test Method KT-61), and Texas has developed its own method using the Los Angeles (LA) abrasion machine to determine the abrasion loss of compacted HMA specimens (Tex-245-F).

2.2.1.1 ASTM D7196

Figure 2-1 and Figure 2-2 show the experimental setup for ASTM D7196 that uses either a field blended mixture (Method A) or a laboratory blended mixture (Method B). The mixture is compacted in a gyratory compactor and cured at the specified conditions for a designated period of time. After the assigned curing time, a rotating rubber hose exerts an abrasion force on the specimen for 15 minutes and the abraded loss of material is calculated. The abrasion potential is evaluated by calculating the total mass loss percentage based on the initial sample mass. The percent mass loss is calculated as follows:

$$\%\ Mass\ Loss = 100 * \frac{A - B}{A}$$

where
A = specimen mass (prior to test)
B = specimen mass (abraded)

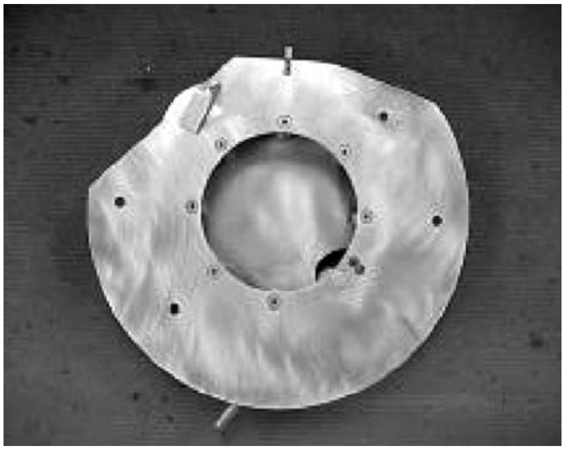


Figure 2-1. Raveling test adapter base (ASTM D7196-06).

Figure 1-2. Raveling test apparatus setup (ASTM D7196-06).

2.2.1.2 Cantabro Loss Test

The Cantabro loss test is used in the Texas Department of Transportation (Texas DOT) to determine the abrasion loss of compacted HMA specimens. This test procedure measures the breakdown of compacted specimens utilizing the LA abrasion machine. The percentage of weight loss (Cantabro loss) is an indication of permeable friction course (PFC) durability and relates to the quantity and quality of the asphalt binder. The Cantabro loss test is conducted using laboratory- or plant-produced mixes. After compaction, the specimen is cooled to the room temperature and the weight is measured. The specimen is then placed in the LA testing machine at a constant rotation speed of 30 to 33 rpm for 300 revolutions. After 300 revolutions, the loose material broken off the test specimen is discarded and the remaining specimen is weighed. The Cantabro loss is calculated as follows:

$$\mathrm{C}L = 100 * \frac{A - B}{A}$$

where
CL = Cantabro Loss%
A = initial weight of test specimen
B = final weight of test specimen

2.2.2 Field Testing for Raveling

In the past, raveling was measured by visual surveys. Now the Automatic Road Analyzer (ARAN) laser shown in Figure 2-3 is being used to detect the severity level of raveling. An algorithm looks for data on missing aggregates with five rocks in a row missing. This algorithm is based on the texture data measured by the ARAN.

Figure 2-3. Automatic Road Analyzer (ARAN) (www.maine.gov).

2.3 MOISTURE SUSCEPTIBILITY (STRIPPING)

Moisture susceptibility is a primary cause of distress in HMA pavements. Moisture susceptibility testing is needed to determine how susceptible a HMA mixture's internal asphalt binder-to-aggregate bond is to weakening in the presence of water. This weakening may result in stripping as shown in Figure 2-4.

Figure 2-4. Fatigue cracking caused by stripping (pavementinteractive.org).

2.4 TEST METHOD FOR MOISTURE SUSCEPTIBILITY (STRIPPING)

Results from the moisture susceptibility test may be used to predict long-term stripping susceptibility of HMA mixtures and to evaluate anti-stripping additives, which are added to the asphalt binder, aggregate, or HMA mixture. The most common moisture susceptibility test is commonly called the "modified Lottman" test and is a combination of the Lottman and Root-Tunnicliff tests. It compares the split tensile strength of unconditioned samples to samples partially saturated with water. Although it is expected that the water-conditioned samples will have a lower tensile strength, excessively low values indicate the potential for moisture damage.

The test used to evaluate HMA for moisture susceptibility is AASHTO T283 or ASTM D4867. This test serves two purposes. It identifies whether a combination of asphalt binder and aggregate is moisture susceptible and also measures the effectiveness of anti-stripping additives.

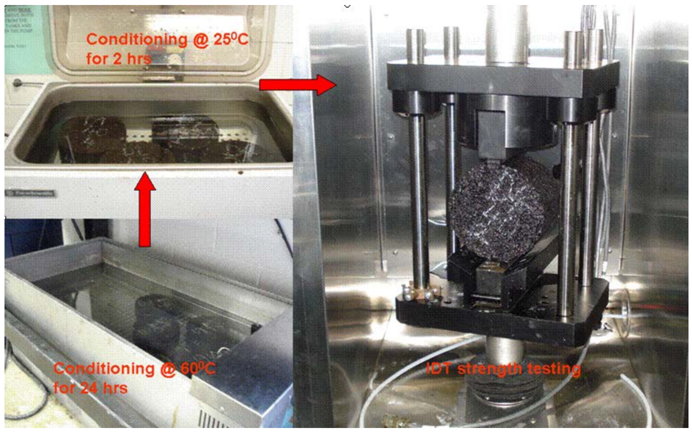


Figure 2-5. Moisture susceptibility test.

A load is applied to the specimen at the rate of 2 in. per minute and the maximum load is recorded. The maximum load is the indirect tensile strength of the specimen. Both the moisture-conditioned and the unconditioned specimens are tested. The average strengths obtained from the conditioned and the unconditioned specimens are used to determine the tensile-strength ratio (TSR). A minimum TSR of 80% is required by Superpave. Values lower than 80% indicate the potential for stripping problems after construction. Figure 2-5 shows the TSR testing equipment.

2.5 HMA CHARACTERISTICS

Mix designs will be performed for each candidate mixture to determine the optimum blends for each mixture type. Once the designs have been completed and the volumetric properties have been determined, complex modulus testing will be performed to compare each candidate mixture and to make sure it is durable and strong.

The complex modulus test is performed by measuring the recoverable vertical strain when sinusoidal (haversine) vertical loads are applied to the specimen at different frequencies. The test procedure follows AASHTO TP2 or ASTM D3497. The axial stress and strain are used to calculate the complex modulus by dividing the repeated stress by the repeated strain. The main purpose of this test is to describe the viscoelastic behavior of asphalt materials under axial loading. The complex modulus test is shown in Figure 2-6.

Figure 2-6. Complex modulus test.

3 PAVEMENT FRICTION

Pavement–tire friction (or, simply, pavement friction) is one of the primary factors determining highway safety and, in particular, the probability of wet skidding crashes. In this chapter, a brief introduction is provided for pavement friction, friction mechanisms, and laboratory and field methods to measure the friction of the pavement, and factors affecting pavement friction are discussed.

3.1 PAVEMENT FRICTION

Friction is an important pavement parameter. Poor friction causes higher incidences of skid-related accidents, and its measurements can be used to evaluate various types of materials and construction practices. Friction is influenced by many factors: tire-related properties such as rubber compound tread design and condition, inflation pressure, and operating temperature; pavement-surface properties such as mixture type, microtexture and macrotexture, and surface temperature; and intervening-substance-related factors such as quantity of water, presence of loose particulate matter, and oil contaminants on the pavement surface (Luo 2003).

Pavement friction usually decreases as the pavement ages. This is due to two mechanisms: under traffic the aggregate polishes, which decreases the microtexture, and the aggregate wears, which decreases the macrotexture. This general trend is observed as pavements age and is the reason for conducting regular surveys (Henry and Kazuo 1983). Friction is also typically higher in the fall and winter (assuming snow and ice are not present) and lower in the spring and summer. This seasonal variation is quite significant and can severely skew skid resistance data if not compensated for (Jayawickrama et al. 1996).

Friction has two major components: adhesion and hysteresis. Adhesion results from the shearing of molecular bonds formed when the tire rubber is pressed into close contact with the pavement surface particles. Hysteresis results from energy dissipation when the tire rubber is deformed while passing across a rough pavement surface. These two components of friction are related to the two key properties of asphalt pavement surfaces—that is, microtexture and macrotexture. Microtexture refers to irregularities in the surfaces of the stone particles (fine-scale texture) that affect adhesion. These irregularities are what make the stone particles feel smooth or harsh to the touch. Macrotexture refers to the larger irregularities in the road surface (coarse-scale texture) that affects hysteresis. These larger irregularities are associated with voids between stone particles. The initial macrotexture on a pavement surface will be determined by the size, shape, and gradation of coarse aggregates used in pavement construction, as well as the particular construction techniques used in the placement of the pavement surface layer (Noyce et al. 2007).

The friction of a pavement surface varies with many factors including the porosity of the surface, wear, polishing, rutting, bleeding, and surface contamination (Haas et al. 1994). During the first 2 years after construction there are increases in friction due to wearing away of the surface asphalt (Corley-Lay 1998). Long-term friction is reduced as the exposed aggregate is worn and some of its microtexture and macrotexture properties are lost as traffic loads compact the HMA in the wheel paths (Anderson et al. 1986).

3.2 PAVEMENT TEXTURE

Studies performed in Europe have indicated that good pavement macrotexture decreases accidents in both wet and dry conditions (Henry 2000). The World Road Association (PIARC) Technical Committee on Surface Characteristics classified the texture of pavement surfaces in terms of relative wavelengths as shown in Figure 3-1(Descornet 1989).

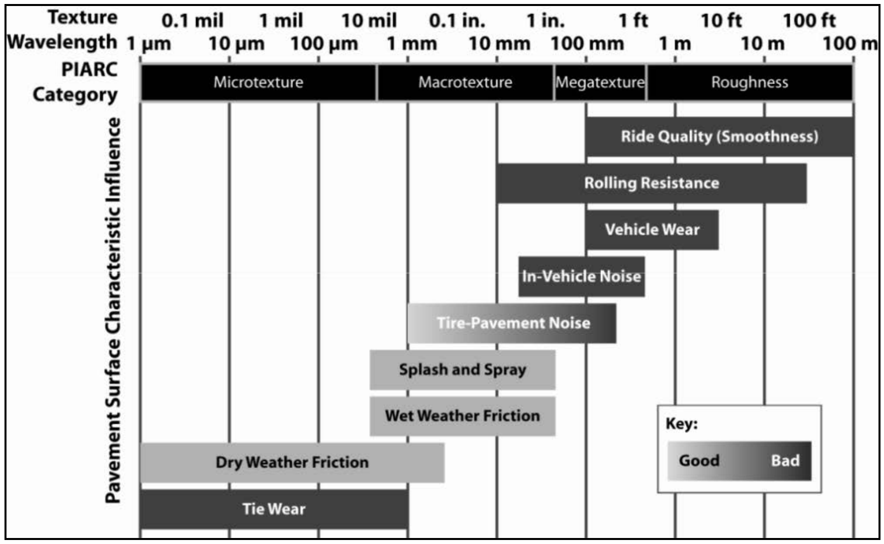


Figure 3-1. PIARC pavement surface characteristics (Rasmussen et al. 2004).

Studies performed in the United Kingdom concluded that macrotexture depth is influential in the high-speed friction of pavement surfaces and that when the macrotexture depth of the pavement was less than 1.0 mm (0.039 in.), there was a large decrease in the high-speed friction (Elsenaar et al. 1976). Additionally, previous experience has shown that pavements with a smooth texture (both low microtexture and low macrotexture) have the lowest skid numbers at any speed (Beaton 1976). Megatexture, a result of pavement surface deteriorations such as alligator cracking, spalling, plucking, and scabbing, is a major factor in pavement noise (Descornet 1989).

In 1992, PIARC conducted extensive tests with pavement friction and texture measurement devices. As a result of these tests, an International Friction Index (IFI) was proposed. The IFI is a harmonized index comprising a friction number (F_{60}) and a speed constant (S_p). The speed constant was found to be linearly related to macrotexture measurements, whereas the friction number is computed from both a friction measurement and the speed constant. The preferred macrotexture measure for the computation of the speed constant is the mean profile depth (MPD), for which both ASTM and the International Standards Organization (ISO) have developed standards (Wambold et al. 1995).

3.2.1 Pavement Macrotexture

The macrotexture of a pavement surface results from the large aggregate particles in the mixture. Macrotexture plays a major role in wet weather friction characteristics, especially at high vehicle speeds. Therefore, pavements that are constructed to accommodate vehicles traveling at speeds of 50 mph (80 kph) or greater require good macrotexture to help prevent hydroplaning (Hibbs, B., and Larson, R. 1996).

Macrotexture can be measured using laser profiling methods, volumetric methods, outflow meters, and other techniques. Traditionally, macrotexture measurements have been made using a volumetric test known as the sand patch test. The sand patch test, as shown in Figure 3-2, is used by many transportation departments in the United States and is a relatively simple test to perform. However, the results from this test are dependent upon the individual performing the test, and therefore are not very repeatable (Henry 2000). Currently, the specifications in ASTM E965 recommend the use of glass spheres because of the consistency of the particle shapes and the commercial availability of the spheres.

Vehicle-mounted laser profilers can now be used to collect pavement texture information while driving. The laser profiler consists of a laser attached to the front of the test vehicle, as shown in Figure 3-3. The laser emits a beam toward the pavement surface that travels to the pavement and is reflected back from the surface to the test vehicle. A receiver records the time for the light to travel from the test vehicle to the pavement and back. The distance of the traveled light is recorded and used to calculate the surface profile. The mean profile depth of the pavement surface can then be computed.

Figure 3-2. Sand patch test (Davis 2001).

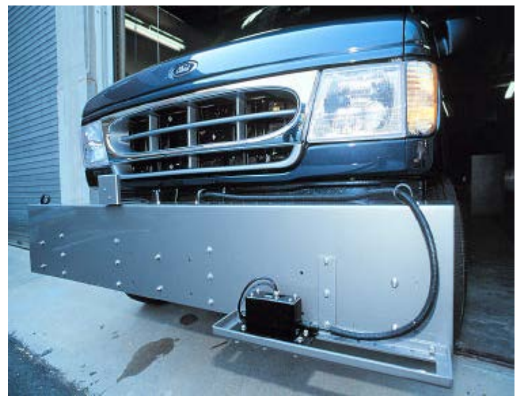


Figure 3-3. Laser texture measuring device (Davis 2001).

3.2.2 Pavement Microtexture

Good microtexture is necessary to provide adequate stopping on dry pavements at typical vehicle operational speeds and on wet (but not flooded) pavements when vehicle speeds are less than 50 mph (80 kph). When higher vehicle speeds are expected, good microtexture and macrotexture are generally required to provide adequate wet-pavement friction. Microtexture is not generally considered to be a factor in the development of pavement noise or splash and spray (Hibbs 1996). Microtexture, a function of the aggregate particle properties, is not measured directly in the field. Microtexture levels are commonly estimated using low-speed friction measurement devices (Wambold et al. 1995). The locked wheel skid trailer can be used to estimate the microtexture properties when testing is performed at low speeds.

3.3 PAVEMENT FRICTION MEASUREMENTS

3.3.1 Laboratory Tests for Friction

Laboratory methods have been developed for evaluating the friction of core samples or laboratory-prepared samples. The British pendulum tester (BPT), Japanese dynamic friction tester (DF-Tester), variable-speed friction tester (VSFT), and small-wheel circular track polish machine can be used for friction measurements in the laboratory.

3.3.1.1 British Pendulum Tester (BPT)

The BPT measures the energy loss when a rubber slider edge is propelled over a test surface. The tester is equipped with a standard rubber slider, as shown in Figure 3-4. During testing, the pendulum is raised to the locked point with a height that is adjusted so that the rubber slider is exposed to the pavement surface. When the pendulum is released and reaches the test surface, its potential energy becomes its maximum kinetic energy. As the rubber slider slides over the test surface, the friction reduces the kinetic energy of the pendulum in proportion to the level of friction. When the slider leaves the test surface, the reduced kinetic is converted to potential energy as the pendulum swings to its maximum height. The difference between the height before the release and the recovered height is equal to the loss of the kinetic energy due to the friction. The test method is covered in ASTM E303. The test result is reported as the British Pendulum Number (BPN). BPN is measured directly using a drag pointer. The greater the friction between the rubber slider and the test surface, the greater the BPN. The BPN depends primarily on the microtexture because the slip speed is very low.

Figure 3-4. British pendulum tester (BPT)
(www.highwaysmaintenance.com).

3.3.1.2 Japanese Dynamic Friction Tester (DF-Tester)

The DF-Tester, as shown in Figure 3-5, is a portable device for measuring friction. The test procedures are covered in ASTM E1911. The fundamental principle is Coulomb's friction law. This device consists of a horizontal spinning disk fitted with three spring-mounted rubber sliders. During testing, the disk is lowered so that the three sliders are in contact with the test surface under a constant force perpendicular to the test surface. The disk is driven by a motor and rotates at a tangential speed varying from 0 to 50 mph (80 kph), which is determined from the rotary speed of the disk. Water is delivered to the test surface by a water supply unit. The horizontal force required to overcome the friction is measured by a transducer. The test result is reported as the coefficient of friction and is plotted against the speed. The DF-Tester has the advantage of being able to measure friction over a range of test speeds. This enables users to create speed gradient curves quickly. Like the BPT, use of the DF-Tester in field pavement friction tests requires traffic control.

Figure 3-5.Japanese dynamic friction tester
(DF-Tester) (www.nippou.com).

3.3.1.3 *Variable-Speed Friction Tester*

The VSFT (shown in Figure 3-6) covers the measurement of friction for paved surfaces or laboratory-prepared specimens using the North Carolina State University variable-speed friction tester. The VSFT is a pendulum-type tester with a locked-wheel smooth rubber tire at its lower end. A stream of water at a selected water test velocity is directed by a nozzle along the specimen surface in the path of contact between the locked-pendulum tire and the specimen. The friction between the tire and the specimen is measured from the energy lost in the pendulum. The tester is suitable for field tests on pavement surfaces as well as laboratory use. The values measured, variable-speed number (VSN), represent the frictional properties obtained with the apparatus.

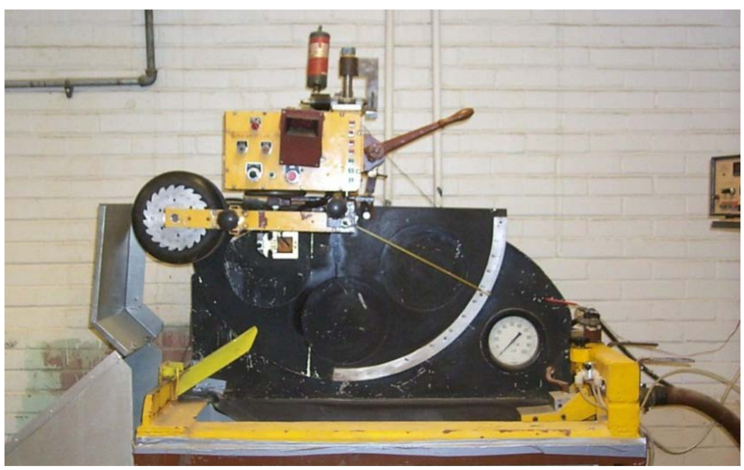


Figure 3-6. Variable speed tester (Illinois DOT).

