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# DESIGNING, PRODUCING, AND CONSTRUCTING FINE-GRADED HOT MIX ASPHALT ON ILLINOIS ROADWAYS

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**Fine-Graded Hot Mix Asphalt on Illinois Roadways**

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16. Abstract
Fine-graded (F-G) asphalt concrete mixtures are composed of an aggregate structure in which the fine fraction controls the load-carrying capacity of the mix. Other states have reported benefits in using F-G mixtures, including improved compaction, lower segregation, and lower permeability—resulting in longer life. Rutting concerns have been mitigated through the use of manufactured sand. This study investigates the feasibility of using F-G mixtures for IL 19.0 mm (3/4 in) asphalt binder courses in Illinois. A careful laboratory investigation, including mix designs guided by the Bailey Method, was conducted, then followed by extensive laboratory performance testing. Performance tests indicated that the F-G mixtures had equivalent or superior rut and crack resistance to a reference coarse-graded (C-G) control mixture. Limited field trials demonstrated the F-G mixtures were easier to compact, led to higher pay factors, and had significantly lower permeability than traditional C-G mixtures, while being similar in cost. Whether designing a coarse- or fine-graded mixture, optimum asphalt content will be the same when using the same materials and targeting the same VMA and voids level, provided asphalt absorption remains constant. Limited full-scale accelerated pavement tests also demonstrated similarity in rutting resistance between the C-G and F-G mixtures investigated. Recommendations for implementation of F-G mixtures are provided, along with a revised draft specification for 19.0 mm mixtures. The revised specification provides upward adjustments to the lower side of the gradation band at the primary control sieve to lessen the likelihood of designing segregation-prone binder mixtures and raises the upper band to permit F-G mixtures to be designed. These principles apply to surface course mixtures as well.

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This publication is based on the results of ICT-R27-079, **Design, Producing, and Constructing Fine-Graded Hot Mix Asphalt on IL Roadways.** ICT-R27-079 was conducted in cooperation with the Illinois Center for Transportation; the Illinois Department of Transportation; and the U.S. Department of Transportation, Federal Highway Administration.

Members of the Technical Review Panel were as follows:

- Matt Mueller, Illinois Department of Transportation, Chair
- Steve Hefel, Illinois Department of Transportation
- Scott Lackey, Illinois Department of Transportation
- Frank Mathewson, Iroquois Paving Corporation
- Steve Robinson, Illinois Department of Transportation
- James Trepanier, Illinois Department of Transportation
- Brett Williams, Rock Roads Companies, Inc.

# EXECUTIVE SUMMARY

This report summarizes the findings of a major, multi-phase study conducted at the Illinois Center for Transportation to assist the Illinois Department of Transportation (IDOT) in considering fine-graded asphalt mixtures for Illinois roadways. More specifically, the research project was designed to assist IDOT in the modification of existing asphalt mixture specifications to allow the use of fine-graded (F-G) hot mix asphalt (HMA) as an alternative to coarse-graded (C-G) HMA in Illinois for binder and level binder asphalt pavement layers. The current C-G gradation limits in Illinois were originally established based on the viewpoint that C-G HMA is more stable than F-G HMA, which was true in the past due to the use of natural sand, which produced inappropriate angularity and gradations in F-G HMA. However, manufactured sand is a common component of HMA today, especially in mixes designed for high traffic. Moreover, traffic loads are transmitted primarily through the fine aggregate fraction of F-G mixtures (i.e., the fine aggregates are densely packed and control the skeletal structure of these mixes). Until these mixtures were better understood through the development of new analysis tools, such as the Bailey Method, and through use in field trials, there was a general belief that they had relatively low rutting resistance. Other states have developed high-performing F-G mixtures with benefits such as consistency of field compaction to higher density (creating lower permeability) and improved smoothness. Recent, early demonstration projects conducted in Illinois District 5 (east-central Illinois) using F-G binder courses have produced favorable results, in terms of mix workability, better than normal density, segregation resistance, low permeability, and good performance to date.

After conducting a detailed literature review and survey of practitioners, a careful laboratory investigation, including mix designs guided by the Bailey Method, followed by extensive laboratory performance testing was conducted. Mix designs were carried out with the goal of achieving very similar VMA in the control, coarse-graded mix, and the three study fine-graded mixes investigated (ranging in degree of “fineness”). These tests included Hamburg wheel track rut evaluation, moisture sensitivity testing, disk-shaped compact tension fracture testing (DC(T)), flexural beam fatigue testing, and dynamic (complex) modulus testing. Limited full-scale accelerated pavement tests also demonstrated the similarity in rutting resistance between the C-G and F-G mixtures investigated. The overall conclusions of the performance tests indicated that the F-G mixtures had equivalent or superior rut and crack resistance to the control coarse-graded (C-G) binder course mixture. Limited field trials demonstrated the F-G mixtures were easier to compact, led to very high pay factors (bonuses) in pay-for-performance specifications, and had significantly lower permeability than traditional C-G mixtures, while being similar in cost.

A common misunderstanding is that fine-graded mixtures require significantly more binder than coarse-graded mixtures. This stems from the fact that F-G mixtures have more aggregate surface area than C-G mixtures, which leads to the incorrect conclusion that this results in a higher optimum asphalt content. Optimum asphalt content is driven by VMA, air voids, and asphalt absorption. Asphalt absorption is a function of several parameters, such as

- combined aggregate blend water absorption;
- porosity index of the aggregates;
- mixture moisture content;
- asphalt binder properties;
- cure time or silo time;
- truck haul time, and;
- mixture temperature,

with combined aggregate blend water absorption being the most critical factor. Water absorption drives asphalt absorption, which in turn creates different optimum asphalt contents between two mixtures that have equal VMA and air voids.

Recommendations for implementation of F-G mixtures in Illinois were developed by the research team in close consultation with the project Technical Review Panel. These recommendations are provided herein, along with a draft specification for the 19.0 mm NMAS binder course HMA mixtures. The revised specification provides upwards adjustments to the lower side of the gradation band near the primary control sieve to lessen the likelihood of designing segregation-prone binder course mixtures and raises the upper gradation band to permit F-G mixtures to be designed. It is thought that contractors will begin to target the finer side of this specification over time, to capture the benefits of F-G mixtures in terms of compactability, workability, lower permeability, smoothness, tighter longitudinal joints, and lower segregation potential. Again, all of these principles apply to surface course mixtures.

# CONTENTS

# CHAPTER 1 INTRODUCTION

This report summarizes the findings of the project entitled ICT-R27-079, "Designing, Producing, and Constructing Fine-Graded Hot Mix Asphalt (HMA) on Illinois Roadways." This project was conducted in cooperation with the Illinois Center for Transportation, the Illinois Department of Transportation, and the U.S. Department of Transportation, Federal Highway Administration, and focused on assessing the feasibility of allowing fine-graded asphalt mixtures for Illinois roadways. More specifically, the research project was designed to assist IDOT in the modification of existing asphalt mixture specifications to allow the use of fine-graded (F-G) hot mix asphalt (HMA) as an alternative to coarse-graded (C-G) HMA in Illinois for binder and level binder roadway layers. In conjunction with IDOT, the Illinois Center for Transportation (ICT) of the University of Illinois at Urbana-Champaign (UIUC), Murphy Pavement Technology (MPT), and Heritage Research Group (HRG) joined together to conduct this collaborative research project. The current C-G gradation limits in Illinois were originally established based on the view point that C-G HMA is more stable than F-G HMA, which was true in the past due to the use of natural sand, which produced inappropriate angularity and gradations in F-G HMA. However, manufactured sand is a common component of HMA today, especially in mixes designed for high traffic. Other states have developed high-performing F-G mixtures with benefits such as consistency of field compaction to achieve better density, lower permeability, and improved smoothness.

One of the motivating factors leading Illinois to re-consider F-G mixes was the relative abundance of success stories reported by other states. For example, a study conducted by the North Carolina Department of Transportation (NCDOT) concluded that F-G Superpave surface course mixtures had superior fatigue life compared to C-G mixes (1). In addition, all mix gradations passing above the restricted zone had better rutting resistance than mixtures passing below the restricted zone in this study. It was also found that the addition of natural sands did not significantly affect the performance of these mixtures. An evaluation of coarse- and fine-graded mixtures evaluated under accelerated pavement testing (APT) was carried out by the Florida Department of Transportation (FDOT). The study demonstrated that F-G mixtures performed as well as or slightly better than C-G mixtures in terms of rutting performance (2). Similar conclusions regarding the equal or superior performance of F-G mixtures as determined through laboratory testing were reported in a study conducted by the West Virginia Department of Transportation (WVDOT) (3). In addition, early demonstration projects conducted in Illinois District 5 (east-central Illinois) using F-G binder courses have produced favorable results, in terms of mix workability, high density, segregation resistance, low permeability, and good early performance.

The work plan for this study consisted of four phases designed to assist the Illinois Department of Transportation (IDOT) in moving toward the use of F-G HMA through a comprehensive research program. The research effort included state and national surveys, careful F-G and C-G mixture designs and mixture evaluations, laboratory performance tests, accelerated pavement testing, limited field testing and case studies, and the development of a revised HMA binder course specification to help move the research toward implementation.

Phase 1 consisted of information gathering (literature review) with three defined tasks:

> **Task 1:** Review historical development of the IDOT HMA specifications. Perform national literature review with a focus on F-G HMA design, field experience, and field performance data.
>
> **Task 2:** Conduct interviews with IDOT Central and District offices, retired IDOT materials engineers, and other researchers who have played a role in the development of the current specification as the best practice. Identify key considerations used in the development of the

current gradation bands used by IDOT to assist in the selection of a range of mixtures and a suite of lab and ATLAS tests for subsequent study phases.

**Task 3:** Gather information on the related practices of other states, the Federal Aviation Administration, and intermodal uses with similar traffic, climate, and aggregate resources. Conduct written and verbal follow-up interviews to help understand the various HMA design, production, and construction philosophies followed in other states.

Phase 2 used the information gathered in Phase 1 to organize, prioritize, and develop job mix formula (JMF) blends to be evaluated in the laboratory. This phase included a comprehensive aggregate analysis through use of the Bailey Method. It was determined that one C-G control mixture and three F-G mixtures with varying proportions of manufactured and natural sand would be studied (all 19 mm binder course mixtures).

Phase 3 involved laboratory analysis of the various HMA blends selected for use in Phase 2. These included Hamburg wheel track testing, disc-shaped compact tension (DC(T)) testing, dynamic modulus (E*) testing, four-point flexural fatigue testing, and moisture sensitivity testing.

Phase 4 involved testing of the four study mixtures under accelerated pavement testing (APT), using the Advanced Testing and Loading System, or ATLAS, at the Advanced Transportation Research and Engineering Laboratory (ATREL), which is home to the Illinois Center for Transportation (ICT). Both dry and wet condition testing was conducted. Due to time and equipment limitations, only the C-G control section and F-G section 1 materials were tested by the time of this report.

In addition, the Technical Review Panel used the results of the study to develop recommendations for a revised 19.0 mm NMAS binder HMA specification that

- Permits F-G mixes, which reduces segregation potential by raising lower gradation bands, and
- Includes slight adjustments to mixture volumetric requirements to improve durability.

Results of limited case studies of ongoing experimental use of F-G binder in IDOT District 5 are presented, which demonstrate that F-G binder can be used in a cost effective manner, and produce desired benefits in terms of increased compactability and lower permeability.

# CHAPTER 2 LITERATURE REVIEW, SURVEY, AND INTERVIEWS

## 2.1 INTRODUCTION

The current coarse-graded (C-G) gradation limits used in Illinois were originally established based on the view point that C-G HMA was thought to be more stable than fine-graded (F-G) HMA, which was probably true in the past due to the use of natural sand, which produced inappropriate angularity and angularity and gradations in F-G HMA. However, manufactured sand is a common component of HMA today, especially in higher $N_{design}$ mixes. Other states have developed high-performing F-G mixtures with benefits such as consistency of field compaction to higher density (creating lower permeability) and improved smoothness. The information was collected through: (1) a detailed literature search; (2) face-to-face and phone conversations with experts from the asphalt and aggregate industry; and (3) an online search and review of state specifications.

## 2.2 LITERATURE SEARCH

ICT and Murphy Pavement Technology, Inc. performed a literature search that included a detailed review of IDOT asphalt specifications, a thorough review of state specifications throughout the Midwest, and a comprehensive search for regional and national research primarily found published in electronic format online. Sources were the following:

- American Association of State Highway and Transportation Officials (AASHTO)
- American Society for Testing and Materials (ASTM)
- American Society of Civil Engineers (ASCE)
- Asphalt Institute (AI)
- Association of Asphalt Paving Technologists (AAPT)
- Federal Aviation Administration (FAA)
- Federal Highway Administration (FHWA)
- Illinois Asphalt Pavement Association (IAPA)
- National Center for Asphalt Technology (NCAT)
- Transportation Research Board (TRB)
- National Technical Information Service (NTIS)
- State Department of Transportation (DOT)
- Transportation Research Information Service (TRIS)

More than 40 documents and ten state DOT specifications were reviewed. Numerous phone calls and face-to-face conversations have complemented the historical, factual, and practical approaches to be taken throughout this research effort specific to HMA and specifically fine-graded HMA. A detailed summary of the literature is included in Appendix A, along with a summary of key interview findings. The literature review was helpful in understanding the historical context of HMA specifications in Illinois and the rationale and approach taken by other states in developing Superpave gradation specifications. Research on fine-graded mixture design and performance was also reviewed. In addition, the FAA P-401 specification was reviewed, demonstrating that the P-401 specification is in fact a F-G specification, having a good track record in terms of performance under very heavy aircraft loading. This review was helpful in establishing an experimental plan for the testing carried out in this study, and also helped with the development of recommended changes to Illinois' 19.0 mm HMA specification (to allow F-G mixes and to avoid segregation-prone C-G mixes). The review also helped to solidify a definition for F-G mixes, as described in the following section.

## 2.3. DEFINITION OF F-G HMA

This section presents a definition for F-G HMA, as discerned from literature review, interviews, and through discussions and consensus reached during meetings of the project Technical Review Panel. ***As a starting point, the demarcation between F-G and C-G mixtures is a function of the primary control sieve, not the maximum particle size.*** Therefore, an HMA mixture can have a small top size, such as an IL-9.5 mm (3/8 in) surface course, and still be classified as coarse or fine-graded. For the purposes of our efforts, the research team considered the standard definitions of the nominal maximum aggregate size and maximum aggregate size from work done by Huber and Schuler (1992). They state:

> The definition of nominal maximum aggregate size (NMAS) is, in a standard set of sieves, one size larger than the first sieve to retain more than ten percent of the total aggregate. The maximum aggregate size (MAS) is defined as the smallest sieve opening through which the entire amount of aggregate is required to pass, one sieve size larger than the NMAS. In addition, the maximum density line on a 0.45 power chart should be drawn through the origin to 100% passing MAS. (Asphalt Institute, 1992)

Many quality and process control measures for HMA follow from these definitions, including

- Voids in mineral aggregate (VMA)
- Quality requirements for the coarse aggregates
- Compacted lift thickness
- Aggregate gradation control limits

For this project, it was deemed important to further distinguish between the definitions of coarse and fine-graded HMA mixtures. This was accomplished by considering the FHWA 0.45 power chart for various NMAS mixes allowed in Illinois versus the Superpave NMAS control points used around the Midwest and nationally. The specification review done of states surrounding Illinois shows that all have adopted the Superpave NMAS control points (Table 1). A quick review of the data shows that these states allow both coarse and fine gradations for HMA. None of the states reviewed are entertaining aggregate gradation control point changes at this time. It should be noted that the national control point limits are from ASTM 3515. A thorough understanding of HMA production and construction efforts suggests the minus #200 values published by IDOT are more realistic than ASTM 3515.

This research evaluated the current IDOT gradation bands versus those recommended by Superpave from coarse through fine gradations. It is suggested that both the lower and upper limits specified by IDOT on the primary control sieves are coarser than is necessary or desired and that an opportunity existed to revise the gradation controls due to the increased experience of contractors through QC/QA, added production capabilities at aggregate producers, new material uses of reclaimed asphalt pavement (RAP) and 1/4 in (FM-22), and better understanding of performance challenges presented from segregation, permeability, low joint density, and reduced performance, which are all outcomes of coarse-graded HMA mixtures.

Table 1. Comparison of Superpave vs. IDOT Gradation Requirements

| | **High ESAL HMA (Superpave) NMAS Control Points versus IDOT Gradation Bands [*40% for $N_{des}$ >= 90] (Percent Passing) and VMA Requirements** | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Sieve Size** | **1 in (25 mm)** | | **3/4 in (19 mm)** | | **1/2 in (12.5 mm)** | | **3/8 in (9.5 mm)** | |
| | **Superpave** | **IDOT** | **Superpave** | **IDOT** | **Superpave** | **IDOT** | **Superpave** | **IDOT** |
| 1-1/2 in | 100 | 100 | — | | — | | — | |
| 1 in | 90–100 | 90–100 | 100 | 100 | — | | — | |
| 3/4 in | 90 max | 90 max | 90–100 | 82–100 | 100 | 100 | — | |
| 1/2 in | — | 45–75 | 90 max | 50–85 | 90–100 | 90–100 | 100 | 100 |
| 3/8 in | 42–70 | | 52–80 | | 90 max | 89 max | 90–100 | 90–100 |
| No. 4 | — | 24–42* | — | 24–50* | — | 28–65 | | 28–65 |
| No. 8 | 19–45 | 16–31 | 23–49 | 20–36 | 28–58 | 28–48* | 32–67 | 28–48* |
| No. 16 | — | 10–22 | — | 10–25 | — | 10–32 | — | 10–32 |
| No. 30 | — | | — | | — | | — | |
| No. 50 | — | 4–12 | — | 4–12 | — | 4–15 | — | 4–15 |
| No. 100 | — | 3–9 | — | 3–9 | — | 3–10 | — | 3–10 |
| No.200 | 0–6 | 3–6 | 2–8 | 3–6 | 2–10 | 4–6 | 2–10 | 4–6 |
| Min. VMA | 12.0 | 12.0 | 13.0 | 13.0 | 14.0 | 14.0 | 15.0 | 14.5 |

The 0.45 Power Chart shown in Figure 1 illustrates the Superpave control points and two typical gradations, one coarse and one fine, that can potentially meet all HMA volumetric requirements. The area shaded (in red) is the restricted zone currently in the IDOT specification that serves Illinois well, especially for work with coarse-graded mixes. The intent of the restricted zone was to keep mix gradations from developing a sand hump that is typical in a “tender” mixture. Sand humps will enter the restricted zone from below when a large amount of natural sand is introduced into the mixture. The usefulness of the restricted zone when producing fine-graded mixtures is redundant, because traveling through the restricted zone from above will be accomplished by introducing a coarse-graded manufactured sand, which is not a concern for HMA nor will it be a tender mix.

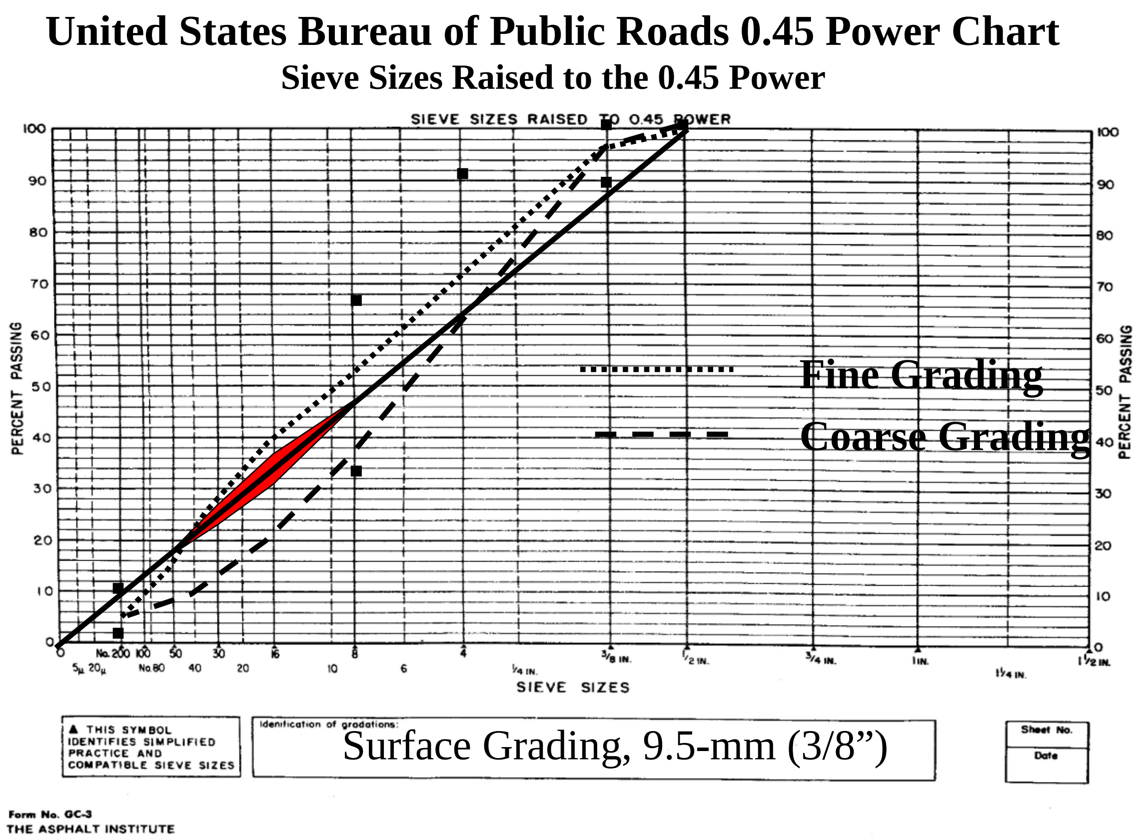



Figure 1. Typical F-G and C-G mixtures that could meet volumetric requirements.

For uniformity of defining coarse and fine-graded HMA mixtures, we will identify the point at which the mixture crosses the maximum density line on the #4 sieve for binder mixtures and the #8 sieve for surface mixtures. This is summarized in Table 2. The cross over point is visually taken from the 0.45 Power Chart shown for the 3/8 in (9.5 mm), 1/2 in (12.5 mm), 3/4 in (19.0 mm), and 1 in (25.0 mm) mixes and is in agreement with Superpave definitions for primary control sieves (Figure 2).

Table 2. C-G/F-G Mixture Thresholds as Defined in This Project

| NMAS (Sieve Size) | Primary Control Sieve | Percent Passing Threshold (equal or above is fine-graded, below is coarse-graded) |
|---|---|---|
| 3/8 in (9.5 mm) | #8 (2.36 mm) | 47% |
| 1/2 in (12.5 mm) | #8 (2.36 mm) | 39% |
| 3/4 in (19.0 mm) | #4 (4.75 mm) | 47% |
| 1 in (25.0 mm) | #4 (4.75 mm) | 40% |

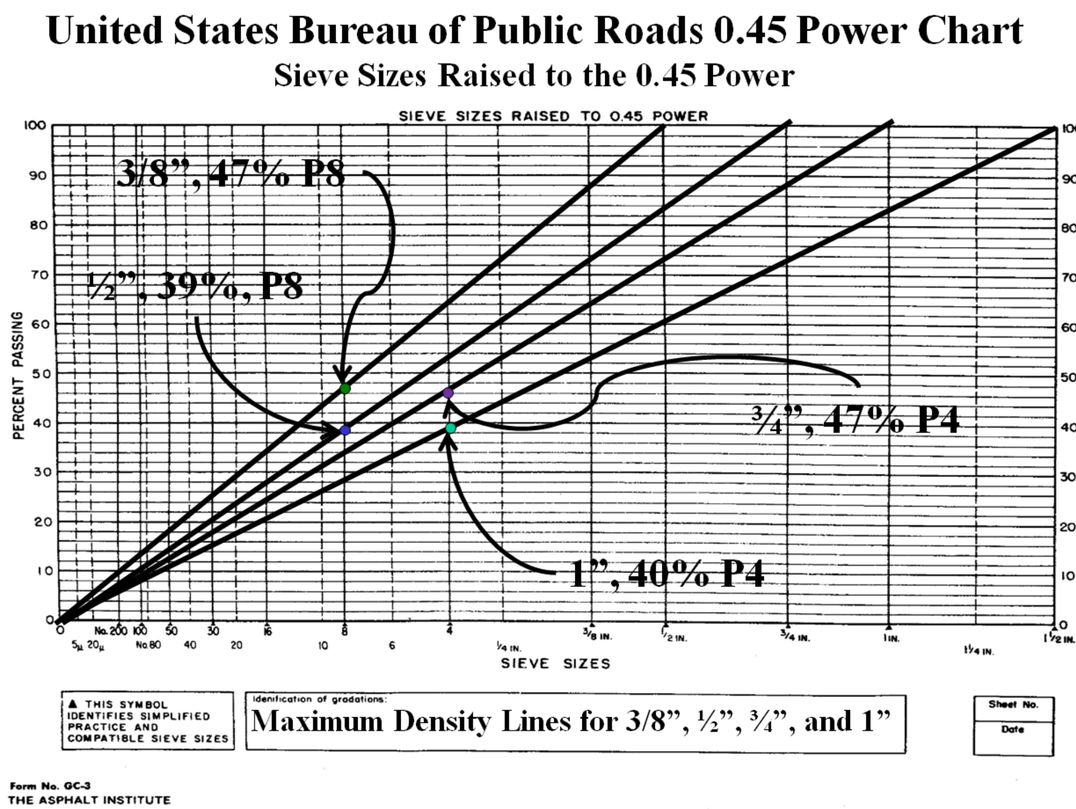



Figure 2. Illustration of C-G/F-G thresholds as shown on 0.45 power chart.

# CHAPTER 3 MATERIALS

This chapter provides information on the aggregate and asphalt binder materials used in this project. First, sampling source information and physical properties of the asphalt binder are provided. Sampling procedures and physical properties of the aggregates are then presented. As a result of Technical Review Panel meetings early in the project, it was decided to focus efforts on the development of 19.0 mm NMAS binder course mixtures for laboratory and ATLAS investigation and to utilize a single, commonly used asphalt binder type.

## 3.1 ASPHALT BINDER

Based upon consensus of the project Technical Review Panel, a Superpave Performance Grade PG 64-22 binder was selected. The PG 64-22 is a commonly used binder in the state of Illinois, and is an unmodified grade. For research purposes, 300 one-gallon metal cans of the PG 64-22 were sampled from a local binder supplier, Emulsicoat, Inc., a certified binder producer for the state of Illinois. Physical properties of the PG 64-22 were measured and provided by Emulsicoat, as presented in Table 3. All Superpave PG binder test results met the AASHTO MP-1 requirements.

Table 3. Physical Properties of the PG 64-22 Binder

| | | | | | |
|---|---|---|---|---|---|
| Sampled From | Emulsicoat, Inc. | | | | |
| Plant Location | Urbana, IL | | | | |
| Sample Date | 07/02/2010 | | | | |
| Material | PG 64-22 | | | | |
| Viscosity Graded Tests | | | | | |
| Pen at 25°C, 100 g, 5 sec, | 67 | | | | |
| Absolute Viscosity at 60°C, (Pa·s) | 205.5 | | | | |
| PG Asphalt Binder Tests | | | | | |
| Specific Gravity at 15.6°C | 1.036 | | | | |
| Flash Point, °C | 332 | | | | |
| Rotational Viscosity at 135°C, (Pa · s) | 0.412 | | | | |
| PAV Aging Temp, °C | 100 | | | | |
| | Temp (°C) | G* (kPa) | Phase Angle, δ, degrees | G*/sin(δ) (kPa) | G*sin(δ) (kPa) |
| DSR Original Binder | 64 | 1.303 | 87.35 | 1.305 | — |
| DSR RTFO Residue | 64 | 3.078 | 84.18 | 3.094 | — |
| DSR PAV Residue | 25 | 5,802 | 47.77 | — | 4,296 |
| | Temp (°C) | m-value | | Creep Stiffness, MPa | |
| Bending Beam Rheometer (BBR), PAV Residue | –12 | 0.314 | | 192 | |

### 3.2 AGGREGATES

Four different types of local central Illinois aggregates were used in the project, which were sampled from Open Road Paving Company in Urbana, IL. These were used to produce the four study gradation types: one coarse-graded and three fine-graded 19.0 mm NMAS binder course mixtures. These included: 19.0 mm NMAS crushed dolomite designated by IDOT as CM11; a 9.5 mm NMAS crushed dolomite (CM16); a manufactured dolomite sand (FM20); and a natural sand (FM02). Additionally, mineral filler (MF) manufactured by Hanson, in Thornton, IL, was used to represent plant breakdown. The aggregate sampling procedure followed is specified in ASTM D75 "Standard Practice of Sampling Aggregates." Briefly, the steps used to obtain large sampling quantities were as follows.

1. An open trailer pulled by a pickup truck was weighed empty as shown in Figure 3.

Figure 3. Truck and trailer at weigh station.

2. Aggregate was sampled from multiple, random locations in the stockpile using a front-end loader as illustrated in Figure 4. Bulk sampling of the material for mix design and performance testing was achieved by collecting multiple trailer loads of each material.

Figure 4. Loading of sampling trailer with bulk aggregate samples.

3. Multiple loads and hauls were performed until the required amounts of bulk aggregate samples were obtained. The trailer was cleaned to avoid contamination before switching materials. Sampled aggregates were kept inside a tent at ATREL, as shown in Figure 5.

Figure 5. Sampled aggregate stockpile at ATREL.

4. To obtain representative samples for physical property testing, stockpiles at Open Road Paving were leveled per approved IDOT methods with a bucket loader to obtain a large, flat sampling surface, and sampled from quarters following IDOT stockpile sampling procedures, as shown in Figure 6.

Figure 6. Stockpile sampling for physical property testing.

5. A total of 15 bags of each of the coarse aggregates (CM11 and CM16) and ten bags of each of the fine aggregates (FM20 and FM02) were collected for physical property testing (Figure 7).

Figure 7. Bagged aggregate samples for physical property testing.

Aggregate samples were then tested and physical properties and gradations determined by researchers at ATREL and IDOT's Bureau of Materials and Physical Research. Results obtained were consistent with values reported by the paving contractor.

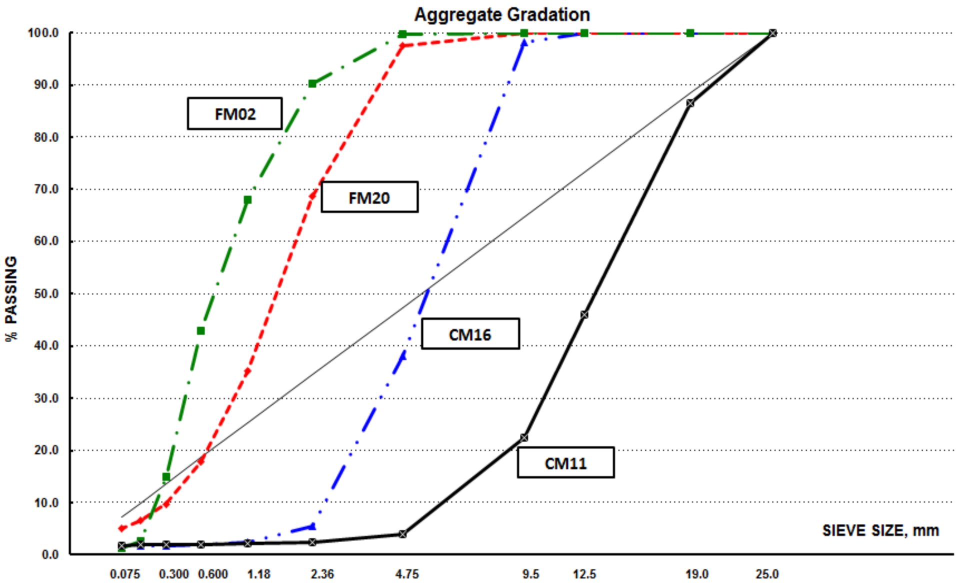



Figure 8. Aggregate gradations.

Figure 8 presents the grain-size distribution plots for the aggregates used in the study, as displayed on a 0.45-power chart (percent passing vs. sieve size raised to the 0.45 power). Table 4 summarizes the gradations and physical properties measured on the four study aggregates. Additionally, bulk (dry) specific gravity of all aggregates and fine aggregate angularity (FAA) of the natural and manufactured sands as measured by uncompacted void content of fine aggregates AASHTO T304 (Method A) was provided by the IDOT Bureau of Materials and Physical Research (BMPR).