3.3.4 *Small-Wheel Circular Track Polish Machine*

The skid resistance of aggregate is actually the polishing resistance of the aggregate. High speeds and high traffic volume can polish the aggregate, leading to a smooth surface and a reduction in the pavement surface friction. Aggregates specified for wearing courses for highways must possess some resistance to polishing. The polished stone value (PSV) is a measure of the resistance of the coarse aggregate to the polishing action of a vehicle's tires under conditions that are similar to those occurring on the pavement surface. The higher the polished stone value, the more resistant the aggregate is to polishing. The small-wheel circular track can accelerate polishing of highly textured surfaces. The small-wheel track polishing machine polishes the aggregate prior to determining the PSV of the stone. ASTM E660 is the method of accelerating the polishing of the aggregate in the laboratory. The small-wheel circular track polishing machine is shown in Figure 3-7.

Figure 3-7. The small-wheel circular track polishing machine (Illinois DOT).

3.3.2 Field Tests for Friction

There are four basic types of full-scale friction measuring devices: locked wheel, side force, fixed slip, and variable slip testers. Differences in friction measurements using the same device could be in the range of 5% between two consecutive measurements of the same road surface. Therefore, results from one device are not equivalent or directly comparable to those obtained with another device. Friction is also sensitive to the test tire (ribbed or smooth), and measurements can also differ from two tires of the same type. Additional variations also arise because of seasonal effects.

3.3.2.1 Locked-Wheel Testers

Locked-wheel systems produce a 100% slip condition. The relative velocity between the surface of the tire and the pavement surface (the slip speed) is equal to the vehicle speed. The brake is applied and the force is measured and averaged for 1 second after the test wheel is fully locked. Because the force measurement is continuous during the braking process, these systems usually can detect the peak friction. Locked-wheel testers simulate emergency braking conditions for vehicles without anti-lock brakes by dragging a locked wheel on a pavement wetted with a specified amount of water. Locked-wheel testers are usually fitted with a self-watering system for wet testing; a nominal water film thickness of 0.5 mm (0.02 in.) is commonly used. Figure 3-8 shows the ASTM locked-wheel tester (ASTM E274).

Figure 3-8. Locked-wheel tester (pavementinteractive.org).

Figure 3-9. Side-force tester (mu meter) (www.airport-technology.com).

3.3.2.2 Side-Force Devices

Side-force devices simulate a vehicle traveling through a curve. They function by maintaining a test wheel in a plane at an angle to the direction of motion (the yaw angle) while the wheel is allowed to roll freely. Side force is measured perpendicularly to the plane of rotation. The main advantage of this method is that it can measure friction continuously through a test section. Examples of specific side-force testing equipment include the mu meter, as shown in Figure 3-9, and the Sideways-Force Coefficient Routine Investigation Machine (SCRIM), both of which originated in the United Kingdom.

3.3.2.3 Fixed-Slip Devices

Fixed-slip devices operate at a constant slip, usually between 10% and 20%. The test wheel is driven at a lower angular velocity than its free-rolling velocity. This is usually accomplished by incorporating a gear reduction or chain drive of the test wheel drive shaft from the drive shaft of the host vehicle. In some cases, it is accomplished by hydraulic retardation of the test wheel. These devices also measure low-speed friction. Like the side-force method, the fixed-slip method can also be operated continuously over the test section without excessive wear of the test tire. An example of a fixed-slip tester is the Griptester shown in Figure 3-10. Most fixed-slip devices are designed to operate at only one slip ratio; however, the slip ratio can be varied on some fixed-slip devices. An ASTM standard for fixed -lip devices is not currently available.

Figure 3-10. Fixed-slip tester (the Griptester) (www.tradewindscientific.com).

3.3.2.4 Variable-Slip Devices

Variable-slip devices sweep through a predetermined set of slip ratios. This is usually accomplished by driving the test wheel through a programmed slip ratio using a hydraulic motor. ASTM Standard E1859 has been developed for devices that perform a controlled sweep through a range of slip ratios. Some locked-wheel testers can operate in a mode that captures the friction as the test tire proceeds from free rolling to the fully locked wheel condition (0% to 100% slip). Locked-wheel testers can also be programmed to operate in accordance with ASTM E1337, in which the brake is released just after the peak is reached. A variable-slip Norsemeter ROAR is shown in Figure 3-11.

Figure 3-11.Variable-slip tester (the Norsemeter ROAR) (www.norsemeter.no).

3.3.3 Current Usage in the United States

AASHTO published guidelines for the design of skid-resistant pavements in 1976. However, there were no practical devices capable of measuring macrotexture at highway speeds at that time. With the development of high-speed laser devices capable of measuring macrotexture at speeds of 60 kph (100 mph) or more, it is now possible to include macrotexture measurements in routine surveys of the road network. There has been no effort to update the guidelines to include data currently obtainable since the AASHTO guidelines were issued in 1976 (Henry 2000).

The ASTM Standard Test Method E274, "Skid Resistance of Paved Surfaces Using a Full-Scale Tire" is used by 39 states and Puerto Rico; however, they do not all use the same type of tire (see Table 3-1). The use of the same test method, but with different standard test tires, could lead to very different strategies for providing skid-resistant pavements (Henry 2000). At low slip speeds, the effect of microtexture dominates the measurement, whereas at high slip speeds the effect of macrotexture becomes important. For this reason, fixed-slip and side-force measurements are usually accompanied by a macrotexture measurement (Leu 1978).

Table 3-1. Friction-Measuring Devices in Use by Agencies (Henry 2000)

State	Tester Type	Tire Type	Test Speed (kph)
Alaska	Locked-Wheel Tester	Ribbed Tire	65
Arizona	Mu Meter	Mu Meter	65
Arkansas	Locked-Wheel Tester	Ribbed Tire	65
California	Locked-Wheel Tester	Ribbed Tire	Posted Speed
Colorado	Locked-Wheel Tester	Ribbed Tire & Smooth Tire	65
Connecticut	Locked-Wheel Tester	Ribbed Tire	65
Florida	Locked-Wheel Tester	Ribbed Tire	65
Georgia	Locked-Wheel Tester	Ribbed Tire & Smooth Tire	65
Hawaii	Locked-Wheel Tester	Ribbed Tire	65
Idaho	Locked-Wheel Tester	Smooth Tire	65
Illinois	Locked-Wheel Tester	Ribbed Tire & Smooth Tire	65
Kansas	Locked-Wheel Tester	Ribbed Tire	65 & 90
Kentucky	Locked-Wheel Tester	Ribbed Tire	65
Louisiana	Locked-Wheel Tester	Ribbed Tire & Smooth Tire	65
Maine	Locked-Wheel Tester	Ribbed Tire	65
Maryland	Locked-Wheel Tester	Ribbed Tire	65
Michigan	Locked-Wheel Tester	Ribbed Tire	35 & 65
Minnesota	Locked-Wheel Tester	Ribbed Tire & Smooth Tire	65
Mississippi	Locked-Wheel Tester	Ribbed Tire	65
Missouri	Locked- Wheel Tester	Smooth Tire	65
Montana	Locked-Wheel Tester	Ribbed Tire	65
Nebraska	Locked-Wheel Tester	Ribbed Tire	65 & 80
New Jersey	Locked-Wheel Tester	Ribbed Tire	65 & 80
New Mexico	Locked-Wheel Tester	Ribbed Tire	80
New York	Locked-Wheel Tester	Ribbed Tire	65
North Carolina	Locked-Wheel Tester	Ribbed Tire & Smooth Tire	65
Oklahoma	Locked-Wheel Tester	Ribbed Tire	40–50
Oregon	Locked-Wheel Tester	Ribbed Tire	65
Pennsylvania	Locked-Wheel Tester	Ribbed Tire & Smooth Tire	65
Rhode Island	Locked-Wheel Tester	Ribbed Tire	65
South Carolina	Locked-Wheel Tester	Ribbed Tire	65

Table 3-1. Friction-Measuring Devices in Use by Agencies (Henry 2000) (cont.)

State	Tester Type	Tire Type	Test Speed (kph)
South Dakota	Locked-Wheel Tester	Smooth Tire	65
Texas	Locked-Wheel Tester	Smooth Tire	80
Utah	Locked-Wheel Tester	Ribbed Tire	65
Vermont	Locked-Wheel Tester	Ribbed Tire	40 & 65
Virginia	Locked-Wheel Tester	Smooth Tire	65
Washington	Locked-Wheel Tester	Ribbed Tire	65
Wisconsin	Locked-Wheel Tester	Ribbed Tire	65
Wyoming	Locked-Wheel Tester	Ribbed Tire	75
Denmark	ROAR	Smooth & Tread Tire	60
France	SCRIM	Avon	60
Hungary	SCRIM	Avon	50
Japan	Locked-Wheel Tester	165 SR 13	60–80
Netherlands	DWW Tester	PIARC Smooth	50
New Zealand	SCRIM	Avon	50
Poland	Polish SRT-3	Patterned	60
Portugal	SCRIM	PIARC Smooth	50
Switzerland	BV-8 & SRM	PIARC Ribbed	40, 60, & 80
United Kingdom	SCRIM	Avon	50 & 20

3.4 FRICTION AND HMA

3.4.1 HMA Characteristics

Many HMA design parameters influence pavement friction. Several studies have tried to determine the effects of the aggregate on friction, microtexture, and macrotexture of a paved surface. Other properties, such as asphalt content and void content, also affect friction. If an excessive amount of asphalt is incorporated into the pavement mixture, there is a tendency for bleeding of the pavement surface. Bleeding prevents aggregate from properly contacting the vehicle tire and causes large decreases in friction properties of the surface (Beaton 1976). High void content in the HMA allows for rapid drainage of the pavement surface, theoretically increasing the pavement friction by removing the water.

Open-graded mixes have higher friction values compared to dense- and gap-graded mixes because open-graded mixes have higher macrotexture. Mixes that are less than 8 years old generally provide satisfactory friction. Sections with higher truck traffic as well as high ESALs have lower microtexture values. Microtexture is sensitive to individual aggregate properties and seasonal changes. Microtexture is not affected by binder type and gradation properties. Therefore, there is no significant difference in microtexture values for different mix types. The increase in macrotexture values over time is greater for sections with higher air void content and coarser gradations (Aybike Ongel et al. 2008).

The primary advantage of stone mastic asphalt (SMA) is resistance to deformation, but it has been shown to have better frictional characteristics than traditional asphalt. SMA is gap-graded, dense asphalt cement concrete with a high percentage of coarse aggregate, typically 10 to 15 mm (0.4 to 0.6 in.). The mix contains a high percentage of mineral filler, and modified asphalt and/or fibers are often used to prevent draindown of the binder. As a result of the aggregate gradation, SMA has excellent macrotexture (Henry 2000).

3.4.2 Aggregate Properties

Numerous aggregate properties affect the friction of pavement surface mixtures. In 1976, a study performed in California suggested that there are four aggregate characteristics that are important, in terms of friction, in the design of pavement surfaces: texture, shape, size, and wear resistance (Beaton 1976). The gradation of the aggregate is also important in that larger aggregates incorporated into the mixture allow for increased surface projections, which in turn increase the contact between the tire and pavement. To allow for adequate friction over the lifetime of the pavement, it is important to ensure that the aggregate does not polish rapidly (Beaton 1976).

AASHTO recommends the use of specific types of aggregates to obtain optimum friction levels. According to AASHTO, the incorporation of blast furnace slag, expanded shale, slate, or lightweight aggregates into the HMA mixture provides adequate levels of friction. Additionally, AASHTO has determined that sharp silica sand and some types of granite provide good friction when used in mixtures (AASHTO 1976).

Research at Pennsylvania State University established characteristics of aggregate that would allow for adequate skid resistance over time. That study showed that, while a wear-resistant aggregate is desired in the mixture, some wearing of the pavement surface must occur in order to ensure good levels of friction. It was reiterated that angular particles performed better in terms of pavement friction than particles that were rounded (Henry and Dahir 1979).

The study established that the use of the polish value of the aggregate is adequate in predicting the long-term frictional properties of the pavement when used in conjunction with the soundness of the aggregate. When the soundness of the aggregate is not adequate, a high polish value may not be enough to ensure good frictional properties (Ongel 2008; Corley-Lay 1998; Descornet 1989; Luo 2003; Noyce et al. 2007). Kandhal (1998) confirmed the importance of aggregate texture properties to low speed friction. The angularity of the aggregate particles contributes to the friction of the pavement by creating points of contact between a road covered with a thin film of water and the tread rubber of the tire. Additionally, the idea that pavement macrotexture is a function of the aggregate gradation in the mixture was confirmed (Kandhal 1998).

4 PAVEMENT NOISE

In general, highway traffic noise is generated by three subsources of highway vehicles: vehicle unit noise, aerodynamic noise, and tire–pavement interaction noise. The vehicle unit noise refers to noise generated from the engine, exhaust, power train, and cooling system. Aerodynamic noise is generated from turbulent airflow around the vehicle in motion. Tire–pavement interaction noise is emitted from a rolling tire as it interacts with the pavement. As shown in Figure 4-1, the noise generated from tire–pavement interaction has been identified as primary noise source at speeds above 30 mph (50 kph) (Donavan 2007; Hibbs and Larson 1996; Sandberg 1979; Sandberg and Ejsmont 2002). In this section, the mechanisms and factors that may affect noise generation related to tire–pavement interaction and the methods to measure noise levels will be discussed.

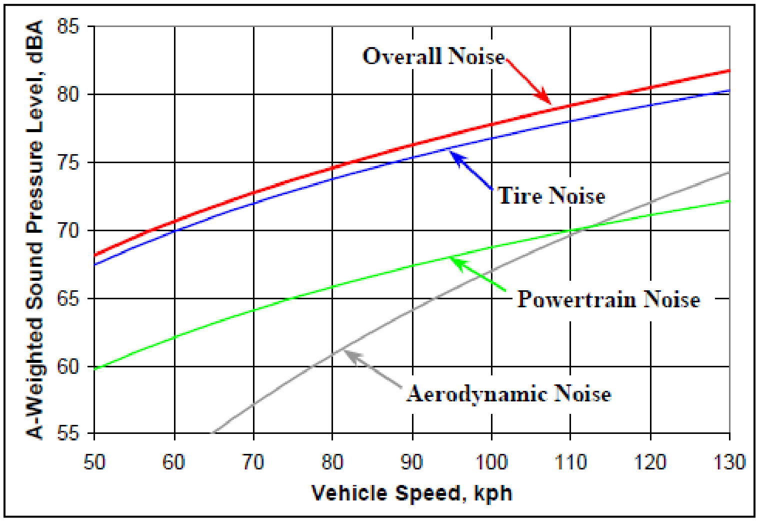


Figure 4-1. Contributions of the various subsources of highway traffic noise (Donavan 2007).

4.1 TIRE–PAVEMENT NOISE MECHANISM

In general, the noise caused by tire–pavement interaction can be categorized into two categories: mechanical vibration mechanisms and aerodynamic phenomena. Sandberg (2002) concluded that mechanical vibration mechanisms control the low-frequency noise and that the aerodynamic mechanisms control the high-frequency noise. The mechanical vibrations mechanism can be further divided into several submechanisms such as tread impact, texture impact, running deflection, and stick/slip mechanisms. The aerodynamic mechanism can also be categorized into air turbulence, air pumping, pipe resonances, and Helmoholtz resonance mechanisms. The detail description of each sound generation mechanism can be found in Sandberg and Ejsmont (2002).

4.2 PAVEMENT VARIABLES AFFECTING NOISE

As mentioned earlier, the major source of noise comes from the pavement–tire interaction. Therefore, pavement characteristics that affect tire–road noise will be briefly discussed. In general, six parameters could affect tire–pavement noise, including pavement texture, roughness, air voids content, age, and temperature.

Texture is the deviation of a road surface from a true planar surface with a maximum dimension (wavelength) of 0.5 m (World Road Association 1987). Texture can be controlled by aggregate size, aggregate shape, and aggregate gradation. Examples of pavement texture and roughness are shown in Figure 4-2. Texture is divided into three types: microtexture, macrotexture, and megatexture. Studies conducted on the effect of microtexture on tire–road noise levels have failed to detect any correlation (Sandberg and Ejsmont 2002). Descornet and Sandberg (1980) reported that sound pressure levels at low frequencies increase with increasing texture amplitudes for texture wavelengths from 10 to 50 mm, while at high frequencies sound pressure levels decrease with increasing texture amplitudes for texture wavelengths between 0.5 and 10 mm. Therefore, macrotexture does affect tire–pavement noise; however, the high and low frequencies are affected differently. Megatexture is a major contributor to tire–pavement noise. Increasing megatexture (the presence of distresses on the pavement surface) increases tire–pavement noise; therefore, megatexture should be removed to reduce noise levels (Morgan et al. 2003; Descornet and Sandberg 1980).

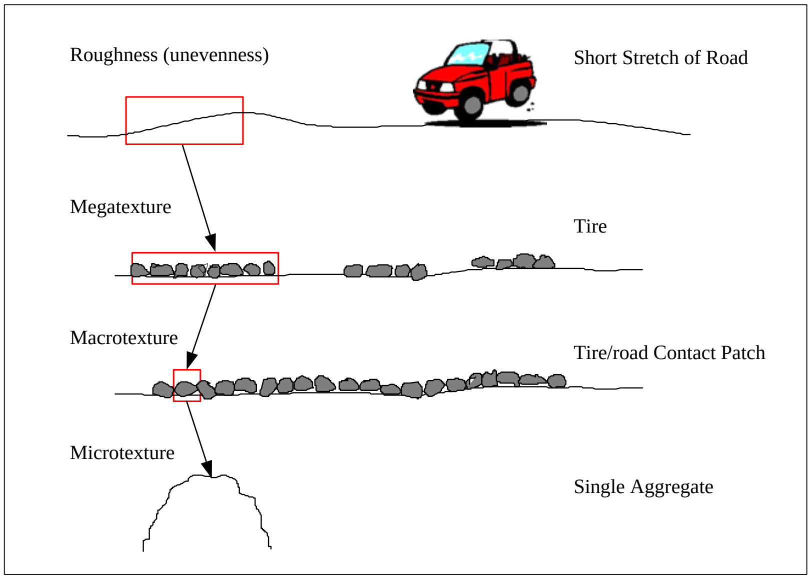


Figure 4-2. Pavement texture and roughness (Aybike Ongel et al. 2008).

Pavement roughness (unevenness) is the deviation of a road surface from a true planar surface with dimensions between 0.5 and 50 m (Sandberg and Ejsmont 2002). Higher roughness at lower-texture wavelengths (0.5 to 0.8 m) has been found to increase tire–pavement noise (Wayson 1998).

Mixtures with high air void content (porosity) can reduce generated noise levels. There are two noise reduction mechanisms in pavements with open-graded (porous) surfaces: noise absorption and noise propagation. With noise absorption, the presence of air voids in the surface layer helps dissipate trapped air in the tire's tread grooves. This results in reduced air pumping and therefore reduces noise emission (Nelson 1994). Porosity also gives the pavement surface acoustical absorption properties in which sound waves are dissipated into heat within the voids of the surface layer (Sandberg and Ejsmont 2002). With noise propagation, noise propagating from a sound source into a free surface attenuates as it travels farther from the source, and the rate of attenuation depends on the shape of the wave front (Nelson 1994). Therefore, the higher the air voids, the better the noise attenuation.

Noise levels of a pavement change over its service life. The rate of change of noise with time is different for different pavements (Sandberg and Ejsmont 2002). Age effects are mainly caused by texture, air void changes, and cracks formed on the surface due to traffic loads and climate effects (Sandberg and Ejsmont 2002). Studies have shown that asphalt concrete pavement noise levels increase with increasing age (Bendtsen 2002, 2008,; Hanson and James 2004; Nelson and Abott 1990).

Pavement and air temperatures also affect noise levels. Studies have found that increasing pavement temperatures decreases noise levels (Anfosso-Lédée and Pichaud 2007; Sandberg 2005). This decrease in noise is caused by the reduction in stiffness of tires and pavement with increasing temperature, and, hence, the reduction of contact stresses.

In summary, many studies have considered how pavement surface characteristics affect the overall tire–pavement noise levels as well as the frequency content of tire–pavement noise. Air void content was shown to be one of most important variables that determine the acoustical absorption coefficient and affect the noise levels.

4.3 METHODS FOR MEASURING TIRE–PAVEMENT NOISE

There are several approaches for measuring highway noise. In the field, the most common way is to measure the overall roadside noise level. It is now possible to measure pavement–tire noise directly on board a moving vehicle. Additionally, the noise potential of a particular mixture can be measured indirectly in the laboratory by characterizing the acoustic absorption properties of the mixture.

4.3.1 Direct Measurement of Roadside Noise Levels

The most common way to measure highway noise in the field is to measure the overall roadside noise levels in order to provide a useful determination of the noise impact on the receivers (i.e., the homes, businesses, and people experiencing the traffic noise from the nearby road). Direct measurement of roadside noise using calibrated Type I instruments (handheld or tripod-mounted sound pressure–level meters) is the standard by which environmental traffic noise is measured. However, “pass-by” testing requires extensive amounts of time and is labor intensive (requiring a 240-vehicle minimum sample). It will never be feasible or reasonable to perform the large number of these tests that would be required to evaluate an entire pavement network. Several methods to measure roadside noise levels include the statistical pass-by (SPB) method, control pass-by (CPB) method, and time-average traffic noise method.

SPB methods utilize a random sample of typical vehicles measured one at a time (ISO 819-1). The maximum sound-pressure level is captured for each pass-by using a sound measurement system such as a sound-level meter (SLM). SPB methods account for all aspects of traffic noise at the sideline of the highway, including engine, exhaust, and aerodynamic noises. The method also takes into account the variation that occurs across vehicles of the same type. However, the measurement is not tightly controlled since random vehicles will be involved at different sites. Therefore, the measurement site must be selected to avoid background noise, reflections, or terrain that might affect the measurement. In general, the background noise levels must be 10 dB(A) less than the measured vehicle noise.

For CPB measurements, the same measurement setup as SPB is used. For CPB measurements, relatively few selected vehicles are driven at a controlled speed past the measurement location. No standards currently exist in the United States for CPB, but the European Union is currently developing a method for EU standardization, and possibly for the ISO, based on a French national standard (NF S S1 119). Compared to the SPB method, the CPB method takes less time; however, it does not account for the variation that might occur in vehicles of the same type.

Under the condition of heavy traffic density, neither SPB nor CPB can be used to evaluate tire–pavement noise because vehicle pass-by's are not sufficiently isolated. Therefore, the time-average traffic noise method can be used so that the sound-pressure level is averaged and converted to the equivalent noise level. The method can be used only where background noise of the testing site is at least 10 dB(A) lower than traffic noise and there are no significant reflections or complex terrain.

4.3.2 Measurement of Noise Levels from a Moving Vehicle

With advanced technology, it is now possible to measure tire–pavement noise directly in a moving vehicle. The systems are either mounted on the vehicle itself or on a towed trailer. Within the trailer systems, there are open, or "free-field," trailers and enclosed trailers. The most well-known systems are the close-proximity method (CPX) and the on-board sound intensity (OBSI) system.

The CPX method was developed with the measurement focused on the tire–pavement interaction noise. Details of the CPX measurement procedure are described in ISO/CD 11819-2 (ISO 11819-2). The measurement is taken on a trailer, as shown in Figure 4-3, using microphones located near the tire. The trailer includes a hood over the microphones so that wind noise is reduced and noise from other traffic is reflected. Thus, this measurement can be made in the traffic stream.

Figure 4-3. Close-proximity trailer (Maher 2004).

The OBSI system provides a more sophisticated measurement of sound than sound pressure. Intensity is the sound power per unit area and is generally a smoother function of position than of sound pressure. The OBSI system uses the industry-standard sound intensity measurement technique adapted for a moving vehicle. The intensity probe is mounted near the tire, as shown in Figure 4-4. The measurement can be made in the traffic stream at normal traffic speeds.

Figure 4-4. OBSI jig being attached to wheel
hub on test vehicle (Trevino and Dossey 2003).

4.3.3 Estimation of Pavement Noise Characteristics from Material Specimens

The noise potential of a particular mixture can be estimated indirectly by measuring the acoustic property of a specimen in the laboratory. A standard method (Crocker and Hanson 2004) for evaluating noise absorption characteristics of materials used in many fields is the standing wave method. This technique has been standardized by the ISO for use in determining the acoustical properties of road surface materials. The sound absorption of a roadway core sample can be checked by mounting it at the end of a specially designed impedance tube. A loudspeaker mounted at the end of the tube emits white noise (sound with different frequencies combined). The sound waves produced by the loudspeaker propagate along the tube and are reflected or absorbed by the sample. Two microphones that are flush mounted in the impedance tube wall measure the resulting sound field in the tube. The signals from the microphones are used to calculate the sound absorption coefficient of the roadway core sample. Figure 4-5 is a schematic of the impedance tube built by the National Center of Asphalt Technology (NCAT) for noise studies. Early work with this tube indicates that the use of the equipment has promise (Crocker and Hanson 2004).

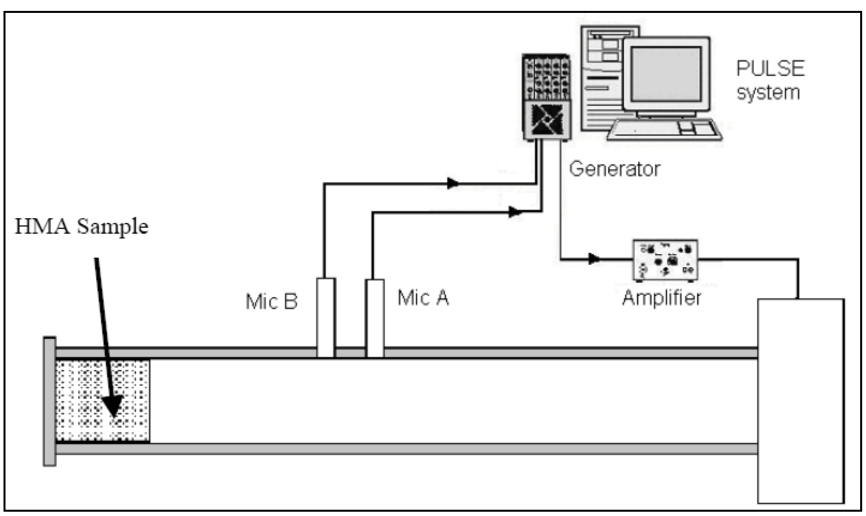


Figure 4-5. Experimental setup of sound absorption of
HMA samples (Crocker and Hanson 2004).

5 MATERIALS IN ILLINOIS

The IDOT Standard Specifications for HMA Surface Layers are provided in Appendix B.

5.1 AGGREGATE DISTRIBUTION IN ILLINOIS

The IDOT standard specifications allows sand, stone sand, chats, slag sand, and steel slag sand for fine aggregate; and gravel, crushed gravel, crushed stone, crushed sandstone, crushed air-cooled blast furnace slag (ACBFS), and crushed steel slag for coarse aggregate used in surface layers. Limestone is not allowed in E and F surface mixes; however, it may be used in C surface mixes and also in D surface mixes when blended with other aggregates. Dolomite is not allowed in F surface mixes, but it may be used in C and D surface mixes and also in E surface mixes when blended with other aggregates.

Limestone and gravel are well distributed in Illinois. Recycled materials such as steel slag and ACBFS, reclaimed asphalt pavement (RAP), and recycled concrete material (RCM) have potential to be used in the surface layer in terms of cost efficiency. Gravel is relatively expensive, roughly twice as expensive as limestone. Table 5-1 shows the aggregates commonly used or produced in Illinois.

Table 5-1. Aggregates Used in Illinois (IDOT)

District	Limestone	Dolomite	Gravel	Sandstone	Steel Slag	Air-Cooled Blast Furnace Slag
1		X	X		X	
2		X	X		X	
3	X	X	X		X	
4	X	X	X		X	
5	X		X		X	
6	X		X			
7	X		X	X		
8	X		X			X
9	X		X	X		
Produced in or imported from	South of I-80	North of I-80, Wisconsin	Statewide (Peoria, Indiana)	District 9, Ohio River	Northwest Indiana, Peoria, St. Louis	Northwest Indiana, St. Louis

5.2 MATERIAL COST INFORMATION

5.2.1 Aggregate Price

The prices for some selected aggregates that meet the IDOT requirements for HMA are shown in Table 5-2. The gravel price is roughly twice that of typical stone, while the price for RAP shows that it can be a cost-efficient alternative. The price of recycled materials such as steel slag and ACBFS will also need to be considered in the cost analysis of the candidate mixtures. This table will be refined as the project progresses, based on the actual materials used and the cost fluctuations over time.

Table 5-2. Aggregate Prices in Illinois ($/ton, Effective January 1, 2009)

Aggregate Type	Company A	Company B	Company C
CA11	12.95	12.95	
CA16	12.95	9.75	
CM11		11.25	11.25
CM16		9.75	13.00
FA20		7.55 (Columbia only)	
FM20			7.5
Gravel	22.5		
RAP (surge stone)		8.1	
RAP (shovel run stone)		7.7	
Shipping Cost			
Shipping($/mile)			2.85

5.2.2 Asphalt Binder Price

The fluctuation in price for PG 64-22 from January 2008 to April 2009 is shown in Figure 5-1.

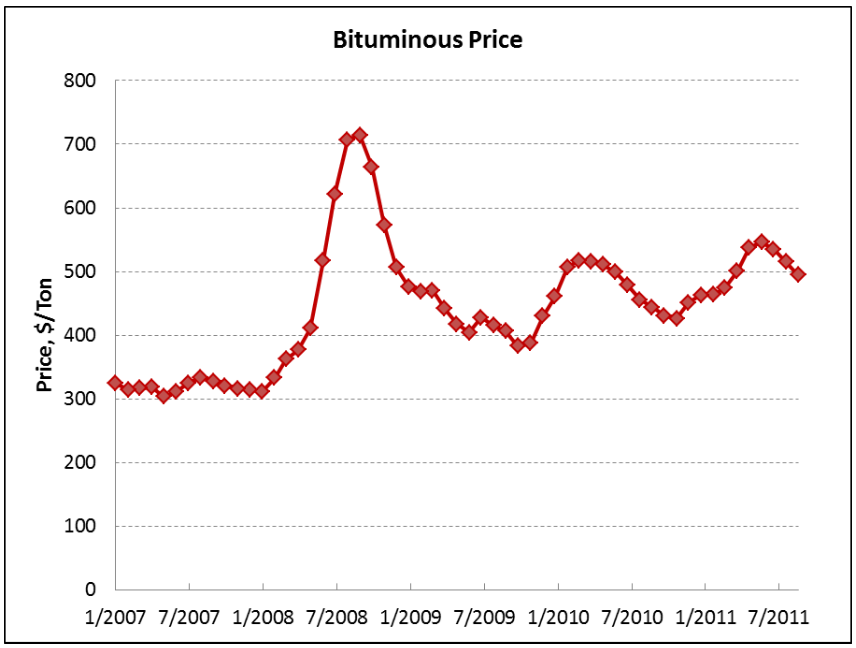


Figure 5-1. Bituminous (PG 64-22) price ($/ton) (www.dot.il.gov/desenv/asphaltpi.html).

Asphalt prices (valid only between January and March 2009) are shown in Table 5-3. Polymer modified asphalt is more expensive than virgin asphalt binder, and the price of emulsified asphalt varies depending on the type. The price of recycled rubber or natural rubber should also be considered for the cost analysis for the candidate mixtures. This table will be refined as the project progresses, based on the actual materials used and the cost fluctuations over time.

Table 5-3. Asphalt Prices (Valid Only Between January and March 2009)

Binder	Company D ($/ton)	Company E ($/ton)
Virgin Binder		
PG 64-22	500	400
PG 46-28	560	
PG 58-22	525	410
Polymer Modified		
SBS PG 70-22	600	
SBS PG 70-28	640	
SBS PG 64-28	630	
Emulsions / Cutbacks		
CRS 2	481 (573)*	
HFE 90, 150	481 (573)*	
HFRS 2	481 (573)*	
HFE 300	516 (630)*	
HF-P	561 (653)*	
MC-30	745 (859)*	
SC 3000	688 (859)*	
*Price if fewer than 3,000 gallons are purchased		
Shipping Cost	Freight (24 ton min.) can be estimated at $2.00 per mile plus fuel surcharge or $ 85/hr (Emulsicoat Inc.)	

6 CANDIDATE MIXTURES FOR WEARING SURFACES

Several candidate mixtures with good friction, durability, and cost efficiency are suggested in this appendix. The current surface mixes used in Illinois are shown as control mixes to compare with the candidates mixes. The candidate mixes are categorized into three groups:

- Better-performing mixtures (friction, durability)
- Cost-efficient mixtures (recycled materials)
- Innovative mixtures

6.1 CURRENT SURFACE MIXTURES IN ILLINOIS

6.1.1 Stone Mastic Asphalt (SMA)

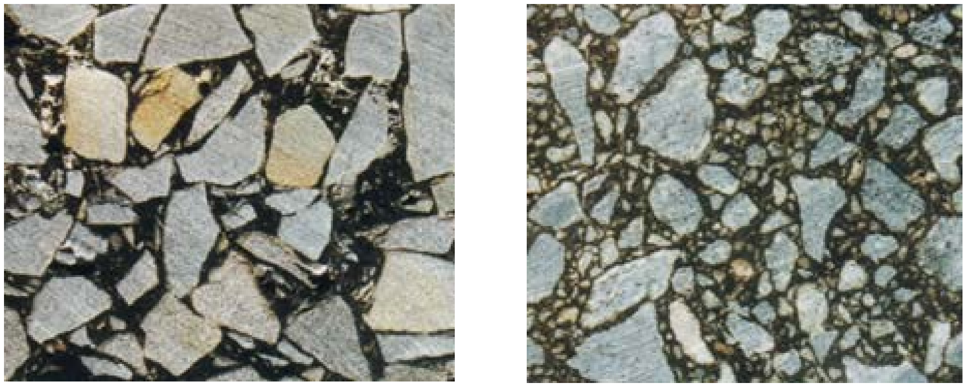
6.1.1.1 Description

Stone mastic asphalt is a gap-graded HMA surfacing material. This mixture was developed for high resistance to the wear of studded tires over 25 years ago in Germany. SMA consists of coarse aggregate and mastic that consists of crushed rock fines, filler, bitumen, and fibers. The combination of these two components increases stability and durability of the pavement. SMA can provide an extremely rut resistant and durable HMA mixture because of the stone-to-stone aggregate structure (Qiu and Lum 2006).

6.1.1.2 Properties

More stone-to-stone contact of the coarse aggregate in SMA results in a very durable mixture with high resistance to deformation. SMA also combines good acoustic properties with good durability, good friction, and resistance against mechanical forces. The rutting resistance of SMA stems from a coarse stone skeleton that provides more stone-to-stone contact than in conventional dense-graded asphalt mixes, as shown in Figure 6-1. Improved binder durability is a result of higher asphalt content, a thicker asphalt film and, lower air void content. The high asphalt content in SMA also improves the flexibility of the mixture. The addition of a small quantity of cellulose or mineral fiber prevents drainage of asphalt during transport and placement. Noise is decreased by a maximum 2 dB(A) at 50 kmh with a stone size of 6 mm maximum. Finer or coarser stone sizes leads to less noise reduction.

The surface texture characteristics of SMA are similar to open-graded mixtures, and the noise level in SMA is lower than that of dense-graded asphalt mixes; however, it is equal to or slightly higher than the noise levels of open-graded asphalt mixes. SMA can be produced and compacted with the same plant and equipment used for conventional HMA, and it can be used where open-graded asphalt is unsuitable. SMA reduces reflection cracking from underlying cracked pavements due to the flexible mastic. The durability of SMA is equal or greater than dense-graded asphalt mixes and is significantly greater than open-graded asphalt mixes.



(a) (b)

Figure 6-1.SMA and dense-graded HMA mixes: (a) stone mastic asphalt (SMA) and (b) dense-graded asphalt (David 2004).

SMA increases material costs related to higher binder and filler contents, and fiber additives. The increased mixing time required to add extra fillers could result in reduced productivity. Possible delays opening to traffic may occur because the mixes need to be cooled to 40°C to prevent flushing of the binder to the surface. Additionally, the initial friction of SMA may be low until the thick binder film is worn off the top of the surface by traffic. In critical situations, a small and clean grit may need to be applied to the surface prior to opening to traffic.

6.1.2 Dense-Graded HMA

6.1.2.1 Description

Dense-graded HMA is a well-graded mixture, and it can be classified as either fine-graded or coarse-graded. A fine-graded mix has smaller particles than a coarse-graded mix. When properly designed and constructed, a dense-graded mix is relatively impermeable. Dense-graded mix works very well for structural, frictional, leveling, and patching needs for all pavement layers and for all traffic conditions. Dense-graded mix is the most common asphalt mix used in wearing courses and structural layers. It is also used as pavement surfacing for parking lots, bike paths, and walking paths.