Table 4. Gradation and Physical Properties of the Study Aggregates

| Material Code No. | CM11 | CM16 | FM20 | FM02 | MF |
|---|---|---|---|---|---|
| Sieve Size | % Passing Sieve | | | | |
| 1 in (25.0 mm) | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |
| 3/4 in (19.0 mm) | 86.6 | 100.0 | 100.0 | 100.0 | 100.0 |
| 1/2 in (12.5 mm) | 46.0 | 100.0 | 100.0 | 100.0 | 100.0 |
| 3/8 in (9.5 mm) | 22.5 | 98.2 | 100.0 | 100.0 | 100.0 |
| No. 4 (4.75 mm) | 3.9 | 38.1 | 97.5 | 99.7 | 100.0 |
| No. 8 (2.36 mm) | 2.5 | 5.5 | 68.6 | 90.2 | 100.0 |
| No. 16 (1.18 mm) | 2.2 | 2.5 | 35.2 | 68.1 | 100.0 |
| No. 30 (600 µm) | 2.0 | 1.9 | 17.8 | 42.9 | 100.0 |
| No. 50 (300 µm) | 2.0 | 1.8 | 9.8 | 14.9 | 100.0 |
| No. 100 (150 µm) | 1.9 | 1.7 | 6.7 | 2.6 | 95.0 |
| No. 200 (75 µm) | 1.8 | 1.7 | 5.1 | 1.4 | 90.0 |
| Aggregate Physical Property | | | | | |
| Bulk (Dry) Spec Gravity | 2.632 | 2.620 | 2.606 | 2.565 | 2.900 |
| Apparent Spec Gravity | 2.763 | 2.796 | 2.811 | 2.689 | 2.900 |
| Absorption (%) | 1.8 | 2.4 | 2.8 | 1.8 | 1.000 |
| FAA (%) | — | — | 38.9 | 44.7 | — |
| Unit Weight Values Required for Bailey Method | | | | | |
| Loose Unit Weight (LUW), Kg/m$^3$ | 1,447.1 | 1,447.7 | 1,616.8 | 1,639.4 | — |
| Rodded Unit Weight (RUW) Kg/m$^3$ | 1,614.4 | 1,581.1 | 1,794.3 | 1,770.5 | — |
| Solid Unit Weight (SUW), Kg/m$^3$ | 2,632.0 | 2,620.0 | 2,606.0 | 2,565.0 | — |
| Voids for LUW (%) | 45.0 | 44.7 | 38.0 | 36.1 | — |
| Voids for RUW (%) | 38.7 | 39.7 | 31.1 | 31.0 | — |

### 3.3 FRACTIONATION OF AGGREGATES

To ensure the highest degree of accuracy in mix designs for research purposes, each aggregate stockpile was carefully fractionated over a broad range of sieves, including the following mesh sizes: 3/4 in (19.0 mm), 1/2 in (12.5 mm), 3/8 in (9.5 mm), No. 4 (4.75 mm), No. 8 (2.36 mm), No. 16 (1.18 mm), and No. 30 (600 µm). Figure 9 shows the high-capacity fractionator used to screen coarse aggregates.

Figure 9. Fractionator (left) and fractionated coarse aggregates (right).

When required, material passing the No. 4 (4.75 mm) screen deck of the fractionator was re-fractionated using a Mary Ann shaker, as shown in Figure 10. The Mary Ann shaker was also used for fractionating the FM20 and FM02 fine aggregates.

Figure 10. Mary Ann shaker and 12 in diameter sieves used.

# CHAPTER 4 MIX DESIGN

Following the guidance of the project Technical Review Panel, a comparison of three fine-graded (F-G) mixes and a reference, traditional coarse-graded (C-G) mix was undertaken. The philosophy used was to hold all variables except gradation as constant as possible, with particular emphasis on using similar voids in mineral aggregate (VMA) levels. In addition to IDOT modified Superpave mix design methods, the Bailey Method was used to systematically design the four study aggregate structures. A summary of mix design parameters used is as follows:

- NMAS: 3/4 in (19.0 mm)
- Design gyrations ($N_d$): 90
- Design air voids: 4.0%
- Minimum VMA: 13.0%
- Voids filled with asphalt (VFA): 65%–75%
- Dust-to-total AC ratio: 0.6–1.0

Even though a design target minimum VMA of 13.0% was specified, mixtures were designed in a narrow range of 13.4 ± 0.1% to be consistent with mix design practices in Illinois. For mix design, a GYR-001 (IPC) servo-type gyratory compactor was used.

## 4.1 BAILEY METHOD

According to standard Superpave mix design practices, the aggregate structure is designed using a trial-and-error process to achieve mix volumetric targets, without regard to particle packing or detailed attention to aggregate structure. Inspired by techniques developed by Robert D. Bailey, of IDOT District 5 throughout the course of his career as an asphalt mix design specialist, Pine et al. (2001) introduced the Bailey Method for systematic design of the aggregate skeleton. The method is based on principles of particle packing, as described by parameters that investigate coarse fraction volume, along with ratios across various fractions of the mix gradation under design. The technique can be used to design coarse-graded, fine-graded, and stone-mastic asphalt (SMA); in addition to mixtures including RAP.

The Bailey Method is not a mix design method; rather, it is used in conjunction with the mix design procedure to systematically design and control certain aspects of particle packing. Principle 1 of the Bailey Method focuses on the bulk volume of coarse fraction (i.e., plus primary control sieve [PCS]) and the resulting volume of voids, which are filled with an equal volume of fine fraction. For dense-graded mixtures (coarse or fine-graded), the Bailey Method references the CA loose unit weight (LUW) for establishing C-G vs. F-G gradation blends. Principles 2, 3, and 4, are ratios that describe specific portions of the aggregate blend (i.e., CA ratio, $FA_c$ ratio, and $FA_f$ ratio).

In this method, the primary control sieve (PCS) is determined based upon the NMAS of the mixture and the assumption that particles having a diameter ratio of 0.22 or less of a reference particle size will fit into the voids of the reference particle size. For a mixture with 19.0 mm NMAS, the PCS is determined by multiplying 19.0 x 0.22, which yields 4.180 mm. The closest sieve in size (to the resulting value of 4.180 mm) typically used in a standard set of sieves in the United States is the 4.75 mm, therefore, the 4.75 mm sieve represents the PCS of this blend (PCS values can also be simply obtained from pre-solved tables described in the Bailey Method).

Other important Bailey Method principles utilize the half sieve, secondary control sieve, and tertiary control sieve. Figure 11 presents a conceptual descripton of the various Bailey control sieve definitions. The Bailey Method utilizes a series of ratios computed using various combinations of the aforementioned parameters. The coarse aggregate ratio (CA ratio) is used to quantify the degree of packing in the coarse fraction (i.e., plus PCS). The CA ratio is calculated as follows:

$$CA\ ratio = \frac{\%\text{passing half sieve} - \%\text{ passing primary control sieve (PCS)}}{100\% - \%\text{ passing half sieve}}.$$

The fine fraction (i.e., minus PCS) is characterized by two principles: one describing the overall fine fraction of the minus PCS, $FA_c$, and one describing the fine portion of the fine fraction, $FA_f$, which are computed as follows:

$$FA_c = \frac{\%\text{ passing secondary control sieve (SCS)}}{\%\text{ passing primary control sieve (PCS)}},$$

$$FA_f = \frac{\%\text{ passing tertiary control sieve (TCS)}}{\%\text{ passing secondary control sieve (SCS)}}$$

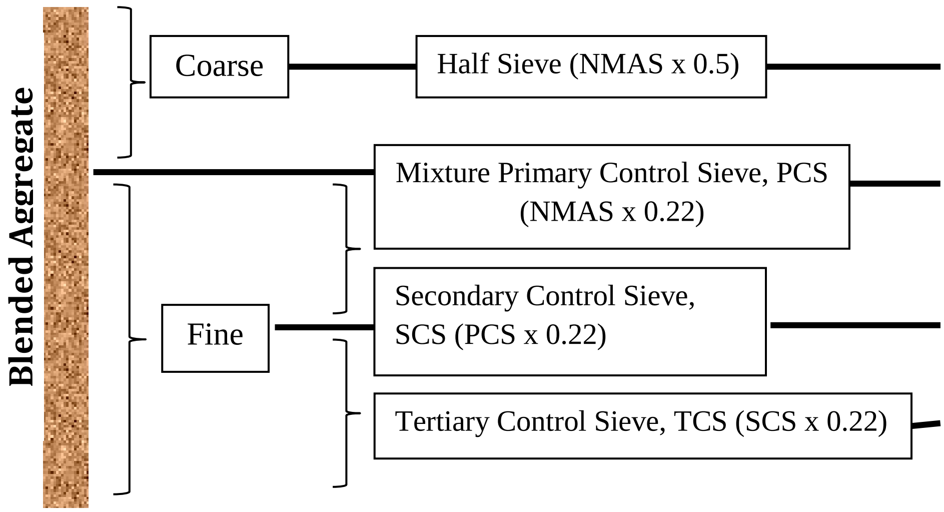



Figure 11. Summary of ratios used in Bailey Method.

Table 5 provides a summary of the formulas used to compute CA, $FA_c$, and $FA_f$ for the coarse and fine-graded mixtures.

Table 5: Formulas for Bailey Ratios Used in Study

| NMAS = 19.0 mm | | | |
|---|---|---|---|
| Mixture | CA ratio | $FA_c$ ratio | $FA_f$ ratio |
| Coarse-Graded | $\frac{(9.5\ \text{mm} - 4.75\ \text{mm})}{(100\% - 9.5\ \text{mm})}$ | $\frac{1.18\ \text{mm}}{4.75\ \text{mm}}$ | $\frac{0.300\ \text{mm}}{1.18\ \text{mm}}$ |
| Mixture | New CA ratio | New $FA_c$ ratio | New $FA_f$ ratio |
| Fine-Graded | $\frac{(2.36\ \text{mm} - 1.18\ \text{mm})}{(4.75\ \text{mm} - 2.36\ \text{mm})}$ | $\frac{0.300\ \text{mm}}{1.18\ \text{mm}}$ | $\frac{0.075\ \text{mm}}{0.300\ \text{mm}}$ |

Throughout this report, mixtures are identified by a numeric identification code that represents mixture variables. From coarsest to finest, the four study mixtures are denoted: coarse-graded (C-G) mix or control mix, fine-graded mix No.1 (FG01), fine-graded mix No. 2 (FG02), and fine-graded mix No. 3 (FG03).

Table 6 summarizes final results of the four Bailey principles for all four mixes. Although some of the values listed are outside of the corresponding recommended range for a F-G mixture, it should be noted that the suggested range for each Bailey principle provides a starting point for designers with field compactability and segregation susceptibility in mind, based on historical data. Designers are encouraged to evaluate existing blends to determine more appropriate ranges for their specific (local) materials. Additionally, the suggested Bailey principle ranges provide initial targets for mix designs, guidance for mix design adjustment, and insight when used in forensic mixture /pavement investigation.

Table 6. Bailey Principles for the 19.0 mm NMAS Mixtures

| Bailey | Fine-Graded Mix | | | | Coarse-Graded Mix | |
|---|---|---|---|---|---|---|
| Principle | FG01 | FG02 | FG03 | Rec.* | C-G | Rec.* |
| CA LUW | 85.0 | 73.9 | 62.5 | 60–85 | 103.8 | 95–105 |
| CA | | | | | 0.750 | 0.60–0.75 |
| $FA_c$ | | | | | 0.403 | 0.35–0.50 |
| $FA_f$ | | | | | 0.456 | 0.35–0.50 |
| New CA | 0.561 | 0.668 | 0.870 | 0.60–1.00 | | |
| New $FA_c$ | 0.403 | 0.363 | 0.339 | 0.35–0.50 | | |
| New $FA_f$ | 0.630 | 0.584 | 0.547 | 0.35–0.50 | | |

Rec.* = Recommended range

### 4.2 SUPERPAVE MIX DESIGNS

Tables 7 through 10 summarizes results of mix design trials leading to final mix designs meeting Superpave criteria and study objectives of achieving similar volumetric parameters for the four distinctly different aggregate structures. According to the Bailey Method, after the first mix design trial blend for a given aggregate structure was completed, the trial blend was examined in terms of compliance to target gradation and required and target volumetric properties. If unsatisfactory, the trial blend was adjusted based on the test results of the first trial and Bailey principles. Mix design iterations required no more than three trials.

Table 7. Development of Mix Design for Fine-Graded Mix No.1 (FG01)

| ***MIX ID*** | **FG01 - Trial 1** | | **FG01 - Trial 2** | | **FG01 - Trial 3** | | **Three-Point Air Void Determination** | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Fine-Graded - Mix 1** | Design | Batched | Design | Batched | Design | Batched | Design | OPT AC - 0.5% | OPT AC | OPT AC - 0.5% |
| **(FG01)** | Target 1 | Blend 1 | Target 2 | Blend 2 | Target 3 | Blend 3 | Target 3 | 4.9% | 5.4% | 5.9% |
| **Blend Percentages** | | | | | | | | | | |
| **Dust Adjustment** | | **Yes** | | **Yes** | | **Yes** | | **Yes** | | |
| CM 11 | 32.3 | 32.4 | 32.3 | 32.3 | 32.3 | 32.5 | 32.3 | 32.5 | 32.5 | 32.5 |
| CM 16 | 34.0 | 34.2 | 34.0 | 34.0 | 34.0 | 34.2 | 34.0 | 34.2 | 34.2 | 34.2 |
| FM 20 | 22.2 | 22.3 | 22.0 | 22.5 | 21.8 | 21.9 | 21.8 | 21.9 | 21.9 | 21.9 |
| FM 02 | 9.7 | 9.7 | 9.6 | 9.7 | 9.4 | 9.4 | 9.4 | 9.4 | 9.4 | 9.4 |
| Mineral Filler | 1.8 | 1.4 | 2.1 | 1.5 | 2.5 | 2.0 | 2.5 | 2.0 | 2.0 | 2.0 |
| Combined Agg Gsb | 2.620 | | 2.621 | | 2.622 | | 2.622 | | | |
| Percent Asphalt | | 5.4 | | 5.5 | | 5.5 | | 4.9 | 5.4 | 5.9 |
| **Percent Passing from Washed Gradations** | | | | | | | | | | |
| 25.0 mm | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | | |
| 19.0 mm | 95.7 | 96.1 | 95.7 | 95.5 | 95.7 | 95.7 | 95.7 | 96.0 | | |
| 12.5 mm | 82.6 | 82.7 | 82.6 | 82.5 | 82.6 | 82.6 | 82.6 | 82.3 | | |
| 9.5 mm | 74.4 | 74.9 | 74.4 | 74.9 | 74.4 | 74.8 | 74.4 | 74.9 | | |
| 6.25 mm | - | 57.8 | - | 58.8 | 58.1 | 58.7 | 58.1 | 58.2 | | |
| 4.75 mm | 47.3 | 48.0 | 47.3 | 48.3 | 47.3 | 48.0 | 47.3 | 47.4 | | |
| 2.36 mm | 28.5 | 28.9 | 28.5 | 29.2 | 28.6 | 29.3 | 28.6 | 29.0 | | |
| 1.18 mm | 17.8 | 19.2 | 17.9 | 18.0 | 18.1 | 18.2 | 18.1 | 18.3 | | |
| 0.6 mm | 11.2 | 11.6 | 11.4 | 11.6 | 11.7 | 12.0 | 11.7 | 11.9 | | |
| 0.3 mm | 6.7 | 6.7 | 6.9 | 6.7 | 7.3 | 7.3 | 7.3 | 7.3 | | |
| 0.15 mm | 4.6 | 4.9 | 4.9 | 4.9 | 5.3 | 5.5 | 5.3 | 5.5 | | |
| 0.075 mm | 4.0 | 4.2 | 4.3 | 4.2 | 4.6 | 4.7 | 4.6 | 4.6 | | |
| **Volumetrics** | | | | | | | | | | |
| Average Gmb | | 2.389 | | 2.382 | | 2.408 | | 2.381 | 2.402 | 2.414 |
| Average Gmm | | 2.503 | | 2.504 | | 2.503 | | 2.518 | 2.504 | 2.484 |
| Gb | | 1.036 | | 1.036 | | 1.036 | | 1.036 | 1.036 | 1.036 |
| Gse | | 2.723 | | 2.729 | | 2.728 | | 2.718 | 2.724 | 2.723 |
| Pba | | 1.50 | | 1.57 | | 1.53 | | 1.40 | 1.48 | 1.46 |
| Pbe | | 3.98 | | 4.02 | | 4.05 | | 3.57 | 4.00 | 4.53 |
| Voids | | 4.6 | | 4.9 | | 3.8 | | 5.4 | 4.1 | 2.8 |
| VMA | | 13.7 | | 14.1 | | 13.2 | | 13.6 | 13.3 | 13.4 |
| VFA | | 66.9 | | 65.5 | | 71.3 | | 60.1 | 69.5 | 78.9 |
| Dust / Total AC | | 0.7 | | 0.8 | | 0.8 | | 0.9 | 0.9 | 0.8 |
| Dust / Effective AC | | 1.0 | | 1.1 | | 1.1 | | 1.3 | 1.2 | 1.0 |

## Table 8. Development of Mix Design for Fine-Graded Mix No.2 (FG02)

| ***MIX ID*** | **FG02 - Trial 1** | | **FG02 - Trial 2** | | **FG02 - Trial 3** | | **Three-Point Air Void Determination** | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Fine-Graded - Mix 2** | Design | Batched | Design | Batched | Design | Batched | Design | OPT AC - 0.5% | OPT AC | OPT AC - 0.5% |
| **(FG02)** | Target 1 | Blend 1 | Target 2 | Blend 2 | Target 3 | Blend 3 | Target 3 | 5.0% | 5.5% | 6.0% |
| **Blend Percentages** | | | | | | | | | | |
| **Dust Adjustment** | | **Yes** | | **Yes** | | **Yes** | | **Yes** | | |
| CM 11 | 29.3 | 29.5 | 29.3 | 29.5 | 29.3 | 29.5 | 29.3 | 29.5 | 29.5 | 29.5 |
| CM 16 | 29.4 | 29.6 | 29.4 | 29.6 | 29.4 | 29.6 | 29.4 | 29.6 | 29.6 | 29.6 |
| FM 20 | 27.1 | 27.2 | 27.3 | 27.2 | 27.2 | 27.3 | 27.2 | 27.3 | 27.3 | 27.3 |
| FM 02 | 11.8 | 11.9 | 11.9 | 11.9 | 11.9 | 11.9 | 11.9 | 11.9 | 11.9 | 11.9 |
| Mineral Filler | 2.4 | 1.8 | 2.1 | 1.8 | 2.2 | 1.7 | 2.2 | 1.7 | 1.7 | 1.7 |
| Combined Agg Gsb | 2.619 | | 2.618 | | 2.619 | | 2.619 | | | |
| Percent Asphalt | | 5.5 | | 5.6 | | 5.6 | | 5.0 | 5.5 | 6.0 |
| **Percent Passing from Washed Gradations** | | | | | | | | | | |
| 25.0 mm | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | | |
| 19.0 mm | 96.1 | 96.3 | 96.1 | 96.3 | 96.1 | 95.8 | 96.1 | 96.1 | | |
| 12.5 mm | 84.2 | 84.0 | 84.2 | 84.1 | 84.2 | 84.2 | 84.2 | 84.0 | | |
| 9.5 mm | 76.8 | 77.4 | 76.8 | 77.4 | 76.8 | 77.3 | 76.8 | 77.2 | | |
| 6.25 mm | 62.5 | 62.7 | 62.5 | 62.8 | 62.5 | 62.4 | 62.5 | 62.5 | | |
| 4.75 mm | 52.9 | 52.9 | 52.9 | 52.9 | 52.9 | 53.3 | 52.9 | 52.9 | | |
| 2.36 mm | 34.0 | 34.1 | 33.9 | 34.3 | 33.9 | 34.4 | 33.9 | 34.3 | | |
| 1.18 mm | 21.3 | 21.7 | 21.2 | 21.7 | 21.2 | 21.5 | 21.2 | 21.2 | | |
| 0.6 mm | 13.4 | 13.6 | 13.2 | 13.6 | 13.2 | 13.7 | 13.2 | 13.5 | | |
| 0.3 mm | 7.9 | 7.5 | 7.6 | 7.5 | 7.6 | 7.7 | 7.6 | 7.6 | | |
| 0.15 mm | 5.5 | 5.4 | 5.2 | 5.4 | 5.2 | 5.5 | 5.2 | 5.4 | | |
| 0.075 mm | 4.7 | 4.6 | 4.5 | 4.6 | 4.5 | 4.6 | 4.5 | 4.5 | | |
| **Volumetrics** | | | | | | | | | | |
| Average Gmb | | 2.409 | | 2.402 | | 2.399 | | 2.387 | 2.396 | 2.418 |
| Average Gmm | | 2.509 | | 2.504 | | 2.498 | | 2.519 | 2.504 | 2.485 |
| Gb | | 1.036 | | 1.036 | | 1.036 | | 1.036 | 1.036 | 1.036 |
| Gse | | 2.735 | | 2.734 | | 2.726 | | 2.724 | 2.729 | 2.729 |
| Pba | | 1.68 | | 1.68 | | 1.56 | | 1.53 | 1.60 | 1.59 |
| Pbe | | 3.91 | | 4.01 | | 4.13 | | 3.55 | 3.99 | 4.51 |
| Voids | | 4.0 | | 4.1 | | 4.0 | | 5.2 | 4.3 | 2.7 |
| VMA | | 13.1 | | 13.4 | | 13.5 | | 13.4 | 13.5 | 13.2 |
| VFA | | 69.5 | | 69.6 | | 70.7 | | 60.9 | 68.2 | 79.6 |
| Dust / Total AC | | 0.9 | | 0.8 | | 0.8 | | 0.9 | 0.8 | 0.8 |
| Dust / Effective AC | | 1.2 | | 1.1 | | 1.1 | | 1.3 | 1.1 | 1.0 |

## Table 9. Development of Mix Design for Fine-Graded Mix No.3 (FG03)

| ***MIX ID*** | **FG03 - Trial 1** | | **FG03 - Trial 2** | | **FG03 - Trial 3** | | **Three-Point Air Void Determination** | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Fine-Graded - Mix 3** | Design | Batched | Design | Batched | Design | Batched | Design | OPT AC - 0.5% | OPT AC | OPT AC - 0.5% |
| **(FG03)** | Target 1 | Blend 1 | Target 2 | Blend 2 | Target 3 | Blend 3 | Target 3 | 5.0% | 5.5% | 6.0% |
| **Blend Percentages** | | | | | | | | | | |
| **Dust Adjustment** | | **Yes** | | **Yes** | | **Yes** | | | **Yes** | |
| CM 11 | 26.3 | 26.5 | 27.7 | 27.9 | 30.3 | 30.5 | 30.3 | 30.5 | 30.5 | 30.5 |
| CM 16 | 24.4 | 24.6 | 22.9 | 23.1 | 18.5 | 18.6 | 18.5 | 18.6 | 18.6 | 18.6 |
| FM 20 | 32.7 | 32.8 | 32.8 | 32.9 | 34.1 | 34.3 | 34.1 | 34.3 | 34.3 | 34.3 |
| FM 02 | 14.5 | 14.5 | 14.5 | 14.5 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 | 15.0 |
| Mineral Filler | 2.1 | 1.6 | 2.1 | 1.6 | 2.1 | 1.6 | 2.1 | 1.6 | 1.6 | 1.6 |
| Combined Agg Gsb | 2.616 | | 2.616 | | 2.616 | | 2.616 | | | |
| Percent Asphalt | | 5.6 | | 5.7 | | 5.7 | | 5.0 | 5.5 | 6.0 |
| **Percent Passing from Washed Gradations** | | | | | | | | | | |
| 25.0 mm | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | | 100.0 | |
| 19.0 mm | 96.5 | 96.2 | 96.3 | 96.3 | 95.9 | 96.4 | 95.9 | | 95.7 | |
| 12.5 mm | 85.8 | 86.0 | 85.0 | 84.8 | 83.6 | 83.5 | 83.6 | | 83.2 | |
| 9.5 mm | 79.2 | 79.7 | 78.1 | 78.4 | 76.2 | 76.6 | 76.2 | | 76.3 | |
| 6.25 mm | 67.0 | 67.0 | 66.2 | 66.0 | 65.3 | 65.2 | 65.3 | | 65.3 | |
| 4.75 mm | 58.8 | 58.8 | 58.3 | 58.3 | 58.5 | 58.7 | 58.5 | | 58.6 | |
| 2.36 mm | 39.6 | 40.1 | 39.6 | 40.1 | 40.8 | 41.3 | 40.8 | | 41.3 | |
| 1.18 mm | 24.7 | 25.3 | 24.7 | 25.0 | 25.4 | 25.8 | 25.4 | | 25.7 | |
| 0.6 mm | 15.1 | 15.6 | 15.1 | 15.3 | 15.6 | 15.8 | 15.6 | | 15.8 | |
| 0.3 mm | 8.4 | 8.3 | 8.4 | 8.1 | 8.6 | 8.4 | 8.6 | | 8.3 | |
| 0.15 mm | 5.5 | 5.6 | 5.5 | 5.5 | 5.6 | 5.7 | 5.6 | | 5.6 | |
| 0.075 mm | 4.6 | 4.7 | 4.6 | 4.6 | 4.7 | 4.8 | 4.7 | | 4.7 | |
| **Volumetrics** | | | | | | | | | | |
| Average Gmb | | 2.392 | | 2.392 | | 2.407 | | 2.384 | 2.399 | 2.408 |
| Average Gmm | | 2.504 | | 2.497 | | 2.495 | | 2.523 | 2.497 | 2.479 |
| Gb | | 1.036 | | 1.036 | | 1.036 | | 1.036 | 1.036 | 1.036 |
| Gse | | 2.734 | | 2.730 | | 2.727 | | 2.729 | 2.720 | 2.721 |
| Pba | | 1.71 | | 1.65 | | 1.61 | | 1.64 | 1.52 | 1.53 |
| Pbe | | 3.99 | | 4.14 | | 4.18 | | 3.44 | 4.06 | 4.56 |
| Voids | | 4.5 | | 4.2 | | 3.5 | | 5.5 | 3.9 | 2.9 |
| VMA | | 13.7 | | 13.8 | | 13.2 | | 13.4 | 13.3 | 13.5 |
| VFA | | 67.3 | | 69.5 | | 73.3 | | 59.0 | 70.6 | 78.7 |
| Dust / Total AC | | 0.8 | | 0.8 | | 0.8 | | 0.9 | 0.9 | 0.8 |
| Dust / Effective AC | | 1.2 | | 1.1 | | 1.1 | | 1.4 | 1.2 | 1.0 |

Table 10. Development of Mix Design for the Coarse-Graded (C-G) Control Mix

| ***MIX ID*** | **CG - Trial 1** | | **CG - Trial 2** | | **CG - Trial 3** | | **Three-Point Air Void Determination** | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Control Mix** | Design | Batched | Design | Batched | Design | Batched | Design | OPT AC - 0.5% | OPT AC | OPT AC - 0.5% |
| **(CG)** | Target 1 | Blend 1 | Target 2 | Blend 2 | Target 3 | Blend 3 | Target 3 | 4.9% | 5.4% | 5.9% |
| **Blend Percentages** | | | | | | | | | | |
| **Dust Adjustment** | | **Yes** | | **Yes** | | **Yes** | | | **Yes** | |
| CM 11 | 46.5 | 46.9 | 45.8 | 46.1 | 45.8 | 46.1 | 45.8 | 46.1 | 46.1 | 46.1 |
| CM 16 | 26.6 | 26.8 | 30.1 | 30.4 | 30.1 | 30.4 | 30.1 | 30.4 | 30.4 | 30.4 |
| FM 20 | 14.2 | 14.2 | 12.6 | 12.6 | 12.6 | 12.5 | 12.6 | 12.5 | 12.5 | 12.5 |
| FM 02 | 10.0 | 10.0 | 8.7 | 8.7 | 8.7 | 8.6 | 8.7 | 8.6 | 8.6 | 8.6 |
| Mineral Filler | 2.7 | 2.1 | 2.8 | 2.2 | 2.8 | 2.4 | 2.8 | 2.4 | 2.4 | 2.4 |
| Combined Agg Gsb | 2.625 | | 2.626 | | 2.626 | | 2.626 | | | |
| Percent Asphalt | | 5.5 | | 5.3 | | 5.5 | | 4.9 | 5.4 | 5.9 |
| **Percent Passing from Washed Gradations** | | | | | | | | | | |
| 25.0 mm | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | | 100.0 | |
| 19.0 mm | 93.8 | 94.3 | 93.9 | 94.3 | 93.9 | 93.9 | 93.9 | | 93.6 | |
| 12.5 mm | 74.9 | 74.8 | 75.3 | 75.5 | 75.3 | 75.2 | 75.3 | | 75.3 | |
| 9.5 mm | 63.5 | 64.0 | 64.0 | 64.4 | 64.0 | 64.7 | 64.0 | | 64.5 | |
| 6.25 mm | 47.4 | 47.3 | 46.8 | 47.0 | 46.8 | 47.1 | 46.8 | | 46.8 | |
| 4.75 mm | 38.5 | 38.5 | 37.0 | 37.0 | 37.0 | 37.0 | 37.0 | | 38.8 | |
| 2.36 mm | 24.1 | 24.3 | 22.1 | 22.1 | 22.1 | 22.2 | 22.1 | | 22.2 | |
| 1.18 mm | 16.2 | 16.5 | 14.9 | 14.7 | 14.9 | 14.7 | 14.9 | | 15.2 | |
| 0.6 mm | 11.0 | 11.1 | 10.3 | 10.0 | 10.3 | 10.3 | 10.3 | | 10.4 | |
| 0.3 mm | 7.0 | 6.7 | 6.8 | 6.3 | 6.8 | 6.5 | 6.8 | | 6.5 | |
| 0.15 mm | 5.1 | 5.2 | 5.1 | 5.0 | 5.1 | 5.2 | 5.1 | | 5.1 | |
| 0.075 mm | 4.6 | 4.6 | 4.6 | 4.4 | 4.6 | 4.6 | 4.6 | | 4.6 | |
| **Volumetrics** | | | | | | | | | | |
| Average Gmb | | 2.405 | | 2.392 | | 2.404 | | 2.390 | 2.404 | 2.418 |
| Average Gmm | | 2.493 | | 2.511 | | 2.498 | | 2.518 | 2.496 | 2.483 |
| Gb | | 1.036 | | 1.036 | | 1.036 | | 1.036 | 1.036 | 1.036 |
| Gse | | 2.715 | | 2.728 | | 2.722 | | 2.718 | 2.714 | 2.721 |
| Pba | | 1.31 | | 1.48 | | 1.38 | | 1.34 | 1.28 | 1.38 |
| Pbe | | 4.26 | | 3.90 | | 4.20 | | 3.63 | 4.19 | 4.60 |
| Voids | | 3.5 | | 4.7 | | 3.8 | | 5.1 | 3.7 | 2.6 |
| VMA | | 13.4 | | 13.7 | | 13.5 | | 13.4 | 13.4 | 13.4 |
| VFA | | 73.7 | | 65.5 | | 72.1 | | 62.2 | 72.5 | 80.4 |
| Dust / Total AC | | 0.8 | | 0.9 | | 0.8 | | 0.9 | 0.9 | 0.8 |
| Dust / Effective AC | | 1.1 | | 1.2 | | 1.1 | | 1.3 | 1.1 | 1.0 |

A summary of the final mix designs is provided in Table 11 and Figure 12. The four study mixtures were designed at 13.4% VMA, and optimum asphalt content was chosen at 4.0% air voids. Therefore, the volume of effective asphalt was the same for all four mixtures. However, the F-G mixtures had approximately 0.2% more absorbed asphalt (by weight) than the C-G control mix, which resulted in slightly higher optimum asphalt contents for the fine mixtures. The slight increase in absorbed asphalt was because the fine aggregates had slightly higher water absorption compared to the coarse aggregates. A common misunderstanding is that fine-graded mixtures require significantly more binder than coarse-graded mixtures for a given NMAS. This belief stems from the fact that F-G mixtures have more aggregate surface area than C-G mixtures, which leads to the incorrect conclusion that it results in a higher optimum asphalt content. Optimum asphalt content is driven by VMA, voids, and asphalt absorption. Asphalt absorption is a function of several factors, such as

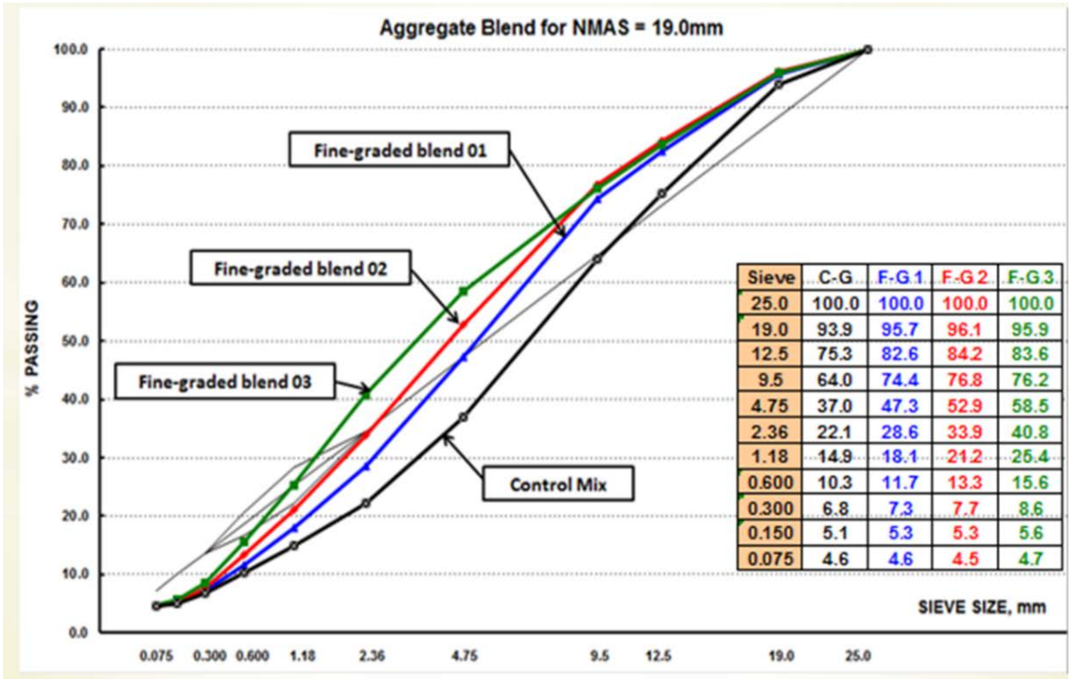
- combined aggregate blend water absorption (the most critical factor);
- porosity index of the aggregates;
- mixture moisture content;
- asphalt binder properties;

- cure time or silo time;
- truck haul time; and
- mixture temperature.