6.1.2.1 Properties

The main advantage of dense-graded mixtures over other mixture types is a lower initial cost. Another advantage is that most contractors and producers are familiar with them. However, the disadvantage is that dense-graded mixtures use high asphalt contents. Relatively low amounts of asphalt are typically used in dense-graded mixtures, and this makes the mixtures more susceptible to cracking and more permeable. Dense-graded mixtures can be generally designed for high rutting resistance or high crack resistance but not both. Dense-graded mixtures are not designed to have a stone-to-stone skeleton. The strength and stability is derived primarily from the quality of the intermediate and fine aggregate. Coarser mixes are sometimes dryer and more difficult to compact, more permeable, and more susceptible to segregation. The low texture of dense-graded mixtures can also affect wet weather friction depending on aggregate type, size, and mineralogy.

6.2 BETTER-PERFORMING MIXTURES

6.2.1 HMA with Trap Rock (Diabase)

6.2.1.1 Description

Diabase is a fine- to medium-grained intrusive igneous rock (see Figure 6-2). Chemically and mineralogically, diabase has a similar composition to basalt but is slightly more coarse grained. Diabase is extremely hard and tough, and it is commonly quarried for crushed stone, under the name of trap.

Figure 6-2. Trap rock (diabase)
(www.plaistedcompanies.com)

6.2.1.2 Properties

When the wearing surface is composed of trap rock, the binding material should be composed either of trap rock screenings or a mixture of trap rock screenings and sand. Trap rock is not as elastic as limestone; however, it is more durable and requires much more rolling during construction (Tillson 1912). Table 6-1 shows the properties of trap rock from a previous study (Cooley and Brown 2003).

Table 6-1. Properties of Coarse and Fine Trap Rock Aggregate (Cooley and Brown 2003)

Coarse Aggregate Properties		
Property	**Test Method**	**Value**
Bulk Specific Gravity	AASHTO T85	2.973
Apparent Specific Gravity	AASHTO T85	3.021
Absorption, %	AASHTO T85	0.7
Los Angeles Abrasion, % Loss	AASHTO T96	17
Flat or Elongated Particles	ASTM D4791	
2 to 1		54
3 to 1		15
5 to 1		1
Soundness, % Loss	AASHTO T104	1.1
Crushed Content, %	ASTM D5821	
One Face		100
Two Face		100
Fine Aggregate Properties		
Property	**Test Method**	**Value**
Bulk Specific Gravity	AASHTO T84	2.919
Apparent Specific Gravity	AASHTO T84	3.001
Absorption, %	AASHTO T84	1
Soundness, % Loss	AASHTO T104	1.1
Angularity, %	AASHTO TP33	48.3
Liquid Limit, %	AASHTO T89	*
Plastic Limit, %	AASHTO T90	NP

*Liquid limit could not be determined.
NP: Non-plastic

6.2.2. Sprinkle Treatment

6.2.2.1 Description

Sprinkle treatment is a surface application of pre-coated aggregate chips with high friction to HMA immediately behind the paver where both are then rolled for embedment and compaction. This treatment has been used successfully in England and in several states in the U.S.

Its first use in Illinois was on Route 185, a two-lane pavement with average daily traffic (ADT) of 1,300 to 1,600 in September and October 1980. A 3-in. binder course and a 1.5-in. surface course with the sprinkle treatment were placed on 8.8 miles of pavement. The construction was done by the following process: The aggregate chips were coated with 1.3% of the same asphalt cement as the HMA at 300°F in a batch plant. An anti-stripping agent in the amount of 0.5% by weight of the asphalt was used. The pre-coated chips were then stockpiled 1 to 9 days before construction. Spreading of the chips behind a paver was done with a Bristowes spreader. Three different spreading rates (6 lb/yd^2, 9 lb/yd^2, and 12 lb/yd^2) were used to compare the surface friction of each rate. For breakdown and chip embedment, a two-wheel vibratory roller was used. Further rolling was done with a tandem roller. The chip embedment was generally good; however, poor embedment was shown at a few locations with a heavy chip application, a thin mat, or perhaps a combination of the two.

6.2.2.2 Properties

Costly aggregate chips with high friction are required; however, the HMA mixture can contain a lower quality and less expensive aggregate. A technical report from IDOT (1981) summarized the properties of the sprinkle treatment as follows:

- Sprinkle treatments are an effective means of providing pavements with high quality frictional properties.
- Fine material in the sprinkle treatment aggregate is detrimental. A coarser, one-size aggregate yields the best sprinkle applications.
- Sprinkle aggregates should be produced from hard, durable materials with a history of good frictional properties.
- Good coating of the sprinkle aggregate is achieved more consistently in batch plants than in drum plants.
- Sprinkle treatments result in a substantial increase in macrotexture.
- Sprinkle treatments may result in a monetary savings in the construction of pavements where special aggregates are required to help ensure durable friction characteristics.
- Sprinkle treatments conserve high quality aggregate.

The IDOT report also contains results of friction tests for the various chip application rates, as shown in Table 6-2.

Table 6-2. Friction Number by Chip Application Rate (IDOT 1981)

Chip Application Rate (lb/yd^2)	Friction Number
6	35
9	36
12	38

Low friction numbers appeared to be attributed to an asphalt film not yet worn away by tire–aggregate contact and to a tendency of the hard but fine-grained trap rock chips to become oriented with their smooth flat faces upward and the sharp edges rotated away from tire contact. The texture values are shown in Table 6-3.

Table 6-3. Texture Values by Sand-Patch Method (IDOT 1981)

Chip Application Rate (lb/yd^2)	Texture Value (in.)
6	0.033
9	0.046 (good)
12	0.062 (excellent)

6.3 COST-EFFICIENT MIXTURES

6.3.1 HMA with Reclaimed Asphalt Pavement (RAP)—Low RAP and High RAP

6.3.1.1 Description

RAP is the removed and/or reprocessed pavement materials containing asphalt and aggregates. These materials are obtained when asphalt pavements are removed for reconstruction, resurfacing, or to obtain access to buried utilities. When it is properly crushed and screened, RAP consists of high quality, well-graded aggregates coated by asphalt cement, as shown in Figure 6-3.

Figure 6-3. HMA cores from a RAP mix and a non-RAP mix (www.pavementinteractive.org).

For asphalt pavement removal, either milling or full-depth removal is possible. Milling machines can remove up to 2 in. in a single pass. A rhino horn on a bulldozer and/or pneumatic pavement breakers can be used for full-depth removal by ripping and breaking the pavement. The broken material is picked up and loaded into haul trucks by a front-end loader and transported to a central facility for processing. The RAP is crushed, screened, conveyed, and stockpiled. In terms of cost efficiency, the higher the cost of asphalt binder, the more RAP is worth. RAP contains about 5% asphalt binder that replaces new and more expensive asphalt binder.

6.3.1.2 Properties

The properties of RAP largely depend on the properties of the constituent materials and the type of asphalt (surface or binder). There can be substantial differences between asphalt concrete mixes in aggregate quality, size, and consistency.

The RAP may give off gaseous hydrocarbons when heated. To reduce these emissions, HMA plants generally heat RAP indirectly by adding RAP after the aggregate is heated, therefore heating the RAP through contact with the already hot aggregate. Longer HMA heating times are required for RAP addition, and this process can reduce the output by as much as half. RAP generally contains 3% to 7% asphalt by weight, or about 10% to 20% asphalt by volume. Generally, RAP will be more viscous than new HMA due to asphalt binder aging. Therefore, when enough RAP is added, a softer asphalt binder should be used. Table 6-4 shows the AASHTO MP2 Superpave asphalt binder selection guidelines for RAP mixtures.

In general, state DOTs allow more RAP in base courses than in surface courses. The degradation that occurs during removal and processing makes RAP finer than pure virgin aggregate.

Table 6-4. Superpave Asphalt Binder Selection Guidelines for RAP Mixtures (AASHTO 2001)

RAP Content (%)	Recommended Virgin Asphalt Binder Grade
<15	No change from basic Superpave PG binder requirements
15–25	Select virgin binder one grade softer than normal (e.g., select at PG 58-22 if a PG 64-22 would normally be used).
>25 (High RAP)	Follow recommendations from blending charts.

The unit weight of milled or processed RAP is dependent on the type of aggregate in the reclaimed pavement and the moisture content of the stockpiled material. The moisture content of the RAP generally increases while in storage, and when exposed to rain the crushed or milled RAP may pick up a considerable amount of water. Moisture contents up to 5% or higher have been measured for stored crushed RAP (Smith 1980). During periods of extensive precipitation, the moisture content of some RAP stockpiles may be as high as 7% to 8% (Decker and Young 1996). Therefore, lengthy stockpiling of RAP should be kept to a minimum.

The asphalt cement adhering to the aggregate is somewhat harder than new asphalt cement because of oxidation during use and weathering. Oxidation of aged asphalt causes the oils to convert to resins and the resins to convert to asphaltenes, which are more viscous than either resins or oils and play a major role in determining asphalt viscosity, resulting in age hardening and a higher viscosity binder (Noureldin and Wood 1989).

The definitive study on the effects of RAP on binder quality was done under the National Cooperative Highway Research Program, Project 9-12, “Incorporation of Reclaimed Asphalt Pavement in the Superpave System.” The final report includes the statement, “Use of RAP has proven to be economical and environmentally sound. In addition, mixtures containing RAP have, for the most part, been found to perform as well as virgin mixtures” (NCHRP 2000).

6.3.2 HMA with Recycled Concrete Material (RCM)

6.3.2.1 Description

RCM consists of high quality and well-graded mineral aggregates bonded by a hardened cementitious paste. The aggregates comprise approximately 60% to 75% of the total volume of concrete. RCM is obtained through the demolition of portland cement concrete roads, runways, and structures during road reconstruction, utility excavations, or demolition operations. The RCM excavation may include 10% to 30% subbase soil material and asphalt pavement. Therefore, the RCM is not pure 0ortland cement concrete, but a mixture of concrete, soil, and small quantities of bituminous concrete.

Recycled concrete materials have been used through the United States as aggregates for various courses, as shown in Figure 6-4. However, because of poor bonding with asphalt binder, RCM is not recommended for wearing surfaces that required higher durability and high friction.

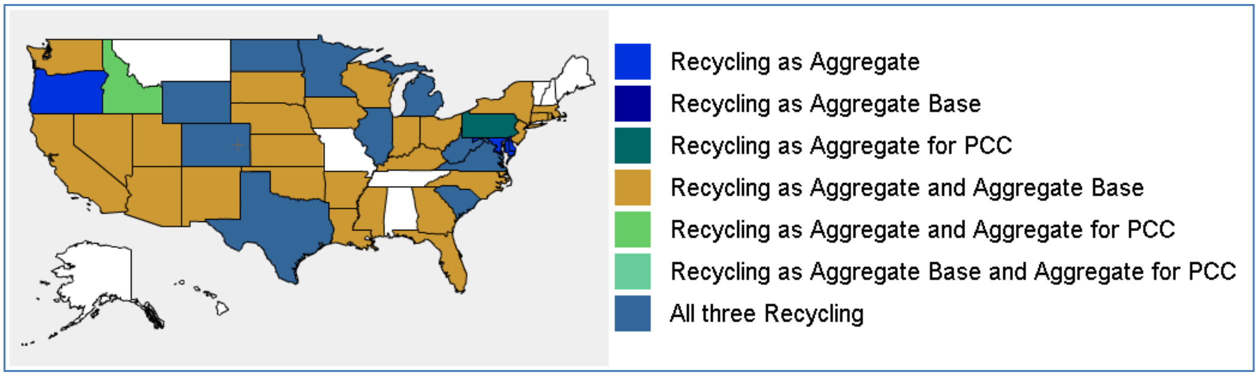


Figure 6-4. Recycled concrete in the United States (www.recycledmaterials.org).

6.3.2.2 Properties

The recycled concrete aggregate is derived from crushing portland cement concrete, producing a range of grain sizes from about 40 to 50 mm and smaller. The crushed aggregate consists of pieces of the original gravel with a cement mortar coating. The mortar attached to the aggregate causes the concrete aggregate to have weaker bond strength, lower abrasion resistance, lighter unit weight, and greater water absorption compared to its constituent mineral gravel.

RCM is rougher and more absorbent than its virgin constituents because it is composed of highly angular conglomerates of crushed quality aggregate and hardened cement. Furthermore, different sources of concrete mixes and uses result in varying aggregate qualities and sizes; for example, pre-cast concrete is less variable than cast-in-place (Chesner et al. 1998).

The physical characteristics of recycled concrete make it a viable substitute for aggregate, and it can be used as such in granular bases and for fill, such as riprap. Ultimately, RCM obtained on site may be employed immediately for construction or stockpiled for future use. The cementitious component contains a high amount of alkalinity by nature, and chlorides from deicing salts may be present. RCM may also have aggregates susceptible to alkali–silica reactions or D cracking (Chesner et al. 1998).

Illinois DOT allows the use of RCM as a coarse aggregate in aggregate surface courses, granular embankments, stabilized bases, and subbase courses provided the project material specifications are not compromised. This material has also been widely used as aggregates in membrane waterproofing and in drainage layers for protection against erosion (IDOT 2002; Schutzbach 1993).

6.3.3 HMA with Steel Slag

6.3.3.1 Description

Steel slag is a by-product of steel making and is produced during the separation of the molten steel from impurities in furnaces. The slag occurs as a molten liquid and is a complex solution of silicates and oxides that solidifies upon cooling. Steel slag can be processed into a coarse or fine aggregate material for use in dense- and open-graded hot-mix asphalt and in cold-mix or surface-treatment applications. Special quality-control procedures and appropriate processing of steel slag are extremely important when selecting steel slag for use in asphalt concrete pavements. Because of the potential expansion due to free lime or magnesia in the slag, particular consideration should be required, which could result in pavement cracking if ignored. The use of steel slag in pavements should be limited to replacement of either the fine or coarse aggregate fraction but not both. Hot-mix asphalt containing 100% steel slag is susceptible to high void space and bulking problems due to the angular shape of steel slag aggregate. Mixes with high void space are susceptible to over-asphalting during production and subsequent flushing due to in-service traffic compaction.

6.3.3.2 Properties

The positive features of steel slag aggregates in hot-mix asphalt are good friction, stripping resistance, high stability, and resistance to rutting/plastic deformation. However, the use of unsuitable or improperly processed slag may result in performance problems. Moreover, hydration of the free lime or magnesia results in expansion and cracking of the slag particle. This reaction in pavements results in cracks or popouts. Ohio and Illinois reported problems relating to deterioration, raveling, and the coarse texture of hot-mix asphalt containing steel slag aggregate and placed some restrictions on steel slag use (Noureldin 1990).

The Recycled Materials Resource Center summarized the properties of mixtures with steel slag as follows:

- Specific Gravity: Because of the higher specific gravity (3.2 to 3.6) of steel slag, steel slag aggregate can be expected to yield a higher-density product compared to conventional mixes. Bulk relative densities are 15% to 25% greater than most conventional mixes.
- Durability: Steel slag aggregate is very hard and abrasion resistant and has very good durability with resistance to weathering.
- Moisture Content: The relatively rough surface texture (deep pores) of steel slag increases the susceptibility of the aggregate to differential drying and potential retention of moisture in the hot-mix asphalt. Moisture retention coupled with the presence of hydratable oxides could result in volumetric instability. To minimize drying requirements and the potential for hydration reactions, steel slag aggregate moisture contents should be limited to 5% prior to use in hot-mix asphalt. The moisture content of the steel slag aggregate after drying should be no greater than 0.1%.
- Absorption: Steel slag has somewhat higher absorption than conventional aggregate. This can result in an increased asphalt cement demand. Asphalt cement extractability in lab tests can be more difficult than for conventional aggregate.
- Frictional Properties: Several countries with experience using steel slag in hot-mix asphalt suggest that very satisfactory frictional resistance can be anticipated. The high frictional resistance, as well as the abrasion resistance of steel slag aggregate, is advantageous in applications where high wear resistance is required, such as industrial roads, intersections, and parking areas subjected to heavy traffic.
- Thermal Properties: Steel slag aggregates have been reported to retain heat considerably longer than conventional natural aggregates. Due to the heat retention of steel slag aggregates, hot-mix asphalt repair can be performed in colder weather with steel slag aggregates than with conventional mixes.
- Stability: Steel slag aggregate mixes have very high stabilities—1.5 to 3 times higher than conventional mixes—with good flow properties.
- Stripping Resistance: Steel slag mixes typically exhibit excellent resistance to stripping of asphalt cement from the steel slag aggregate particles. Resistance to stripping is probably enhanced because of the presence of free lime in the slag.
- Rutting Resistance: The higher stability with good flow properties results in a mix that resists rutting. Rutting resistance is advantageous for highways, industrial roads, and parking areas subjected to heavy axle loads.

Asi (2007) performed a BPN friction test for different mixes, and the results showed that asphalt concrete mixes containing 30% slag have the highest skid number, followed by Superpave, SMA, and Marshall mixes, respectively (see Table 6-5).

Table 6-5. Friction Evaluation Results for Different Mixes (Asi 2007)

Sample no.	Trial No.	Marshall	Marshall +0.5%	Marshall +1%	Superpave	SMA	30% slag
1	1	92	84	77	100	93	102
	2	89	80	71	110	90	97
	3	97	77	70	97	90	100
	4	100	80	73	93	96	100
	5	80	81	75	92	94	100
2	1	80	82	72	90	94	100
	2	80	83	77	90	92	99
	3	85	84	70	98	93	99
	4	82	82	77	95	92	99
	5	87	80	77	92	90	100
Average		87.2	81.3	73.9	95.7	92.4	99.6
Standard deviation		7.3	2.2	3	6.1	2	1.3

6.3.4 HMA with Air-Cooled Blast Furnace Slag (ACBFS)

6.3.4.1 Description

ACBFS, one of various slag products, is formed when the liquid slag is allowed to cool under atmospheric conditions. It can later be broken down with typical aggregate processing equipment to meet gradation specifications (Chesner et al. 1998). Crushed ACBFS is angular, roughly cubical materials with textures ranging from rough, porous surfaces to smooth, shell-like fractured surfaces, as shown in Figure 6-5. A schematic of the production of ACBFS is shown in Figure 6-6.

Figure 6-5. Air-cooled blast furnace slag
(www.tarmac.co.uk)

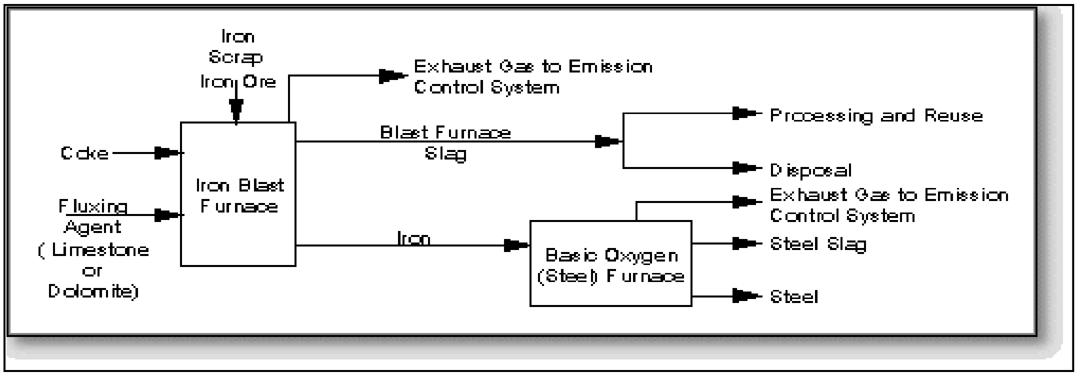


Figure 6-6. General schematic of blast furnace operation and production (www.tfhrc.gov).

6.3.4.2 Properties

Considerable variability in the physical properties of blast furnace slag depends on the iron production process. Though vesicular, the structure's cells are not interconnected, and little absorption to the interior is likely. Physical properties such as unit weight and size can vary considerably depending on the method of production. For instance, the high use of scrap iron can lead to higher unit weights (Chesner et al.1998; www.nationalslagassoc.org). Some blast furnace slag was reported to have a compacted unit weight as high as 1,940 kg/m^3 (120 lb/ft^3) (Schutzbach 1993). Higher unit weights are reported generally due to increased metal and iron content in the slag and tend to occur in slags that are generated from blast furnaces with higher scrap metal additions.

The water absorption of ACBFS can be as high as 6%. Although ACBFS can exhibit these high absorption values, it can be readily dried since little water actually enters the pores of the slag and most is held in the shallow pits on the surface.

6.3.5 Rubberized Gap-Graded HMA (Wet Process)

6.3.5.1 Description

Asphalt rubber gap-graded hot-mix asphalt is a surface course with a gap-graded aggregate gradation, which is referred to as ARHM-GG or RAC-G. The gap-graded HMA with the high viscosity of the binder allows a high binder content (7% to 9%). This makes a very flexible mix that is highly resistant to reflective cracking. RAC-G mixes have been used to address raveling, aging, reflective cracking, minor surface irregularities, bleeding, and load-associated cracking (California DOT 2005). Figure 6-7 shows an example of rubberized asphalt binder.

Figure 6-7. Rubberized asphalt binder (www.rubberpavements.org).

There are two processes to introduce the crumb rubber in asphalt mixtures: dry process and wet process. In the dry process, the crumb rubber is mixed together with the aggregates as an aggregate prior to addition of the asphalt. In the wet process, the crumb rubber is added to the preheated conventional binder (Pais and Pereria 2007). Asphalt rubber in the wet process is a chemically reacted mix of liquid asphalt binder with 15% to 22% crumb rubber obtained from used tires and added to liquid asphalt. It reacts at high temperatures prior to being mixed with aggregate. The asphalt rubber is described in ASTM D8-88. In the wet process, rubber may also be added as a powder (up to 15 %, typically 7 %) to modify the binder (rubberized asphalt). The wet process is sometimes used to improve binder quality in porous surfaces.

6.3.5.2 Properties

Until 2002 there was no conclusive evidence that the addition of small quantities of rubber to a bituminous wearing course would significantly reduce the noise of it (Sandberg and Ejsmont 2002). Donavan (2005) did a comparative measurement campaign both in the United States (Arizona and California) and Europe with his CPX-like measurement device based on sound intensity technique. He found noise levels for two-layer porous asphalt between 94.5 and 96.5 dB(A) and for RAC between 95.5 and 97.5 dB(A). RAC is a non-porous SMA-like wearing course with a thickness of 2.5 cm and containing from 8% up to 10% binder. RAC contains typically between 14% and 20% rubber by weight of the total asphalt–rubber mixture. Several advantages of rubberized gap-graded asphalt identified by Tom Kuennen (2004) are as follows:

- Improved resistance to surface-initiated cracking due to higher binder contents
- Improved aging and oxidation resistance due to higher binder contents
- Improved resistance to fatigue and reflection cracking due to higher binder contents
- Improved resistance to rutting due to higher viscosity and softening points
- Increased nighttime visibility due to contrast between the pavement and the striping
- Reduced tire noise due to increased binder film thickness and open texture
- Reduced splash and spray during rain storms due to open texture
- Reduced construction times because less material is placed
- Lower pavement maintenance costs due to improved pavement performance
- Better chip retention for chip seals due to thick films of asphalt
- Lower life-cycle costs due to improved performance
- Savings in energy and natural resources by using waste products

According to a Caltrans study (Aybike Ongel et al. 2008), the RAC-G has a lower noise than dense-graded asphalt. Most of the noise benefits of RAC-G come from its air void content, which is higher than that of dense-graded asphalt. However, RAC-G loses its permeability faster than the open-graded mixes, and hence their noise-reducing properties. Based on Caltrans statistics, the noise levels from RAC-G mixes appear to approach those of dense-graded asphalt within 4 years.

The structural performance of the rubberized asphalt is directly related to the mechanical properties of the asphalt rubber binder used. The binder content is around 7.5% to 9.5%, which has an important effect on the material performance, mainly in terms of fatigue response, where it is expected to have at least 10 times more fatigue life than a conventional asphalt mixture for which the binder content is about 5% (Minhoto et al. 2005). This behavior is attributed to the larger flexibility of the mixtures provided by incorporation of crumb rubber into the conventional binder.

In terms of permanent deformation, several studies indicate a satisfactory performance of asphalt rubber hot mixes compared to those produced with conventional binders, mainly due to the aggregate gradation used to produce the mixture and due the thickness of the layers where the mixture is placed (Antunes et al. 2000).

Structural equivalencies and reflection crack retardation equivalencies (thicknesses) used by Caltrans are shown in Table 6-6 and Table 6-7.

Table 6-6. Structural Equivalencies, Thickness (ft.)

Dense-Graded HMA	Rubberized Gap-Graded HMA	Rubberized Gap-Graded HMA on a Stress Absorbing Membrane Interlayer
0.15	0.10	-
0.20	0.10	-
0.25	0.15	0.10
0.30	0.15	0.10
0.35	0.20	0.15
0.40	0.20	0.15
0.45	0.15	0.20
0.50	0.15	0.20
0.55	0.20	0.15
0.60	0.20	0.15

Notes: The maximum allowable non-experimental equivalency for rubberized gap-graded HMA is 2:1. The minimum allowable rubberized gap-graded HMA lift thickness is 0.10 ft.

Table 6-7. Reflection Crack Retardation Equivalencies, Thickness (ft.)

Dense-Graded HMA	Rubberized Gap-Graded HMA	Rubberized Gap-Graded HMA on a Stress Absorbing Membrane Interlayer
0.15	0.10	-
0.20	0.10	-
0.25	0.15	-
0.30	0.15	-
0.35	0.15 or 0.20	0.10

Notes: The minimum allowable rubberized gap-graded HMA lift thickness is 0.10 ft. The dense-graded HMA thickness of 0.35 ft. is the maximum thickness recommended by Caltrans for reflection crack retardation.

Asphalt rubber mixtures have their principal application in the field of pavement rehabilitation, where reflective cracking resistance is essential. Over 30 years of application of these mixtures on cracked pavements has shown their capacity for retarding reflection crack propagation (Way 2000).

Harun summarized the performance of crumb rubber modified asphalt mixes in the United States in the 1970s and 1980s (see Table 6-8).

Table 6-8. United States Experiences in Crumb Rubber (Harun)

State	Remarks
Alaska	Pavement sections placed in 1979~1983 using dry process have superior fatigue resistance but were not as good as conventional in resisting raveling and pothole formation.
Arizona	The longest user of crumb rubber modified mixes, Arizona presently uses dense- and open-graded mixes made with asphalt rubber binders for overlays on existing rigid and flexible pavements.
California	After using crumb rubber for more than 20 years, California recommends: • Asphalt rubber open-graded mixes should no longer be considered an experimental technology. • Asphalt rubber dense- and gap-graded mixes should be used on an experimental basis. • Dry process using devulcanized rubber should not be used.
Connecticut	Based on 9-year performance study of asphalt rubber pavement produced using dry process, Connecticut concludes: • On thick overlays, 2% crumb rubber increase reflection cracking as compared with control sections. • On thin overlays, 1% crumb rubber reduces reflection cracking by two thirds. Increased crumb rubber contents result in more cracking.
Florida	All asphalt rubber dense- and open-graded sections performed well since 1989~1990. Beginning in January 1994, all dense- and open-graded friction courses require an asphalt rubber binder.
Kansas	Two experimental asphalt rubber dense-graded sections placed in 1990 showed more reflection cracking.
Michigan	Eight experimental sections constructed in 1978–1979 performed poorly in terms of reflection cracking and surface disintegration cracking. Michigan does not recommend the use of crumb rubber modified asphalt.
Mississippi	A test section with 6% devulcanized rubber showed little significant difference in crack pattern, friction, and rutting after 2 years, compared with the control section.
Oregon	After 5 years, rubber-modified section showed better resistance to cracking. However, raveling in the section was of concern.
South Dakota	Dry-process rubber modified sections developed some potholes and break-up after 1 year that subsequently developed into large areas of delamination and peeling.
Texas	Of two sections, one raveled shortly after construction while the other performed satisfactorily.
Utah	Dry-process rubber modified section was removed after 3 years because of severe raveling.
Washington	Five open-graded sections showed good to very good performance. Dry-process sections showed poor to average performance.

6.4 INNOVATIVE MIXTURES

6.4.1 Fiber Reinforced HMA

6.4.1.1 Description

Fibers have been used to improve the performance of asphalt mixtures against permanent deformation and fatigue cracking *(*Bueno et al. 2003; Lee et al. 2005*)*. Recent developments in fibershave made it possible to add fibers directly to the mixture and not to the asphalt binder (Kaloush 2008), as shown in Figure 6-8.

6.4.1.2 Properties

Little research has been reported on experiments using synthetic fibers with asphalt concrete. Bueno et al. (2003) studied the addition of randomly distributed synthetic fibers on the mechanical response of a cold-mixed, dense-graded asphalt mixture using the Marshall test, as well as static and cyclic triaxial tests. The results showed that the addition of fibers caused small variations in the mixture's triaxial shear strength parameters. Lee et al. (2005) evaluated the influence of recycled carpet fibers on the fatigue cracking resistance of asphalt concrete using fracture energy. They found that the increase in fracture energy represents a potential for improving asphalt fatigue life.

(a)

(b)

Figure 6-8. Fiber-reinforced HMA: (a) reinforcing fibers and (b) fiber-reinforced asphalt mixture (Kaloush 2008).

In Kaloush's study (2008), a mixture of polypropylene and aramid fibers (Table 6-9) was used in a field and laboratory study to evaluate the performance characteristics of the modified asphalt mixture.

Table 6-9. Properties of Reinforcing Fibers (Kaloush 2008)

Materials	**Polyolefin/Polypropylene**	**Aramid**
Form	Twisted Fibrillated and Monofilament Fibers	
Specific Gravity	0.91	1.44
Tensile Strength (psi)	70,000	400,000
Melting Temperature	212°F (100°C)	800°F (427°C)

The results of the laboratory test showed that the fibers improved the mixture's performance in several unique ways:

- The fiber-reinforced asphalt mixture showed better resistance to shear deformation. The fibers in the mix provided higher residual energy and a gradual drop in strength.
- Permanent deformation tests for the fiber-reinforced mixture showed lower permanent strain accumulation compared to the control. The fibers induced an extended endurance period in the secondary stage of the permanent deformation curve, and the gradual accumulation of permanent strain beyond tertiary flow. Both of these characteristics were attributed to the presence and mobilization of the fibers distributed in the mix.
- The dynamic modulus E* was higher for the fiber-reinforced mix. The difference between the two mixtures was less at the lowest temperature (20% increase) because of the

dominant effect of the binder and less contribution of the role of fibers. The largest difference was observed at 100°F (80% higher), where the reinforcement effect of the fibers was observed to be the highest. At 130°F, the increase in modulus was also substantial, at about 50%.

- The fatigue cracking test was different in that, unlike the other tests, the strain level was held constant. The fatigue life was higher for the control mixture at high strain values, while the fiber-reinforced mixture had higher fatigue life at lower strain values. The shift in predicted fatigue life suggests that the fiber-reinforced mix will perform better on roads where traffic speeds are higher. However, it was concluded that the fatigue cracking results are inconclusive and need further evaluation.
- The tensile strength and fracture energy showed that the fiber-reinforced mix exhibited the highest values at all test temperatures: an increase of 25% to 50% for the tensile strength, and 50% to 75% for the fracture energy. Generally, lower thermal cracking should be expected as tensile strength and fracture energy are increased.
- Relationships between crack growth rates and C* line integral values showed that the fiber-reinforced mix had about 40 times higher resistance to crack propagation than the control mix.
- No cracks were observed in the fiber-reinforced pavement sections, while a field condition survey after approximately 1 year revealed that there were a couple of low severity cracks, 1 to 2 ft long, in the control section.

6.4.2 Fiber Reinforced Warm-Mix Asphalt (WMA)

6.4.2.1 Description

Fiber-reinforced WMA has not been developed yet for actual field construction. The basic concept of fiber-reinforced WMA comes from fiber-reinforced concrete pavements. The raw fibers used in concrete pavements commonly have low melting temperatures which would not work with typical HMA temperatures.

Several different types of fibers (Figure 6-9) have been used in concrete pavements to reinforce the cement-based matrices, while the fibers used in HMA are limited due to temperatures. However, various types of fibers can be considered with the lower temperatures in WMA. The choice of fibers varies from synthetic organic materials such as polypropylene or carbon, synthetic inorganic such as steel or glass, natural organic such as cellulose or sisal to natural inorganic asbestos.

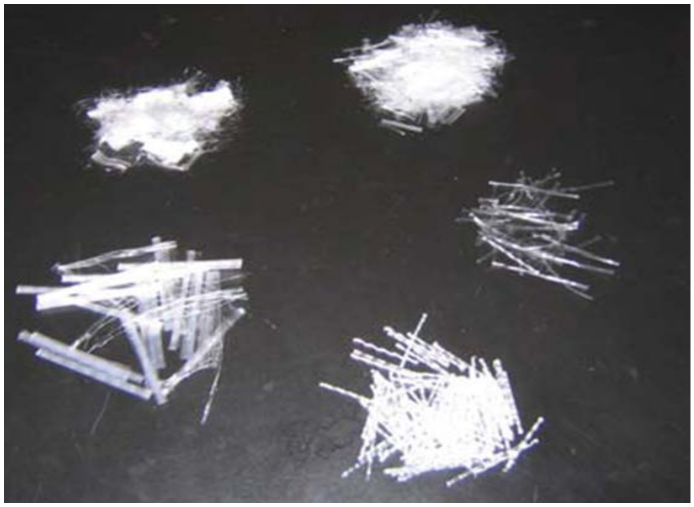


Figure 6-9. Synthetic fibers in a variety of shapes and sizes (www.acpa.org).

WMA, developed in Europe, has gained strong interest in the United States. By lowering the viscosity of asphalt binder and/or increasing the workability of mixtures using minimal heat, WMA technology allows mixing, transporting, and paving at significantly lower temperatures compared to conventional HMA. Typically, WMA is defined as asphalt mixtures for which the plant mixing temperature range is from 212°F to 275°F (100°C to 135°C). Using a WMA process, mixes can be produced at temperatures as much as 37°C (100°F) lower than traditional methods, as shown Figure 6-10.

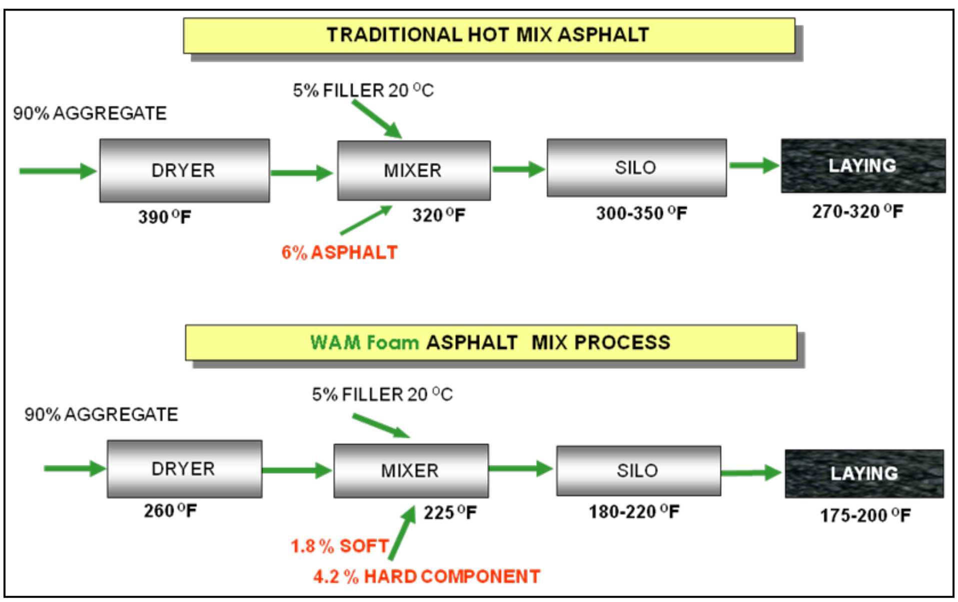


Figure 6-10. Schematics of the traditional HMA and WMA process (Roberts 2007).

6.4.2.2 Properties

The mixture properties are similar to the fiber-reinforced HMA suggested above. For fiber-reinforced HMA, special fibers with a higher melting temperature are needed to prevent them from melting during the mixing process. However, for WMA, the fibers commonly used in concrete pavements are available due to the low temperature mixing process.

The benefits of fibers are shown in the fiber-reinforced HMA section; the additional advantages of WMA are as follows:

- Significantly lower production and placement temperatures
- Less aging of binder during plant mixing and placement, thus improving longevity of pavement service life
- Reduced thermal segregation in the mat
- Less fuel/energy consumption, thus lowering fuel/energy costs
- Decreased emissions/odors from mixing plant and during placement
- Decreased dust production due to lower temperatures and shorter heating time
- Extended paving season (i.e., paving during cooler weather)
- Extended mix haul distance (due to less difference between ambient temperature and mix temperature), thus providing expanded market areas and decreased mobilization cost
- Facilitates compaction, which is beneficial for stiff mixes, RAP mixes, low-temperature paving, and reducing compaction effort
- Faster construction of pavements made of deep lifts of asphalt (e.g., intersections, which need to be opened as soon as possible; less time is required to cool the mix before the next lift is placed)
- Improved working conditions for plant/paving crew
- Improved thin-lift capabilities (i.e., lower cooling rate from maximum temperature or lower compaction cessation temperature)
- Quicker opening to traffic for some WMA products (a particularly important factor for some airports)
- Easier permitting for plant sites in urban areas

6.4.3 Fine Dense-Graded Hot-Mix Asphalt (HMA)

6.4.3.1 Description

According to the Bailey method’s definition, a fine dense-graded mixture is the mixture that has a volume of coarse aggregate less than the loose unit weight condition (Vavrik 2002). Therefore, the fine fraction carries most of the load because the fine dense-graded mixture does not have enough coarse aggregate particles to form a skeleton and the coarse fraction is spread apart and floating in the fine fraction.