Water absorption drives asphalt absorption, which in turn creates different optimum asphalt contents between two mixtures that have equal VMA and air voids. Figure 12 shows that the four study mixtures span across the aggregate gradation range typically associated with coarse-graded and fine-graded mixes. Although in practice some 19.0 mm NMAS coarse-graded binder course mixtures in Illinois have even coarser gradations (below 37% passing the 4.75 mm sieve), the C-G mixture designed is typical of binder course mixtures used in central Illinois; moreover, similar to known field trials in central Illinois where both C-G and experimental F-G mixtures are used. The finest mixture, FG03, was designed to be at the upper range anticipated in Illinois as determined by the Technical Review Panel.

Table 11. Volumetric Properties of the Mixtures

| Property | Mixture | | | |
|---|---|---|---|---|
| | FG01 | FG02 | FG03 | Control |
| Asphalt Content (% of Total Mix), $P_b$ | 5.4 | 5.6 | 5.5 | 5.3 |
| Specific Gravity of Asphalt, $G_b$ | 1.036 | 1.036 | 1.036 | 1.036 |
| Aggregate Bulk (Dry) Specific Gravity, $G_{sb}$ | 2.622 | 2.619 | 2.616 | 2.626 |
| Aggregate Effective Specific Gravity, $G_{se}$ | 2.722 | 2.727 | 2.723 | 2.718 |
| Maximum Theoretical Specific Gravity, $G_{mm}$ | 2.502 | 2.499 | 2.500 | 2.502 |
| Mixture Bulk Specific Gravity, $G_{mb}$ | 2.402 | 2.399 | 2.400 | 2.402 |
| Asphalt Absorption, $P_{ba}$ | 1.4 | 1.6 | 1.6 | 1.3 |
| Effective Asphalt Content, $P_{be}$ | 4.1 | 4.1 | 4.1 | 4.1 |
| Percent Air Voids (%), $V_a$ | 4.0 | 4.0 | 4.0 | 4.0 |
| Percent VMA (%) | 13.4 | 13.4 | 13.4 | 13.4 |
| Percent VFA (%) | 70 | 70 | 70 | 70 |
| Dust/Total AC | 0.9 | 0.8 | 0.9 | 0.9 |
| Dust/Effective AC | 1.1 | 1.1 | 1.1 | 1.1 |




| Sieve | C-G | F-G 1 | F-G 2 | F-G 3 |
|---|---|---|---|---|
| 25.0 | 100.0 | 100.0 | 100.0 | 100.0 |
| 19.0 | 93.9 | 95.7 | 96.1 | 95.9 |
| 12.5 | 75.3 | 82.6 | 84.2 | 83.6 |
| 9.5 | 64.0 | 74.4 | 76.8 | 76.2 |
| 4.75 | 37.0 | 47.3 | 52.9 | 58.5 |
| 2.36 | 22.1 | 28.6 | 33.9 | 40.8 |
| 1.18 | 14.9 | 18.1 | 21.2 | 25.4 |
| 0.600 | 10.3 | 11.7 | 13.3 | 15.6 |
| 0.300 | 6.8 | 7.3 | 7.7 | 8.6 |
| 0.150 | 5.1 | 5.3 | 5.3 | 5.6 |
| 0.075 | 4.6 | 4.6 | 4.5 | 4.7 |

Figure 12. Aggregate structures of the four study mixtures.

## 4.3 EVALUATION OF MOISTURE SUSCEPTIBILITY

Once mixture volumetrics are met, the last step of the Superpave mix design procedure usually involves a check of the moisture sensitivity of the designed mixture. The test was performed in accordance with the IDOT Modified Test Procedure AASHTO T283-07, "resistance of compacted bituminous mixtures to moisture-induced damage." Briefly, six replicates of each mixture were compacted to 7.0 ± 0.5% air voids and approximately 95 mm height. Three replicates were left unconditioned, which is termed the "dry subset," while the other three replicates were moisture conditioned, which is termed the "wet subset." The wet subset of specimens were conditioned using the Lottman procedure by applying a vacuum level of 250–650 mmHg for a short period of time (between 5 and 10 min) to achieve the target degree of saturation range of 70%–80%. After vacuum saturation, the wet subset specimens were soaked in a water bath at 60 ± 1°C for 24±1 hr. After the 24-hr soak, the specimens were placed into a water bath maintained at 25 ± 0.5°C for 2 ± 1 hr. Finally, the indirect tensile (IDT) test was performed on the wet subset to obtain the wet tensile strengths of the specimens, or "wet strength." Specimens in the dry subset were not subjected to vacuum saturation and required only 2 ± 1 hr of soaking in the water bath at 25 ± 0.5°C prior to conducting the IDT test, resulting in "dry strengths." According to the IDOT modified test procedure, a freeze-thaw cycle is not required. The tensile strength ratio (TSR) was computed by taking a ratio of the average wet strength to the average dry strength. A minimum requirement on the tensile strength is 60 psi and the minimum tensile strength ratio ($TSR_{min}$) is 85%.

Table 12. Results of Moisture Susceptibility Testing

| Parameters | Mixture ID | | | |
|---|---|---|---|---|
| | Control | FG01 | FG02 | FG03 |
| Air Void (%) | 6.8 | 6.8 | 6.7 | 6.9 |
| Partial Pressure (mm. Hg) | 254 | 254 | 254 | 254 |
| Time (min) | 10 | 10 | 10 | 10 |
| Level of Saturation (%) | 70 | 63 | 61 | 62 |
| Wet Tensile Strength, psi | 82.6 | 102.3 | 106.6 | 118.5 |
| Dry Tensile Strength, psi | 93.6 | 119.7 | 128.1 | 149.0 |
| TSR (%) | 88.2 | 85.5 | 83.2 | 79.5 |

Table 12 presents the results of the TSR tests conducted on all mixes. Based on the results, interestingly, only the control mix achieved the desired saturation level of the 70%–80%. None of the fine-graded mixes reached the specified saturation level. This was likely due to the reduced interconnectivity of voids inside fine-graded mixes. In terms of the tensile strength results, the finer the mixer, the greater the absolute tensile strength of the mix. This is perhaps a counter-intuitive result. This trend held true for both the wet and dry conditions. However, in terms of the TSR result, and quite interestingly, a reverse ranking was observed. This illustrates a potential flaw in the TSR test for fine-graded mixtures (and perhaps in general), in that mixtures with higher absolute strength sometimes have lower TSR values than mixtures with lower absolute strength, but a relatively high ratio of strengths in the wet and dry condition. A possible explanation is fine-graded mixtures would not be expected to take on much moisture in service due to better compactability and less interconnectivity of voids. This raises the question of whether or not the TSR test or specified requirements are applicable to F-G mixes. Additional moisture sensitivity testing was conducted to evaluate different saturation techniques, as reported in Appendix B. It should also be noted that, based on consensus of the Technical Review Panel, an anti-strip agent was not specified for any of the study mixtures for the purposes of accelerated pavement testing.

# CHAPTER 5 LABORATORY PERFORMANCE TESTING

This chapter presents an evaluation of laboratory performance testing including Hamburg wheel tracking (HWT), disc-shaped compact tension (DC(T)), dynamic modulus, and 4-pt flexural beam fatigue testing. In general, the literature suggests a wide range of opinions as to the relative rutting and crack resistance of coarse-graded and fine-graded mixtures. Thus, a suite of modern performance tests were run to characterize and compare the three fine-graded and one coarse-graded control mix designed in this study. This will also allow future comparison of laboratory performance test results to field performance results of field demonstration projects constructed in central Illinois with similar mixtures as used herein. Also, the HWT test results obtained herein also provide a basis for later comparison to performance results under accelerated pavement testing, as presented in Chapter 6.

## 5.1 HAMBURG WHEEL TRACKING (HWT) TEST

A Hamburg wheel tracking device (HWTD) was employed to evaluate the relative rutting resistance of the four study mixtures. Figure 13 illustrates the ATREL HWT test device and an example of the specimen configuration used. The HWT test procedure is specified by Texas Department of Transportation (TxDOT) in procedure Tex-242-F. The HWT test assesses rutting resistance subjecting compacted asphalt mixtures to loaded, rolling steel wheels on test specimens immersed in hot water (typically a test temperature of 50°C is used). A load level of 158 lb is used (Yetkin, 2007). Loading was applied for a total of 20,000 passes or until 20 mm deformation was reached, whichever occurred first. In terms of data interpretation, a maximum rut depth of 12.5 mm for 10,000 passes is recommended, when using a PG 64-*XX* binder.

Figure 13. Hamburg wheel tracking device (HWTD) and test setup.

Table 13 shows selected results from the HWT rutting test. All mixes were below the rutting threshold of 12.5 mm (approximately half of the threshold value was reached for each mixture). Therefore, both coarse-graded and fine-graded mixes are comparable in terms of rutting resistance as characterized by the HWT. It was not evident why the FG02 mix showed significantly different rutting behavior in the test at higher wheel pass levels. Retesting of mix FG02 led to essentially the same results. It was decided to revisit this result once field-produced mixture was obtained during construction of accelerated pavement test sections and tested in the HWT then analyzed.

Table 13. Rut Depth at 10,000 Passes for PG 64-22

| Mix ID | Rut Depth (mm) at 10,000 passes | | | Average (mm) |
|---|---|---|---|---|
| | Rep 1 | Rep 2 | Rep 3 | |
| Control | 6.90 | 4.53 | 4.71 | 5.38 |
| FG01 | 4.85 | 4.97 | 5.35 | 5.05 |
| FG02 | 6.53 | 6.46 | 6.64 | 6.54 |
| FG03 | 3.36 | 4.55 | 3.54 | 3.84 |

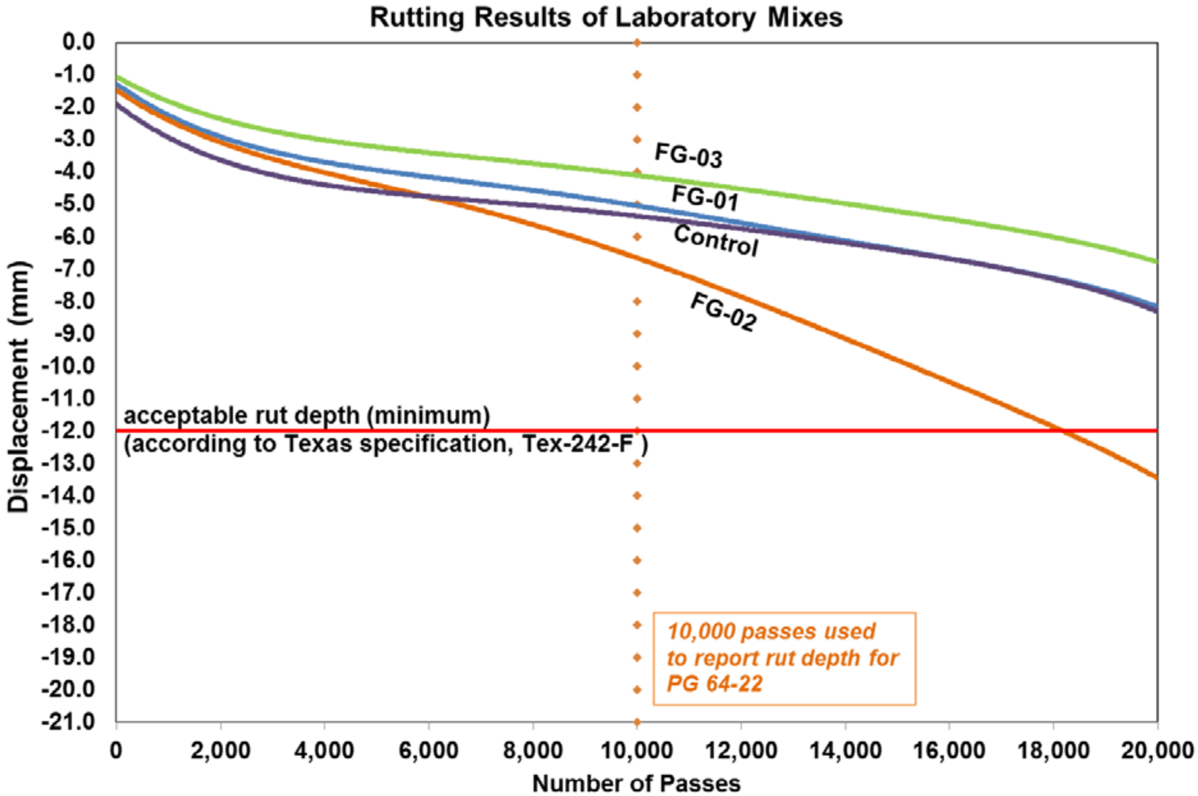



Figure 14. Comparison of Hamburg wheel track testing results for lab prepared mixes.

## 5.2 DISK-SHAPED COMPACT TENSION (DC(T)) FRACTURE TEST

The disk-shaped compact tension (DC(T)) test was used to evaluate the fracture behavior of the four study mixtures. At the onset of the study, there was a general lack of consensus on how the F-G mixtures would rank as compared to the C-G control mixtures in terms of fracture resistance. On one hand, the C-G mix has lower aggregate surface area yet the same effective asphalt content as F-G mixtures, suggesting higher film thickness. Because higher asphalt film thickness is often thought to be related to cracking resistance, some thought that the C-G mix might outperform the F-G mixes in terms of fracture resistance. Yet others were unconvinced that the general concept of film thickness would apply in the comparison of C-G vs. F-G mixes with similar effective asphalt and aggregate mineralogical composition.

The DC(T) test was developed by Wagoner et al. (2005), and later standardized in ASTM 7313-07 “Determining fracture energy of asphalt-aggregate mixtures using the disk-shaped compacted tension geometry." The DC(T) test device and setup are shown in Figure 15. Following the ASTM specification, test specimens were conditioned in a cooling chamber at the selected test temperature for at least two hr. The standard test temperature was recommended to be set at 10°C warmer than the low-temperature grade of the performance grade (PG) binder. For example, for the PG 64-22 binder used in this study, a test temperature of –12°C would be recommended. However, each of the four mixtures were also tested at temperatures of 0°C and –24°C to evaluate fracture resistance across a wider range of test temperatures for research purposes. A seating load of 0.2 kN was used (load control), followed by testing at a standard loading rate of 1mm of Crack Mouth Opening Displacement (CMOD) per minute (external displacement, or CMOD control mode). The test was stopped when the applied load had peaked and then decreased to 0.1 kN, as specified in ASTM D7313.

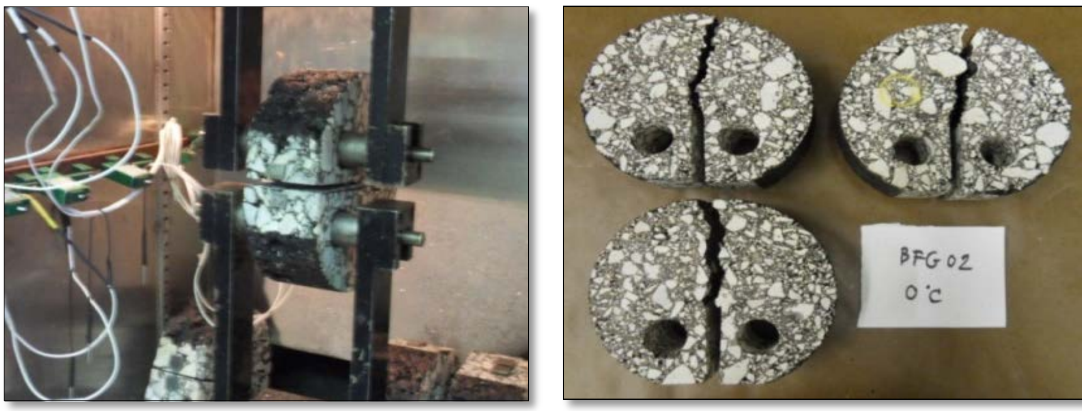



Figure 15. DC(T) Test device and setup (left) and tested specimens (right).

Fracture energy is a measure of the resistance of a material to crack formation and movement (work of fracture per unit of new crack surface created). Fracture energy ($G_f$) is computed as the area under the load-CMOD curve (work) divided by the fracture area (unit area), which is simply formulated as:

$$G_f = \frac{A_f}{T \ x \ L}$$

where $G_f$ is fracture energy (J/m$^2$), $A_f$ is the area under load-CMOD curve (kN·mm), T is thickness of specimen (mm), usually around 50 mm, and L is the fractured ligament length (mm), usually around 83 mm.

Three test temperatures (0°, –12°, and –24°C) and six test replicates (double the minimum specified number of test replicates) were made and tested for each study mixture. Table 14 presents measured fracture energy and coefficient of variation (CoV) of the mean for the four mixtures tested. CoV is simply the standard deviation of the fracture energy of the test replicates divided by the mean fracture energy for the set of replicates. Taking advantage of the extra replicates, a trimmed mean was used to compute average values of the fracture energy of all mixtures.

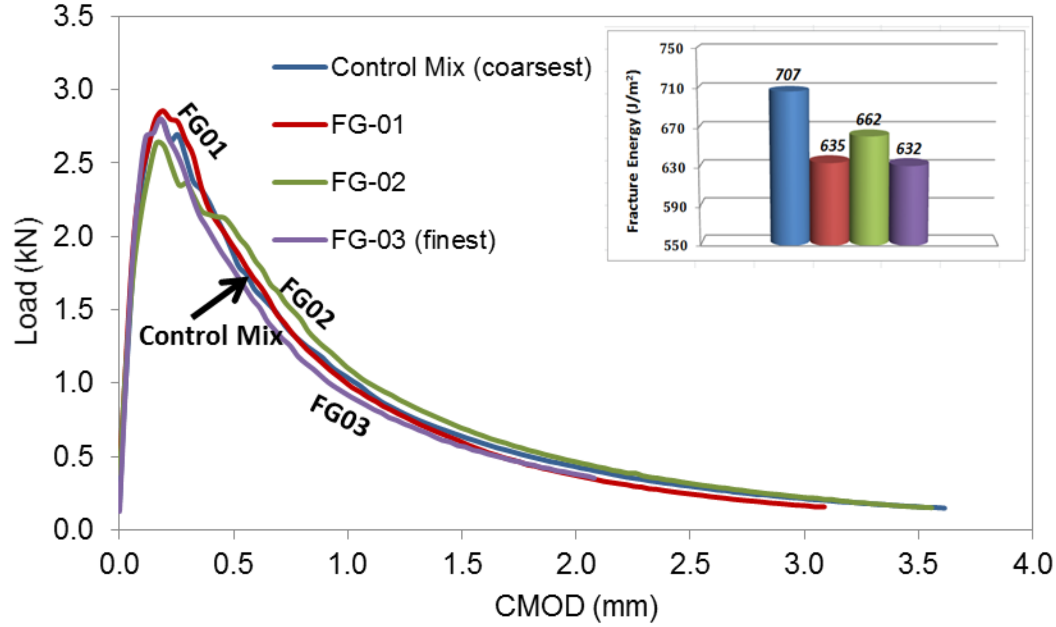
Table 14. Fracture Energy and Associated CoV for the Study Mixtures

| Test Temperature | Trimmed Mean of Fracture Energy (J/m$^2$) and CoV (%) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Control | CoV | FG01 | CoV | FG02 | CoV | FG03 | CoV |
| 0$^o$C | 707 | 5.8 | 635 | 7.4 | 662 | 2.3 | 632 | 3.7 |
| -12$^o$C | 415 | 3.7 | 427 | 6.5 | 416 | 5.4 | 429 | 8.5 |
| –24$^o$C | 249 | 3.8 | 244 | 4.0 | 259 | 4.7 | 248 | 6.7 |

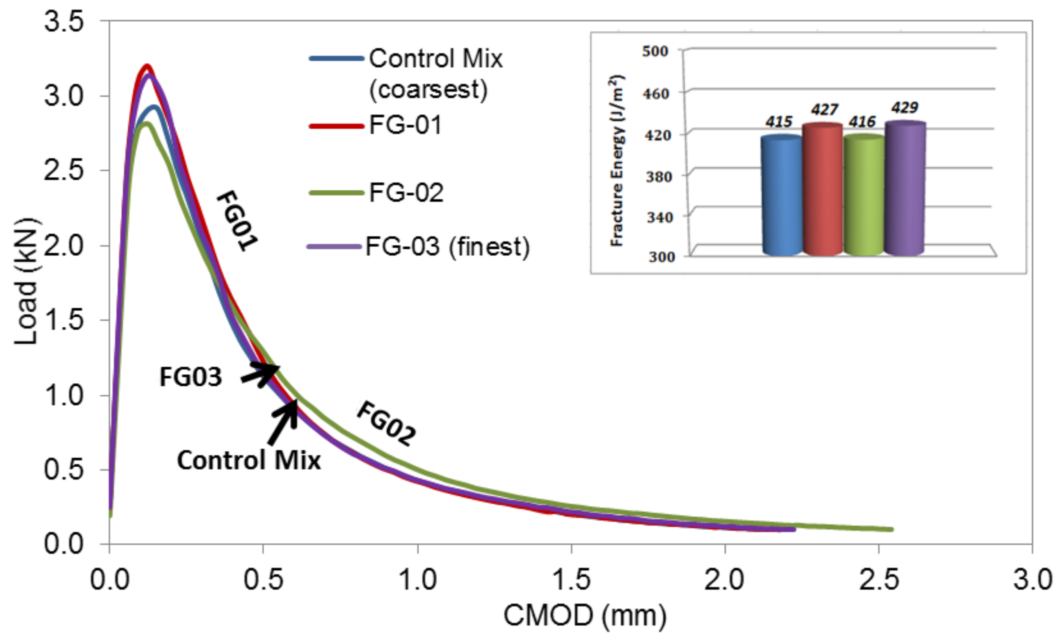
Figure 16 presents plots of load-CMOD curves and fracture energy of the mixes for each test temperature. For the standard test temperature of –12°C of a PG 64-22 binder (standard recommends testing at 10 degrees Celsius above the PG low-temperature binder grade, or PGLT+10), fracture energies of all F-G mixes were slightly higher than the C-G control mix; but within the experimental error. While a standard fracture energy has not been established for binder course mixtures, conservatively, the recommended minimum fracture energy for surface materials used under low traffic as established in the National Pooled Fund Study #776 on Low-Temperature Cracking in Asphalt Pavements (Marasteanu et al., 2010) can be conservatively applied. In this case, a minimum fracture energy of 400 J/m$^2$ at PGLT+10 is specified. The four study mixtures were found to have similar results and be just above this minimum threshold.

For the test temperature of 0°C, the control mix provided the highest fracture energy, perhaps because the fracture path was observed to go around aggregates at this temperature, which consumes energy. Again, at –24°C, no significant differences in fracture energy was observed for the four mixtures. At –24°C, the specimens were subjected to a test temperature which was lower than the PGLT grade of –22°C. A fairly straight crack path is observed at this temperature, indicating a brittle failure mode. Because the manufactured sand was obtained from the same quarry as the coarse aggregate in all study mixes, the fracturing of the coarse and fine aggregate in the C-G and F-G mixes and brittle mastic was similar and the DC(T) could not distinguish between them.

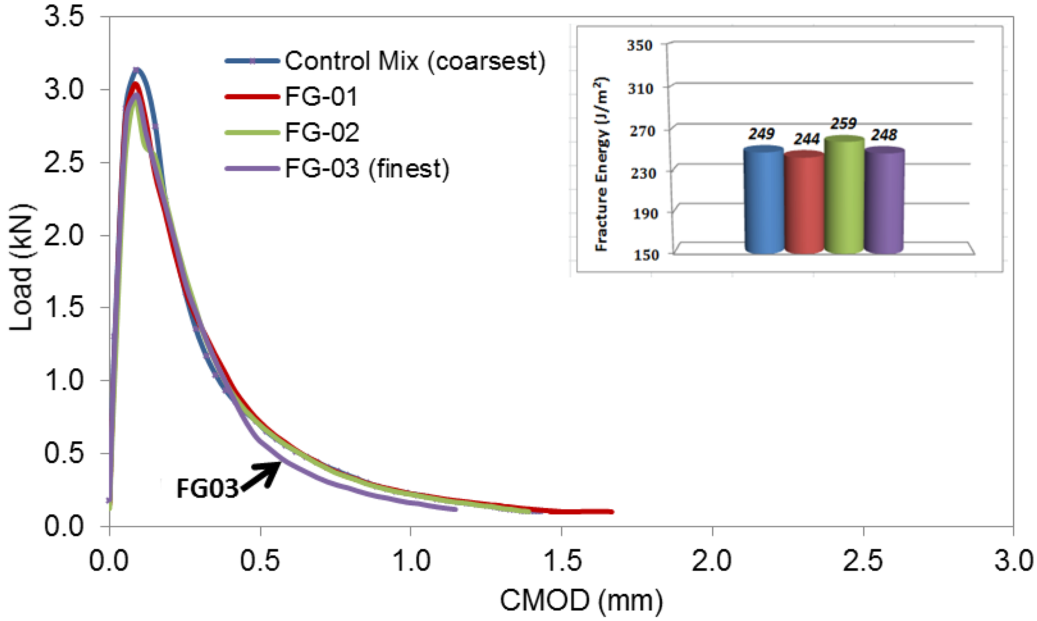
In summary, the low-temperature cracking performance testing indicated that fine-graded mixtures can perform as good as or better than coarse-graded mixtures of similar composition at low temperatures. Given the fact that binder course mixtures will be thermally insulated due to their lower position in the pavement structure, it appears that low-temperature cracking will likely not be of concern for F-G mixtures used as binder course materials. It is recommended that research be conducted to establish less conservative thresholds for fracture energy for binder course mixtures (either by lowering fracture energy requirements or, preferably, by raising the required test temperature) to account for the fact that binder course mixtures do not reach the same extreme cold temperatures as surface courses. More realistic specification limits might then allow higher amounts of reclaimed asphalt pavement and stiffer binder grades to be used in these layers with confidence. The results also suggest that the concept of film thickness needs to be revisited (or dropped from consideration) when dealing with comparisons of C-G vs. F-G mixtures when all other variables are held constant. In other words, by implementing a low-temperature cracking performance test and having reasonably conservative VMA requirements, it is probably not necessary to include a film thickness requirement to control low-temperature cracking.




(a) DC(T) Test Results: 0°C




(b) DC(T) Test Results: –12°C.




(c) DC(T) Test Results: –24°C

Figure 16. Load vs. CMOD Plots and fracture energy at each test temperature.

## 5.3 DYNAMIC MODULUS

Cyclic compressive testing at low strains was performed across a wide range of temperatures to characterize the complex or "dynamic" modulus (E*) of the study mixtures according to AASHTO TP 63-07(2009). Although this is not a standard mixture performance test in Illinois, the method produces data which can be fed into the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG). As this was considered to be a potential long-term need for research on F-G mixtures, E* testing was performed. For brevity, results are presented. Very little difference in E* was measured between the four study mixtures. This indicates that the four mixtures would be expected to produce similar deflection basins and stress-strain profiles not only in binder course layers, but would also lead to similar pavement deflection, stress, and strain characteristics in surrounding pavement layers, all other factors being equal. This finding serves to dispel the incorrect generalization that fine-graded binder courses would tend to produce higher pavement deflections due to their relatively low content of larger coarse aggregate particles.

## 5.4 FLEXURAL BEAM FATIGUE TEST

Traditional "bottom-up" fatigue cracking is of particular concern in binder course mixtures, as they are in the critical tension zone of a pavement in bending under traffic loads under this analysis scenario. Experimentally, a four-point bending fatigue test is often used to assess the fatigue life of HMA mixtures in accordance with AASHTO T321-03. The IPC flexural fatigue test device at ATREL used in this study is shown in Figure 17. Six replicates of each mix were produced at 7.0% ± 0.5% air voids. Beam specimens were produced in the ATREL rolling wheel slab compactor and cut with a masonry saw to 380 ± 6 mm in length, 63 ± 6 mm in width, and 50 ± 6 mm in height. The six specimens were tested at six different strain levels including, 300, 400, 500, 700, 800, and 1,000 microstrain. A standard test temperature of $20^oC$ was used.

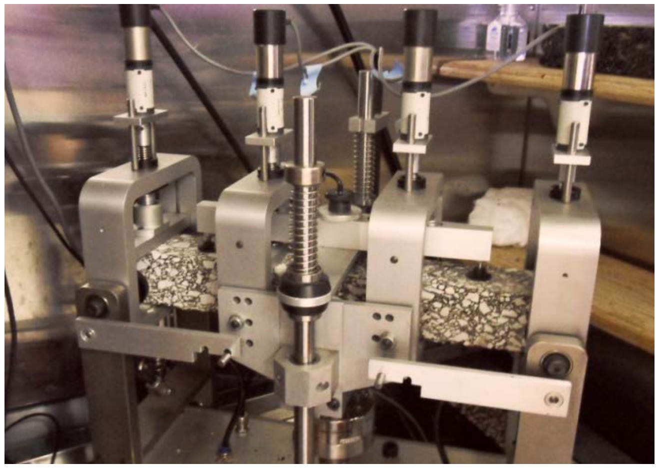


Figure 17. Four-point bending fatigue test device.

For an analysis of the test results, two approaches were used: (1) traditional approach and (2) energy-based approach (presented in Appendix C).

### 5.4.1 Traditional Approach

In the traditional approach, the fatigue failure criterion is defined as the number of cycles to reach a 50% reduction in initial beam stiffness. Figure 18 presents plots of strain vs. number of cycles to failure for the four study mixes. A simple power lab based fatigue model was fit to test data as follows:

$$N_{f50} = f_1 \left(\frac{1}{\varepsilon}\right)^{f_2}$$

where $N_{f50}$ = number of load repetitions to failure, ε = asphalt concrete flexural strain at outer fiber of bending (mm/mm), and $f_1$ and $f_2$ = experimental fatigue parameters.

Table 15 presents the traditional fatigue model parameters for the four study mixes. The models, which represent the average behavior of the mixtures (there is generally a high amount of scatter in fatigue data, so model fitting provides average response), can then be used to interpolate allowable numbers of cycles to failure at various strain levels. Based on the predictions shown in the table, all fine-graded mixtures would be expected to withstand a higher number of cycles to failure than the control mix at the 300 and 500 microstrain levels. At the 1,000 microstrain level, less difference between mixtures was observed, with the C-G mix having the highest fatigue life by a slight amount. It is speculated that this might have some resemblance to the fracture energy trend when comparing the highest test temperature to the lowest. But, because strains at the bottom of the binder course are expected to be much lower than even 300 microstrains in most pavement structures, it appears that the F-G mixtures have equal or superior performance to C-G mixtures with respect to resisting traditional bottom-up fatigue when used as a binder course in a flexible pavement structure.

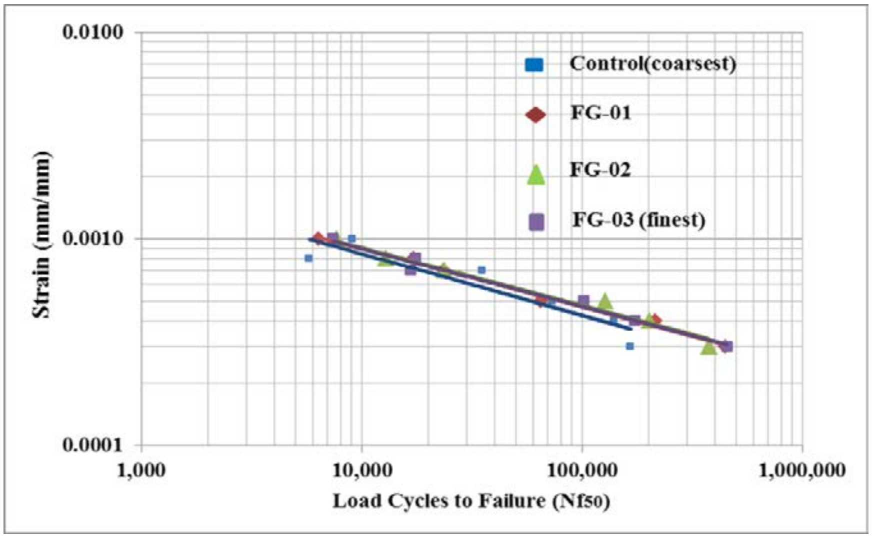



Figure 18. Strain vs. number of load cycles to failure and fitted fatigue models.

Table 15. Traditional Fatigue Model and Predicted Number of Cycles to Failure

| | | Number of cycles to failure at different strain levels | | |
|---|---|---|---|---|
| Mix | Model | 300 με | 500 με | 1,000 με |
| CG Control | $N_{f50} = 2x10^{-5}(\frac{1}{\varepsilon})^{2.875}$ | 268,724 | 61,872 | 8,434 |
| FG01 | $N_{f50} = 2x10^{-7}(1/\varepsilon)^{3.532}$ | 554,419 | 91,257 | 7,889 |
| FG02 | $N_{f50} = 3x10^{-7}(\frac{1}{\varepsilon})^{3.481}$ | 549,873 | 92,898 | 8,320 |
| FG03 | $N_{f50} = 2x10^{-7}(\frac{1}{\varepsilon})^{3.509}$ | 460,057 | 76,620 | 6,730 |

# CHAPTER 6 ACCELERATED PAVEMENT TESTING WITH ATLAS

## 6.1 BACKGROUND

The Advanced Transportation Research and Engineering Laboratory (ATREL) at University of Illinois Urbana-Champaign (UIUC) is home to the Accelerated Transportation Loading Assembly System (ATLAS) as shown in Figure 19. The ATLAS device is 156-kips in weight, 124 ft long, 12 ft high, and 12 ft wide. ATLAS transmits load to the pavement structure through a hydraulic ram attached to a cable-driven wheel carriage. The maximum loading length of the ATLAS device is 85 ft, with approximately 65 ft of travel available for testing under constant velocity.