In the Bailey method, the chosen unit weight is selected as a percentage of the loose unit weight of coarse aggregate. The fine-graded mixture should have the chosen unit weight less than 90% of the loose unit weight, as shown in Figure 6-11. Because the coarse aggregate particles do not touch each other in the fine-dense graded mixture, VMA is controlled by the fine aggregate. As the chosen unit weight of the coarse aggregates decreases, the volume of fine aggregate increase, which results in an increase of VMA.

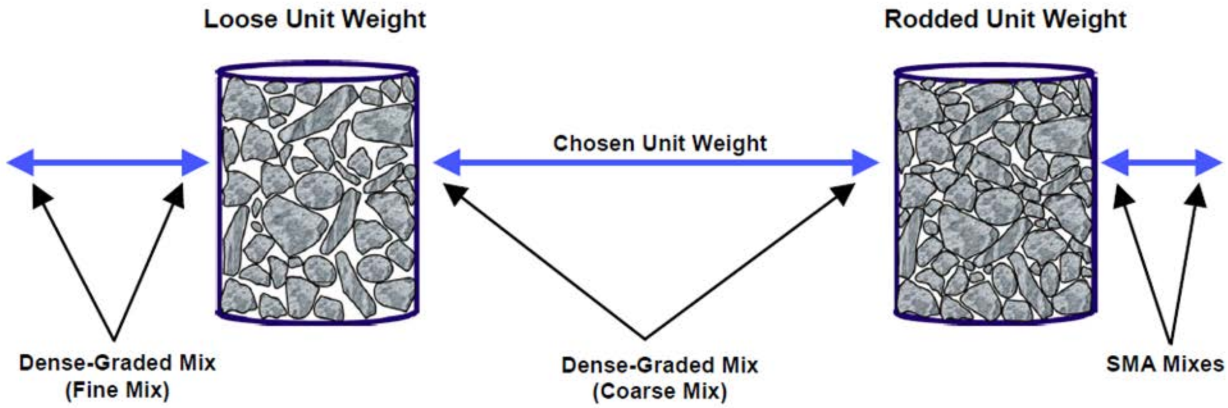


Figure 6-11. Selection of chosen unit weight of coarse aggregates (Vavrik 2002).

6.4.3.2 Properties

NCHRP Synthesis (284) suggests fine-graded surface mixes as alternative mixtures and showed several advantages and disadvantages. The fine-graded mixes have lower initial costs because of higher natural sand content. The fine texture and high sand content in this mixture make it easy to place and easy to compact with a smooth finish. Handwork is easy and blends in well without leaving surface blemishes. The smooth surface texture with small aggregates yields less distortion of the tires around the aggregate particles, reducing tire vibrations and noise.

However, the fine-graded mixes are generally less rut resistant than conventional mixes. The high natural sand content creates a weak aggregate skeleton. There is also a lower hydroplaning threshold because the surface texture is very fine. The macrotexture of this mix does not provide an escape route for the water. It requires good aggregates for rut resistance and frictional properties. An example of a fine dense-graded mixture is shown in Figure 6-12.

Figure 6-12. Fine dense-graded HMA (9.5 mm) (www.pavementinteractive.org).

Kandhal and Cooley (2002) compared coarse-graded mixtures with fine-graded mixtures in terms of resistance to rutting to determine whether restrictions on gradation type, either coarse- or fine-graded mixtures, are justified. Three different rutting susceptibility tests (Asphalt Pavement Analyzer, Superpave shear tester, and repeated load confined creep test), indicated that no significant differences in rut potential occurred between the two gradation types in all three performance tests.

6.5 ADDITIONAL NON-CANDIDATE MIXTURES

Many mixture types were reviewed during the literature review. The mixtures identified as candidates for this study were selected based on their feasibility for use in Illinois and their respective costs. The mixes that were reviewed but not considered feasible to include in this study are briefly described as follows.

6.5.1 Resin-Bound Surface Dressing

Resin-bound surface dressings consist of a layer of resinous binder densely spread and covered with small size aggregates with a high polish stone value. Crushed natural rock and artificial aggregates such as milled steel slag can be used as chippings (Descornet et al. 2000). This surface dressing has good durability, high friction, and very quiet surface noise due to the smoothening of the megatexture by the liquid resin and the macrotexture of the fine array of small aggregates. However, this surface dressing is very expensive.

6.5.2 Microlayers

Microlayers have the advantages of porous asphalt and stone mastic asphalt. Due to the porous nature of the pavement, it has low splash and spray, and the durability is comparable to SMA. The noise reduction is comparable to porous asphalt.

6.5.3 Single-Layer Porous Asphalt

Single-layer porous asphalt consists of gap-graded aggregate and polymer modified binder to form a matrix with interconnecting voids through which water can pass. It has a high stone content of 81% to 85%, a high void content of 18% to 22%, and a thickness of about 1.5 in. The difference between this mix and SMA is that SMA usually has 3% to 6% air voids. Single-layer porous asphalt requires high quality aggregate to provide good durability and high friction. Due to the high air void content, it provides good drainage and low noise. The main problem is water freezing in the pavement in the winter when the voids are clogged. Raveling has also been a serious problem on porous asphalt pavements in European countries.

6.5.4 Two-Layer Porous Asphalt

Two layer porous asphalt consists of a 1.5- to 2-in. sublayer of coarse-graded porous asphalt and a 1-in. wearing course with a fine aggregate on the top of the sublayer. To prevent the fine aggregates on the top layer from leaving, a binder with very high viscosity is needed. The top layer prevents coarse dirt from entering into the pavement and offers acoustic advantage due to the porosity and the fine surface texture. However, two-layer porous asphalt can have freezing problems in the winter due to clogging. Raveling is also a serious problem.

7 SUMMARY

A comprehensive literature review was performed to examine various asphalt mixtures and their feasibility for use in Illinois. The factors affecting HMA durability, friction, and noise were also reviewed. Various testing procedures were considered to compare the properties of the asphalt mixtures.

At this point, the list of candidate mixtures will be reviewed with IDOT to select the final group of mixes that will be studied. Table 7-1 provides a summary of the candidate mixtures and their general properties. Table 7-2 provides information on the general advantages and disadvantages of each candidate mixture. Table 7-3 is intended to be used in discussion with IDOT while finalizing the mixture selection for the project and will be used to weigh the advantages and disadvantages of each mixture.

Once the final mixtures are selected, mix designs will be performed, and the volumetric properties of each mix will be determined. The control mixtures will be obtained from IDOT and will include existing SMA and dense-graded mixtures that are currently in use in Illinois. The complex modulus will be performed to compare each mixture. Durability testing will include the Cantabro loss test to compare the mixtures for raveling potential and moisture susceptibility testing (AASHTO T283) to compare the mixtures for stripping potential. The variable-speed friction tester (VSFT) and the small-wheel circular track polishing machine will be used to compare the friction and surface polishing potential for each mixture.

Table 7-1. Mixture Properties of Candidate Surface Mixtures

Mixture Type	Gradation	Binder	NMAS (mm)	Thickness (mm)	Air Voids (%)	Binder Content (%)	Durability	Friction	Life Cycle (years)
Current Surface Mixtures in Illinois (The Noted Properties are General and Not Specific to Illinois Mixtures)									
Stone Matrix Asphalt (SMA)	Gap-graded (nearly 75% with coarse agg. high quality and crushed agg.)	Modified binder stabilized with cellulose fiber	9.5–19	30–100	6 (4–8)	More than 5	Good	Good (good in wet conditions as well)	10–12
Dense-Graded HMA	Dense-graded	Asphalt binder or modified binder	9.5–19 (generally 9.5)	19–75	4 (5–9)	3–7	Good	Good	8–11
Better Performing Mixtures (Higher Friction, Higher Durability, or Higher Resistance to Cracking and Rutting)									
HMA w/ Trap Rock (Diabase)	Dense-graded (possibly SMA and fine-graded)	Asphalt binder or modified binder	9.5–12.5	Variable	Depends on trap rock content	Depends on trap rock content	Very good	Superior friction	
Sprinkle Treatment	IDOT: chips (66% between ¾-in. and ½-in. sieves, 32.5% between ½-in. and No. 4 sieves)	Same as basic mixes (IDOT: coated with 1.3% of asphalt)	Coarser one sized agg. yields the best sprinkle applications (high quality agg.: trap rock, steel slag, air-cooled furnace slag)	Depends on basic mixes (IDOT: 1.5-in. surface, binder course)	Depends on basic mixes	Depends on basic mixes	Anti-stripping agent needed	Superior friction	
Cost Efficient Mixtures									
HMA w/ Reclaimed Asphalt Pavement (RAP) (Low RAP / High RAP)	Dense-graded (10–35% RAP by IL spec., gap- or single- graded RAP is not allowed)	Asphalt binder (one grade down for mixes with more than 15% RAP)	9.5–12.5	Variable	4 (5–9)	3–7	Lower RAP % results in better performance	Same as dense-graded	
HMA w/ Steel Slag	Dense-graded (possibly SMA and fine-graded)	Asphalt binder or modified binder	9.5–12.5	19–37.5	4	4.5–6 (depends on steel slag content)	Very good	Superior friction	

Table 7-1. Mixture Properties of Candidate Surface Mixtures (cont.)

Mixture Type	Gradation	Binder	NMAS (mm)	Thickness (mm)	Air Voids (%)	Binder Content (%)	Durability	Friction	Life Cycle (years)
<u>HMA w/ Air-Cooled Blast Furnace Slag</u>	Dense-graded (possibly SMA and fine-graded)	Asphalt binder or modified binder	9.5–12.5	Variable	Depends on slag content	Requires more binder due to high surface absorption	Very good	Very good	
<u>HMA w/ Recycled Concrete Material (RCM)</u>	Dense-graded				4–5 (depends on RCM content; Wong 2006)	5–7 (depends on RCM content; Wong 2006)	Not good for surface course (poor bond with asphalt)		
Rubberized Gap-Graded HMA (Wet Process)	Gap-graded (possibly SMA)	Rubberized binder (wet process) 15% rubber at Caltrans	12.5	30–60 at Caltrans (half of dense-graded HMA and 2~3 times MAS)	3–6 (4 at Caltrans)	7-9 at Caltrans	Good	Good	Longer than conventional mixes
Innovative Mixtures									
Fiber-Reinforced HMA (w/ Steel Slag)	Dense-graded (possibly SMA and fine-graded)	Asphalt binder or modified binder	Variable	Reduced up to 35% of dense-graded HMA	Variable (7 at Arizona State Univ.)	Variable	Very good	(Very high friction due to steel slag)	Longer than conventional mixes
Fiber-Reinforced WMA (w/ Steel Slag)	Dense-graded (possibly SMA and fine-graded)	Asphalt binder with WMA additive	Variable (9.5 at NCAT test)	Reduced up to 35% of dense-graded HMA	Variable (4.7 at Florida WMA)	Variable (5.6 at Florida WMA)	Very good	(Very high friction due to steel slag)	Longer than conventional mixes
<u>Fine Dense-Graded HMA</u>	Fine dense-graded	Asphalt binder or modified binder	High quality fine aggregate	Same as dense-graded HMA	Same as dense-graded HMA	More than conventional coarse-graded mixes due to fine aggregate	Good (needs more asphalt binder)	Good (requires good aggregate)	

Note: Underlined mixtures were suggested by IDOT.

Table 7-2. Advantages and Disadvantages of Candidate Surface Mixtures

Mixture Type	Advantages	Disadvantages
Current Surface Mixtures in Illinois		
Stone Matrix Asphalt (SMA)	• Excellent rut resistance and crack resistance due to stone-to-stone skeleton • Good wet weather friction due to coarser surface texture • Lower tire noise • Good durability • Used at intersections and other high traffic stress situations	• Increased material cost associated with higher binder and filler contents and fiber additive • Requires higher quality aggregates • Requires a significant compactive effort • Initial friction may be low until the thick binder film is worn off of the surface by traffic
Dense-Graded HMA	• Lower initial cost • Most contractors and HMA producers are generally familiar with the production and placement of dense-graded mixtures	• Cannot accommodate high asphalt contents without becoming unstable and susceptible to rutting • Relatively low amounts of asphalt are typically used in dense-graded mixtures, which in turn makes them more susceptible to cracking and more permeable
Better Performing Mixtures (Higher Friction, Higher Durability, or Higher Resistance to Cracking and Rutting)		
HMA w/ Trap Rock (Diabase)	• Very hard aggregate	• Relatively expensive (must be imported from other states)
Sprinkle Treatment	• Very high friction • Can be cost effective if the basic mixture allows use of a lower quality and less expensive aggregate	• Requires very high quality aggregates • Cost of sprinkle treatment is 16% over conventional treatments
Cost-Efficient Mixtures		
HMA w/ Reclaimed Asphalt Pavement (RAP)	• Cost efficient • Environmentally responsible • Performs well	• Need to control RAP variability to meet production tolerances • Difficult to identify the optimum RAP content
HMA w/ Steel Slag	• Superior friction due to its angularity • Generally low cost because it is a by-product of the steel-making process; however, this may not be true if the material is not locally available • Mixed with limestone (cost effective) • High shear resistance and rutting resistance • Available to pave during colder weather because steel slag retains heat longer than conventional aggregates	• Volume expansion due to the hydration of free lime or magnesia in the slag
HMA w/ Air-Cooled Blast Furnace Slag	• High resistance to polishing and weathering • Low cost because it is a by-product of the steel-making process • More stable than steel slag	• Considerable variability in the physical properties depending on the iron production process • Lower thermal conductivities than conventional aggregates because of a more porous structure
HMA w/ Recycled Concrete Material (RCM)	• Cost efficient due to recycled material	• Poor bond with asphalt binder • Potential stripping or wearing problems
Rubberized Gap-Graded HMA (Wet Process)	• Increased resistance to reflection cracking and rutting • Good surface friction • Can be used as a structural layer • Half the thickness of dense-graded HMA • Reduces maintenance costs • Decreases HMA stiffness at low temperatures which resists thermal cracking • Increases pavement life • Decreases noise levels (5dB(A)) • Beneficially uses 500–2,000 scrap tires per lane mile	• Difficult to control rubber quality • High cost for wet process ($16/ton more than conventional mixes) • Needs more compaction than dense-graded HMA

Table 7-2. Advantages and Disadvantages of Candidate Surface Mixtures (cont.)

Mixture Type	Advantages	Disadvantages
Innovative Mixtures		
Fiber-Reinforced HMA (w/ Steel Slag)	• High friction due to steel slag • High resistance to rutting and cracking due to both steel slag and fibers • Reduces thickness up to 35% compared to conventional mixes • Cost-effective due to good performance with less asphalt thickness • Extended life • Improves tensile strength, resilient modulus, and stability	• Requires special fibers with high melting point • Fiber cost is $6/lb and the mixture needs 1lb of fiber per ton of mixture
Fiber-Reinforced WMA (w/ Steel Slag)	• There are many choices of fibers with a low melting temperature • High friction due to steel slag • High resistance to rutting and cracking due to both steel slag and fibers • Reduces thickness up to 35% compared to conventional mixes • Cost-effective due to good performance with less asphalt thickness • Extended life • Reduced emissions, fuel/energy usage (25~30%) • Allows paving in colder temperatures • Able to incorporate higher percentages of RAP • Able to open to traffic in a short time	• Increases initial cost due to the WMA additive however the fuel cost is reduced • May reduce tensile strength due to more water remaining in WMA than in HMA (increased moisture susceptibility)
<u>Fine Dense-Graded HMA</u>	• Low initial cost—a higher natural/local sand content in this mixture results in a less expensive mix. • Easy to construct—the fine texture and high sand content in this mixture makes it easy to place and easy to compact with a smooth finish; handwork is easy and blends in well without leaving surface blemishes. • Smooth surface texture with small aggregate—less distortion of the tires around the aggregate particles reduces tire vibrations resulting in lower noise.	• Less rut resistant than other mixes—the high natural sand content creates a weak aggregate skeleton. • There is a lower hydroplaning threshold because the surface texture is very fine. The macrotexture of this mix does not provide an escape route for the water. • Requires good aggregates for rut resistance and frictional properties • Requires more binder

Note: Underlined mixtures were suggested by IDOT.

Table 7-3. Comparison of Candidate Mix Properties

Mixture Type	Friction	Thin	Durability	Cost Effective
Current Surface Mixtures in Illinois				
Stone Matrix Asphalt (SMA)				
Dense-Graded HMA	Control	Control	Control	Control
Better-Performing Mixtures (Higher Friction, Higher Durability, or Higher Resistance to Cracking and Rutting)				
<u>HMA w/ Trap Rock (Diabase)</u>				
<u>Sprinkle Treatment</u>				
Cost-Efficient Mixtures				
HMA w/ Reclaimed Asphalt Pavement (RAP) (Low RAP / High RAP)				
HMA w/ Steel Slag				
<u>HMA w/ Air-Cooled Blast Furnace Slag</u>				
<u>HMA w/ Recycled Concrete Material (RCM)</u>				
Rubberized Gap-Graded HMA (Wet Process)				
Innovative Mixtures				
Fiber-Reinforced HMA (w/ Steel Slag)				
Fiber-Reinforced WMA (w/ Steel Slag)				
<u>Fine Dense-Graded HMA</u>				

Note: Underlined mixtures were suggested by IDOT.

APPENDIX A REFERENCES

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APPENDIX B IDOT SPECIFICATIONS, SPECIAL PROVISIONS, POLICIES, AND PROCEDURES FOR HMA

Specific information pertinent to this study was selected from several Illinois Department of Transportation documents and is included in this appendix. The areas in the tables that are shaded gray contain information that is particularly relevant to this study.

(NOTE: Not all of information from the source material is included in this appendix. Source material is indicated by quotation marks. Selected tables are included in their entirety but are not always shown in the same order as in the source documents. The names of the complete documents are listed, and links or locations where they may be referenced are shown below.)

Standard Specifications for Road and Bridge Construction (2012)

NOTE: The mix designs for HMA mixes used in this study were completed according to the requirements in the 2012 edition of the Standard Specifications. However, the mix designs were completed prior to publishing of the 2012 edition of the Standard Specifications (http://www.dot.il.gov/desenv/stdspecs12.html). Refer to Division 1000 Materials, with attention paid to:

Section 1003 Fine Aggregates

Section 1004 Coarse Aggregates

Section 1031 Reclaimed Asphalt Pavement

Section 1032 Bituminous Materials

BDE Special Provision for Reclaimed Asphalt (RAP) and Reclaimed Asphalt Shingles (RAS)

NOTE: When the mix designs for the HMA mixes used in this study were completed, separate special provisions for RAP and RAS were current. However, any RAP that was utilized in mixes in this study was done according to the single Special Provision for RAP and RAS referenced at http://www.dot.il.gov/desenv/specrev/specprovarchive.html.

BMPR Policy Memorandum for Performance Graded Asphalt Binder Acceptance Procedure **and BMPR Policy Memorandum for Emulsified Asphalt Acceptance Procedure (http://www.dot.il.gov/materials/index2.html)**

Illinois Test Procedure 403, Calibration of the Ignition Oven for the Purpose of Characterizing Reclaimed Asphalt Pavement (RAP) found in Appendix B13 of the Illinois Department of Transportation *Manual of Test Procedures*.

AGGREGATE SPECIFICATIONS FOR HMA

“1003.03 Fine Aggregate for Hot-Mix Asphalt (HMA).

The aggregate shall be according to Article 1003.01 and the following.

(a) Description. Fine aggregate for HMA shall consist of sand, stone sand, chats, slag sand, or steel slag sand. For gradation FA 22, uncrushed material will not be allowed.

(b) Quality: The fine aggregate for all HMA shall be Class B Quality or better.”

[Table from Article 1003.01(b)]

“Fine Aggregate Quality

Quality Test	Class		
	A	B	C
Na_2SO_4 Soundness 5 Cycle, Illinois Modified AASHTO T 104, % Loss max.	10	15	20
Minus No. 200 (75 µm) Sieve Material, Illinois Modified AASHTO T 11, % max.[4/]	3	6[1/]	10[1/]
Organic Impurities Check, Illinois Modified AASHTO T 21	Yes[2/]	-	-
Deleterious Materials:[3/ 5/]			
Shale, % max.	3.0	3.0	-
Clay Lumps, % max.	1.0	3.0	-
Coal, Lignite, & Shells, % max.	1.0	3.0	-
Conglomerate, % max.	3.0	3.0	-
Other Deleterious, % max.	3.0	3.0	-
Total Deleterious, % max.	3.0	5.0	-

1/ Does not apply to Gradation FA 20 or FA 21.

2/ Applies only to Sand. Sand exceeding the colorimetric test standard of 11 (Illinois Modified AASHTO T 21) will be checked for mortar making properties according to the Illinois Modified AASHTO T 71, and shall develop a compressive strength at the age of 14 days when using Type I or II Cement of not less than 95 percent of the comparable standard.

3/ Applies only to sand.

4/ Fine aggregate used for hot-mix asphalt (HMA) shall not contain more than three percent clay (2 micron or smaller) particles as determined by Illinois Modified AASHTO T 88.

5/ Tests shall be run according to Illinois Test Procedure 204.”

"(c) Gradation: The fine aggregate gradation for all HMA shall be FA 1, FA 2, FA 20, FA 21 or FA 22.

Gradation FA 1, FA 2, or FA 3 shall be used when required for prime coat aggregate application for HMA."

[Table from Article 1003.01(c)]

"Fine Aggregate Gradations

Grad No.	Sieve Size and Percent Passing										
	3/8 in.	No. 4	No. 8 [4/]	No. 10	No. 16	No. 30 [5/]	No. 40	No. 50	No. 80	No. 100	No. 200 [1/]
	9.5 mm	4.75 mm	2.36 mm [4/]	2.00 mm	1.18 mm	0.6 mm [5/]	0.425 mm	0.3 mm	0.18 mm	0.15 mm	0.075 mm [1/]
FA 1	100	97±3			65±20			16±13		5±5	
FA 2	100	97±3			65±20			20±10		5±5	
FA 3	100	97±3		80±15			50±20		25±15		3±3
FA 4	100				5±5						
FA 5	100	92±8								20±20	15±15
FA 6		92±8 [2/]								20±20	6±6
FA 7		100		97±3			75±15		35±10		3±3
FA 8			100				60±20			3±3	2±2
FA 9			100					30±15		5±5	
FA 10				100			90±10		60±30		7±7
FA 20	100	97±3	80±20		50±15			19±11		10±7	4±4
FA 21[3/]	100	97±3	80±20		57±18			30±10		20±10	9±9
FA 22	100	[6/]	[6/]		8±8						2±2

1/ Subject to maximum percent allowed in Fine Aggregate Quality Table.

2/ 100 percent shall pass the 1 in. (25 mm) sieve, except that for bedding material 100 percent shall pass the 3/8 in. (9.5 mm) sieve. If 100 percent passes the 1/2 in. (12.5 mm) sieve, the No. 4 (4.75 mm) sieve may be 75 ± 25.

3/ For all HMA mixtures. When used, either singly or in combination with other sands, the amount of material passing the No. 200 (75 µm) sieve (washed basis) in the total sand fraction for mix design shall not exceed ten percent.

4/ For each gradation used in HMA, the aggregate producer shall set the midpoint percent passing, and the Department will apply a range of ±15 percent. The midpoint shall not be changed without Department approval.

5/ For each gradation used in HMA, the aggregate producer shall set the midpoint percent passing, and the Department will apply a range of ±13 percent. The midpoint shall not be changed without Department approval.

6/ For the fine aggregate gradation FA 22, the aggregate producer shall set the midpoint percent passing, and the Department will apply a range of ± ten percent. The midpoint shall not be changed without Department approval."

“1004.03 Coarse Aggregate for Hot-Mix Asphalt (HMA).

The aggregate shall be according to Article 1004.01 and the following.

(a) Description. The coarse aggregate for HMA shall be according to the following table.

Use	Mixture	Aggregates Allowed
Class A	Seal or Cover	Gravel Crushed Gravel Crushed Stone Crushed Sandstone Crushed Slag (ACBF) Crushed Steel Slag Crushed Concrete
HMA All Other	Stabilized Subbase or Shoulders	Gravel Crushed Gravel Crushed Stone Crushed Sandstone Crushed Slag Crushed Concrete The coarse aggregate for stabilized subbase, if approved by the Engineer, may be produced by blending aggregates according to Article 1004.04 (a).
HMA High ESAL Low ESAL	IL-25.0, IL-19.0, or IL-19.0L	Crushed Gravel Crushed Stone Crushed Sandstone Crushed Slag (ACBF)
HMA High ESAL Low ESAL	C Surface IL-12.5,IL-9.5, or IL-9.5L	Crushed Gravel Crushed Stone Crushed Sandstone Crushed Slag (ACBF) Crushed Steel Slag (except when used as leveling binder)
HMA High ESAL	D Surface IL-12.5 or IL-9.5	Crushed Gravel Crushed Stone (other than Limestone) Crushed Sandstone Crushed Slag (ACBF) Crushed Steel Slag (except when used as leveling binder) Limestone may be used in Mixture D if blended by volume in the following coarse aggregate percentages: Up to 25% Limestone with at least 75% Dolomite. Up to 50% Limestone with at least 50% any aggregate listed for Mixture D except Dolomite. Up to 75% Limestone with at least 25% Crushed Slag (ACBF) or Crushed Sandstone.
HMA High ESAL	E Surface IL-12.5 or IL-9.5	Crushed Gravel Crushed Stone (other than Limestone and Dolomite) Crushed Sandstone No Limestone. Dolomite may be used in Mixture E if blended by volume in the following coarse aggregate percentages: Up to 75% Dolomite with at least 25% Crushed Sandstone, Crushed Slag (ACBF), or Crushed Steel Slag. When Crushed Slag (ACBF) or Crushed Steel Slag are used in the blend, the blend shall contain a minimum of 25% to a maximum of 75% of either Slag by volume. Up to 50% Dolomite with at least 50% of any aggregate listed for Mixture E. If required to meet design criteria, Crushed Gravel or Crushed Stone (other than Limestone or Dolomite) may be blended by volume in the following coarse aggregate percentages: Up to 75% Crushed Gravel or Crushed Stone (other than Limestone or Dolomite) with at least 25% Crushed Sandstone, Crushed Slag (ACBF), or Crushed Steel Slag. When Crushed Slag (ACBF) or Crushed Steel Slag are used in the blend, the blend shall contain a minimum of 25% to a maximum of 50% of either Slag by volume.
HMA High ESAL	F Surface IL-12.5 or IL-9.5	Crushed Sandstone No Limestone. Crushed Gravel or Crushed Stone (except Limestone) may be used in Mixture F if blended by volume in the following coarse aggregate percentages: Up to 50% Crushed Gravel or Crushed Stone with at least 50% Crushed Sandstone, Crushed Slag (ACBF), or Crushed Steel Slag. When Crushed Slag (ACBF) or Crushed Steel Slag are used in the blend, the blend shall contain a minimum of 50% to a maximum of 75% of either Slag by volume.

(b) Quality. For surface courses and binder courses when used as surface course, the coarse aggregate shall be Class B quality or better. For Class A (seal or cover coat), and other binder courses, the coarse aggregate shall be Class C quality or better. For All Other courses, the coarse aggregate shall be Class D quality or better."

[Table from Article 1004.01(b)

"Coarse Aggregate Quality				
Quality Test	Class			
	A	B	C	D
Na_2SO_4 Soundness 5 Cycle, Illinois Modified AASHTO T 104[1/], % Loss max.	15	15	20	25[2/]
Los Angeles Abrasion, Illinois Modified AASHTO T 96, % Loss max.	40[3/]	40[4/]	40[5/]	45
Minus No. 200 (75 µm) Sieve Material, Illinois Modified AASHTO T 11	1.0[6/]	-	2.5[7/]	-
Deleterious Materials[10/]				-
Shale, % max.	1.0	2.0	4.0[8/]	
Clay Lumps, % max.	0.25	0.5	0.5[8/]	-
Coal & Lignite, % max.	0.25	-	-	-
Soft & Unsound Fragments, % max.	4.0	6.0	8.0[8/]	-
Other Deleterious, % max.	4.0[9/]	2.0	2.0[8/]	-
Total Deleterious, % max.	5.0	6.0	10.0[8/]	-

1/ Does not apply to crushed concrete.

2/ For aggregate surface course and aggregate shoulders, the maximum percent loss shall be 30.

3/ For portland cement concrete, the maximum percent loss shall be 45.

4/ Does not apply to crushed slag or crushed steel slag.

5/ For hot-mix asphalt (HMA) binder mixtures, except when used as surface course, the maximum percent loss shall be 45.

6/ For crushed aggregate, if the material finer than the No. 200 (75 µm) sieve consists of the dust from fracture, essentially free from clay or silt, this percentage may be increased to 2.5.

7/ Does not apply to aggregates for HMA binder mixtures.

8/ Does not apply to Class A seal and cover coats.

9/ Includes deleterious chert. In gravel and crushed gravel aggregate, deleterious chert shall be the lightweight fraction separated in a 2.35 heavy media separation. In crushed stone aggregate, deleterious chert shall be the lightweight fraction separated in a 2.55 heavy media separation. Tests shall be run according to Illinois Modified AASHTO T 113.

10/ Test shall be run according to Illinois Test Procedure 203."

“(c) Gradation. The coarse aggregate gradations shall be as listed in the following table.

Use	Size / Application	Gradation No.
Class A-1, 2 & 3	3/8 in. (10mm) Seal	CA 16 or CA 20
Class A-1	1/2 in. (13mm) Seal	CA 15
Class A-2 & 3	Cover	CA 14
HMA High ESAL	IL-25.0 IL-19.0 IL-12.5 IL-9.5	CA 7 [1/] or CA 8 [1/] CA 11 [1/] CA 16 and/or CA 13 CA 16
HMA Low ESAL	IL-19.0L IL-9.5L	CA 11 [1/] CA 16
HMA All Other	Stabilized Subbase or Shoulder	CA 6 [2/], CA 10, CA 12

[1]/ CA 16 or CA 13 may be blended with the gradations listed.

[2]/ CA 6 will not be permitted in the top lift of shoulders.”

[Table from Article 1004.01(c)]

	“Coarse Aggregate Gradations												
Grad No.	Sieve Size and Percent Passing												
	3 in.	2 1/2 in.	2in.	1 1/2 in.	1 in.	3/4 in.	1/2 in.	3/8 in.	No. 4	No. 8	No. 16	No. 50	No. 200[1/]
	75mm	63mm	50mm	37.5mm	25mm	19mm	12.5mm	9.5mm	4.75mm	2.36mm	1.18mm	0.3mm	0.075mm[1/]
CA 1	100	95±5	60±15	15±15	3±3								
CA 2		100	95±5		75±15		50±15		30±10		20±15		8±4
CA 3		100	93±7	55±20	8±8		3±3						
CA 4			100	95±5	85±10		60±15		40±10		20±15		8±4
CA 5				97±3[2/]	40±25		5±5		3±3				
CA 6				100	95±5		75±15		43±13		25±15		8±4
CA 7				100	95±5		45±15[3/8/]		5±5				
CA 8				100	97±3	85±10	55±10		10±5		3±3[4/]		
CA 9				100	97±3		60±15		30±15		10±10		6±6
CA 10					100	95±5	80±15		50±10		30±15		9±4
CA 11					100	92±8	45±15[5/8/]		6±6		3±3[4/6/]		
CA 12						100	95±5	85±10	60±10		35±10		9±4
CA 13						100	97±3	80±10	30±15		3±3[4/]		
CA 14							90±10[7/]	45±20	3±3				
CA 15							100	75±15	7±7		2±2		
CA 16							100	97±3	30±15		2±2[4/]		
CA 17	100								65±20		45±20	20±10	10±5
CA 18	100				95±5				75±25		55±25	10±10	2±2
CA 19	100				95±5				60±15		40±15	20±10	10±5
CA 20							100	92±8	20±10	5±5	3±3		

1/ Subject to maximum percent allowed in Coarse Aggregate Quality table.

2/ Shall be 100 percent passing the 1 3/4 in. (45 mm) sieve.

3/ When using gradation CA 7 for IL-25.0 binder, the percent passing the 1/2 in. (12.5 mm) sieve may also be 35±10 or 15±10.

4/ When used in HMA (High and Low ESAL) mixtures, the percent passing the No. 16 (1.18 mm) sieve for gradations CA 8, CA 11, CA 13, or CA 16 shall be 4±4 percent.

5/ When using gradation CA 11 for IL-19.0 and IL-19.0L binder, the percent passing the 1/2 in. (12.5 mm) sieve may also be 15±10.

6/ The No. 16 (1.18 mm) requirement will be waived when CA 11 is used in the manufacture of portland cement concrete.

7/ Shall be 100 percent passing the 5/8 in. (16 mm) sieve.

8/ When Class BS concrete is to be pumped, the coarse aggregate gradation shall have a minimum of 45 percent passing the 1/2 in. (12.5 mm) sieve. The Contractor may combine two or more coarse aggregate sizes, consisting of CA 7, CA 11, CA 13, CA 14, and CA 16, provided a CA 7 or CA 11 is included in the blend.”

"**1032.05 ASPHALT BINDER (PREPARED FROM PETROLEUM).**

These materials will be accepted according to the current Bureau of Materials and Physical Research Policy Memorandum, "Performance Graded Asphalt Binder Acceptance Procedure." These materials shall be free from water and shall not foam when heated to any temperature below the actual flash point."

"1032.05(a) Performance Graded (PG) Asphalt Binder.

The asphalt binder shall meet the requirements of AASHTO M 320, Table 1 "Standard Specification for Performance Graded Asphalt Binder" for the grade shown on the plans. Air blown asphalt will not be allowed.

1032.05(b) Modified Performance Graded (PG) Asphalt Binder.

The asphalt binder shall meet the requirements of AASHTO M 320, Table 1 "Standard Specification for Performance Graded Asphalt Binder" for the grade shown on the plans. Elastomers shall be added to the base asphalt binder to achieve the specified performance grade and shall be either a styrenebutadiene diblock or triblock copolymer without oil extension, or a styrenebutadiene rubber. Air blown asphalts, acid modification, and other modifiers will not be allowed. Asphalt modification at hot-mix asphalt plants will not be allowed. The modified asphalt binder shall be smooth, homogeneous, and be according to the requirements shown in Table 1 or 2 for the grade shown on the plans.

Table 1 - Requirements for Styrene-Butadiene Copolymer (SB/SBS) Modified Asphalt Binders

Test	Asphalt Grade SB/SBS PG64-28 SB/SBS PG70-22 SB/SBS PG70-28	Asphalt Grade SB/SBS PG76-22 SB/SBS PG76-28
Separation of Polymer Illinois Test Procedure, "Separation of Polymer from Asphalt Binder" Difference in °F (°C) of the softening point between top and bottom portions.	4 (2) max.	4 (2) max.
Force Ratio AASHTO T 300, (f_2/f_1), 39.2°F (4°C), 50 mm/min., 300 mm elongation.	0.30 min.	0.35 min.
TESTS ON RESIDUE FROM ROLLING THIN FILM OVEN TEST (AASHTO T 240)		
Elastic Recovery ASTM D 6084, Procedure A, 77°F (25°C), 100 mm elongation, %	60 min.	70 min.

Note. When SBS/SBR PG 76-22 or SBS/SBR 76-28 is specified for mixture IL-4.75, the elastic recovery shall be a minimum of 80.

Table 2 - Requirements for Styrene-Butadiene Rubber (SBR) Modified Asphalt Binders		
Test	Asphalt Grade SBR PG64-28 SBR PG70-22 SBR PG70-28	Asphalt Grade SBR PG76-22 SBR PG76-28
Separation of Polymer Illinois Test Procedure, “Separation of Polymer from Asphalt Binder” Difference in °F (°C) of the softening point between top and bottom portions.	4 (2) max.	4 (2) max.
Toughness ASTM D 5801, 77°F (25°C), 20 in./min. (500 mm/min.), in.-lbs (N-m).	110 (12.5) min.	110 (12.5) min.
Tenacity ASTM D 5801, 77°F (25°C), 20 in./min. (500 mm/min.), in.-lbs (N-m).	75 (8.5) min.	75 (8.5) min.
TESTS ON RESIDUE FROM ROLLING THIN FILM OVEN TEST (AASHTO T 240)		
Elastic Recovery ASTM D 6084, Procedure A, 77°F (25°C), 100 mm elongation, %	40 min.	50 min.

Note. When SBS/SBR PG 76-22 or SBS/SBR 76-28 is specified for mixture IL-4.75, the elastic recovery shall be a minimum of 80.

1032.06 Emulsified Asphalts.

Emulsified asphalts will be accepted according to the current Bureau of Materials and Physical Research Policy Memorandum, “Emulsified Asphalt Acceptance Procedure.” These materials shall be homogeneous and shall show no separation of asphalt after thorough mixing, within 30 days after delivery, provided separation has not been caused by freezing. They shall coat the aggregate being used in the work to the satisfaction of the Engineer and shall be according to the following requirements.

a. Anionic Emulsified Asphalt
b. Cationic Emulsified Asphalt
c. High Float Emulsion
d. Penetrating Emulsified Prime (PEP)
e. CSS-1h Latex Modified Emulsified Asphalt
f. Polymer Modified Emulsified Asphalt”

The remainder of Appendix B is taken from the Special Provision for Reclaimed Asphalt Pavement (RAP) and Reclaimed Asphalt Shingles (RAS) and includes a referral to Illinois Test Procedure 403, Calibration of the Ignition Oven for the Purpose of Characterizing Reclaimed Asphalt Pavements (RAP).

“SECTION 1031 RECLAIMED ASPHALT PAVEMENT AND RECLAIMED ASPHALT SHINGLES

1031.01 Description.

Reclaimed asphalt pavement and reclaimed asphalt shingles shall be according to the following.