Figure19. ATLAS.

For this study, ATLAS testing was specified to provide a full-scale comparison of the permanent deformations or rutting resistance of coarse-graded versus fine-graded mixtures under heavy traffic loading. ATLAS testing was designed to be conducted at a constant temperature of 90$^{o}$F and in dry and wet conditions. The proposed testing schematic is shown in Figure 20. Loading areas of the wheel paths were located about 4 ft from both edges of the pavement for the dry and wet or (soaked) conditions, respectively.

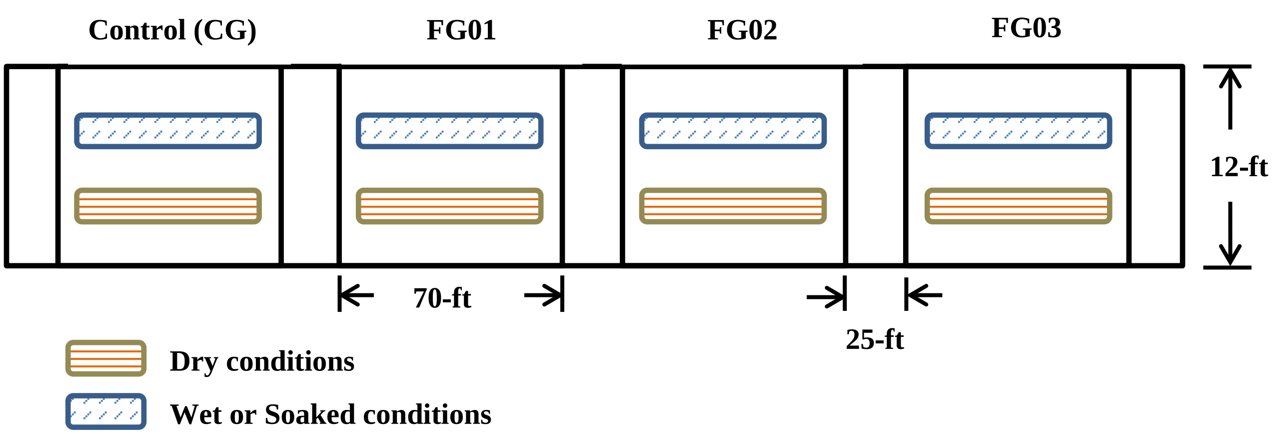



Figure 10. Proposed testing sections for ATLAS experimentation.

Figure 21 shows the ATLAS device with dual-wheel assembly conducting unidirectional loading (wheel is lifted during reverse travel of carriage assembly) on one of the testing sections. Test parameters used were the following:

1. Tire type: Dual-tire assembly
2. Loading speed: 8 mph
3. Tire pressure: 110 psi
4. Tire load: 18 kips used for all passes throughout the test to accelerate rutting
5. Controlled pavement temperature: 90$^{o}$F (32.2$^{o}$C) at 2 in (50.8 mm) below the surface
6. Test location: Dual-tire assembly at 4 ft. (1.3 m) from pavement edge for dry condition and 4 ft. (1.3 m) from the opposite side of the paved lane for wet condition–dry condition testing performed first
7. Rutting measurement: Laser profilometer, as shown in Figure 21

Figure 21. Dual-tire assembly of ATLAS.

According to the proposal, 125-ft sections of the control mix and each of three fine-graded mixes were designed and constructed for ATLAS field testing to evaluate rutting performance. Mix design adjustment, density testing, and permeability testing were conducted before ATLAS testing. Permeability testing results were highly scattered, probably due to roller sequencing and other limitations of paving a short, narrow test strip, and are therefore not reported herein.

Due to the occurrence of several mechanical breakdowns during testing and time limitations, at the time of this report only a portion of the total test results were available, including (1) the control mix in dry condition from 0 to 120,000 passes; (2) the FG01 mix in dry condition from 0 to 60,000 passes (it was decided that 60,000 passes was sufficient); (3) the FG01 mix in wet condition from 0 to 30,000 passes; and (4) the control mix in wet condition. However, the test results for the control section were obtained from testing of an adjacent paving lane (ICT moisture/additive study) constructed at the same time as this study, where both projects utilized an identical C-G control mix. This was done after determining that the constructed C-G control mix for this study was not sufficiently close to design targets (for instance, asphalt content deviated approximately 0.5 percent from the target). Test data of the rutting were collected every 15,000 passes using automated laser technique as shown in Figure 22.

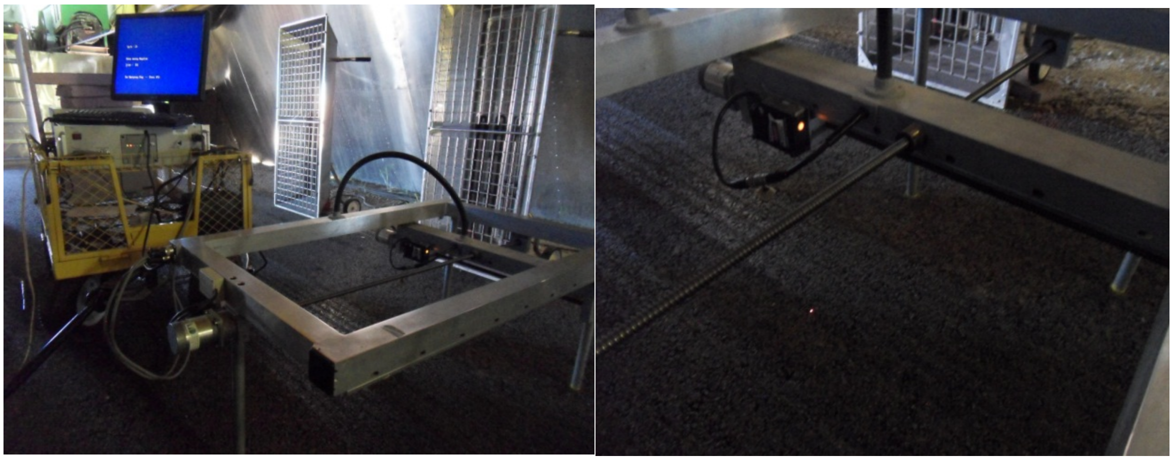


Figure 22. Rutting measurement using laser profilometer

Due to space limitations in the main body of this report following ICT report guidelines, details of pavement construction, testing setup, and test results can be found in Appendix D.

### 6.2 SELECTED ATLAS RESULTS

In this section, ATLAS testing results from the C-G control section and FG01 section in both the wet and dry condition are presented, along with a comparison to Hamburg wheel tracking results for both lab and plant-produced mix.

As shown in Table 16, the data suggests the following findings:

- Both F-G and C-G binder course mixtures were very rut resistant under ATLAS loading in both the dry and wet condition, with less than 1 mm of rutting measured in all cases
- The FG01 mix outperformed the C-G control in both the dry and wet condition under ATLAS testing (0.42mm rut depth vs. 0.67 in dry condition, and 0.68 vs. 0.85 mm rut depth in the wet condition)
- It is acknowledged that neither the HWT nor ATLAS accelerated rut tests perfectly represents field trafficking. However, based on field observations to date, the ATLAS test results are probably more indicative of field performance than the HWT rut depth magnitudes from the standpoint of binder course layer deformation.
- The field-produced mix samples tested in the HWT produced rut depths approximately 40% lower than the lab/design mixtures, which is consistent with observations in practice. This suggests, along with the fact that ATLAS rut magnitudes are far lower than HWT rut magnitudes, that HWT testing of design mixtures may yield very conservative results for binder course mixtures. This might also suggest that future research be directed to potentially easing HWT requirements for binder course mixtures.

- HWT testing of the field-produced mixtures are shown in Figure 23. The close correspondence of all results suggests that all three F-G mixtures should be expected to be very rut resistant when used as binder course mixtures in the field.

Table 16. Comparison of Rut Depths in mm for the HWT (lab, field) and ATLAS (field)

| Mix ID | Measured at (passes) | Dry condition (mm) | | Wet condition (mm) | |
|---|---|---|---|---|---|
| | | Control | FG01 | Control | FG01 |
| Lab Mix (HWT) | 10,000 | n/a | n/a | 5.38 | 5.05 |
| Field Mix (HWT) | 10,000 | n/a | n/a | 2.88 | 2.91 |
| ATLAS (field) | 15,000 | 0.67 | 0.42 | 0.85 | 0.68 |

*Note: for HWT test the mixes were only performed in the soaked condition.*

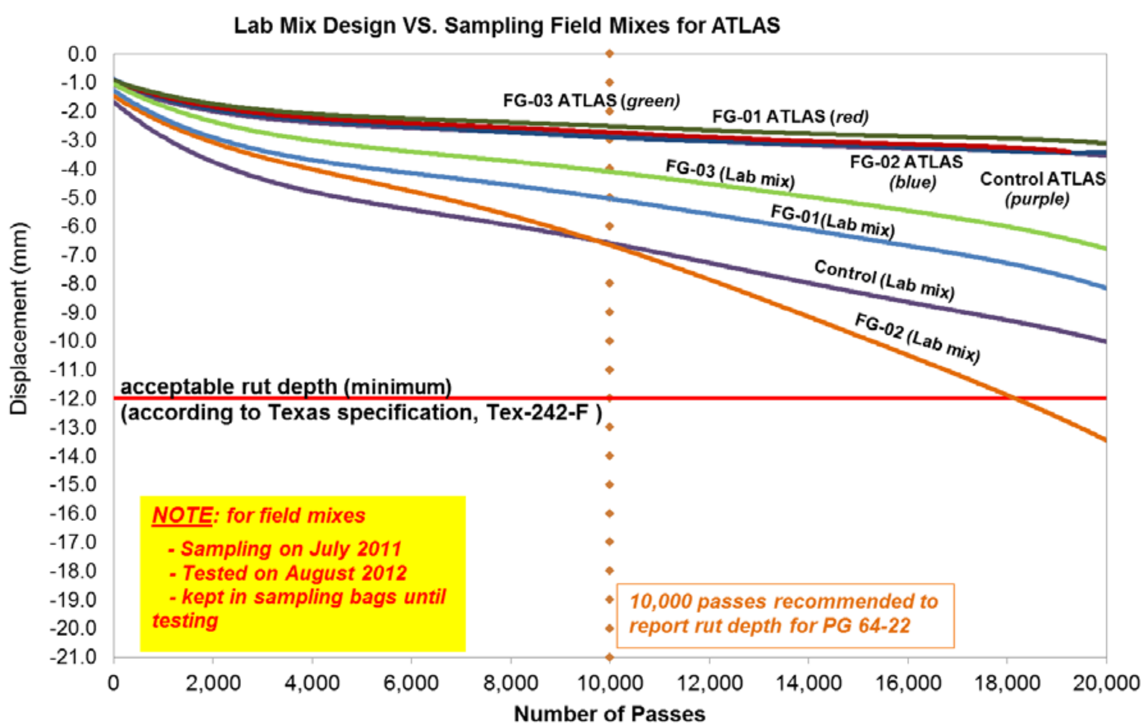



Figure 23. Rutting profiles of HWT tests on lab and field (ATLAS test section) mixes.

# CHAPTER 7 DEVELOPING REVISED HMA BINDER COURSE REQUIREMENTS

In addition to laboratory and ATLAS testing as reported in the previous chapters, other efforts and studies in Illinois had produced sufficient data and experience to the extent that the Technical Review Panel decided that the development of revised requirements for HMA binder courses in Illinois was warranted. Field permeability testing results from a fine-graded binder mixture constructed in IDOT District 5 are presented in Appendix E. These results show a 25-fold decrease in permeability in F-G sections as compared to a traditional C-G binder course. Data from other projects suggest that F-G binder course mixtures can be easily compacted in the field and have led to contractor bonuses in pay-for-performance projects, and that bid prices for F-G mixtures started a bit higher than C-G mixtures, but over time have come in line with C-G mix bid prices (Castro, 2014)

Several meetings of the Technical Review Panel in 2013 were held to develop a revised HMA binder course specification, by a consensus process. The panel consisted of asphalt experts from IDOT districts and the BMPR, FHWA, Wisconsin-based contractors, consulting firms, pavement construction firms, and academia. After reviewing the findings of the literature review, lab and ATLAS test data, field project data, and collective overall experience with C-G and F-G mixtures in Illinois, a draft revised material specification was produced, as shown in Appendix F. The draft specification is a revision to the current BDE Special Mixture Design Composition and Volumetric Requirements. As discussed at the final technical working group (TWG) meeting with industry, a decision was made to abandon the long-standing practice of only allowing coarse-graded 19.0 mm NMAS aggregate structures and adopting some aspects of a fine-graded 19.0 mm mix.

The draft specification does not add “fine-graded 19.0 mm” but rather redefines the existing 19.0 mm gradation bands, resulting in a less coarse mix. This approach was taken to eliminate the need for new terminology in the standard specs, and to mitigate the need for new pay items and to reduce the number of material codes moving forward. Again, the revised gradation bands reflect the changes recommended by the Fine-Graded TRP. While these new bands were deemed agreeable for Districts 2 and 5 at the time of the writing of this report, the other seven of the nine IDOT districts will have an opportunity to experiment with the new specification to ensure it is workable for them as well.

In addition to the revisions to adopt finer 19.0 mm mixes, the elimination of the reference to N105 and 12.5 mm surface and 12.5 mm level binder mixes was made as well. If the revisions are deemed agreeable, then a plan was established to target the November 2014 letting for a BDE Special Provision to impact the 2015 construction season. In the meantime, the new specification is planned to be kept as a BMPR special provision for districts to use as desired for the 2014 construction season.

Given the considerable experience with F-G binder in other states, the results of this study, the experience amassed over the past 5 years in IDOT District 5, and recent experience with F-G binder in District 2, it is believed that this specification can be used with confidence across Illinois. However, it is recommended that plant and field data and the performance of pavements constructed with this specification be monitored over time for further validation.

# CHAPTER 8 SUMMARY

Fine-graded (F-G) binder course HMA mixtures were designed and evaluated as an alternative to typical coarse-graded (C-G) binder course HMA in Illinois. Three different aggregate structures of F-G mix and one C-G control mix were designed, produced, and tested. The Superpave mix design process was used in the study in conjunction with the Bailey Method. After designing the four study mixes to have nearly identical volumetrics (particularly VMA), moisture damage testing was conducted. Two interpretations of moisture sensitivity testing were possible. On one hand, the F-G mixtures possessed higher indirect tensile strength than the C-G control in all conditions, particularly in the case of the finest mixtures tested in the dry condition. On the other hand, the tensile strength ratio (TSR) results showed a reverse ranking (C-G best). From a practical standpoint, because the marginally failing TSR results (which are not uncommon for the aggregates used) would be easily corrected by introducing a liquid anti-strip agent, and because the F-G mixtures had higher overall strength (wet and dry) and should have lower permeability (based on field studies), moisture sensitivity was not deemed as a potential issue for F-G binder courses.

The laboratory testing used in the study included Hamburg wheel tracking (HWT), disc-shaped compact tension (DC(T)) testing, dynamic modulus testing, and four-point flexural beam fatigue testing. In the HWT test, two of the three F-G mixes performed better than the coarse-graded mix in terms of rutting resistance. For the DC(T) test, no significant difference was found among the four study mixes in terms of fracture energy across all test temperatures (0°, –12°, and –24°C). The control mix was found to be slightly stiffer than the three F-G mixes at low temperatures according to the dynamic modulus test results. However, the FG01 (coarsest of the F-G mixes) had the greatest stiffness on the high temperature side, and FG03 (the finest mix) provided the least stiffness in all test temperatures—but not by a significant margin. For the fatigue test, both traditional and energy-based approaches were used to evaluate the performance of the mixes under repeated flexural loading. In this test, all three F-G mixes outperformed the C-G control mix. In summary, laboratory testing indicates that F-G mixes, on the whole, should be expected to outperform traditional C-G mixes.

Because the DC(T) could not distinguish between the C-G and F-G mixes, a practical and reliable test method has been recently developed through an IDOT-sponsored project to screen a mix's capacity for cracking resistance. The test method uses the Semi-Circular Beam (SCB) fracture test conducted at 25°C with a loading head displacement rate of 50 mm/min. This test method was selected by IDOT for the following reasons: (1) correlation to field performance; (2) significant and meaningful spread in test outcome representing a mix's cracking resistance; (3) repeatability, practicality, low cost, and easy implementation by agencies and contractors; and (4) correlation to other independent cracking test methods.

A flexibility index (FI), derived from the SCB test results, was introduced. The FI will allow asphalt mixture designers and contractors to develop a mix that has the potential to resist cracking. Hence, this approach encourages innovation. The FI may be integrated into IDOT's mix design specifications as a performance quality indicator. This will complement existing volumetric specifications and the rutting resistance performance test, resulting in a more balanced mix design protocol.

For further validation of the feasibility of F-G mixes, ATLAS was used to evaluate rutting performance of the mixes when tested in full scale. Test sections 125 ft in length of all four study mixes were constructed at the Advanced Transportation Research and Engineering Laboratory (ATREL). Testing under dry and soaked conditions was performed. However, because of a mechanical breakdown of the ATLAS device and time limitation, testing of only four of the eight test conditions was completed at the time of this report: dry and soaked conditioning of the control mix, and dry and soaked conditioning of

FG01. Based on the ATLAS test results obtained, the fine-graded mix performed slightly better that the control mix in terms of rutting resistance. Although different magnitudes of rutting were observed, ATLAS ranking and permanent deformation trends were similar to laboratory results. In addition, it was observed that HWT tests performed on plant-produced mix had lower rut depth than tests performed on the laboratory-produced mix during mix design and correlated better with ATLAS results. This suggests that HWT tests conducted on lab mix may produce fairly conservative results.

Permeability testing of a demonstration project in IDOT District 5 involving F-G and C-G binder demonstrated a 25-fold decrease in permeability in F-G sections compared to a traditional C-G binder course. Data from other projects suggest that F-G binder course mixtures can be easily compacted in the field and have led to contractor bonuses in pay-for-performance (PFP) projects, and that bid prices for F-G mixtures started a bit higher than C-G mixtures, but over time have come in line with C-G mix bid prices.

Several meetings of the Technical Review Panel in were held in 2013 to develop a revised HMA binder course specification by a consensus process. The panel consisted of asphalt experts from IDOT districts and the BMPR, FHWA, a Wisconsin contractor, consulting firms, pavement construction firms, and representatives from academia. A draft specification was developed as a revision to the current BDE Special Mixture Design Composition and Volumetric Requirements. A decision was made to abandon the long-standing practice of allowing only coarse-graded 19.0 mm NMAS aggregate structures and adopting some aspects of a fine-graded 19.0 mm mix. The draft specification does not add “fine-graded 19.0 mm” but rather redefines the existing 19.0 mm gradation bands, resulting in a less coarse mix. This approach was taken to eliminate the need for new terminology in the standard specifications, to mitigate the need for new pay items, and to reduce the number of material codes moving forward.

Given the considerable experience with F-G binder in other states, the results of this study, the experience amassed over the past 5 years in IDOT District 5, and recent experience with F-G binder in District 2, it is believed that 19.0 mm F-G binder mixes can now be used with confidence across Illinois. It is also believed that F-G binder mixes will significantly improve Illinois roadways by means of improved compactability, workability, lower permeability, increased smoothness, tighter longitudinal joints, and reduced segregation. It is recommended that plant and field data and the performance of pavements constructed with the new 19.0 mm NMAS binder HMA specification be monitored over time for further validation.

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Note: Interviews and phone conversations with asphalt scientists and engineers from around the state, region, and nationally can be found in Appendix A.

# APPENDIX A LITERATURE REVIEW

## Historical Development of the Illinois Department of Transportation HMA Specifications

Many primary and interstate roadways in Illinois were originally built with Portland cement concrete (PCC). Until the late 1970s, these PCC pavements were rehabilitated and resurfaced using hot mix asphalt and performed satisfactorily by industry standards. However, in the early 1980s, when HMA was placed on deteriorated interstate pavements, there were severe rutting and shoving problems almost immediately. Loading on the system had increased substantially due to the expansion of the maximum allowed truck-trailer capacity coupled with the introduction of higher pressure radial tires.

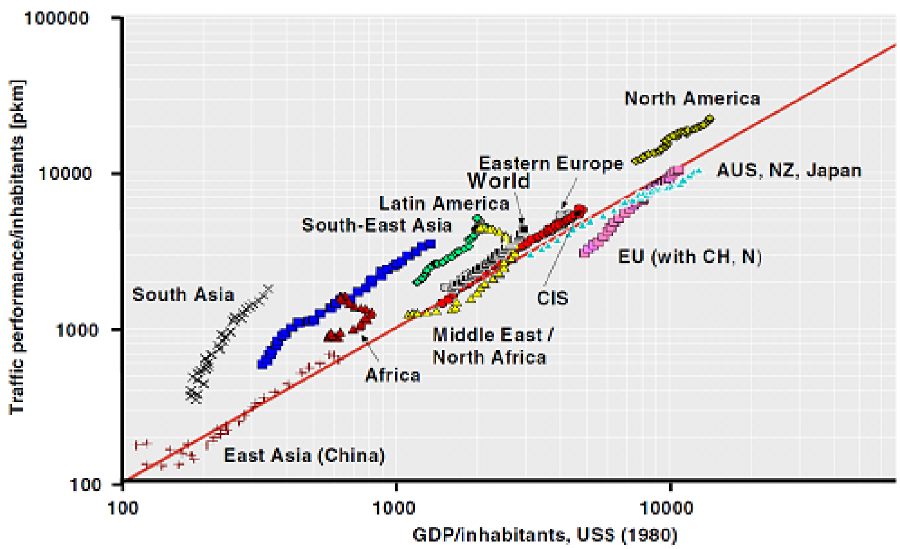


Fig. 1: Development of mobility depending on Gross Domestic Product

National Highway Traffic Safety Association (NTHSA) Report 05-0471

**National Highway Traffic Safety Association (NTHSA) Report 05-0471**

Through the early 1980s, mix designs were developed by IDOT with materials supplied by the contractor and the Marshall method of mix design (reference Asphalt Institute MS-2). Process control at the production facility and acceptance testing were conducted by IDOT. Measuring in-place density on the grade was also an IDOT function. Essentially, up until the mid-1980’s, IDOT was 100% responsible for mix designs, process control, and acceptance as the contractor was not staffed for these functions nor allowed to make changes to the production or construction practices. IDOT was a method specification state, like most of the nation at that time.

In Illinois, a 9-mile section of Interstate 70 performed so poorly that it was removed and replaced within one year. In response to this event, IDOT ceased all paving and formed a task force to investigate the failures. The cause for the failures in Illinois was the explosion of the Average Annual Daily Traffic (AADT) and Average Annual Daily Loading (AADL) in the United States. Maintaining the road system through numerous enhancements to ensure that the highest level of safety is maintained has improved dramatically over the years as well. The Bituminous Task Force on Stability and Durability, formed in February of 1984 to return IDOT to the level of service (LOS) expected by the motoring public, was asked to:

- Review IDOT policies, procedures, and operations associated with the placement of bituminous overlays on interstate routes, and
- Recommend changes necessary to increase the resistance of the overlay against severe and immediate rutting.

The task force consisted of 19 members with extensive HMA experience in design, production, and construction from academia, agency, associations, and industry. Through numerous meetings, they were able to finalize a white paper of recommendations by May 1984 for use in that summer's construction projects.

The task force guided the literature review from that time and showed deficiencies in mix design, as IDOT had no voids in mineral aggregate (VMA) criteria and targeted most production to 2.5% Voids ($V_a$). Many of the HMA mixes were a blend of two crushed coarse aggregates, two natural sands, and mineral filler. Most job mix formulas (JMFs) targeted 40% passing the #10 sieve (60% retained), 2.5% voids, and 2,000-lb Marshall stability (something IDOT has proven does not guarantee stable mixes) with a maximum flow of 15. Again, there was no mention or measure of VMA or Voids Filled with Asphalt (VFA) in the specification of the time. Finally, the practice of the day was to increase oil during production to assist construction with achieving 93% $G_{mm}$ (maximum theoretical) for in-place density.

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## Discussion on Voids in Mineral Aggregate (VMA)

### *Evaluation*

It has long been established that gradation of the aggregate is one of the factors that must be carefully considered in the design of asphalt paving mixtures, especially for heavy duty highways. The purpose for establishing and controlling aggregate gradation is to provide sufficient voids in the asphalt-aggregate mixture to accommodate the proper asphalt film thickness on each particle and provide the design air void system to allow for thermal expansion of the asphalt within the mix. Minimum voids in mineral aggregate (VMA) requirements have been established that vary with the nominal maximum aggregate size.

Traditionally, gradation requirements are so broad that they permit the use of paving mixtures ranging from coarse to fine and to either low or high stability. To further complicate matters, different combinations of sieve sizes are specified to control specific grading ranges. Standardization of sieve sizes and aggregate gradations, which has often been suggested, is not likely to occur because of the practice of using locally available materials to the extent possible.

In the early 1960s, the Bureau of Public Roads introduced a gradation chart (Figure A.1) which is especially useful in evaluating aggregate gradations. The chart uses a horizontal scale, which represents sieve size openings in microns raised to the 0.45 power, and a vertical scale that represents percent passing. The advantage in using this chart is that, for all practical purposes, all straight lines, plotted from the lower left corner of the chart in a directions upwards and toward the right to any specific maximum particle size, represent maximum density gradations. As shown in Figure A.2, the nominal maximum particle sieve size is one size larger than the first sieve to retain more than 10% of material. The gradations depicted in Figure A.3 and A.4 are exaggerated to illustrate the points being made. By using the chart, aggregate gradations can be related to maximum density gradation and used to predict if the mixture will be fine or coarse textured, as shown in Figure A.3.

Soon after the chart was developed, it was used to study gradations of aggregate from several mixtures that had been reported as having unsatisfactory compaction characteristics. These mixtures could not be compacted in the normal manner because they were slow in developing sufficient stability to withstand the weight of the rolling equipment. Such mixtures can be called “tender mixes." This study identified a consistent gradation pattern in these mixes, as illustrated in Figure A.4. Most notable is the hump in the curve near the #30 sieve and the relatively flat slope between the #30 sieve and the #8 sieve. This indicates a deficiency of material in the #30 to #8 sieve range and an excess of material passing the #30 sieve. Mixtures with an aggregate exhibiting this gradation characteristic are susceptible to being tender, particularly if the fines are composed of natural sand.

As part of the bituminous mix design process, the aggregate gradation should be plotted on the 0.45 power gradation chart.

# United States Bureau of Public Roads 0.45 Power Chart

## Sieve Sizes Raised to the 0.45 Power

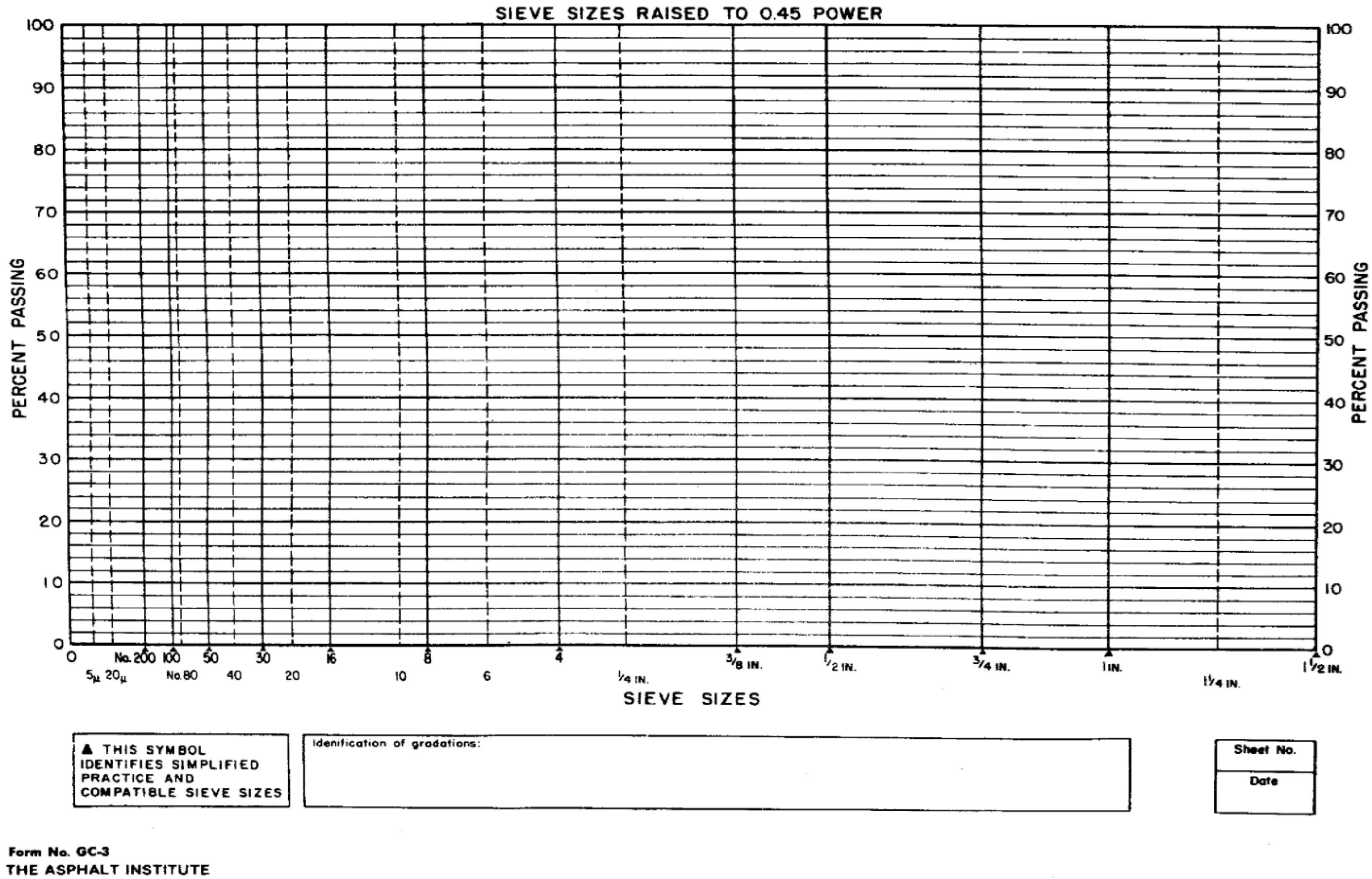


Figure A.1

# United States Bureau of Public Roads 0.45 Power Chart

## Sieve Sizes Raised to the 0.45 Power

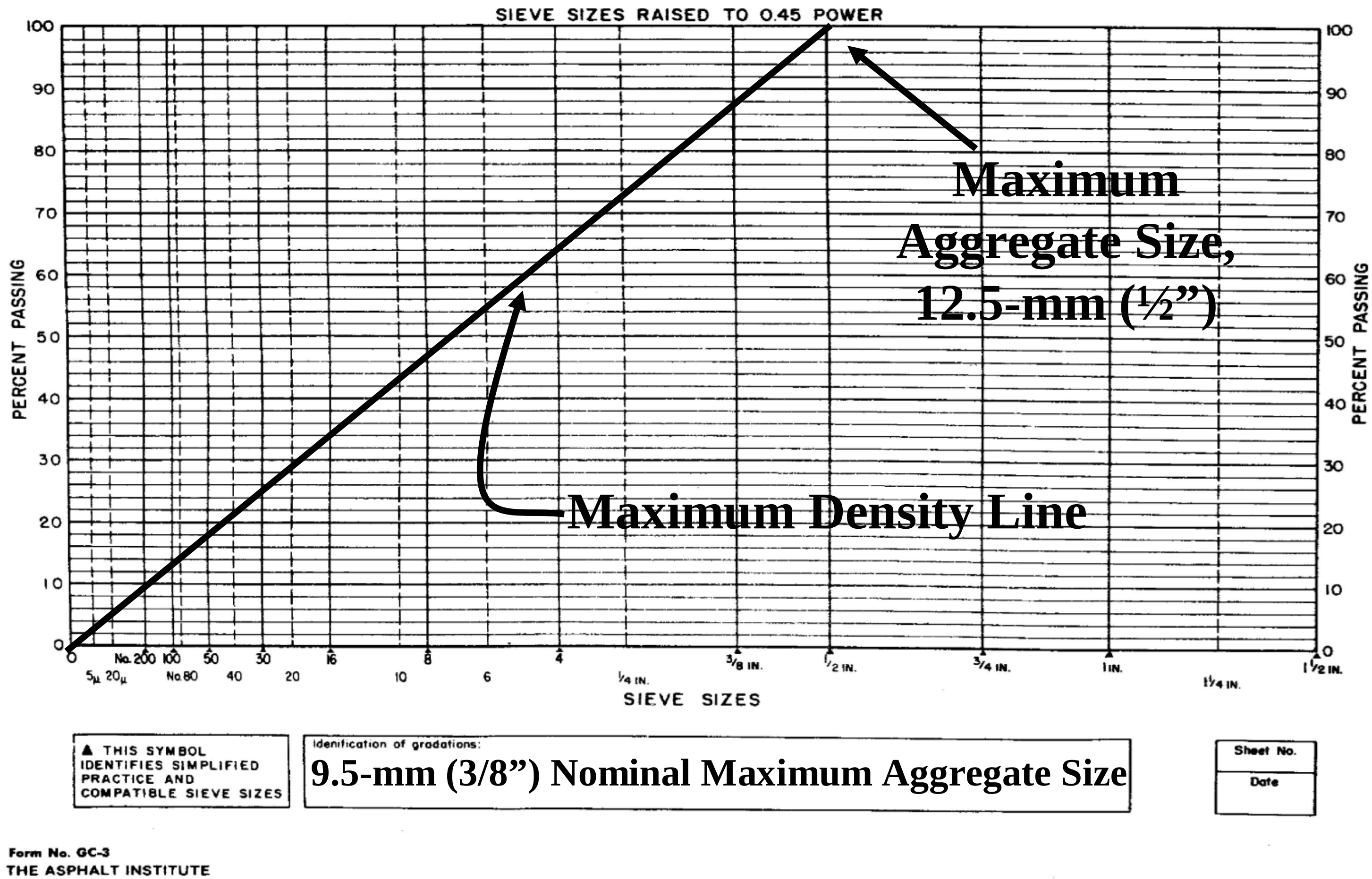


Figure A.2

# United States Bureau of Public Roads 0.45 Power Chart

## Sieve Sizes Raised to the 0.45 Power

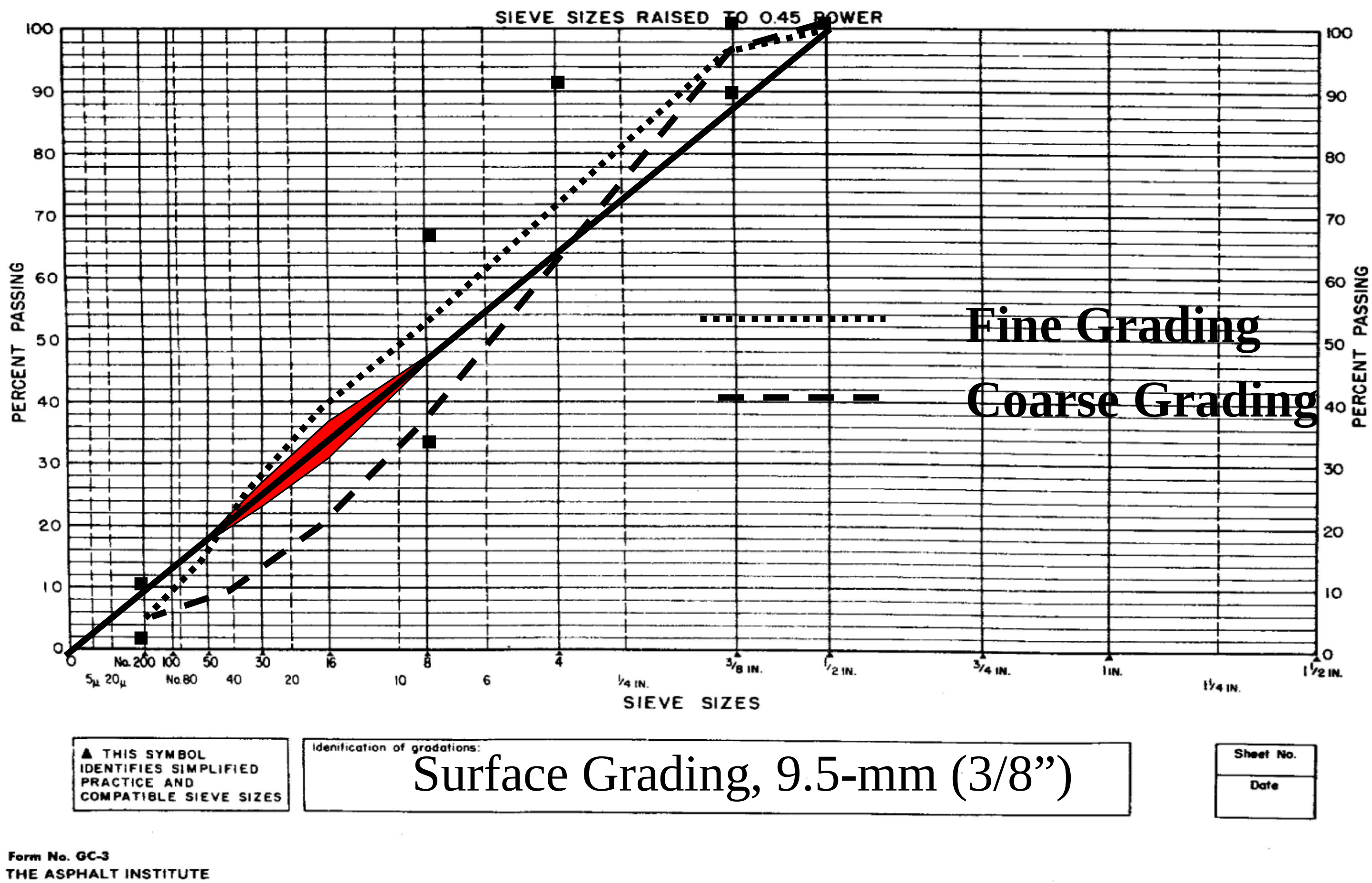


Figure A.3

# United States Bureau of Public Roads 0.45 Power Chart

## Sieve Sizes Raised to the 0.45 Power

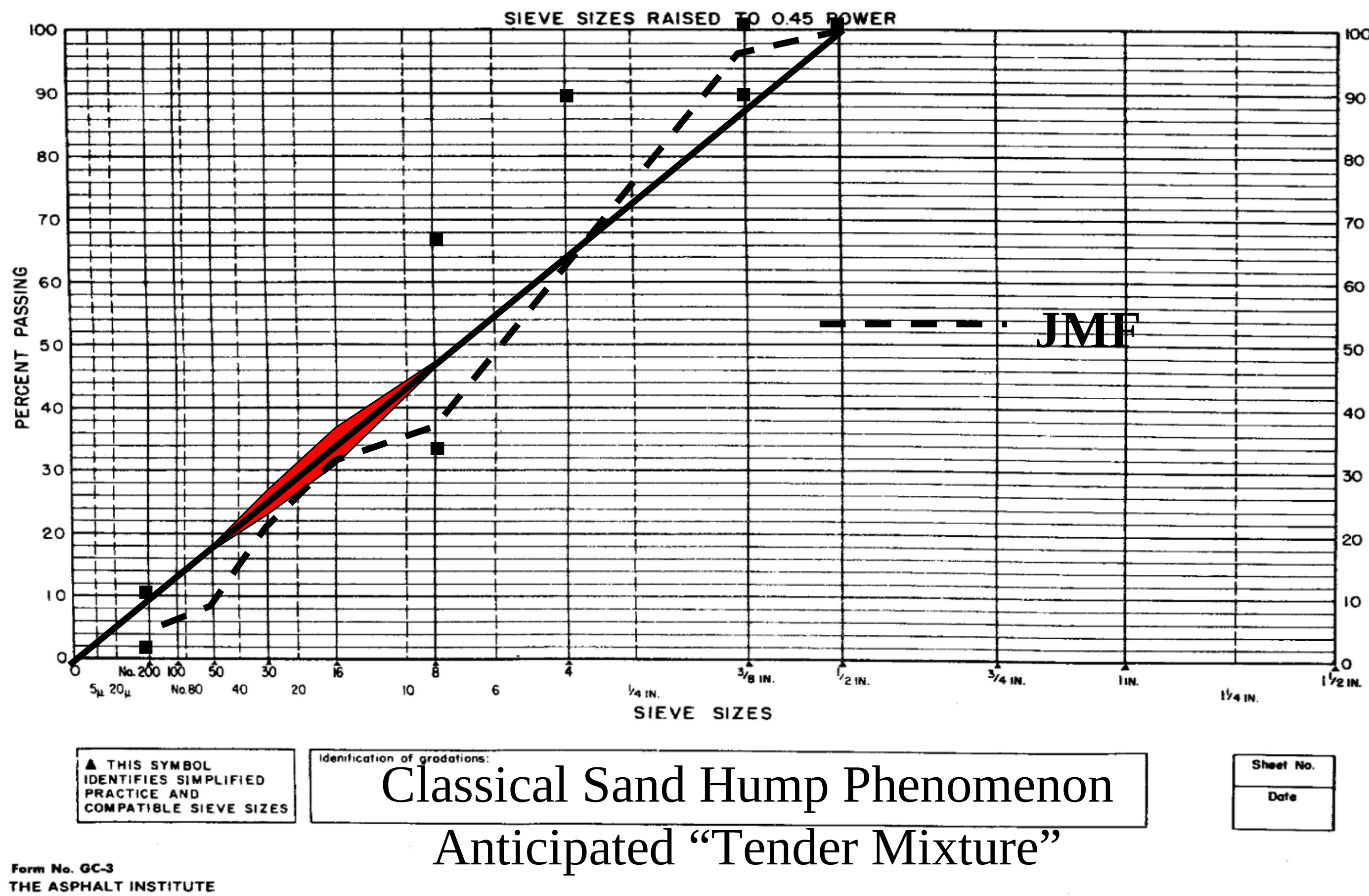


Figure A.4

**Aggregate Gradation and VMA**

1. Adjust VMA by adjusting aggregate gradation:
   a) Change P#200 by increasing or decreasing "dirty" aggregate blend
   b) Change P#30 by balancing manufactured and natural sands
   c) Change P3/8 in. by adjusting coarse aggregate blend especially through varying particle shape and surface texture (crushing change)
   d) Introduce a clean, crushed 1/4 in. aggregate
   e) Introduce a product from another source with different shape and texture

2. With a given aggregate gradation, the VMA will decrease with increasing compactive effort. It is important to note that the VMA criteria do not change based on the level of compaction. The reasoning for having sufficient VMA is consistent regardless of the traffic level for which the mixture is being designed.

3. As VMA goes so goes air voids; this ensures that a mix maintains an adequate asphalt film thickness. Typically, the target air void level is 4.0%; however, if 3.0% is desired, such as for a low volume roadway, then adjust the minimum VMA requirement accordingly. Generally, a slightly higher VFA is desirable when designing for a low volume roadway to allow for more durability and less stability.

4. VMA curves are flat near optimum asphalt contents, and the accurate measurement of VMA becomes difficult as the asphalt content increases. Excess asphalt in the mix holds the aggregate apart because a fluid cannot be compressed. This is floating rock, not a true measure of VMA. Therefore, avoid the plastic side of the VMA curve. Conversely, avoid the dry side of the VMA curve because the area where the asphalt mixture is brittle will tend to segregate, causing premature pavement failure.

5. Adequate VMA is a very good starting point; however, without achieving the proper in-place density in the field, water and air penetration will destroy the asphalt pavement prematurely. Many in industry believe that "Density is King!" and this is not far from the truth. However, adequate film thickness ensures durability and assists in the compaction process while holding all of the materials together, thereby reducing segregation potential.

6. The influence of the pavement structure, climate, and project conditions will influence mixture selection, asphalt, and aggregate specifications as well as the compaction requirement.

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Based on the pavement failures during the early 1980s, the literature review, and findings of the task force, the need for change became paramount and the recommendations given were to:

- Alter the mix gradation to move it away from the maximum density line,
- Adopt VMA minimums in accordance with the Asphalt Institute recommendations, and
- Increase design air voids from 2.5% to 4%.

The Illinois task force recognized that, historically, IDOT made a conscious effort to take advantage of the naturally occurring, locally available, materials. In particular, they noted that Illinois natural sands are round in nature with an excess of material passing the #30 sieve causing sand humps, tender mixes, yet mixes that were not prone to segregation. For a successful implementation, it became important to identify and described coarse and fine aggregates available across the state because strength and abrasion resistance of the main rock controls final product roundness and toughness. It is generally accepted that roundness measures the relative sharpness or angularity of the edges and corners of a particle, which is dependent on the type of crusher and its reduction ratio. As categorized by Topal and Sengoz (2003) a classification used in the United States and Illinois is as follows:

- Well-rounded: No original faces left.
- Rounded: Faces almost gone.
- Sub-rounded: Considerable wear, faces reduced in area.
- Sub-angular: Some wear but faces untouched.
- Angular: Little evidence of wear.

In order to obtain VMA, the task force concluded that only altering gradation away from the maximum density line was not going to be enough to obtain and maintain minimum VMA values; thus the fine aggregate (FA) of the 1980s, FA-10, was eliminated. Replacing the FA-10 with a crushed angular FA with lower percentages passing the #30 sieve became the final task force materials initiative.

Illinois aggregate producers had crushed FA materials available, but it was “dirty” or high in minus #200 material, a known VMA rescuer and a product known to cause excess cracking and rutting in HMA pavements. The task force worked with existing crushed product gradations to develop a new crushed sand product, but limited the minus #200 to 8% or less, which required quarries to wash the screenings of excess fines. Immediately, the new product, FA-20, and a minimum of 50% of the sand fraction went into all HMA mixtures used on heavy route roadways. Aggregate tests, particularly the compacted aggregate resistance (CAR) values, show that crushed sand’s average CAR values are as high as 3,600 lb while natural sands typically average 500 lb.

**TABLE 1 Results of Aggregate Testing on Fine Aggregate Sources**

| Aggregate Type | Sample ID | FAA Method A | FAA Method B | FAA Method C | FAA KT-50 | Friction Angle-Direct Shear (degrees) | CAR Value (lbs) |
|---|---|---|---|---|---|---|---|
| Manufactured (angular) sands | ABE | 49.9 | 54.9 | 41.5 | 49.0 | 43.1 | 5,133 |
| | STV | 49.8 | 54.9 | 42.5 | 50.6 | 44.2 | 4,610 |
| | TRI | 48.2 | 52.3 | 47.4 | 48.6 | 42.4 | 5,033 |
| | MTH | 48.0 | 52.0 | 44.0 | 46.9 | 39.2 | 2,217 |
| | TOX | 47.9 | 52.5 | 43.5 | 47.2 | 43.1 | 2,918 |
| | BMO | 47.5 | 53.0 | 41.6 | 48.6 | 39.9 | 4,090 |
| | BLV | 44.6 | 48.3 | 41.2 | 44.8 | 38.6 | 1,422 |
| Natural (rounded) sands | AMB | 42.2 | 47.1 | 39.8 | 41.6 | 37.3 | 558 |
| | CBL | 42.5 | 45.8 | 44.1 | 41.8 | 35.6 | 370 |
| | BMB | 42.0 | 45.2 | 42.1 | 41.6 | 33.3 | 272 |
| | CWD | 41.8 | 44.9 | 43.6 | 41.5 | 32.9 | 325 |
| | MDL | 41.6 | 44.5 | 44.2 | 41.8 | 39.7 | 775 |
| | CJX | — | — | 46.7 | — | 35.8 | 272 |
| Minus No. 8 from aggregate blends for HMA testing | STV Bld | 46.1 | — | — | — | 36.7 | 1,473 |
| | MTH Bld | 45.2 | — | — | — | 35.1 | 1,050 |
| | TRI Bld | 45.5 | — | — | — | 39 | 1,740 |
| | AMB Bld | 42.4 | — | — | — | 36.3 | 523 |
| | ABE Bld | 46.1 | — | — | — | 37.4 | 1,485 |
| | TOX Bld | 45.3 | — | — | — | 40 | 1,245 |
| | BMO Bld | 45 | — | — | — | 36.9 | 1,425 |
| | BMB Bld | 42.3 | — | — | — | 30.3 | 413 |

According to Thomas Bennert et al. of TRR, “The CAR test is an empirical test method that was developed to evaluate the shear resistance of compacted fine aggregate. The test procedure is similar to that of the California bearing ratio (CBR) test, though on a much smaller scale. And like the CBR test, the results of the CAR test are dependent on the shear strength properties of the aggregate” (1962). Additional work done included testing HMA with the Asphalt Pavement Analyzer (APA), Repeated Shear at Constant Height (RSCH), and Frequency Sweep at Constant Height (FS-CH). The findings from Bennert’s research are summarized in the following chart:

| Physical Measure | CAR Value |
|---|---|
| FAA of 45 | 1,220 lb |
| APA rutting of 5 mm, max. | 1,320 lb |
| RS-CH accumulated shear strain of 2% | 1,430 lb |
| FS-CH G* at 40°C of 36,200 psi | 910 lb |
| Average value to minimize rutting | 1,250 lb |

The work done here using New Jersey and Pennsylvania aggregates is useful to this research because the mixtures analyzed were defined as fine-graded by Bennert and provide for some anticipated similar results by ICT laboratory analysis. The recent purchase of a Hamburg wheel by IDOT BMPR will allow for analysis by either the APA as done by Bennert or the Hamburg wheel. Additionally, many earlier studies have shown that by using manufactured sands the Marshall stability values quite often doubled or tripled by swapping out the natural sands (Murphy, A., 1996).

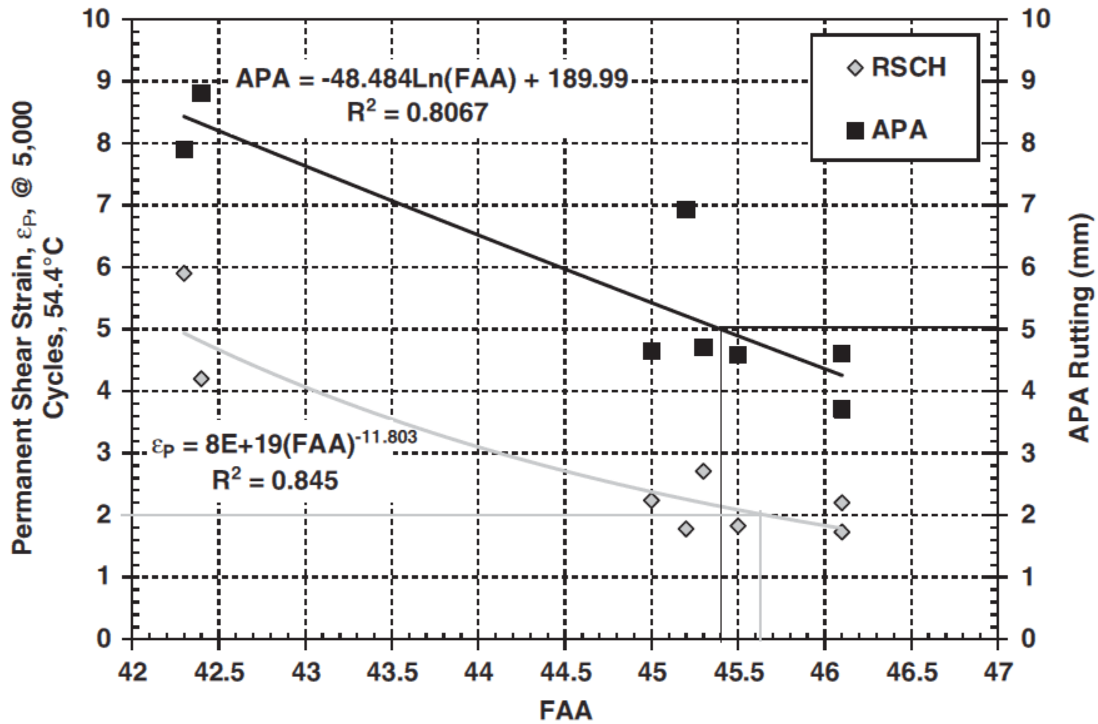
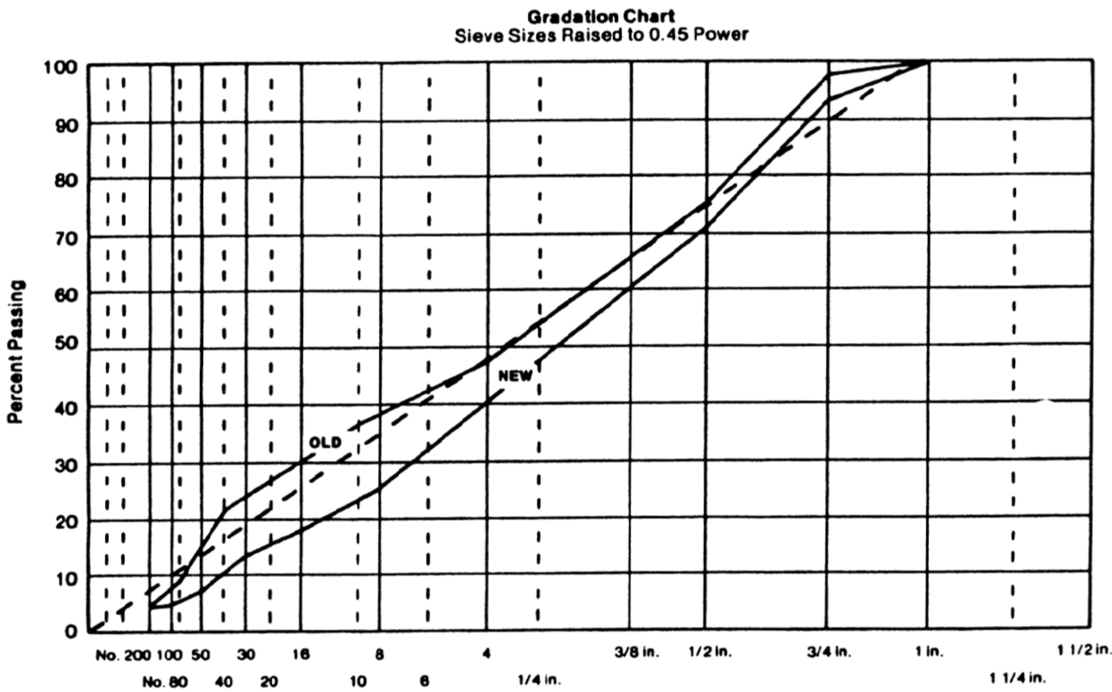



FIGURE 7 FAA versus APA and SST repeated shear.

The gradation changes, adoption of minimum VMA, and the use of FA-20 allowed IDOT to target 4% voids in lieu of 2.5% and, thanks to VMA, to increase the amount of effective asphalt into an HMA mixture. It should be noted at this time that VMA was selected to be 15.0 for 3/8 in surface course mixes for full-depth pavements (Ref. SSRBC, 1996) yet current specification requirements are 14.5. Complementing mix changes, the task force also recommended controlling dust (as introduced by Scheftick and McCain, IDOT; the HMA Level III training course requires the introduction of breakdown [aka extra dust] with all mix designs submitted for verification) and asphalt in the field in order to maintain VMA and $V_a$ by requiring a “start-up” procedure during the first two days of production. Finally, IDOT moved to a statistically reasonable measure for in-place density.

Several recommended changes from the 1984 task force remain within today’s HMA specification for the loading being delivered onto Illinois heavily trafficked roadways. Implementation included decentralization of mix design, start-up, process control, and acceptance to the nine districts. Education and training included workshops, video tapes of best practices, and on-the-job training state wide that continues today. Over the next ten years, IDOT moved to the Quality Control/Quality Assurance (QC/QA) Initiative, which was an action item from the task force based on their findings.




**Gradation; Old = pre-1984, New = 1984**

## Literature Reviewed

A local, regional, and national literature review was performed to collect information specific to hot mix asphalt performance and especially for information that addressed questions on fine-graded HMA from the following sources:

- Standard Specifications,
- Asphalt experts within the Midwest market, and
- Previously completed research work.

### *Standard Specifications*

The review included gathering information from states located in the Midwest with material sources, climate, and traffic loading similar to Illinois. The states reviewed included Iowa, Indiana, Kentucky, Michigan, Minnesota, Missouri, Nebraska, and Wisconsin. Generally speaking, these states have HMA specifications that were adopted after the Superpave recommendations from the 1990s. The states have adopted and/or maintained:

- Superpave aggregate gradation control points,
- Equivalent Single Axle Loading (ESAL) driven items, including:
    - Coarse aggregate angularity
    - Fine aggregate angularity (Method A),
    - Flat and elongated,
    - Depth in structure,

- Voids, VMA, and VFA driven by nominal maximum aggregate size (NMAS),
- Tensile Strength Ratio minimums,
- Dust to asphalt ratios based on effective asphalt.

Potential IDOT areas to research include:

| Superpave | IDOT HMA |
|---|---|
| Control Points | Gradation control limits |
| FAA; 40%/43%/45% | 50% FM-20 for $N_{des} \geq 90$ |
| Dust to Effective Asphalt: 0.6–1.4 | Dust-to-Total Asphalt:<br>1.0 design/1.2 production |

One final finding that is not specific to HMA only for this research, that compliments findings found for the research done for the QA Peer Exchange, is that many states have bonuses and penalties for HMA quality measures such as in-place density, volumetric control, and smoothness. IDOT currently does not have a statewide Pay Within Limits PWL specification for HMA, but it should be noted that a transition to this end began in 2008 with the introduction of pay for performance (PFP).

***Asphalt Experts Within the Midwest Market***
The interviews are summarized and included at the conclusion of this appendix.

***Previously Completed Research Work***
The performance of HMA is driven by quality volumetrics, voids, uniform gradations, consistent production measured by controlled aggregate gradation and asphalt content, and in-place density. As Kim et al. explain: “Coarse gradation, meaning a larger proportion of coarse aggregates with the same NMAS compared to medium gradation, did not show significant effects on permanent deformation. Interactions of aggregate type with gradation, asphalt type, air voids, and temperature were found to be significant for the permanent deformation of asphalt concrete, whereas no interaction appeared to be significant for fatigue with the given size of experimentation” (ASTM STP 1147). From Stiady et al. (2001) on *PURWheel the Effect of Main Factors on Rutting Performance* includes an adequate finding that, “equally adequate performance could be obtained with mixture gradations plotting above, through, and below the restricted zone; i.e., fine, medium, and coarse-graded HMA mixtures.” Many other reports mimic similar results when dealing with “other than” coarse-graded HMA mixtures and ICT will develop a matrix to look at similar opportunities for use in Illinois.

The majority of the literature reviewed deals with the benefits of utilizing crushed coarse and fine materials when building HMA roadways. The Illinois DOT dealt with crushed materials over the years by requiring 100% crushed CA (with friction requirements driving CA type) and a minimum of 50% of the FA portion to be FM-20. Refinement since the implementation of Superpave has led many state agencies to further revise their crush requirements for both CA and FA. For example, many states now specify four different levels for FAA; 45% for highest ESALs, 43% for intermediate ESALs, 40% for moderate ESALs, and N/A for lowest ESALs. These enhancements specific to FAA are critical success points that IDOT should consider researching and implementing. From work done by Anderson, Asphalt Institute and Bahia, University of Wisconsin they found that by using a Delphi process with the Expert Task Group they could come up with a consensus for aggregate properties (reference table within).

Anderson and Bahia (TRR 1583, Paper No. 971295) state, "To ensure that poor aggregates would not be used in Superpave mixes, an expert panel was convened to determine the most important aggregate properties to include in a mixture specification. A number of aggregate properties were identified." In addition they found that, "The general contention that finer gradations have weaker aggregate structures was not confirmed by the results of the Frequency Sweep at Constant Height (FS-CH) and Repeated Shear at Constant Height (RSST-CH) testing in this study. In fact, the opposite is indicated by the [1]$m_{FS}$ values, which are used as inputs to the current Superpave models for predicting rutting."

**TABLE 1 Aggregate Property Rankings From SHRP Delphi Process (Third Questionnaire)**

| | Average Ranking[a] | Standard Deviation |
|---|---|---|
| Gradation Limits: | **6.57** | **0.76** |
| minimum/maximum | 4.50 | 1.95 |
| control points/restricted zone | 4.14 | 1.79 |
| control points only | 5.29 | 1.68 |
| Coarse Aggregate Angularity | **6.43** | **0.76** |
| Fine Aggregate Angularity: | **5.85** | **1.21** |
| natural/manufactured | 3.57 | 1.87 |
| NAA method | 5.29 | 1.49 |
| Aggregate Toughness: | **5.57** | **1.65** |
| LA Abrasion | 4.71 | 2.16 |
| Maximum Clay Content: | **6.07** | **1.00** |
| sand equivalent | 4.29 | 1.68 |
| plasticity index | 4.86 | 1.29 |
| Thin, Elongated Particles: | **4.71** | **1.98** |
| ASTM D4791 | 4.86 | 1.70 |

*a* Scaled rankings based on a range of 1-7 with a "7" meaning the expert "very strongly agreed" with the inclusion of the property in a specification.

[1] The output of the test was a determination of complex shear modulus (G*) and phase angle (δ). The slope of the curve of G* versus frequency (on a log-log graph) was also calculated. This slope, $m_{FS}$, is currently used in Superpave mix analysis models to predict rutting.

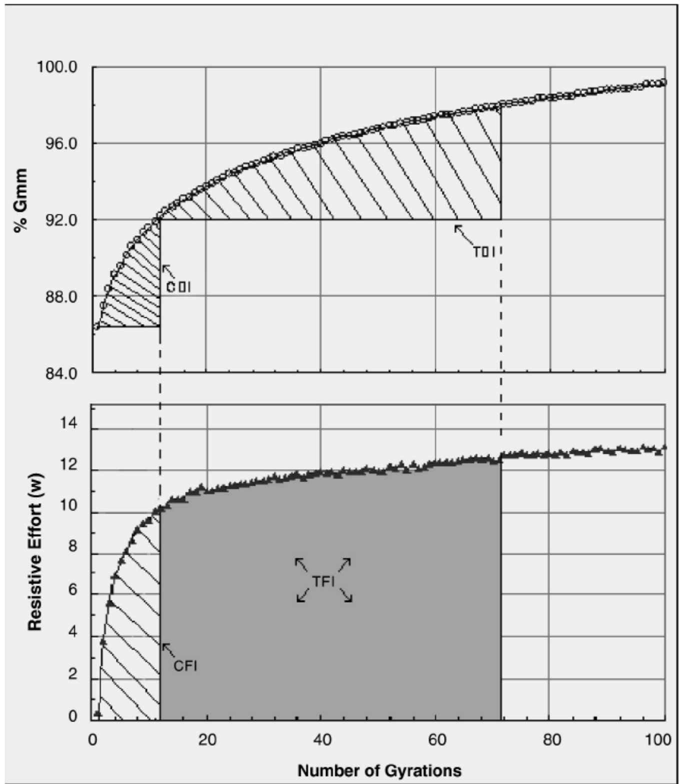



FIGURE 1 Response variables (CDI, TDI, CFI, TFI).

Upper limits on FAA for the entire mix may be appropriate without an upper limit for VMA in some states; however, this is typically not a problem in Illinois because HMA is paid for by the ton in-place so there is no advantage to increasing asphalt content as done in other states that pay for oil separately. In addition, FAA testing may be expedited if an acceptable protocol is developed using imaging shape indices as has been proposed (Eyad Masad et al., TRR 1757, Paper No. 01-2132). If standard FAA, Method A protocol is to be followed, then it is important to train technicians in the proper techniques and to avoid data manipulation, a problem that has been identified and measured by several agencies across the nation (Johnson et al., TRR 1998). Nationally, Cross and Brown (1992) performed an extensive study that focused on 43.3 as uncompacted void content that separated good and poor HMA performers. Developing a more comprehensive sliding scale for FAA, versus only two values, is prudent because it balances engineering need with local material availability. For all practical purposes, as IDOT implemented Superpave mixtures with $N_{des}$ = 105, almost all production was done with only FM-20 as the sand fraction.

HMA testing measures should complement FAA values. The University of Wisconsin looked at HMA energy indices during Superpave Gyratory Compaction efforts to better understand particle shape, surface texture, source characteristics, and the total blend gradation. According to Anthony D. Stakston et al. in TRR 1789, Paper No. 02-3239: “The compaction densification index (CDI) and the compaction force index (CFI) are used to evaluate the performance of mixtures during construction. CDI is measured by integration of the area under the densification curve measured by the SGC between the first gyration and the 92% $G_{mm}$. The 92% $G_{mm}$ is assumed to be the target density at the end construction. CFI is the integration of the area under the resistive work curve measured with the gyratory load plate assembly (GLPA) between the same reference points. The traffic densification index

(TDI) and the traffic force index (TFI) are used to measure mixture resistance to traffic. TDI and TFI are estimated by integrating the area under the densification and resistive work curves, respectively, between 92% $G_{mm}$ and 98% $G_{mm}$, as indicated the figure shown. This index does not represent the effect of binder viscosity but evaluates only the aggregate friction contribution."

Unfortunately, as dense-graded mixes continue to be used on high traffic roadways, IDOT found that permeability, segregation, and workmanship problems led to less than desirable performance. Although much has been written and discussed regarding WesTrack, one fact exposed during the post-mortem of the eventual HMA failures is that all of the coarse-graded mixtures had the largest standard deviation, approaching 7% on the 3/8 in sieve from core samples versus 2% on the fine-graded mixtures, when it came to measuring total aggregate gradation even when following standard sampling and testing practices. "This coupled with the fact that coarse-graded mixtures are typically more sensitive to variability in AC and gradation could lead to more performance problems with coarse-graded mixtures as compared to fine-graded mixtures" (Epps, et al., on WesTrack). Similar results were found at Chicago Testing Laboratory in 1996 when an analysis was performed during mix design to determine the probability of segregation of an asphalt mixture. The procedure included mixing up a batch of aggregates and asphalt to the optimum asphalt content, loading 6,000 grams into a bucket, tipping the bucket at an elevation of 1-meter from the floor over paper to prevent loss of material as it discharged on the floor, placing the bucket over the center of the conical stockpile of HMA that just formed, and analyzing inside versus outside the bucket HMA for asphalt content and gradation. Typical testing would include measuring any difference on the #4 sieve, asphalt content, and voids. Typical differences on the #4 = 15%, AC = 0.3%, and voids = 1.5% for the coarse-graded surface mixes and one half these values for fine-graded (commercial) HMA. The coarse aggregates rolling to the outside led to the high differences in measured values due to segregation. Fine-graded mixtures will reduce the potential for segregation in production, trucking, and construction phases. "Fine mixes at the bottom of a base HMA layer would likely increase the fatigue lives of flexible pavements as well." [Al-Qadi et al., TRR 1823, Paper No. 03-3497].