(a) Reclaimed Asphalt Pavement (RAP). RAP is the material produced by cold milling or crushing an existing hot-mix asphalt (HMA) pavement. The Contractor shall supply written documentation that the RAP originated from routes or airfields under federal, state, or local agency jurisdiction.”

“1031.02 Stockpiles.

RAP and RAS stockpiles shall be according to the following.

(a) RAP Stockpiles. The Contractor shall construct individual, sealed RAP stockpiles meeting one of the following definitions. No additional RAP shall be added to the pile after the pile has been sealed. Stockpiles shall be sufficiently separated to prevent intermingling at the base. Stockpiles shall be identified by signs indicating the type as listed below (i.e. ”Homogeneous Surface”).

Prior to milling, the Contractor shall request the District provide documentation on the quality of the RAP to clarify appropriate stockpile.

(1) Fractionated RAP (FRAP). FRAP shall consist of RAP from Class I, HMA (High and Low ESAL) mixtures. The coarse aggregate in FRAP shall be crushed aggregate and may represent more than one aggregate type and/or quality but shall be at least C quality. All FRAP shall be fractionated prior to testing by screening into a minimum of two size fractions with the separation occurring on or between the #4 (4.75 mm) and 1/2 in. (12.5 mm) sieves. Agglomerations shall be minimized such that 100 percent of the RAP shall pass the sieve size specified below for the mix the FRAP will be incorporated.

Mixture FRAP will be used in:	Sieve Size that 100% of FRAP Shall Pass
IL-25.0	2 in. (50mm)
IL-19.0	1 1/2 in. (40mm)
IL-12.5	1 in. (25mm)
IL-9.5	3/4 in. (20mm)
IL-4.75	1/2 in. (13mm)

(2) Homogeneous. Homogeneous RAP stockpiles shall consist of RAP from Class I, HMA (High and Low ESAL) mixtures and represent: 1) the same aggregate quality, but shall be at least C quality; 2) the same type of crushed aggregate (either crushed natural aggregate, ACBF slag, or steel slag); 3) similar gradation; and 4) similar asphalt binder content. If approved by the Engineer, combined single pass surface/binder millings

may be considered “homogenous” with a quality rating dictated by the lowest coarse aggregate quality present in the mixture.

(3) Conglomerate. Conglomerate RAP stockpiles shall consist of RAP from Class I, HMA (High and Low ESAL) mixtures. The coarse aggregate in this RAP shall be crushed aggregate and may represent more than one aggregate type and/or quality but shall be at least C quality. This RAP may have an inconsistent gradation and/or asphalt binder content prior to processing. All conglomerate RAP shall be processed prior to testing by crushing to where all RAP shall pass the 5/8 in. (16 mm) or smaller screen. Conglomerate RAP stockpiles shall not contain steel slag.

(4) Conglomerate “D” Quality (DQ). Conglomerate DQ RAP stockpiles shall consist of RAP from Class I, HMA (High or Low ESAL), or “All Other” (as defined by Article 1030.04(a)(3)) mixtures. The coarse aggregate in this RAP may be crushed or round but shall be at least D quality. This RAP may have an inconsistent gradation and/or asphalt binder content. Conglomerate DQ RAP stockpiles shall not contain steel slag.

(5) Non-Quality. RAP stockpiles that do not meet the requirements of the stockpile categories listed above shall be classified as “Non-Quality”.

RAP/FRAP containing contaminants, such as earth, brick, sand, concrete, sheet asphalt, bituminous surface treatment (i.e. chip seal), pavement fabric, joint sealants, etc., will be unacceptable unless the contaminants are removed to the satisfaction of the Engineer. Sheet asphalt shall be stockpiled separately.”

“1031.03 Testing.

RAP/FRAP and RAS testing shall be according to the following.

(a) RAP/FRAP Testing. When used in HMA, the RAP/FRAP shall be sampled and tested either during or after stockpiling.

(1) During Stockpiling. For testing during stockpiling, washed extraction samples shall be run at the minimum frequency of one sample per 500 tons (450 metric tons) for the first 2000 tons (1800 metric tons) and one sample per 2000 tons (1800 metric tons) thereafter. A minimum of five tests shall be required for stockpiles less than 4000 tons (3600 metric tons).

(2) After Stockpiling. For testing after stockpiling, the Contractor shall submit a plan for approval to the District proposing a satisfactory method of sampling and testing the RAP/FRAP pile either in-situ or by restockpiling. The sampling plan shall meet the minimum frequency required above and detail the procedure used to obtain representative samples throughout the pile for testing.

Each sample shall be split to obtain two equal samples of test sample size. One of the two test samples from the final split shall be labeled and stored for Department use. The Contractor shall extract the other test sample according to Department procedure. The Engineer reserves the right to test any sample (split or Department-taken) to verify contractor test results.”

“1031.04 Evaluation of Tests.

Evaluation of test results shall be according to the following.

(a) Evaluation of RAP/FRAP Test Results. All of the extraction results shall be compiled and averaged for asphalt binder content and gradation and, when applicable G_{mm}. Individual extraction test results, when compared to the averages, will be accepted if within the tolerances listed below.

Parameter	FRAP/Homogeneous /Conglomerate	Conglomerate “D” Quality
1 in. (25 mm)		± 5 %
1/2 in. (12.5 mm)	± 8 %	± 15 %
No. 4 (4.75 mm)	± 6 %	± 13 %
No. 8 (2.36 mm)	± 5 %	
No. 16 (1.18 mm)		± 15 %
No. 30 (600 µm)	± 5 %	
No. 200 (75 µm)	± 2.0 %	± 4.0 %
Asphalt Binder	± 0.4 % 1/	± 0.5 %
G_{mm}	± 0.03	

1/ The tolerance for FRAP shall be ± 0.3 %.

If more than 20 percent of the individual sieves and/or asphalt binder content tests are out of the above tolerances, the RAP/FRAP shall not be used in HMA unless the RAP/FRAP representing the failing tests is removed from the stockpile. All test data and acceptance ranges shall be sent to the District for evaluation.

With the approval of the Engineer, the ignition oven may be substituted for extractions according to the Illinois Test Procedure, “Calibration of the Ignition Oven for the Purpose of Characterizing Reclaimed Asphalt Pavement (RAP)”.

“1031.05 Quality Designation of Aggregate in RAP/FRAP.

(a) RAP. The aggregate quality of the RAP for homogenous, conglomerate, and conglomerate ‘D’ quality stockpiles shall be set by the lowest quality of coarse aggregate in the RAP stockpile and are designated as follows.

(1) RAP from Class I, Superpave/HMA (High ESAL), or (Low ESAL) IL-9.5L surface mixtures are designated as containing Class B quality coarse aggregate.

(2) RAP from Superpave/HMA (Low ESAL) IL-19.0L binder mixture is designated as Class D quality coarse aggregate.

(3) RAP from Class I, Superpave/HMA (High ESAL) binder mixtures, bituminous base course mixtures, and bituminous base course widening mixtures are designated as containing Class C quality coarse aggregate.

(4) RAP from bituminous stabilized subbase and BAM shoulders are designated as containing Class D quality coarse aggregate.

(b) FRAP. If the Engineer has documentation of the quality of the FRAP aggregate, the Contractor shall use the assigned quality provided by the Engineer.

If the quality is not known, the quality shall be determined as follows. Coarse and fine FRAP stockpiles containing plus #4 (4.75 mm) sieve coarse aggregate shall have a maximum tonnage of 5,000 tons (4,500 metric tons). The Contractor shall obtain a representative sample witnessed by the Engineer. The sample shall be a minimum of 50 lb (25 kg). The sample shall be extracted according to Illinois Modified AASHTO T 164 by a consultant prequalified by the Department for the specified testing. The consultant shall submit the test results along with the recovered aggregate to the District Office. The cost for this testing shall be paid by the Contractor. The District will forward the sample to the BMPR Aggregate Lab for MicroDeval Testing, according to Illinois Modified AASHTO T 327. A maximum loss of 15.0 percent will be applied for all HMA applications.

1031.06 Use of RAP/FRAP and/or RAS in HMA.
"The use of RAP/FRAP and/or RAS shall be a Contractor's option when constructing HMA in all contracts.

(a) RAP/FRAP. The use of RAP/FRAP in HMA shall be as follows.

(1) Coarse Aggregate Size. The coarse aggregate in all RAP shall be equal to or less than the nominal maximum size requirement for the HMA mixture to be produced.

(2) Steel Slag Stockpiles. Homogeneous RAP stockpiles containing steel slag will be approved for use in all HMA (High ESAL and Low ESAL) Surface and Binder Mixture applications.

(3) Use in HMA Surface Mixtures (High and Low ESAL). RAP/FRAP stockpiles for use in HMA surface mixtures (High and Low ESAL) shall be FRAP or homogeneous in which the coarse aggregate is Class B quality or better. RAP/FRAP from Conglomerate stockpiles shall be considered equivalent to limestone for frictional considerations. Known frictional contributions from plus #4 (4.75 mm) homogeneous RAP and FRAP stockpiles will be accounted for in meeting frictional requirements in the specified mixture.

(4) Use in HMA Binder Mixtures (High and Low ESAL), HMA Base Course, and HMA Base Course Widening. RAP/FRAP stockpiles for use in HMA binder mixtures (High and Low ESAL), HMA base course, and HMA base course widening shall be FRAP, homogeneous, or conglomerate, in which the coarse aggregate is Class C quality or better.

(5) Use in Shoulders and Subbase. RAP/FRAP stockpiles for use in HMA shoulders and stabilized subbase (HMA) shall be FRAP, homogeneous, conglomerate, or conglomerate DQ.

(6) When the Contractor chooses the RAP option, the percentage of RAP shall not exceed the amounts indicated in Article 1031.06(c)(1) below for a given N Design.

(b) RAS. RAS meeting Type 1 or Type 2 requirements will be permitted in all HMA applications as specified herein.

(c) RAP/FRAP and/or RAS Usage Limits. Type 1 or Type RAS may be used alone or in conjunction with RAP or FRAP in HMA mixtures up to a maximum of 5.0% by weight of the total mix.

(1) RAP/RAS. When RAP is used alone or RAP is used in conjunction with RAS, the percentage of virgin asphalt binder replacement shall not exceed the amounts listed in the Max RAP/RAS ABR table listed below for the given Ndesign.

RAP/RAS Maximum Asphalt Binder Replacement (ABR) Percentage

HMA Mixtures 1/, 2/	RAP/RAS Maximum ABR %		
Ndesign	**Binder/Leveling Binder**	**Surface**	**Polymer Modified**
30	30	30	10
50	25	15	10
70	15	10	10
90	10	10	10
105	10	10	10

1/ For HMA "All Other" (shoulder and stabilized subbase) N-30, the RAP/RAS ABR shall not exceed 50 percent of the mixture.

2/ When RAP/RAS ABR exceeds 20 percent, the high and low virgin asphalt binder grades shall each be reduced by one grade (i.e. 25 percent ABR would require a virgin asphalt binder grade of PG64-22 to be reduced to a PG58-28). If warm mix asphalt (WMA) technology is utilized, and production temperatures do not exceed 275 °F (135 °C) the high and low virgin asphalt binder grades shall each be reduced by one grade when RAP/RAS ABR exceeds 25 percent (i.e. 26 percent RAP/RAS ABR would require a virgin asphalt binder grade of PG64-22 to be reduced to a PG58-28).

(2) FRAP/RAS. When FRAP is used alone or FRAP is used in conjunction with RAS, the percentage of virgin asphalt binder replacement shall not exceed the amounts listed in the FRAP/RAS tables listed below for the given Ndesign.

Level 1 – FRAP/RAS Maximum Asphalt Binder Replacement (ABR) Percentage

HMA Mixtures 1/, 2/	Level 1 – FRAP/RAS Maximum ABR %		
Ndesign	**Binder/Leveling Binder**	**Surface**	**Polymer Modified 3/, 4/**
30	35	35	10
50	30	25	10
70	25	20	10
90	20	15	10
105	10	10	10

1/ HMA "All Other" (shoulder and stabilized subbase) N30, the FRAP/RAS ABR shall not exceed 50 percent of the mixture.

2/ When FRAP/RAS ABR exceeds 20 percent for all mixes the high and low virgin asphalt binder grades shall each be reduced by one grade (i.e. 25 percent ABR would require a virgin asphalt binder grade of PG64-22 to be reduced to a PG58-28). If warm mix asphalt (WMA) technology is utilized, and production temperatures do not exceed 275°F (135 °C) the high and low virgin asphalt binder grades shall each be reduced by one grade when FRAP/RAS ABR exceeds 25 percent (i.e. 26 percent ABR would require a virgin asphalt binder grade of PG64-22 to be reduced to a PG58-28).

3/ For SMA the FRAP/RAS ABR shall not exceed 20 percent.

4/ For IL-4.75 mix the FRAP/RAS ABR shall not exceed 20 percent.

Level 2 – FRAP/RAS Maximum Asphalt Binder Replacement (ABR) Percentage

HMA Mixtures 1/, 2/	Level 2 – FRAP/RAS Maximum ABR %		
Ndesign	Binder/Leveling Binder	Surface	Polymer Modified 3/, 4/
30	40	40	10
50	40	30	10
70	30	20	10
90	30	20	10
105	30	15	10

1/ For HMA "All Other" (shoulder and stabilized subbase) N30, the FRAP/RAS ABR shall not exceed 50 percent of the mixture.

2/ When FRAP/RAS ABR exceeds 20 percent for all mixes the high and low virgin asphalt binder grades shall each be reduced by one grade (i.e. 25 percent ABR would require a virgin asphalt binder grade of PG64-22 to be reduced to a PG58-28). If warm mix asphalt (WMA) technology is utilized, and production temperatures do not exceed 275 °F (135 °C) the high and low virgin asphalt binder grades shall each be reduced by one grade when FRAP/RAS ABR exceeds 25 percent (i.e. 26 percent ABR would require a virgin asphalt binder grade of PG64-22 to be reduced to a PG58-28).

3/ For SMA the FRAP/RAS ABR shall not exceed 20 percent.

4/ For IL-4.75 mix the FRAP/RAS ABR shall not exceed 30 percent.

1031.07 HMA Mix Designs.

At the Contractor's option, HMA mixtures may be constructed utilizing RAP/FRAP and/or RAS material meeting the above detailed requirements.

FRAP/RAS mix designs exceeding the Level 1 FRAP/RAS Maximum ABR percentages shall be tested prior to submittal for verification, according to Illinois Modified AASHTO T324 (Hamburg Wheel) and shall meet the following requirements:

Asphalt Binder Grade	# Repetitions	Max Rut Depth (mm)
PG76-XX	20,000	1/2 (12.5)
PG70-XX	15,000	1/2 (12.5)
PG64-XX	10,000	1/2 (12.5)
PG58-XX	10,000	1/2 (12.5)"

[NOTE: The criteria in the table above were in effect at the time this study was conducted. However, since that time, the number of repetitions required for PG64-XX and PG58-XX has been modified to 7,500 and 5,000 respectively.]

"(a) RAP/FRAP and/or RAS. RAP/FRAP and/or RAS designs shall be submitted for volumetric verification. If additional RAP/FRAP stockpiles are tested and found that no more than 20 percent of the results, as defined under 'Testing' herein, are outside of the control tolerances set for the original RAP/FRAP stockpile and HMA mix design, and meets all of the requirements herein, the additional RAP/FRAP stockpiles may be used in the original mix design at the percent previously verified."

"1031.08 HMA Production

Mixture production where the FRAP/RAS ABR percentage exceeds the Level 1 limits shall be sampled within the first 500 tons (450 metric tons) on the first day of production with a split reserved for the Department. The mix sample shall be tested according to the Illinois Modified AASHTO T324 and shall meet the requirements specified herein. Mix production shall not exceed 1,500 tons (1,350 metric tons) or one day's production, whichever comes first, until the testing is completed and the mixture is found to be in conformance. The requirement to cease mix production may be waived if the plant produced mixture conformance is demonstrated prior to start of mix production for a State contract.

(a) RAP/FRAP. The coarse aggregate in all RAP/FRAP used shall be equal to or less than the nominal maximum size requirement for the HMA mixture being produced.

To remove or reduce agglomerated material, a scalping screen, gator, crushing unit, or comparable sizing device approved by the Engineer shall be used in the RAP feed system to remove or reduce oversized material. If material passing the sizing device adversely affects the mix production or quality of the mix, the sizing device shall be set at a size specified by the Engineer.

If the RAP/FRAP control tolerances or QC/QA test results require corrective action, the Contractor shall cease production of the mixture containing RAP/FRAP and either switch to the virgin aggregate design or submit a new RAP/FRAP design."

[…]

"(c) RAP/FRAP and/or RAS. HMA plants utilizing RAP/FRAP and/or RAS shall be capable of automatically recording and printing the following information.

1. Dryer Drum Plants.

   a. Date, month, year, and time to the nearest minute for each print.
   b. HMA mix number assigned by the Department.
   c. Accumulated weight of dry aggregate (combined or individual) in tons (metric tons) to the nearest 0.1 ton (0.1 metric ton).
   d. Accumulated dry weight of RAP/FRAP/RAS in tons (metric tons) to the nearest 0.1 ton (0.1 metric ton).
   e. Accumulated mineral filler in revolutions, tons (metric tons), etc. to the nearest 0.1 unit.
   f. Accumulated asphalt binder in gallons (liters), tons (metric tons), etc. to the nearest 0.1 unit.
   g. Residual asphalt binder in the RAP/FRAP material as a percent of the total mix to the nearest 0.1 percent.
   h. Aggregate and RAP/FRAP moisture compensators in percent as set on the control panel. (Required when accumulated or individual aggregate and RAP/FRAP are printed in wet condition.)

2. Batch Plants.

   a. Date, month, year, and time to the nearest minute for each print.
   b. HMA mix number assigned by the Department.
   c. Individual virgin aggregate hot bin batch weights to the nearest pound (kilogram).
   d. Mineral filler weight to the nearest pound (kilogram).
   f. RAP/FRAP/RAS weight to the nearest pound (kilogram).

g. Virgin asphalt binder weight to the nearest pound (kilogram).
h. Residual asphalt binder in the RAP/FRAP/RAS material as a percent of the total mix to the nearest 0.1 percent.

The printouts shall be maintained in a file at the plant for a minimum of one year or as directed by the Engineer and shall be made available upon request. The printing system will be inspected by the Engineer prior to production and verified at the beginning of each construction season thereafter.

1031.09 RAP in Aggregate Surface Course and Aggregate Shoulders.

The use of RAP in aggregate surface course and aggregate shoulders shall be as follows.

(a) Stockpiles and Testing. RAP stockpiles may be any of those listed in Article 1031.02, except “Non-Quality” and “FRAP”. The testing requirements of Article 1031.03 shall not apply.

(b) Gradation. One hundred percent of the RAP material shall pass the 1 1/2 in. (37.5 mm) sieve. The RAP material shall be reasonably well graded from coarse to fine. RAP material that is gap-graded or single sized will not be accepted.”