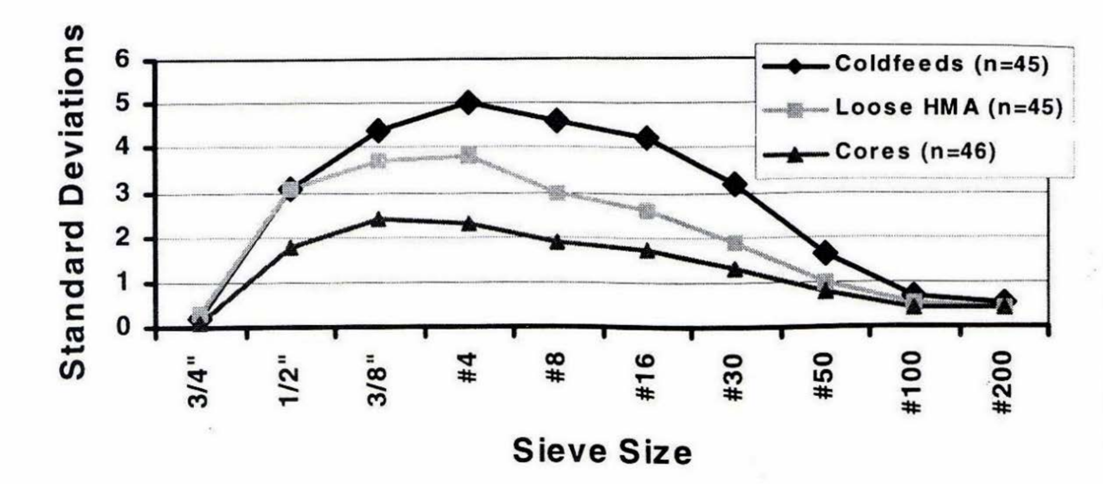



*Figure 4 - Fine Gradation Variability*

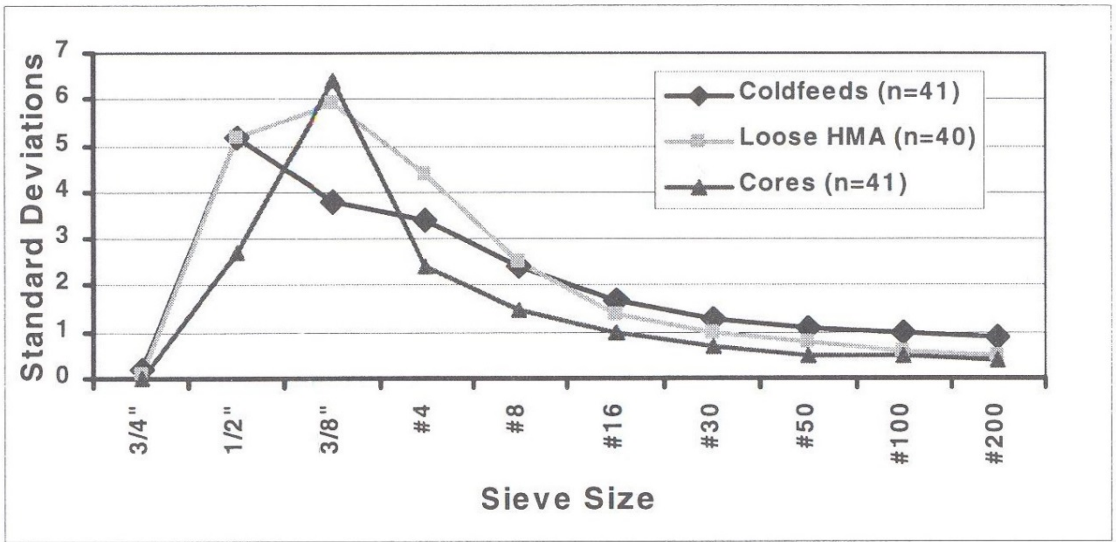


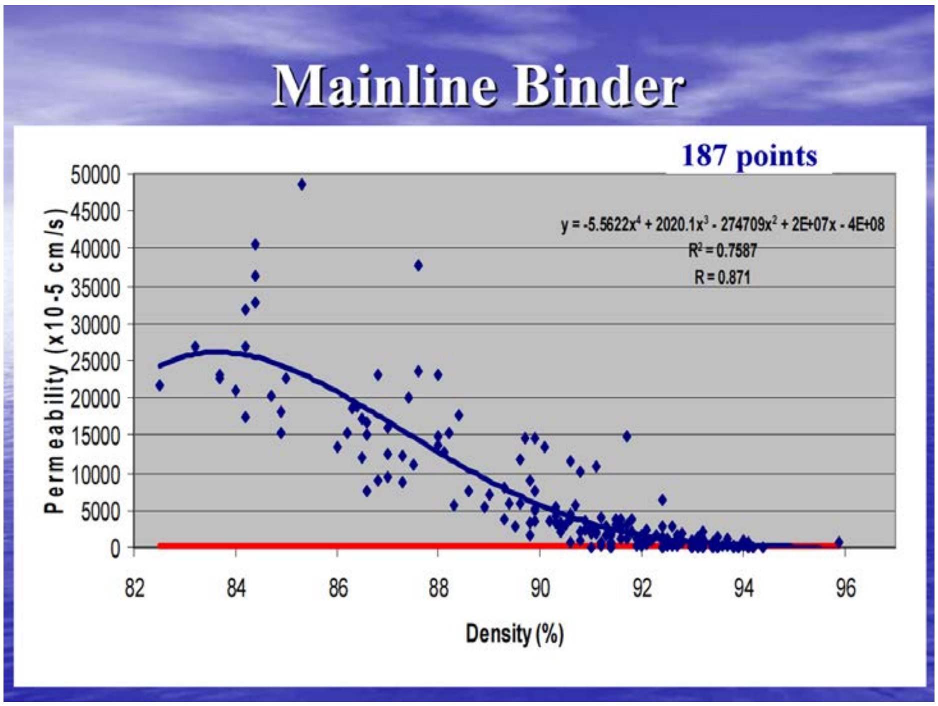

Figure 6 - *Coarse Gradation Variability*

Excessive oxidation leads to cracking and raveling while stripping led to rutting, shoving, and raveling. Permeability was, and still is, prevalent in segregated areas, level binder and mainline binder, and longitudinal joints. Some solutions over the years were to:

- Specify Material Transfer Devices to ensure homogenous mixture was placed on the roadway,
- Specify polymers to improve strength and reduce stripping,
- Research, write, and implement a segregation specification, and
- Study permeability causes and effects; both in the lab and field using a permeameter (Trepanier et al., IDOT, 2005),
- Utilize IL-4.75mm level course with high crush sand and polymer amounts,

- Increase compacted lift thickness,
- Increase in-place density requirements,
- Implement a longitudinal joint density specification, and
- Install joint sealants/adhesives.

It is anticipated that many of these items can be eliminated by developing high strength, fine-graded HMA.




According to Bailey et al. (TRB Circular ec044, 2002): “In a coarse-graded HMA, coarse aggregate interlock plays a significant role in resisting permanent deformation. However, in fine-graded mixes, the fine aggregate plays the predominant role in resisting permanent deformation.” Understanding the role that aggregate quality plays in both coarse or fine-graded HMA mixtures is essential to the long-term performance of the pavement. In terms of laboratory testing, we look at LA Abrasion, soundness, impurities, and fracture, just to name a few. During review of FAA measures in Kansas, they discovered that one “chat” product met the FAA minimums and increased VMA but reduced stability when compared to limestone. “Its strength and performance was on an even keel to natural sand, which had a measured FAA 9-points lower.” [Cross and Purcell, STP 1412, 2001.] Reviewing Illinois quarry FA shape will be a performance test that will ensure quality products are not only specified but used in HMA. In the Bailey Method of mix design, tools are used to assist in better understanding, achieving, and controlling aggregate interlock (whether defined as a coarse or fine job mix formula) through using loose and rodded unit weights of both the coarse and fine aggregates.

NCAT looked at Superpave mixes with coarse and fine gradations that meet CAA, FAA, and VMA requirements after NCAT realized that some state agencies had begun to specify only coarse or only fine gradations for HMA. According to Kandahl and Cooley, (TRR 1789, Paper No. 02-2403) "Results of this study, using three different rutting susceptibility tests, indicate that no significant differences in rut potential occurred between the two gradation types. This was true for all three performance tests. Based on the results of this study, mix designers should not be limited to designing Superpave mixes on the coarse or fine side of the restricted zone. Mixes with either gradation type can perform well. Therefore, it is recommended that gradation specifications use both coarse and fine-graded mixes. Regardless of the gradation type, some type of rutting torture test should be used to verify the rut resistance of the mixture." It is interesting to note that the conclusion of one NCAT study, "Rutting performance of mixes having gradations below the restricted zone, which was commonly recognized to be rut resistant, appears to be more sensitive to aggregate properties than does rutting performance of mixes having gradations above or through the restricted zone" (TRR 1891).

It is a fallacy that coarser graded mixtures always perform better. In Illinois, it is possible to gap-grade an aggregate blend to increase VMA in the mix design process. This is common practice for HMA producers who wish to limit FM-20 (crushed sand) and utilize the less expensive FA-02 (natural sand). Although the testing in the laboratory yields passing results, due to the fact that the mixture is gap graded (i.e., approximately 28% passing on the #8 sieve for a surface course), it is practically impossible to manufacture, truck, and place uniformly. This leads to excessive segregation, a blotchy mat, low in-place density, high permeability, and increased maintenance costs. The most prevalent mixture that allows this behavior is the N70 surface mixture because there is not a manufactured sand requirement.

"Arbitrary use of any gradation band or specification limits may result in a gap-graded or poorly graded aggregate band" (Ruth and Birgisson, STP1412). It may be valuable to look at how to "shift" the current gradations upwards and increase the lowest and highest gradation values currently allowed by specification. For example, in lieu of stating 28–48 on the No. 8 (2.36 mm) for an IL-9.5/IL-12.5, it could be revised to 35–55, but arguably the range should not be the same for different NMAS mixtures. As of this writing, IDOT has revised the IL-9.5 mm passing the No. 8 (2.36 mm) sieve upwards to 32%–52%.

***Whether you design an HMA mixture to be coarse or fine graded, optimum asphalt content will be the same when achieving the same VMA and voids level, provided the asphalt absorption, $P_{ba}$, is the same in the two aggregate blends (Pine, W. and Murphy, T.).***

Voids in Mineral Aggregate = Air Voids + Effective Volume of AC.

Where; VMA: Voids in Mineral Aggregate

$V_a$: Voids

$V_{be}$: Effective Volume of Asphalt

$VMA = V_a + V_{be}$, therefore

$V_{be} = VMA – V_a$.

For a given NMAS, the VMA requirement is fixed. For example, when placing a 1/2 in (12.5 mm) NMAS surface course, the VMA requirement is 14.0 and voids is 4.0. Based on the previous equation, the effective volume of AC = 10.0 regardless of whether the job mix formula is a coarse or fine gradation.

**Lift Thickness**

Research completed by NCAT as part of NCHRP 9-27, Report 531, "Relationships of HMA In-Place Air Voids, Lift Thickness, and Permeability," recommended a "minimum layer thickness to nominal maximum aggregate size (NMAS) of 3:1 for fine-graded mixes and 4:1 for coarse-graded mixtures." The recommendations come from research and field data that shows how the NMAS and gradation characteristics reduce or help in the ability to achieve in-place density, facilitates coated particle reorientation, increases overall mat quality, reduces permeability, and allows for improved smoothness. All measures make for a more durable HMA pavement.

In Illinois the HMA surface and binder, N90 mixtures are generally accepted to be coarse-graded mixtures. Based on definitions and review of typical contractor designs they fall below the maximum density line on the No. 4 (4.75 mm) sieve for binder mixes (less than 40%) and the No. 8 (2.36 mm) sieve for surface mixes (less than 40%). Quite often, almost every HMA mixture in Illinois, from N50 through N105, falls below the maximum density line on the control sieve and is considered to be coarse graded. "The in-place void content is the most significant factor impacting permeability of HMA mixtures. This is followed by coarse aggregate ratio and VMA. As the values of coarse aggregate ratio increases, permeability increases. Permeability decreases as VMA increases for constant air voids" (Report 531).

For Illinois 3/8 in (9.5 mm) surface and 3/4 in (19.0 mm) binder course mixtures a fine- versus coarse-graded mixture, comparison between NCAT and IDOT requirements follows specific to lift thickness.

| HMA Lift Thickness versus NMAS and gradation type | | | |
|---|---|---|---|
| HMA Mixture | NCAT Coarse-Graded HMA Compacted Thickness | NCAT Fine-Graded HMA Compacted Thickness | Illinois Compacted Thickness Specification |
| IL-9.5 mm (3/8") | (4) x 3/8" = 1-1/2" | (3) x 3/8" = 1-1/8" | 40% max. on No. 8, 1-1/4" |
| IL-12.5 mm (1/2") | (4) x 1/2" = 2" | (3) x 1/2" = 1-1/2" | 40% max. on No. 8, 1-1/2" |
| IL-19.0 mm (3/4") | (4) x 3/4" = 3" | (3) x 3/4" = 2-1/4" | 40% max. on No. 4, 2-1/4" |
| IL-25.0 mm (1") | (4) x 1" = 4" | (3) x 1" = 3" | 40% max. on No. 4, 3" |

In each case shown, when compared to NCHRP thickness recommendations for coarse versus fine-graded HMA, there would be a substantial cost savings to performance for all mixtures, especially binder lifts. Based on IDOT's current compacted thickness policy, fine-graded HMA would meet existing criteria by IDOT as well as NCHRP 9-27 recommendations, coarse-graded HMA does not meet Report 531 recommendations: "Coarse-graded mixtures generally have higher permeability values than the fine-graded mixtures for a given air void level." [Report 531, 2004]

Furthermore, according to Russell et al. (WHRP 05-05, 2005), "it was found that there is a good correlation between the gradation of aggregate and permeability. As the ratio of

$$\frac{(\%P12.5\text{ mm} - \%P9.5\text{ mm})}{(\%P4.75\text{ mm} - \%P2.36\text{ mm})}$$

increases, the permeability decreases and as the gaps between the coarse aggregates (%P12.5 mm and %P9.5 mm) and/or the fine aggregates (%P4.75 mm and %P 2.36 mm) increase, the permeability increases; where %P = percentage passing. This could be the effect of differences in aggregate sizes on the internal void structure, and thus measured permeability, of the compacted material. This trend could be used in mix design by controlling the ratio to limit permeability by either reducing the difference between the coarse sieves, fine sieves, or both." The connectivity of voids is reduced with fine-graded mixes, which is more important than the total volume of voids when measuring the permeability of HMA. As the chart below from Russell et al. shows, the fine aggregate difference has the most distinct test results that can only be achieved by reducing and/or eliminating gap-graded mixtures.

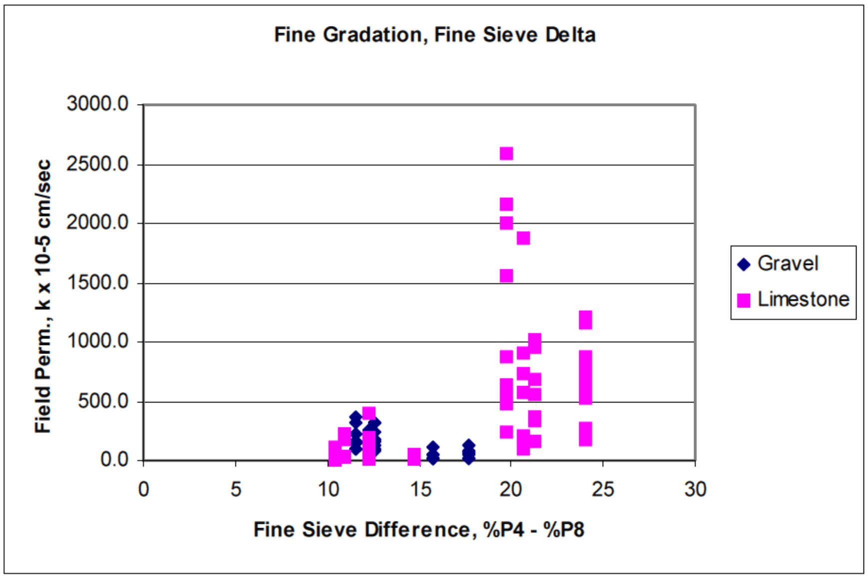



**Figure 2.12 Field Permeability and Fine Sieve Difference (%P4 – %P8)**

Gap-graded mixes have steep slope in their gradation followed by a relatively flat slope, leaving a gap in the gradation. These mixes are often designed by Illinois mix designers when trying to reduce the use (dependence) of crushed manufactured sands (FM-20) as a cost reduction measure. It is widely known and accepted throughout industry that gap-graded mixtures are prone to segregation, difficult to compact, lead to higher permeability, and poor joint construction.

As smoothness, density, and cessation temperature are all important HMA measurements, the research could look at temperature as being a controlling factor in maintaining or increasing the current lift thickness requirements for surface courses. Building ultra-thin HMA surfaces successfully has been partially researched by NCAT (Report 02-10 and TRR 1832, Paper No. 03-2390) and is being refined as a separate research project currently underway at the Illinois Center for Transportation.

## Case Studies

### *Reclamation, C&D Facility*

In the summer of 2007, an Illinois Asphalt Company purchased and began to operate a recycling facility in Chicagoland. The entrance road was granular upon purchase and has a very steep slope into the floor of the quarry (>10%). The existing truck traffic approaches 300 vehicles per day with a moderate growth anticipated. The roadway thickness design was performed in accordance with the Bureau of Design and Environment Chapter 54 as 10 million ESALs will occur over the 20-year design life.

To construct a highly abrasive resistant HMA mixture full-depth in the most economical way, it was decided to produce and install a 6 in thick N90, Binder course full-depth. The mixture was to act as the wear course also, it was produced with 50% P#4 in lieu of the 39% P#4 designed. Additionally, an existing IDOT verified design was reproportioned to increase RAP utilization. The original design is shown below with design proportions in parenthesis beside the actual proportioning was

- CM11: 23% (38.7),
- CM16: 24% (31.8),
- FM20: 21% (11.5),
- FA02: 7% (0.0),
- RAP: 30.5% (10.0), and
- MF: 1.5% (1.0).


The design optimum asphalt content for the coarse-graded HMA was utilized for the fine-graded HMA, and volumetrics and in-place density met IDOT specification limits.

Without violating the IDOT upper limit on the #4 sieve for HMA, it would not have been possible to use 30% RAP in this IL-19.0 mm, N90 binder course. The roadway installed has approximately 500,000 ESALs to date and shows no signs of deformation, cracking, delaminating, or channelization. A visual of the in-service HMA shows a “tight surface”, no segregation, and a smooth roadway. In summary, the HMA pavement installed is performing as designed and constructed.

Material Code Number:

State Assigne Mix Design Number

| Agg No. | #1 | #2 | #3 | #4 | #5 | RAP #6 | ASPHALT |
|---|---|---|---|---|---|---|---|
| Material Code | 042CM11 | 032CM16 | 038FM20 | 037FM02 | 004MF01 | 017CM16 | 10127 |
| Producer Number | 51972-15 | 51972-15 | 51972-15 | 50990-54 | 5116-05 | 5116-05 | 1757-05 |
| Producer Name | JSG | JSG | JSG | Kelly Co. | D Const | D Const | Seneca |
| Producer Location | Rockdale | Rockdale | Rockdale | Norway | Rockdale | Rockdale | Lemont |
| | | | | | | 4.0 | <--%AC in RAP |
| Aggragate Blend | 38.7 | 31.8 | 11.5 | 7.0 | 1.0 | 10.0 | <--% RAP |
| | | | | | | | New AC |
| Mix Blend | 36.8 | 30.2 | 10.9 | 6.7 | 1.0 | 9.9 | 4.5 |

100.0 <--Blend Total

100.0 <--Blend Total

| Agg No. / Sieve Size | #1 | #2 | #3 | #4 | #5 | #6 | Aggregate Blend |
|---|---|---|---|---|---|---|---|
| 1" ( 25.0mm ) | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 |
| 3/4"( 19.0mm ) | 91.3 | 100.0 | 100.0 | 100.0 | 100.0 | 100.0 | 96.6 |
| 1/2" (12.5mm ) | 40.0 | 100.0 | 100.0 | 100.0 | 100.0 | 99.0 | 76.7 |
| 3/8" ( 9.5mm ) | 21.2 | 97.8 | 100.0 | 100.0 | 100.0 | 94.0 | 68.2 |
| No.4 (4.75mm ) | 5.5 | 36.0 | 98.0 | 93.7 | 100.0 | 66.0 | 39.0 |
| No.8 ( 2.36mm ) | 3.2 | 6.9 | 81.0 | 79.8 | 100.0 | 45.0 | 23.8 |
| No.16 ( 1.18mm ) | 2.9 | 4.4 | 52.1 | 65.5 | 100.0 | 35.0 | 17.6 |
| No.30 ( 600μm ) | 2.8 | 3.7 | 27.0 | 43.1 | 100.0 | 28.0 | 12.2 |
| No.50 ( 300μm ) | 2.6 | 3.6 | 19.2 | 13.5 | 95.0 | 20.0 | 8.3 |
| No.100 ( 150μm ) | 2.5 | 3.2 | 8.4 | 5.1 | 90.0 | 14.0 | 5.6 |
| No.200( 75μm ) | 1.9 | 2.9 | 4.0 | 3.8 | 85.0 | 10.4 | 4.3 |

| Mixture Composition Specification | Formula | Formula Range Min | Max |
|---|---|---|---|
| 100 | 100 | | |
| 82-100 | 97 | | |
| 50-85 | 77 | 71 | 83 |
| | 68 | | |
| 24-40 | 39 | 34 | 44 |
| 20-36 | 24 | 19 | 29 |
| 10-25 | 18 | | |
| | 12 | 8 | 16 |
| 4-12 | 8 | | |
| 3-9 | 6 | | |
| 3-6 | 4.3 | 2.8 | 5.8 |

| | #1 | #2 | #3 | #4 | #5 | #6 | Blend | |
|---|---|---|---|---|---|---|---|---|
| Bulk Sp Gr | 2.680 | 2.669 | 2.712 | 2.548 | 2.900 | 2.660 | 2.671 | Dust/AC |
| Apparent Sp Gr | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | Ratio |
| Absorption, % | 1.30 | 1.70 | 1.10 | 2.00 | 1.00 | 1.00 | 1.420 | |
| | | | | | | SP GR AC | 1.040 | 0.87 |

## SUMMARY OF SUPERPAVE GYRATORY DESIGN DATA

BITUMINOUS MIXTURE AGED 2 HOURS @ 305

| DATA for N-initial 8 | AC, %MIX | ( Gmb ) | ( Gmm ) | ( Pa ) | VMA | VFA | Vbe | Pbe | Pba |
|---|---|---|---|---|---|---|---|---|---|
| MIX 1 | 4.3 | 2.177 | 2.564 | 15.1 | 22.0 | 31.4 | 6.90 | 3.30 | 1.05 |
| MIX 2 | 4.8 | 2.184 | 2.541 | 14.0 | 22.1 | 36.6 | 8.12 | 3.87 | 0.98 |
| MIX 3 | 5.3 | 2.188 | 2.522 | 13.2 | 22.4 | 41.0 | 9.17 | 4.36 | 0.99 |
| MIX 4 | 5.8 | 2.211 | 2.514 | 12.1 | 22.0 | 45.2 | 9.98 | 4.69 | 1.17 |

| | |
|---|---|
| Conditioned | 85.6 |
| Unconditioned | 94.6 |
| TSR | 0.91 |
| C A Strip Rating | |
| F A Strip Rating | |

Gse's ARE IN TOLERANCE

| DATA for N-design 90 | | ( Gmb ) | ( Gmm ) | ( Pa ) | VMA | VFA | Vbe | Pbe | Gse | Pba |
|---|---|---|---|---|---|---|---|---|---|---|
| MIX 1 | 4.3 | 2.423 | 2.564 | 5.5 | 13.2 | 58.3 | 7.69 | 3.30 | 2.745 | 1.05 |
| MIX 2 | 4.8 | 2.432 | 2.541 | 4.3 | 13.3 | 67.7 | 9.03 | 3.86 | 2.740 | 0.99 |
| MIX 3 | 5.3 | 2.444 | 2.522 | 3.1 | 13.3 | 76.9 | 10.26 | 4.36 | 2.741 | 0.99 |
| MIX 4 | 5.8 | 2.471 | 2.514 | 1.7 | 12.8 | 86.8 | 11.14 | 4.69 | 2.754 | 1.18 |

Gse Diff 0.014 Gse Tolerance OK

| RAP AC = 4% | NUMBER OF GYRATIONS | %AC | Gmb | Gmm | %VOIDS ( Pa ) Target | VMA | Field VMA | VFA | Gse | Gsb | TSR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| OPTIMUM DESIGN DATA @Ndes: ---> | 90 | 4.9 | 2.435 | 2.536 | 4.0 | 13.34 | 15.55 | 70.0 | 2.741 | 2.671 | 0.91 |
| REMARKS: 0.5% Anti Strip | | | | | | | | | | | |

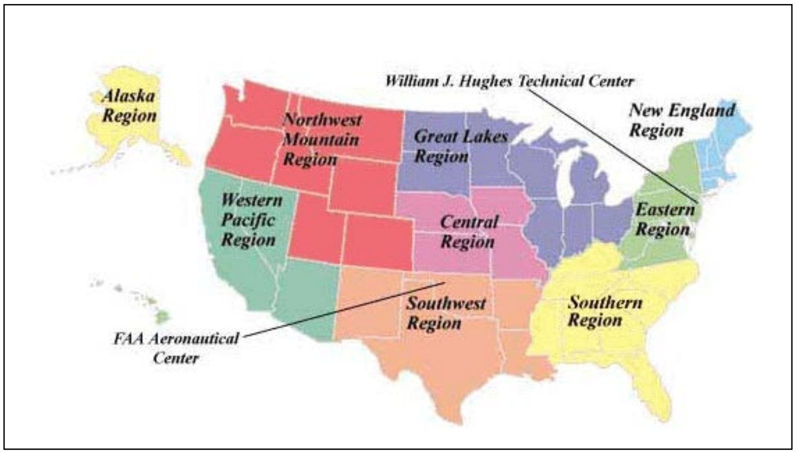
VMA ACCEPTABLE VFA ACCEPTABLE TSR ACCEPTABLE

Tested & Submitted by : Manager

Reviewed by :

Final Approval :

**Airfield HMA (P401)**




There are nine Federal Aviation Administration regions that deliver the airport program throughout the United States. According to Buncher and Duval (2003), "Asphalt pavements have a long history of outstanding performance in airfield applications. One of the first modern airport projects in the country was Washington National Airport. Upon its completion in 1940, more than one million square yards hot mix asphalt (HMA) pavements had been constructed to meet the bright future of commercial aviation. Much has changed in both the asphalt and airfield pavement industries over the last 63 years. Larger and more powerful aircraft have changed how pavement structures and mixes are designed. Enhanced production, laydown, and quality control technology has led to increased quality and higher production capability on airfield paving projects." In dealing with the ever increasing demands of increasing air traffic, the engineers who plan and design new and rehabilitation projects are challenged by handling huge loads. Typical airplane loads can exceed 1.3 million pounds with tire pressures in excess of 400 psi.

Photo from Rototherm instrument control

As spelled out in the current P401 specification used on heavily trafficked airfields:

*"Aggregates shall consist of crushed stone, crushed gravel, or crushed slag with or without sand or other inert finely divided mineral aggregate. Coarse aggregate shall consist of sound, tough, durable particles, free from adherent films of matter that would prevent thorough coating and bonding with the bituminous material and be free from organic matter and other deleterious substances. Fine aggregate shall consist of clean, sound, durable, angular shaped particles produced by crushing stone, slag, or gravel that meets the requirements for wear and soundness specified for coarse aggregate. The aggregate particles shall be free from coatings of clay, silt, or other objectionable matter and shall contain no clay balls. Natural sand may be used to obtain the gradation of the aggregate blend or to improve mix workability. The amount of sand to be added will be adjusted to produce mixtures conforming to requirements of this specification."*

*"The bituminous plant mix shall be composed of a mixture of well-graded aggregate, filler if required, and bituminous material. The several aggregate fractions shall be sized, handled in separate size groups, and combined in such proportions that the resulting mixture meets the grading requirements of the job mix formula (JMF)."*

MARSHALL TEST PROPERTIES AND VALUES

| Test Property | Pavement Designed for Aircraft Gross Weights of 60,000 lb or More or Tire Pressures of 100 psi or More | Pavements Designed for Aircraft Gross Weight Less than 60,000 lb or Tire Pressure Less than 100 psi |
|---|---|---|
| Number of blows | 75 | 50 |
| Stability, lb (N) | 2,150 (9555) | 1,350 (4450) |
| Flow, 0.01 in (0.25 mm) | 10-14 | 10-18 |
| Air Voids (percent) | 2.8-4.2 | 2.8-4.2 |
| Voids in Mineral Aggregate (VMA) | See Table 5 | See Table 5 |

MINIMUM PERCENT VOIDS IN MINERAL AGGREGATE

| Maximum Particle Size | Minimum VMA |
|---|---|
| 1/2 in (12.5 mm) | 16 |
| 3/4 in (19.0 mm) | 15 |
| 1 in (25.0 mm) | 14 |
| 1-1/4 in (31.25 mm) | 13 |

Lower than necessary VMA values lead to poor performance. The low effective asphalt content manifests throughout production and construction activities. This author has analyzed several roadway and runway project failures, many with low field VMA and high in-place air voids because of the inability to compact the mixture efficiently, which leads to permeability problems. Prior analysis by others indicated that poor construction activities were to be blamed for the failure. However, after further investigation, the variable and nonchalant determination of the bulk (dry) aggregate specific gravity presented itself to be the culprit in many cases. Low VMA segregation takes hold at a very early stage of the production process and carries through into storage and trucking, while passing through the paving machine and trying to achieve acceptable in-place densities, and finally as permeability in the asphalt pavement.

The Federal Aviation Administration has developed a specification that is fine-graded, high in stability, demands consistent and substantial in-place density, requires slightly more liquid asphalt than for roadways, and carries millions of pounds of loading with high tire pressures. Airfield pavements do not typically rut, delaminate, crack, or ravel when properly designed, produced, and constructed, yet they are what most in industry would call “fine-graded mixtures.”

AGGREGATE–BITUMINOUS PAVEMENTS

| Sieve Size | Percentage by Weight Passing Sieves 1-1/4 in | 1 in | 3/4 in | 1/2 in |
|---|---|---|---|---|
| 1-1/4 in (30.0 mm) | 100 | — | — | — |
| 1 in (24.0 mm) | 86–98 | 100 | — | — |
| 3/4 in (19.0 mm) | 68–93 | 76–98 | 100 | — |
| 1/2 in (12.5 mm) | 57–81 | 66–86 | 79–99 | 100 |
| 3/8 in (9.5 mm) | 49–69 | 57–77 | 68–88 | 79–99 |
| No. 4 (4.75 mm) | 34–54 | 40–60 | 48–68 | 58–78 |
| No. 8 (2.36 mm) | 22–42 | 26–46 | 33–53 | 39–59 |
| No. 16 (1.18 mm) | 13–33 | 17–37 | 20–40 | 26–46 |
| No. 30 (0.600 mm) | 8–24 | 11–27 | 14–30 | 19–35 |
| No. 50 (0.300 mm) | 6–18 | 7–19 | 9–21 | 12–24 |
| No. 100 (0.150 mm) | 4–12 | 6–16 | 6–16 | 7–17 |
| No. 200 (0.075 mm) | 3–6 | 3–6 | 3–6 | 3–6 |
| **Asphalt percent:** | | | | |
| Stone or gravel | 4.5–7.0 | 4.5–7.0 | 5.0–7.5 | 5.5–8.0 |
| Slag | 5.0–7.5 | 5.0–7.5 | 6.5–9.5 | 7.0–10.5 |

The primary concern after construction of asphalt runways is understanding that the safety of air travel may be compromised by foreign object damage (FOD); similar to the primary after construction concern at motor speedways such as Indianapolis, Talladega, and Newport Beach. As stated by Buncher and Duval, FOD is of great concern to the safe operation of aircraft. Loose aggregate particles raveling from deteriorated pavements can be ingested into the high thrust jet engines. It is not simply a theoretical problem. Rather, FOD from loose aggregate particles from the pavement has been identified as the cause of at least one airplane crash that resulted in loss of life. Due to the life-safety implications of this problem, minimizing FOD must be considered one of the primary goals of the pavement design process. For this reason fine-graded dense mixes have been preferred for airfields because their tighter surface texture tends to be less permeable to water and air, leading to increased durability, and the inherent reduction to FOD. Fine-graded mixtures also offer improved workability versus coarse-graded mixes.

Flanigan 1-inch Max P401 HMA 1

Recognizing the specific challenges of airport pavements with regard to traffic volume, traffic loads, and FOD leads the HMA mix designer to ensure that the pavement is stable, durable, impermeable, and workable. Over its intended life, the pavement should adequately resist damaging environmental effects such as raveling, cracking, and stripping. Simultaneously, the HMA mix should be stable enough to resist enormous aircraft loads and impermeable to protect the pavement foundation. It cannot be forgotten to make the mix workable so it is capable of being placed in the field. The bottom line is that proper selection of the asphalt binder, adequate asphalt content, aggregate characteristics, and mix proportioning all greatly affect the ability of the pavement to perform over time.

In general, based on the unique needs of the Federal Aviation Administration for “surface texture” and the known characteristics of aggregates it has been observed for years that the resulting JMF typically:

- Falls on the fine side of the 0.45 power chart,
- Contains a minimum of natural sand and a high percentage of manufactured sand,
- Passes through the restricted zone, as defined by SHRP, from above to below quickly without a sand hump, and
- Measures within Superpave recommendations on the fine aggregate angularity even though this is not normally a measured value by specification.

Airfield pavement mix designs, production, and construction practices provide good pavement life and carry heavy loadings. The Marshall and current P401 specification should be considered a solid reference during the balance of this research project.

**Pavement Friction**

**[References NCHRP W108, Project 01-43; Guide for Pavement Friction and Midwest Regional University Transportation Center Traffic and Operations (TOP) Laboratory]**

***From NCHRP W108***

"Pavement friction design is one of the key elements required for ensuring highway safety, as empirical evidence suggests that vehicle crashes are highly correlated to the amount of pavement friction available at the pavement–tire interface. Although comprehensive guidance covering both the policy and technical aspects of designing for and managing pavement friction was provided in Guidelines for Skid-Resistant Pavement Design, published by AASHTO in 1976, many significant improvements in design and material characterization have taken place since this time. Moreover, although more current information and guidance related to pavement friction is available, it is quite fragmented and has not been integrated into a comprehensive administrative policy and design tool for addressing friction issues. Thus, a new Guide for Pavement Friction was developed to assist highway engineers in:

a) Understanding the complex subject of pavement friction and its importance to highway safety and
b) Instituting pavement management and design practices and processes that optimize friction safety, while recognizing and considering the effects on economics and other pavement–tire interaction issues (e.g., noise, splash/spray, visibility/glare).


Pavement sections with measured friction values at or below an assigned investigatory level are subject to a detailed site investigation to determine the need for remedial action, such as erecting warning signs, performing more frequent testing and analysis of friction data and crash data, or applying a short-term restoration treatment. For pavement sections with friction values at or below the intervention level, remedial action may consist of immediately applying a restoration treatment or programming a treatment into the maintenance or construction work plan and/or erecting temporary warning signs at the site of interest." For all of these reasons there are federal mandates that have been issued and implemented over the years.

***Federal Mandates***

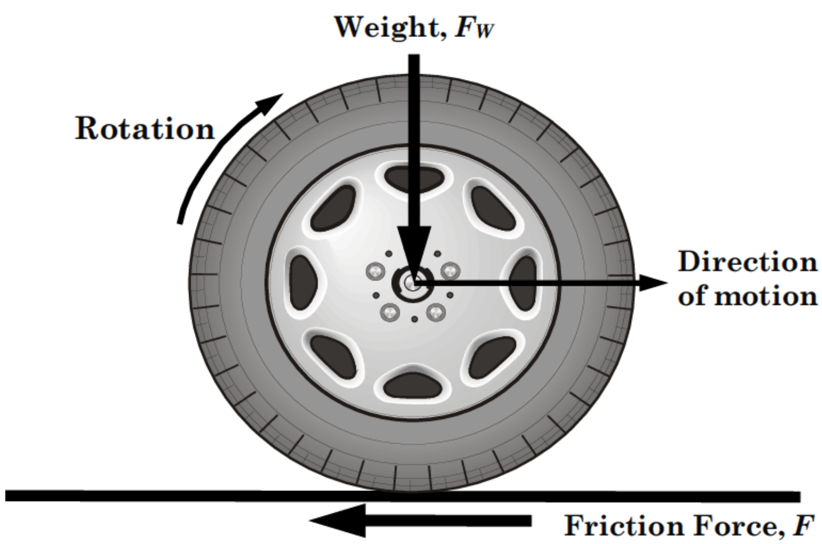
Since 1966, the U.S. Congress has approved several Acts concerning highway safety. A chronological summary of these Acts and associated directives from federal agencies, as listed below, are summarized in the following sections.

- Highway Safety Act of 1966 (23 USC Chapter 4)
- Highway Safety Program Standard 12 (HSPS No. 12) of 1967
- FHWA Instructional Memorandum 21-2-73
- 1975 Federal-Aid Highway Program Manual

- 1980 FHWA Technical Advisory T 5040.17
- Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991
- National Highway System (NHS) Designation Act of 1995
- Transportation Equity Act for the 21st Century of 1998 (TEA-21)
- 2005 FHWA Technical Advisory T 5040.36

**Friction Mechanisms**

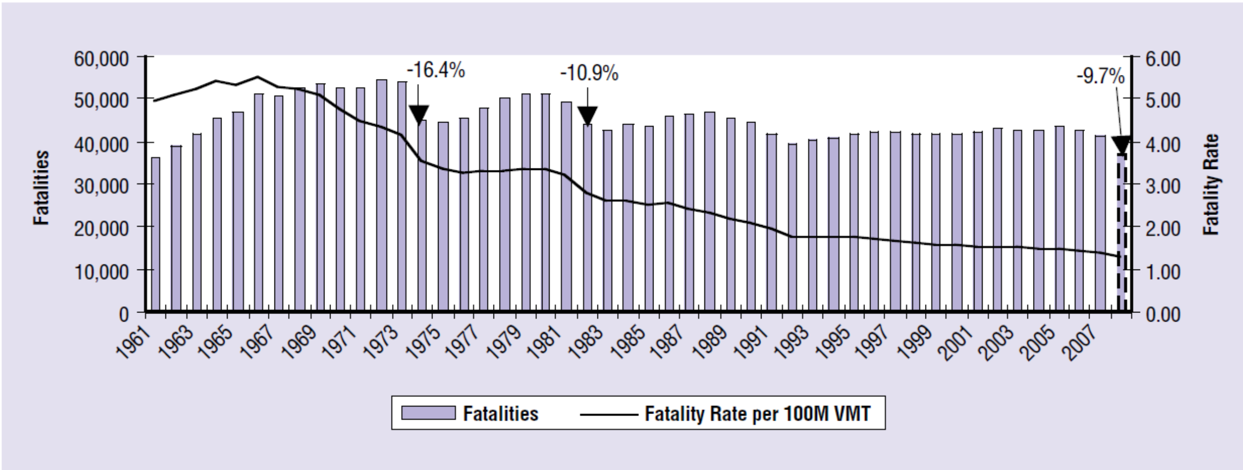
- Pavement friction is the force that resists the relative motion between a vehicle tire and a pavement surface. This resistive force, illustrated below, is generated as the tire rolls or slides over the pavement surface. Below is a simplified diagram of forces acting on a rotating wheel.




The resistive force, characterized using the non-dimensional friction coefficient, μ, is the ratio of the tangential friction force (F) between the tire tread rubber and the horizontal traveled surface to the perpendicular force or vertical load (FW). Pavement friction plays a vital role in keeping vehicles on the road, as it gives drivers the ability to control and/or maneuver their vehicles in a safe manner in both longitudinal and lateral directions. It is a key input for highway geometric design, used in determining the adequacy of the minimum stopping sight distance, minimum horizontal radius, minimum radius of crest vertical curves, and maximum super-elevation in horizontal curves. Generally speaking, the higher the friction available at the pavement–tire interface, the more control the driver has over the vehicle.

Comprehensive guidance covering both the policy and technical aspects of designing for and managing pavement friction has been limited to Guidelines for Skid-Resistant Pavement Design, published by AASHTO in 1976. This document recommended pavement specifications that would yield the desired frictional properties upon completion of construction and that would maintain adequate long-term friction. It also discussed the importance of aggregate selection and mixture design for both asphalt and concrete surfaced pavements, and the role of micro-texture and macro-texture in pavement surface friction. Many methods of reducing skid accidents have been initiated throughout the engineering arena and are ongoing daily. Since the construction of the interstate systems, improvements have been made to roadways and vehicles.

**Figure 1: Fatalities and Fatality Rates per 100 Million VMT From 1961 - 2008**




Driver challenges that have been addressed to improve safety and reduce fatalities are:

- Oversteering or Understeering (YAW),
- Geometrics (curves and horizontal and vertical alignment; A/D lanes),
- Traffic Volume, speed, and composition,
- Roadway maintenance,
- Intersections,
- Vehicle ability, (SUV and center of gravity)
- Drivers age and ability (requirements for training),
- Attenuators and barriers,
- Sight distance,
- Electronics (better signals),
- Reflectivity (brighter signs),
- Rollover Mitigation Systems,
- Anti-lock brakes,
- Longitudinal and traversal brake lock,
- Education and enforcement of driving while under the influence,
- Stiffer penalties, and
- Advertising campaigns.

***Auto Electronic Stability Control [Bosch]***

"The results of several independent studies show a consistent picture of the ESP with remarkable safety benefits and proof the positive impact. Further potential is available with functional extensions especially for SUV and light trucks concerning rollover mitigation and 4WD adaptations. The ESP with Rollover Mitigation functions helps the driver to stay on the road and to avoid tripping obstacles by a specific yaw control. It also supports the driver with an optimized lateral acceleration control to manage rollover critical on-road situations. In cooperation with four wheel drive train concepts, ESP delivers at the same time the expected safety benefits and excellent off-road and handling functionality."

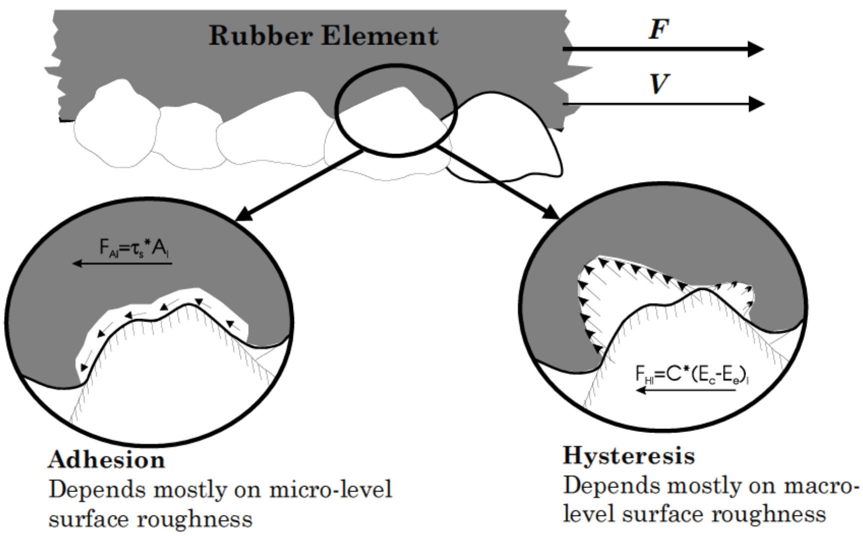



Figure 15. Key mechanisms of pavement–tire friction.

Pavement friction is the result of a complex interplay between two principal frictional force components: adhesion and hysteresis, as shown in Figure 15. Adhesion is the friction that results from the small-scale bonding/interlocking of the vehicle tire rubber and the pavement surface as they come into contact with each other. It is a function of the interface shear strength and contact area. The hysteresis component of frictional forces results from the energy loss due to bulk deformation of the vehicle tire. The deformation is commonly referred to as enveloping of the tire around the texture. When a tire compresses against the pavement surface, the stress distribution causes the deformation energy to be stored within the rubber. As the tire relaxes, part of the stored energy is recovered, while the other part is lost in the form of heat (hysteresis), which is irreversible. That loss leaves a net frictional force to help stop the forward motion.

Macro-texture is defined by the type of surface paving mixtures and/or surface texturing techniques applied. Several different surface mix types and finishing/texturing techniques are available for use in constructing new pavements and overlays, or for restoring friction on existing pavements. The more commonly used mix types and texturing techniques are presented in this report along with the typical macro-texture levels achieved. Pavement-tire considerations, such as noise, splash/spray, and hydroplaning, as well as general considerations, such as constructability, cost, and structural performance, are not directly discussed in this report, however, they must be an integral part of any policies developed for the application of these mixes and texturing techniques.

### *Testing Equipment*

Testing the pavement surface for micro-texture and macro-texture can be evaluated using various types of equipment, including:

- Micro-texture, which can be evaluated using any of the following:
    - Locked-wheel friction tester.
    - British Pendulum Tester (BPT).
    - Dynamic Friction Tester (DFT).
- Macro-texture, which can be evaluated using any of the following:
    - High-speed laser.
    - Circular Texture Meter (CTM).
    - Sand Patch Method (SPM).

### *Aggregate Measures*

Pavement friction design is basically a process of selecting the right combination of pavement surface micro-texture and macro-texture to optimize available pavement friction for a given design situation. For both asphalt and concrete surfaces, micro-texture is defined by the surface aggregate material properties. Historically aggregate measures used in road construction included establishing, measuring, and ensuring minimum values for abrasion resistance (mechanical properties), angularity (physical and geometrical properties), hardness, mineralogy and petrographic analysis (i.e., mineral composition and structure), polish resistance, shape, soundness, and texture. Based on recent thorough evaluations of aggregate tests related to performance (Kandhal and Parker, 1998; Folliard and Smith, 2003) and the proactive work of various states—Maryland, Michigan, Ohio, New York, and Texas, to name a few—the following tests (with methods) are considered most relevant in characterizing frictional properties and potential performance with aggregates in use today.

- Acid Insoluble Residue (AIR) (ASTM D 3042).
- Fractured-Face Particles (ASTM D 5821).
- LA Abrasion (AASHTO T 96 or ASTM C 131 for small-sized coarse aggregates;
- ASTM C 535 for large-sized coarse aggregates).
- Magnesium Sulfate Soundness (AASHTO T 104 or ASTM C 88).
- Micro-Deval for Coarse Aggregates (AASHTO TP 58 or ASTM D 6928).
- Micro-Deval for Fine Aggregates (Canadian Standards Assoc. [CSA] A23.2-23A).
- Mineralogy and Petrographic Analysis (ASTM C 295).
- Polished Stone Value (PSV) (AASHTO T 278 and T 279 or ASTM E 303 and D 3319).
- Uncompacted Voids (UV) for Fine Aggregate (AASHTO T 304 or ASTM C 1252).
- UV for Coarse Aggregate (AASHTO T 326).
- Scratch Hardness (Mohs).

**From MRUTC [Noyce, Bahia, Yambo, and Kim]**

"Pavement management usually leaves macro-texture out of the panorama; but the real problem is that sometimes skid resistance as a safety concept is also neglected and not incorporated in the pavement management process. Jayawickrama conducted a study of state practices to control skid resistance on asphalt roadways. Five categories of practice were considered:

I: No specific guidelines to address skid resistance;

II: Skid resistance is accounted for through mix design;

III: General aggregate classification procedures are used;

IV: Evaluate aggregate frictional properties using laboratory test procedures;

V: Incorporates field performance in aggregate qualification."

**Categories of Design Procedures used by State DOTS**

| State DOT | Category I | II | III | IV | V | State DOT | Category I | II | III | IV | V |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Alabama | | | | X | | Nebraska | | X | | | |
| Arizona | | X | | | | Nevada | X | | | | |
| Arkansas | | | X | | | New Hampshire | | X | | | |
| California | X | | | | | New Jersey | | | | X | |
| Colorado | X | | | | | New Mexico | X | | | | |
| Connecticut | X | | | | | New York | | | | X | |
| Delaware | | | X | | | North Carolina | X | | | | |
| Florida | | | | X | X | North Dakota | X | | | | |
| Georgia | | | X | | | Ohio | X | | | | |
| Idaho | X | | | | | Oklahoma | | | | X | |
| Illinois | | | X | | | Oregon | | X | | | |
| Indiana | | | | X | | Pennsylvania | | | | X | X |
| Iowa | | | | X | | Rhode Island | X | | | | |
| Kansas | | | X | | | South Carolina | | | | X | |
| Kentucky | | | | X | X | South Dakota | | X | | | |
| Louisiana | | | | X | | Tennessee | | | | X | |
| Maine | | X | | | | Texas | | | | X | X |
| Maryland | X | | | | | Utah | | | | X | |
| Massachusetts | X | | | | | Vermont | | X | | | |
| Michigan | | | | X | | Virginia | | | X | | |
| Minnesota | | | | X | | Washington | | X | | | |
| Mississippi | | | | X | | West Virginia | | | X | | |
| Missouri | X | | | | | Wisconsin | | | | X | |
| Montana | | X | | | | Wyoming | X | | | | |
| Nebraska | | X | | | | | | | | | |

## State Overview

### *Illinois (VST for Friction Testing)*

- The Illinois DOT selects and designs pavement surfaces in accordance with specific criteria. Traffic levels from the expected year of construction are used to determine the mixture for HMA pavements. For IDOT surface courses, friction measures equivalent to or greater than those provided by the following guidelines must be met.
- Mixture C is used as the Class I surface course on roads and streets having an ADT of 5,000 vehicles/day or less.
- Mixture D is used as the Class I surface course on two-lane roads and streets having an ADT greater than 5,000 vehicles/day, on four-lane highways having an ADT between 5,001 and 25,000 vehicles/day, and on six-lane (or greater) highways having an ADT of 60,000 vehicles/day or less.
- Mixture E is used as the Class I surface course on four-lane highways having an ADT between 25,001 and 100,000 vehicles/day or on six-lane (or greater) highways having an ADT between 60,001 and 100,000 vehicles/day.
- Mixture F is used as the Class I surface course on any facility having an ADT greater than 100,000 vehicles/day.

The Illinois DOT friction policy requires the use of non-polishing coarse aggregates at varying percentages for roadways based on ADT numbers. The typical practice is to have a certain percentage of the aggregate blend containing non-polishing CA. For example, the specification states that for an HMA high ESAL IL-9.5/12.5 mm “D” surface mixture:

Limestone may be used if blended by volume in the following CA percentages:

8. Up to 25% Limestone with at least 75% Dolomite,
9. Up to 50% Limestone with at least 50% Crushed: Gravel, Sandstone, Slag (both),
10. Up to 75% Limestone with at least 25% Crushed: Slag (both) or Sandstone.

If a typical surface course contains 65% CA and 35% FA then friction CA shall make up between approximately 16% (Case 3) and 49% (Case 1) for the entire aggregate mixture. In order to provide the same friction with fine-graded HMA these should become the new specification values when properly adjusted for aggregate specific gravity.

***Indiana***

Polish resistant aggregates are defined as those aggregates in accordance with ITM 214. The amount of crushed limestone sand shall not exceed 20% of the total aggregate used in HMA surface mixtures with ESAL equal to or greater than 3,000,000, except limestone sands manufactured from aggregates on the Department’s list of approved Polish Resistant Aggregates will not be limited. Indiana uses the British Wheel, British Pendulum Tester, Friction Vehicle, and Smooth Tread Standard Tire. [Ref. Indiana Test Method 214].

| Coarse Aggregate Type | Traffic ESALs | | |
|---|---|---|---|
| | < 3,000,000 | < 10,000,000 | ≥ 10,000,000 |
| Air-Cooled Blast Furnace Slag | Yes | Yes | Yes |
| Steel Furnace Slag | Yes | Yes | Yes |
| Sandstone | Yes | Yes | Yes |
| Crushed Dolomite | Yes | Yes | Note 1 |
| Polish Resistant Aggregates | Yes | Yes | Note 1 |
| Crushed Stone | Yes | No | No |
| Gravel | Yes | No | No |
| Note 1. Polish resistant aggregates or crushed dolomite may be used when blended with ACBF or sandstone but cannot exceed 50% of the coarse aggregate by weight (mass), or cannot exceed 40% of the coarse aggregate by weight (mass) when blended with steel furnace slag. | | | |

### *Aggregate Wear Index; Michigan DOT*

Michigan determines the polishing potential of HMAC coarse aggregates for design of high friction pavements through laboratory testing; specifically wear-track testing or petrographic analysis (Skerritt, 2004). The wear-track testing program consists of a large-scale indoor polishing track and a tire-mounted friction tester. Aggregate test specimens are subjected to 4 million wheel passes on the wear track, during which surface friction is measured. The normalized value of friction at the end of the test is used to calculate an Aggregate Wear Index (AWI), which is a measure of the polishing potential of the aggregate source tested. Aggregates are specified for use as follows, based on anticipated traffic (Liang, 2003):


1. ADT < 100 vehicles/day/lane: no AWI requirement.
2. ADT = 100 and < 500 vehicles/day/lane: AWI =220.
3. ADT $\geq$ 500 vehicles/day/lane: AWI =260.

### *Iowa Uses Mohs Hardness Values*

For friction classification L-2, L-3, and L-4.

- Type 1, 7 to 9 is imported,
- Type 2, 5 to 7 are quartz and granites,
- Type 3, traprocks and crushed gravels,
- Type 4, 3 to 5 dolomitic and limestones,
- Type 5, dolomitic and limestone.

### *Missouri*

- Limestone LA Abrasions and Non-Carbonate thresholds by Volume.
- N/A for Dolomite.

Table C-6. Summary of test properties used in aggregate selection.

| Aggregate Property | No. of States |
|---|---|
| Gradation and Size | 8 |
| Angularity, Shape, and Texture | 16 |
| Mineral Composition | 15 |
| Resistance to Degradation and Abrasion | 21 |
| Durability and Soundness | 11 |
| Polish and Frictional Characteristics | 11 |

## Summary of Agency Survey

### Treating Pavement Surfaces

#### *Obtain IDOT Research Information from ICT*

#### *Federal Aviation Administration*

International airfield runways are grooved after completion.

**Conversations**

- John Yzenas, Levy, Bill Pine, HRG.
- Mr. Abdul Dahhan, P.E. IDOT D1: In a face-to-face meeting Mr. Dahhan expressed his desire to ensure that a measure of the fine aggregate surface micro-texture be addressed through either the FAA or one that will give similar comfort for strength measurement.
- Mr. David Lippert, P.E., IDOT BMPR: During several meetings Mr. Lippert indicated that the BMPR is purchasing a Hamburg wheel tester for strength testing capabilities.
- Mr. Marvin Traylor, P.E., Ph. D., Illinois Asphalt Pavement Association: Supplied me with the AAPT paper he co-authored in 1988, explained the historical background of how IDOT moved forward with HMA road construction, and expressed his support for developing longer lasting HMA roadways through research that improves the life-cycle cost analysis for road building and is thus economically feasible for IDOT, the taxpayers of Illinois, and industry.
- Mr. Peterson, P.E., Executive Director of the Colorado Asphalt Pavement Association: In a phone conversation with Mr. Thomas Peterson, P.E., Executive Director of the Colorado Asphalt Pavement Association, on 9/28/09 he discussed the Colorado history of asphalt from pre-Superpave to date. Pre-Superpave had rutting in Colorado with large top size

stone and high natural sand mixes. In Colorado Superpave (CDOT SX mix = 1” top size, CDOT S mix = ¾” top size) introduced high crushed sands with FAA measures. According to Tom, “This is one of the best changes to HMA over the years.” S is the current mixture of choice in Colorado for the past 3–4 years because segregation problems could not be solved with early Superpave mixes. Tom states that Colorado has not had any rutting issues since before the advent of Superpave nearly 15 years ago.

- Mr. Brian Rice, Geologist: Explained the availability of substantial amounts of FM-21, dirty crushed sand, throughout the state. He, like many in industry, understand that the high dust makes the increased use of FM-21, especially with the increased use of reclaimed asphalt pavement (RAP), unlikely however this material can be washed and turned into FM-20. Mr. Rice indicated that several sources in Illinois have tens of millions of tons of FM-20 and FM-21 available for use with fine-graded HMA and that if additional processing is required it will be done.
- Mr. Patrick Koester, Corporate VP of Production for Howell Asphalt; QC and HMA Plants.

**Top Ten**

1. Density in general should in theory be easier with fine-graded than coarse-graded, and this will be good for edge of pavement and middle of the mat.
2. Mix will be less permeable.
3. Should segregate less.
4. For level binders it will help achieve a much less permeable mix.
5. Binders (19.0’s) will be helped with a fine-graded mix, for much the same reasons as surface mixes.
6. Most failures observed are older coarse-graded binders.
7. Most current coarse-graded binders are much finer and contain higher AC contents and the VMA is definitely higher than older designs.
8. These mixes will perform much better that the old designs.
9. But a fine-graded binder will help with density and permeability.
10. The less permeable fine-graded mix will help with the hydro-demolition that occurs with binders that are placed on old concrete that has poor drainage.

# APPENDIX B ADDITIONAL EVALUATION OF EFFECT OF FREEZE-THAW CYCLES

Both C-G and F-G mixtures were evaluated utilizing various levels of freeze-thaw cycles. According to AASHTO T283 (moisture-induced damage evaluation), one freeze-thaw cycle was utilized. Additionally, conditioning with five freeze-thaw cycles was also conducted to evaluate relative performance under extreme conditioning.

The IDOT modified specification was followed, with the exception that specimens were wrapped with at least three layers of plastic to retain moisture, following current practices used at ATREL. Samples were kept in refrigeration at $-18^{o}$C for at least 16 ±1 hr. Samples were then placed in a water bath and held constant at $60^{o}$C for 24 ± 1 hr. After required freeze-thaw cycles were completed, specimens were soaked in a water bath at 25 ± $0.5^{o}$C. Nine replicates were produced for each mixture, producing three replicates for each of the three conditioning regimens.

Figure B.1 shows tensile strength ratio (TSR) results. While very little difference was noted between the specimens subjected to zero and one cycle of conditioning, the five freeze-thaw cycle produced significant differences.

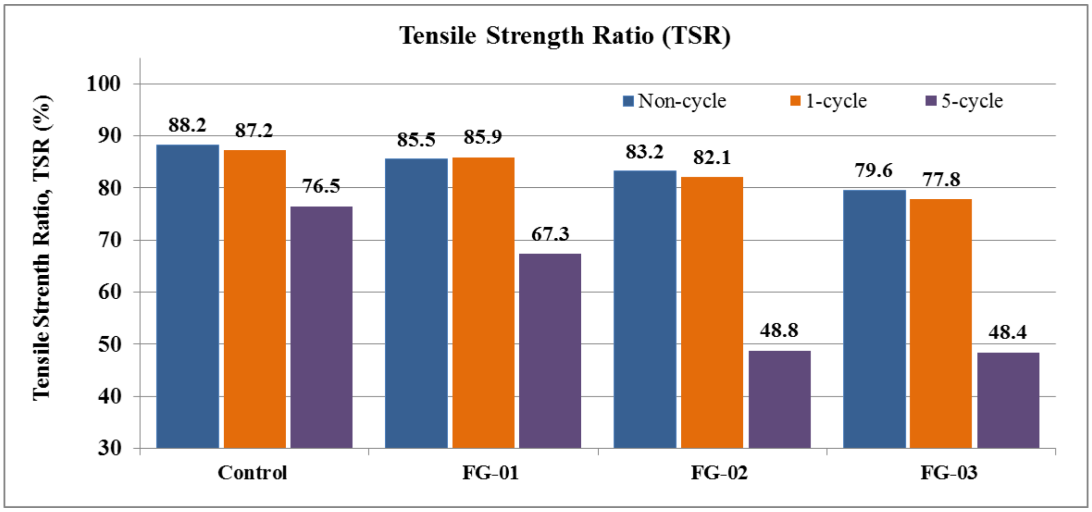



Figure B.1. TSR results for 0, 1- and 5-cycles of freeze-thaw process.

Figures B.2 and B.3 present test results of wet and dry strengths for the three conditioning cases. Unlike TSR results, the absolute tensile strength in both the wet and dry condition tended to increase with mixture fineness. The exception to this trend was noted in the 5 freeze-thaw cycle results, where lower absolute strengths were noted for the two finest mixtures.

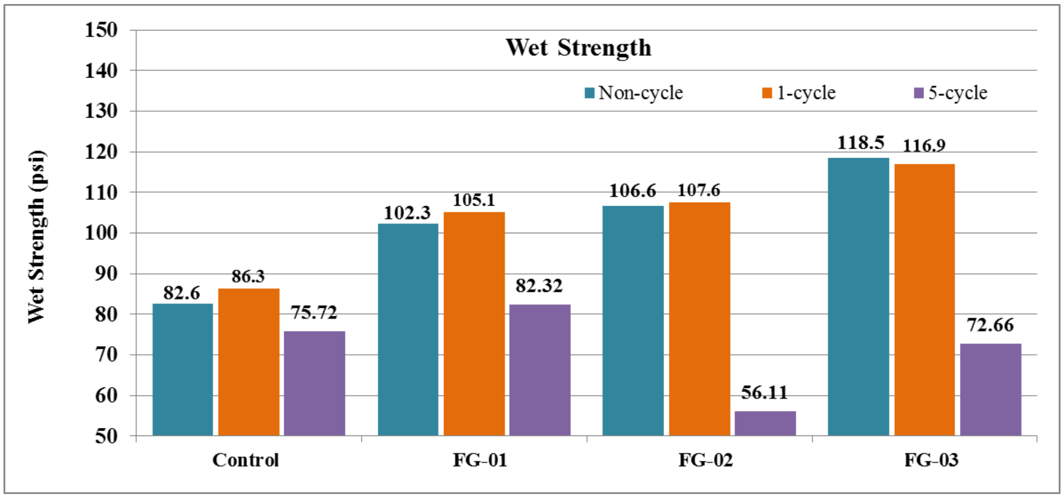



Figure B.2. Wet strength of the mixtures for 0, 1, and 5 cycles of freeze-thaw process.

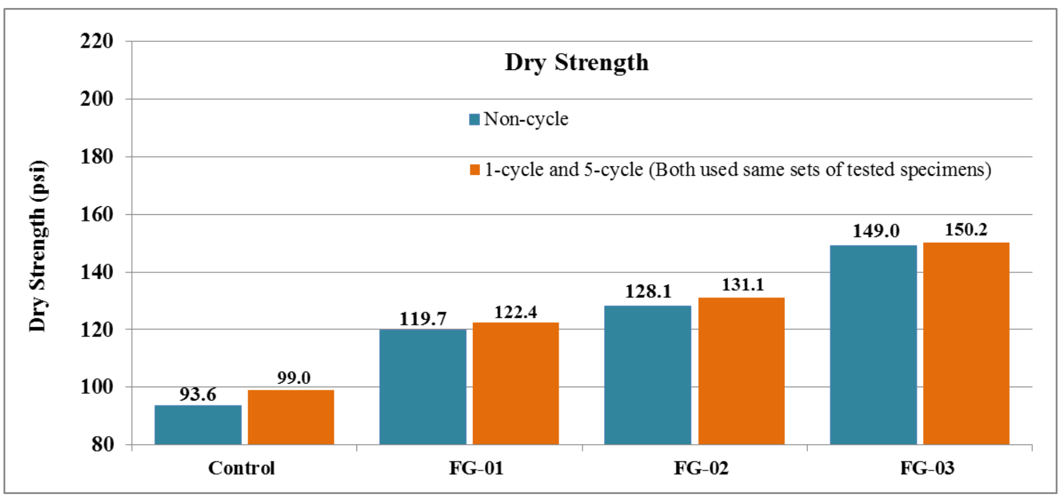



Figure B.3. Dry strength of the mixtures for 0, 1, and 5-cycles of freeze-thaw process.

# APPENDIX C ADDITIONAL PERFORMANCE TEST RESULTS FOR LABORATORY MIXES

## Dynamic Modulus

Dynamic modulus, which is often referred to as E-star (|E*|) testing in the pavements community, was conducted, as it is gaining popularity as an input for mechanistic-empirical pavement design guides such as that developed under NCHRP 1-37A. The dynamic modulus is driven by asphalt viscoelastic properties, and is therefore dependent on test temperature and rate of loading.

The dynamic modulus is computed as the absolute value of the complex modulus, which in turn represents the ratio between applied stress amplitude and the resulting axial strain amplitude in cyclic uniaxial compressive testing of cylindrical specimens (Figure C.1).

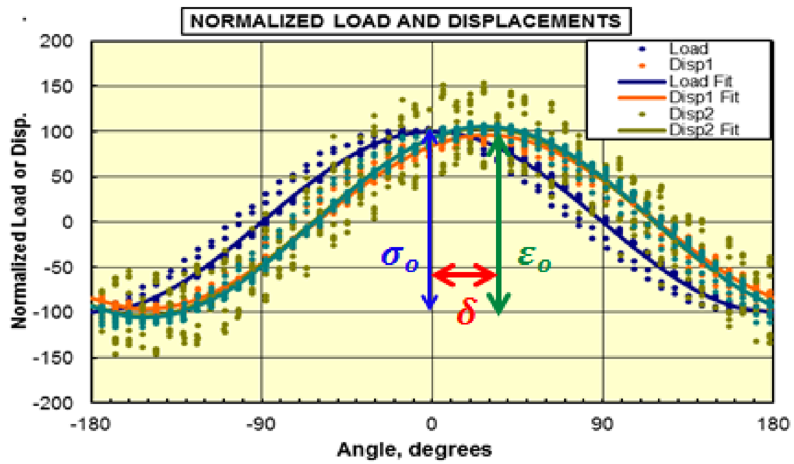



Figure C.1. Plot of normalized load versus displacement.

Dynamic modulus, E*, can be calculated as:

$$|E^*| = \frac{\sigma_0}{\varepsilon_0}$$

where $\sigma_0$ is the stress amplitude and $\varepsilon_0$ is the strain amplitude. Phase angle is computed as

$$\phi = t_{lag} x\ f\ x\ 360^0$$

where $t_{lag}$ is the time difference between the stress and strain peaks (in seconds), and $f$ is the frequency of the applied cyclic load (Hz).

Experimentally, dynamic modulus of the mixes for the study was determined in accordance with AASHTO TP 63-07(2009). Test specimens were produced at 7.0 ± 0.5% air voids, and were 150-mm tall and 100-mm in diameter. Three replicates were made for each mix. Figure C.2 illustrates the test device used. Each specimen was tested at five different temperatures: –10°C, 4°C, 21°C, 37 °C, and 53°C. For each test temperature, test specimens were tested at six different load frequencies: 25, 10, 5, 1, 0.5, and 0.1 Hz. A sinusoidal load waveform was used. The amplitude of loading was initially adjusted to produce an axial strain between 50 and 150 microstrain. This was done to ensure that the material was responding in the linear viscoelastic range.

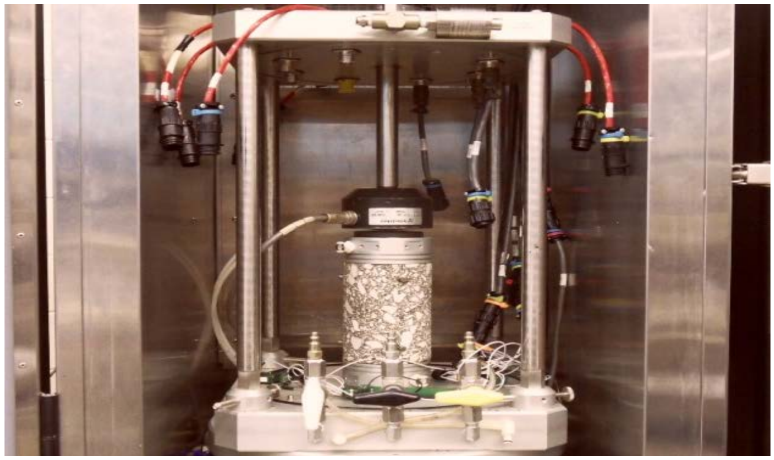


Figure C.2. Test device and setup for complex modulus test.

Table C.1 (next page) presents test results of the dynamic modulus test for all test temperatures and frequencies. For low temperatures of 10°C and -4°C, the control mix (coarsest of all mixes) provided the highest stiffness. At 21, 37, and 53°C, the FG01 mix exhibited the highest stiffness.

Figure C.3 presents a plot of the master curves of each mixture based on the test results, illustrating the rough equivalency of the fine-graded mixes as compared to the control mixture. This suggests that very little total pavement thickness difference would result in an M-E based design of a pavement section regardless of which mix was used.

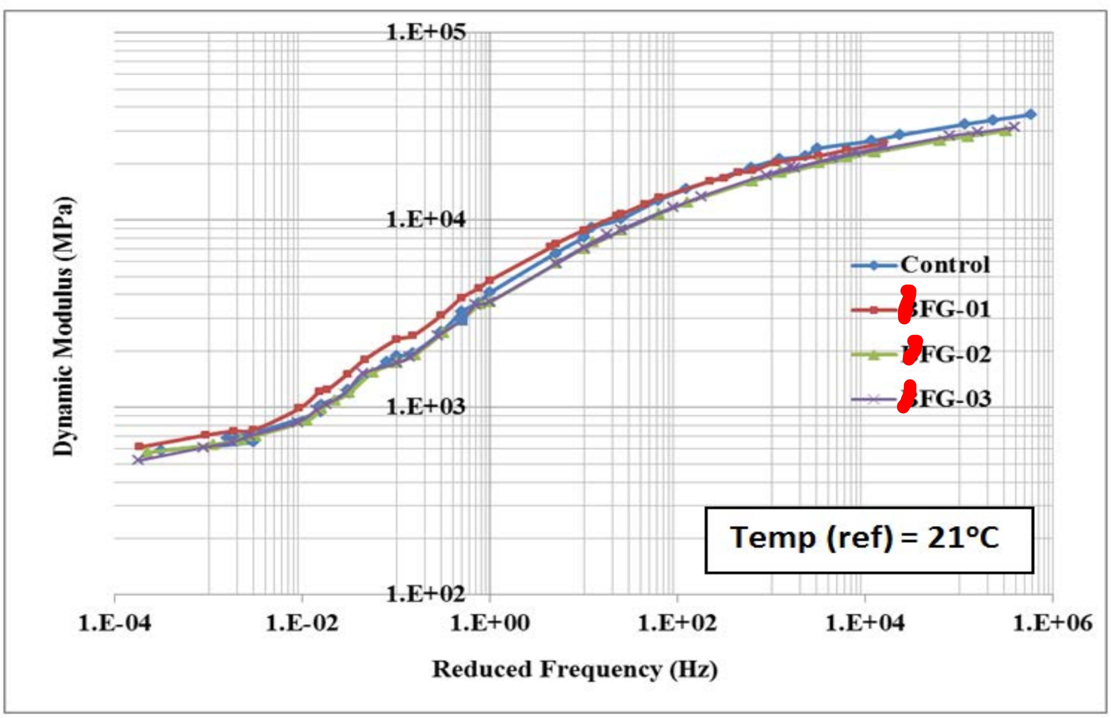



Figure C.3. Master curves of the mixtures.

Table C.1. Results of the Dynamic Modulus and Phase Angle for the Mixtures

| Temperature | | -10 ℃ | | | | 4 ℃ | | | | 21 ℃ | | | | 37 ℃ | | | | 53 ℃ | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Frequency | Mixture | [E*] (MPa) | | Phase Angle (Degree) | | [E*] (MPa) | | Phase Angle (Degree) | | [E*] (MPa) | | Phase Angle (Degree) | | [E*] (MPa) | | Phase Angle (Degree) | | [E*] (MPa) | | Phase Angle (Degree) | |
| (Hz) | | Avg. | CV | Avg, | CV | Avg. | CV | Avg, | CV | Avg. | CV | Avg, | CV | Avg. | CV | Avg, | CV | Avg. | CV | Avg, | CV |
| **25.0** | Control | 36,500 | 8.7 | 6.6 | 15.7 | 24,047 | 6.9 | 11.5 | 6.0 | 10,152 | 2.2 | 51.7 | 5.6 | 3,635 | 6.5 | 30.0 | 0.6 | 1,747 | 4.2 | 26.9 | 5.3 |
| | FG01 | 25,712 | 15.1 | 8.3 | 13.8 | 20,320 | 7.5 | 12.1 | 1.9 | 10,747 | 1.7 | 56.7 | 5.1 | 4,327 | 6.8 | 28.6 | 3.0 | 1,804 | 9.8 | 26.3 | 3.8 |
| | FG02 | 29,970 | 21.5 | 6.0 | 22.3 | 20,227 | 15.5 | 11.4 | 6.0 | 8,826 | 3.5 | 53.3 | 10.8 | 3,575 | 1.6 | 29.1 | 3.0 | 1,535 | 3.6 | 26.1 | 1.3 |
| | FG03 | 31,286 | 12.7 | 5.6 | 6.8 | 21,523 | 0.6 | 11.5 | 3.9 | 9,326 | 3.7 | 56.7 | 13.5 | 3,535 | 2.0 | 28.9 | 0.2 | 1,515 | 7.9 | 24.8 | 4.9 |
| **10.0** | Control | 34,185 | 8.9 | 7.5 | 6.4 | 21,084 | 8.6 | 13.0 | 8.8 | 8,093 | 1.8 | 55.0 | 9.1 | 2,521 | 6.1 | 29.6 | 0.4 | 1,210 | 5.3 | 24.5 | 4.3 |
| | FG01 | 23,631 | 14.6 | 9.3 | 11.9 | 17,960 | 7.5 | 13.6 | 1.9 | 8,848 | 3.0 | 50.0 | 0.0 | 3,105 | 8.8 | 28.9 | 2.5 | 1,246 | 7.7 | 24.2 | 0.7 |
| | FG02 | 27,983 | 22.3 | 6.9 | 20.3 | 17,871 | 15.2 | 12.8 | 3.2 | 7,062 | 3.7 | 53.3 | 5.4 | 2,500 | 1.9 | 29.3 | 1.6 | 1,085 | 4.8 | 23.2 | 0.7 |
| | FG03 | 29,358 | 12.3 | 6.4 | 2.6 | 19,160 | 1.1 | 12.8 | 3.4 | 7,366 | 4.4 | 55.0 | 9.1 | 2,422 | 2.0 | 36.0 | 33.9 | 1,038 | 6.2 | 22.6 | 3.1 |
| **5.0** | Control | 32,463 | 9.3 | 8.2 | 6.1 | 19,105 | 9.1 | 14.2 | 9.0 | 6,658 | 2.0 | 55.0 | 9.1 | 1,958 | 4.8 | 28.5 | 1.3 | 950 | 6.1 | 22.3 | 2.7 |
| | FG01 | 21,992 | 15.1 | 10.0 | 12.3 | 16,156 | 7.4 | 14.8 | 1.7 | 7,460 | 3.0 | 51.7 | 5.6 | 2,423 | 9.3 | 28.4 | 1.5 | 988 | 6.0 | 22.0 | 1.7 |
| | FG02 | 26,640 | 23.2 | 8.0 | 14.9 | 16,183 | 15.4 | 13.9 | 2.9 | 5,862 | 3.3 | 56.7 | 5.1 | 1,909 | 2.9 | 29.0 | 1.7 | 851 | 4.2 | 21.2 | 0.5 |
| | FG03 | 28,047 | 12.3 | 7.2 | 3.0 | 17,399 | 1.3 | 14.0 | 3.2 | 6,079 | 4.0 | 60.0 | 0.0 | 1,858 | 1.7 | 35.4 | 34.4 | 833 | 4.1 | 20.5 | 1.5 |
| **1.0** | Control | 28,376 | 10.0 | 10.3 | 6.4 | 14,586 | 11.2 | 18.1 | 10.0 | 4,096 | 2.0 | 73.3 | 7.9 | 1,245 | 4.0 | 23.5 | 2.3 | 731 | 6.8 | 14.3 | 13.9 |
| | FG01 | 18,275 | 15.9 | 12.5 | 13.1 | 12,191 | 7.6 | 18.6 | 1.9 | 4,768 | 2.1 | 63.3 | 4.6 | 1,509 | 11.0 | 25.0 | 1.0 | 748 | 5.8 | 15.8 | 3.0 |
| | FG02 | 23,111 | 25.7 | 10.0 | 14.2 | 12,377 | 16.3 | 17.5 | 2.2 | 3,660 | 2.8 | 65.0 | 7.7 | 1,194 | 2.7 | 24.6 | 1.3 | 670 | 2.9 | 14.8 | 5.6 |
| | FG03 | 27,356 | 5.7 | 9.0 | 2.2 | 13,353 | 1.1 | 17.6 | 3.2 | 3,780 | 1.8 | 71.7 | 4.0 | 1,184 | 1.4 | 31.8 | 42.9 | 652 | 4.6 | 14.6 | 5.6 |
| **0.5** | Control | 26,426 | 10.6 | 11.4 | 8.2 | 12,715 | 12.5 | 19.9 | 10.2 | 3,235 | 1.8 | 85.0 | 5.9 | 1,019 | 4.1 | 21.6 | 2.6 | 684 | 4.4 | 12.9 | 5.2 |
| | FG01 | 16,687 | 15.6 | 13.6 | 12.4 | 10,528 | 7.5 | 20.3 | 1.6 | 3,854 | 1.8 | 68.3 | 4.2 | 1,211 | 9.8 | 23.3 | 1.5 | 711 | 7.7 | 13.7 | 2.3 |
| | FG02 | 21,611 | 26.9 | 10.9 | 15.1 | 10,821 | 16.8 | 19.1 | 1.8 | 2,937 | 2.1 | 71.7 | 4.0 | 978 | 2.7 | 22.6 | 2.6 | 634 | 5.2 | 12.8 | 8.9 |
| | FG03 | 23,076 | 13.4 | 9.8 | 2.1 | 11,692 | 0.8 | 19.2 | 2.6 | 3,015 | 1.6 | 78.3 | 3.7 | 971 | 1.7 | 29.3 | 42.4 | 611 | 4.6 | 12.5 | 9.6 |
| **0.1** | Control | 21,921 | 12.4 | 15.0 | 11.1 | 9,074 | 15.4 | 24.2 | 9.7 | 1,874 | 2.1 | 61.7 | 4.7 | 654 | 1.2 | 18.2 | 3.8 | 590 | 4.0 | 9.4 | 8.7 |
| | FG01 | 13,134 | 14.7 | 17.1 | 13.1 | 7,160 | 7.1 | 24.8 | 1.8 | 2,295 | 0.4 | 51.7 | 5.6 | 758 | 10.3 | 19.6 | 3.0 | 614 | 4.5 | 10.3 | 4.6 |
| | FG02 | 18,078 | 31.2 | 13.7 | 20.7 | 7,637 | 19.9 | 23.2 | 3.2 | 1,726 | 1.5 | 56.7 | 10.2 | 705 | 5.4 | 17.6 | 1.5 | 574 | 2.4 | 10.1 | 12.2 |
| | FG03 | 19,201 | 16.8 | 12.4 | 3.5 | 8,375 | 0.8 | 23.2 | 2.3 | 1,782 | 1.4 | 60.0 | 15.0 | 706 | 0.2 | 25.7 | 56.7 | 520 | 3.2 | 9.7 | 6.2 |

# APPENDIX D ADDITIONAL DETAILS OF ATLAS CONSTRUCTION AND TESTING

The Advanced Transportation Research and Engineering Laboratory (ATREL) at University of Illinois Urbana-Champaign (UIUC) is home to the Accelerated Transportation Loading Assembly System (ATLAS) as shown in Figure D.1. ATLAS transmits load to the pavement structure through a hydraulic ram attached to the wheel carriage. Additionally, a movable structure was used to protect the ATLAS from the environment to minimize environmental effects on the pavement. The maximum loading length of the ATLAS device is 85 ft, with approximately 65 ft of travel available for testing under constant velocity.

Figure D.1. Accelerated transportation loading assembly system (ATLAS).

ATLAS testing was conducted at constant temperature of 90°F, as controlled through the use of heating elements and a closed loop pavement temperature monitoring and control system. Figure D.2. shows the testing configuration used, where wheel tracks were located at 4 ft (1.3 m) from the edge of pavement.

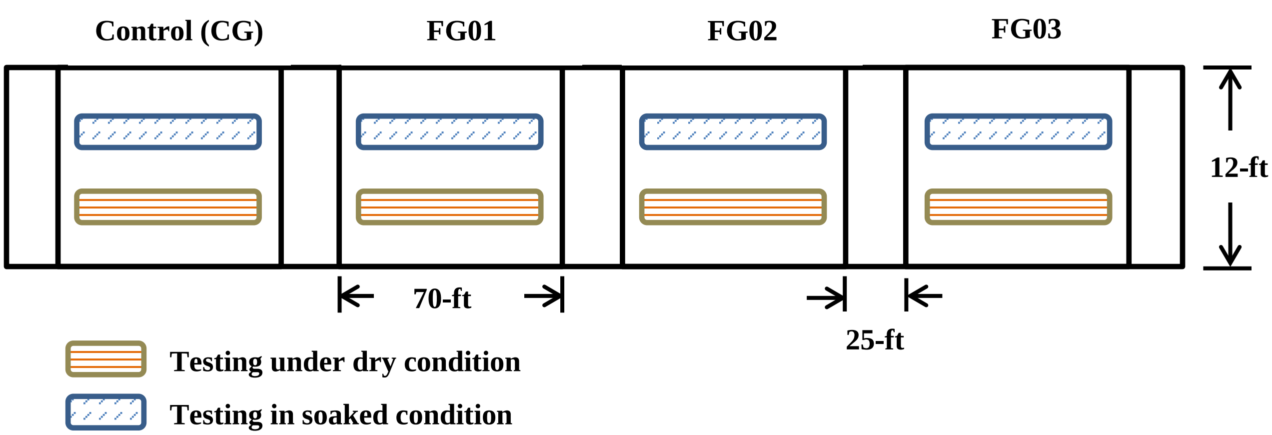
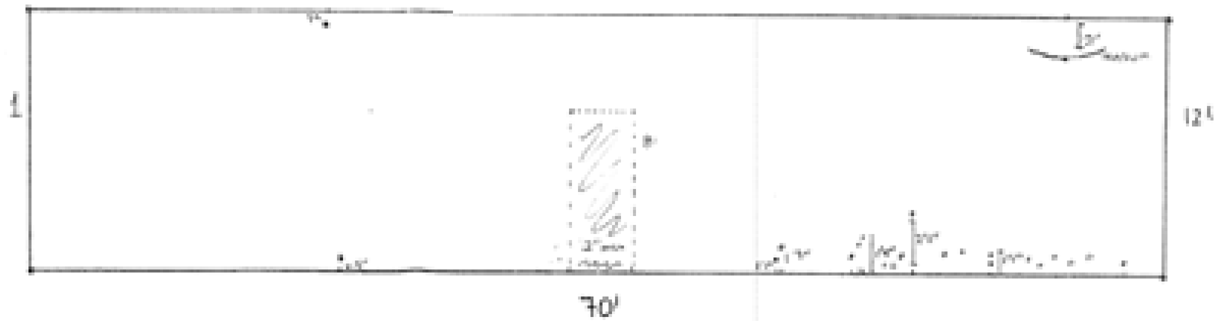
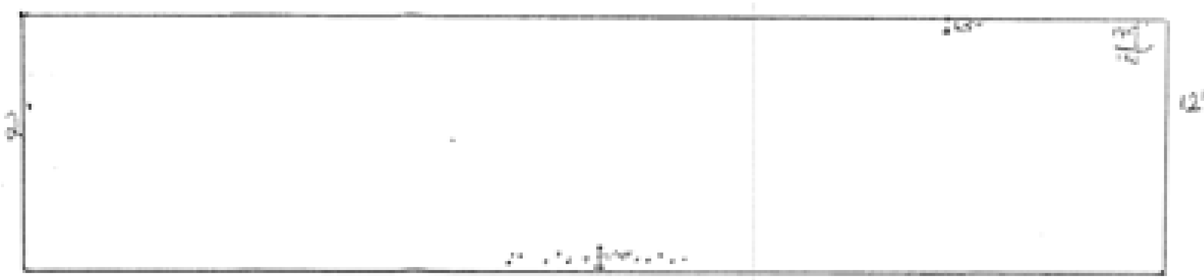



Figure D.2. Loading configuration for ATLAS testing.

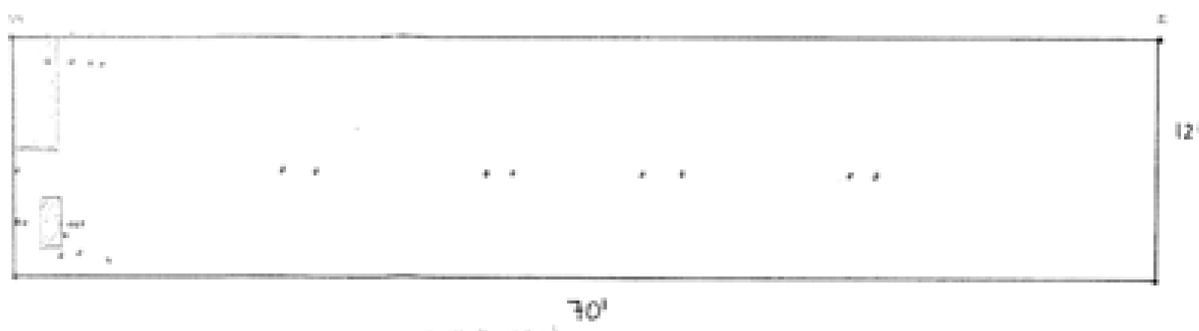
**Control Mix Section**




**Fine-graded Mix #1**




**Fine-graded Mix #2**




**Fine-graded Mix #3**

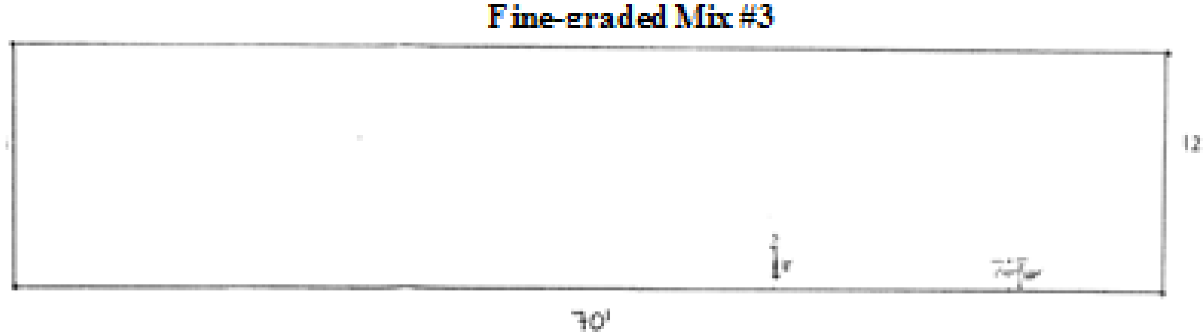



Figure D.3.Typical pre-overlay condition maps of distresses on existing CRC pavement.

After consultation with the project Technical Review Panel, the coarse-graded control mix and three fine-graded mixtures were placed as overlays on an existing continuously reinforced concrete pavement (CRC) at ATREL. Before construction, the CRC pavement was surveyed to identify existing distress types and locations. However, because the CRC pavement was edge-loaded, most distress areas were outside of the areas to be loaded as part of this study. The survey was done after cold-milling the existing pavement. Results are shown in Figure D.3.

### Construction of Overlays

All four mixtures were produced and hauled to ATREL by Open Road Paving, LLC. The thickness of the HMA overlays used throughout was 2.25 in (57 mm). A lane width of 12 ft (4 m) was constructed. Transition areas were allotted between sections for a minimum of 25 ft (8 m) as displayed in Figure D.2. The construction was performed on June 16 and 17, 2011. Key construction details include:

1. Cold-milling: June 16, the existing pavement was milled and swept as shown in Figure D.4 to remove the existing asphalt overlay on the CRCP.

Figure D.4. Milling existing pavement.

2. Tack coat application: after the milling process was completed, SS-1hp tack coat type was applied on the milled surface as shown in Figure D.5. Table D.1 presents the measured residual rate of the tack coat applied on each test section. Small, 12 in (308 mm), square pieces of geotextile fabric were placed in each test section to measure the as-placed tack coat rate.

Figure D.5. Application of tack coat.

Table D.1. Measured In-Place Tack Coat Application Rate

| Mix ID | Replicate | Wt. of Fabric Sheet (g) | | Weight of Tack Coat (g) | Residual rate (gal/yd$^2$) |
|---|---|---|---|---|---|
| | | Before | After | | |
| Control | 1 | 58.4 | 71.3 | 12.9 | 0.03 |
| | 2 | 48.1 | 67.5 | 19.4 | 0.05 |
| FG01 | 1 | 36.5 | 56.9 | 20.4 | 0.05 |
| | 2 | 32.9 | 51.2 | 18.3 | 0.04 |
| FG02 | 1 | 71.9 | 89.9 | 18.0 | 0.04 |
| | 2 | 63.2 | 81.7 | 18.5 | 0.04 |
| FG03 | 1 | 63.5 | 84.3 | 20.8 | 0.05 |
| | 2 | 44.0 | 63.4 | 19.4 | 0.05 |

3. Paving: June 17, the control mix was paved after 24-hour curing of the tack coat (Figure D.6).

Figure D.6. Construction train (left) and paver-placed mixture (right).

4. Compacting and finishing: both static and vibratory steel rollers were used to compact paved mixtures as shown in Figure D.7. Finally, finish rolling was conducted.

Figure D.7. Vibratory steel drum roller (left) and finish roller (right).

5. Density verification: a nuclear gauge as shown in Figure D.8 was used to check density of the paved sections. Five readings were taken across the mat, with results shown in Table D.2.

Figure D.8. Nuclear density test (left) and temperature indicator (right).

Table D.2. Non-Correlated Nuclear Density Data Collection

| Mixture | $G_{mm}$ | Non-Correlated Nuclear Density Readings ($kg/m^3$)–Average of 5 Readings Across the Mat | | | | | Percent $G_{mm}$ |
|---|---|---|---|---|---|---|---|
| | | 1 | 2 | 3 | 4 | 5 | (Average) |
| Control | 2.502 | 2,343 | 2,350 | 2,314 | 2,327 | 2,346 | 93.4 |
| FG01 | 2.502 | 2,333 | 2,372 | 2,331 | 2,356 | 2,363 | 94.0 |
| FG02 | 2.499 | 2,364 | 2,384 | 2,283 | 2,326 | 2,321 | 93.5 |
| FG03 | 2.500 | 2,351 | 2,337 | 2,269 | 2,307 | 2,342 | 92.8 |

**ATLAS Testing**

The ATLAS device was used to evaluate rutting performance of the F-G mixes relative to the control C-G mix under accelerated loading conditions. Figure D.9 presents a typical ATLAS testing configuration. Test parameters used were given in the main body of the report.

Figure D.9. Dual-tire assembly loading applied 4 ft (1.3 m) from pavement edge.

Rutting profile data were collected every 15,000 passes using an automated laser profiler as shown in Figure D.10. Sample rutting profiles are presented in Figures D.11 to D.13.

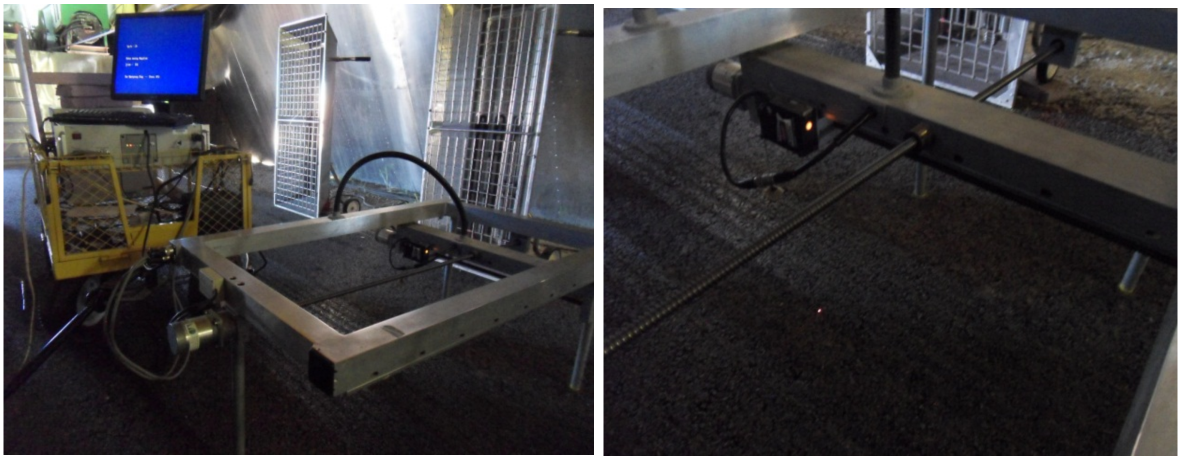


Figure D.10. Rutting measurement using laser technique.

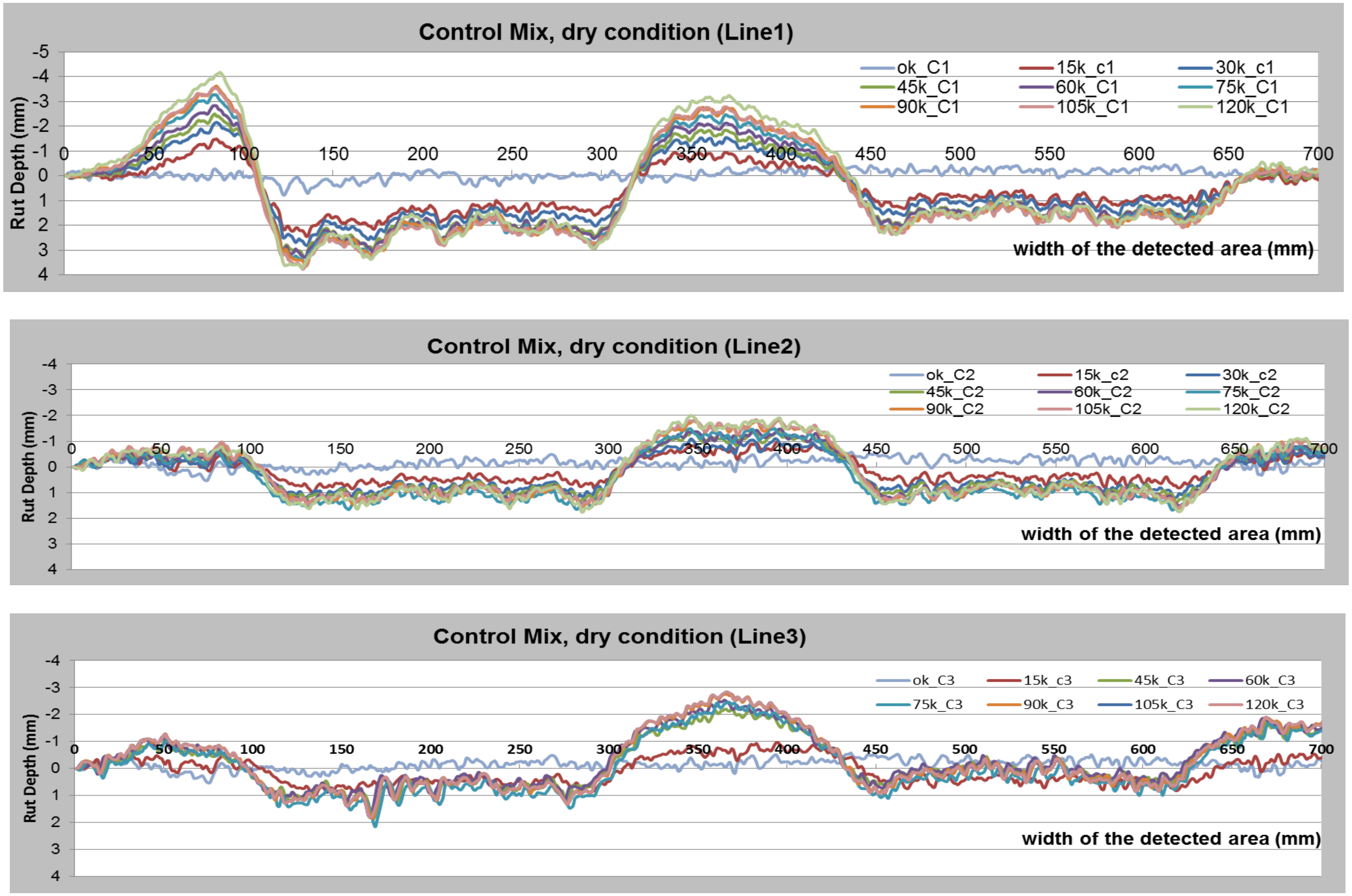



Figure D.11. Rutting profiles of the control mix (dry condition), measured every 15,000 passes (from 0 to 120,000 passes).

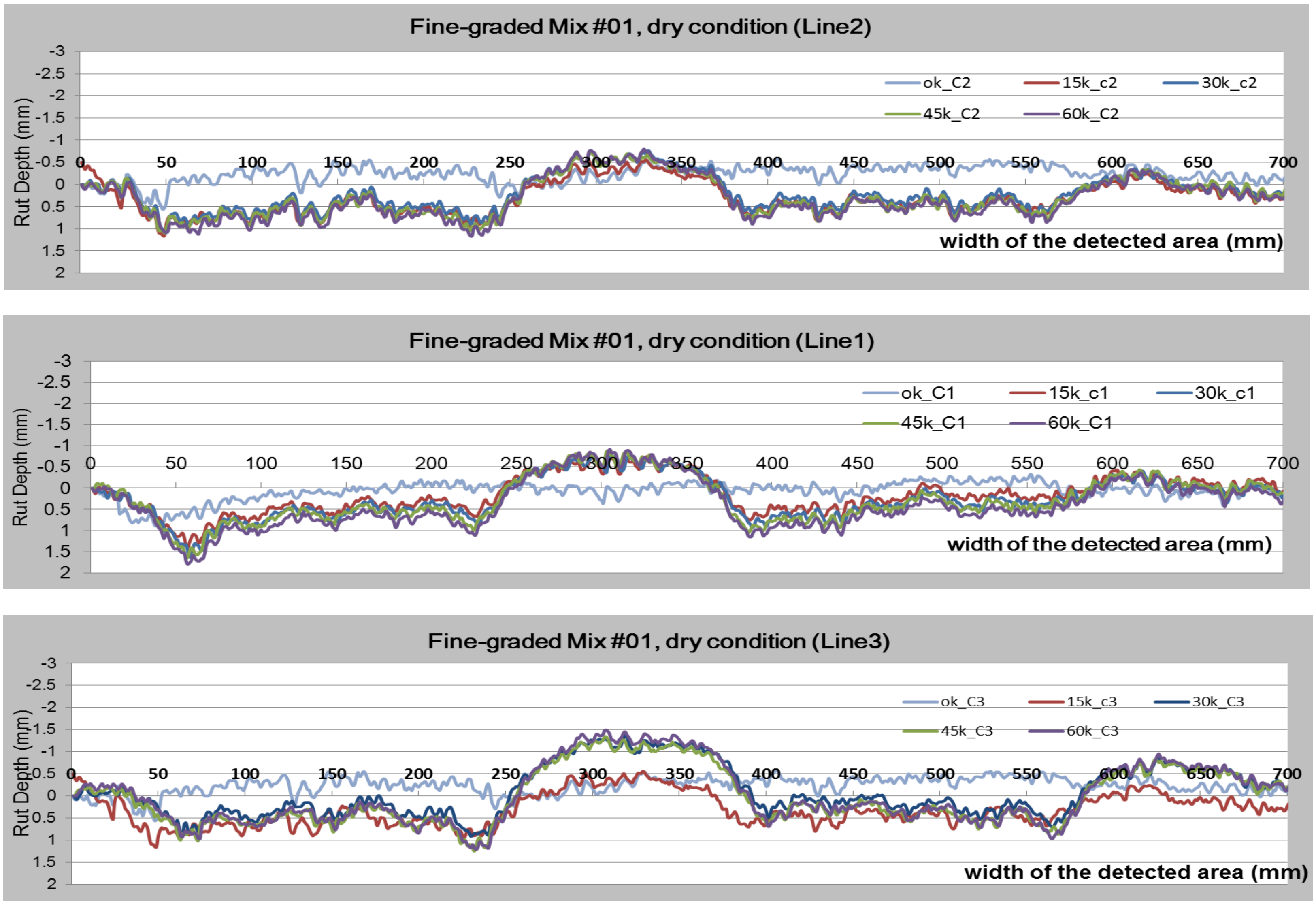



Figure D.12. Rutting profiles of the fine-graded #01 (dry condition), measured every 15,000 passes (from 0 to 60,000 passes).

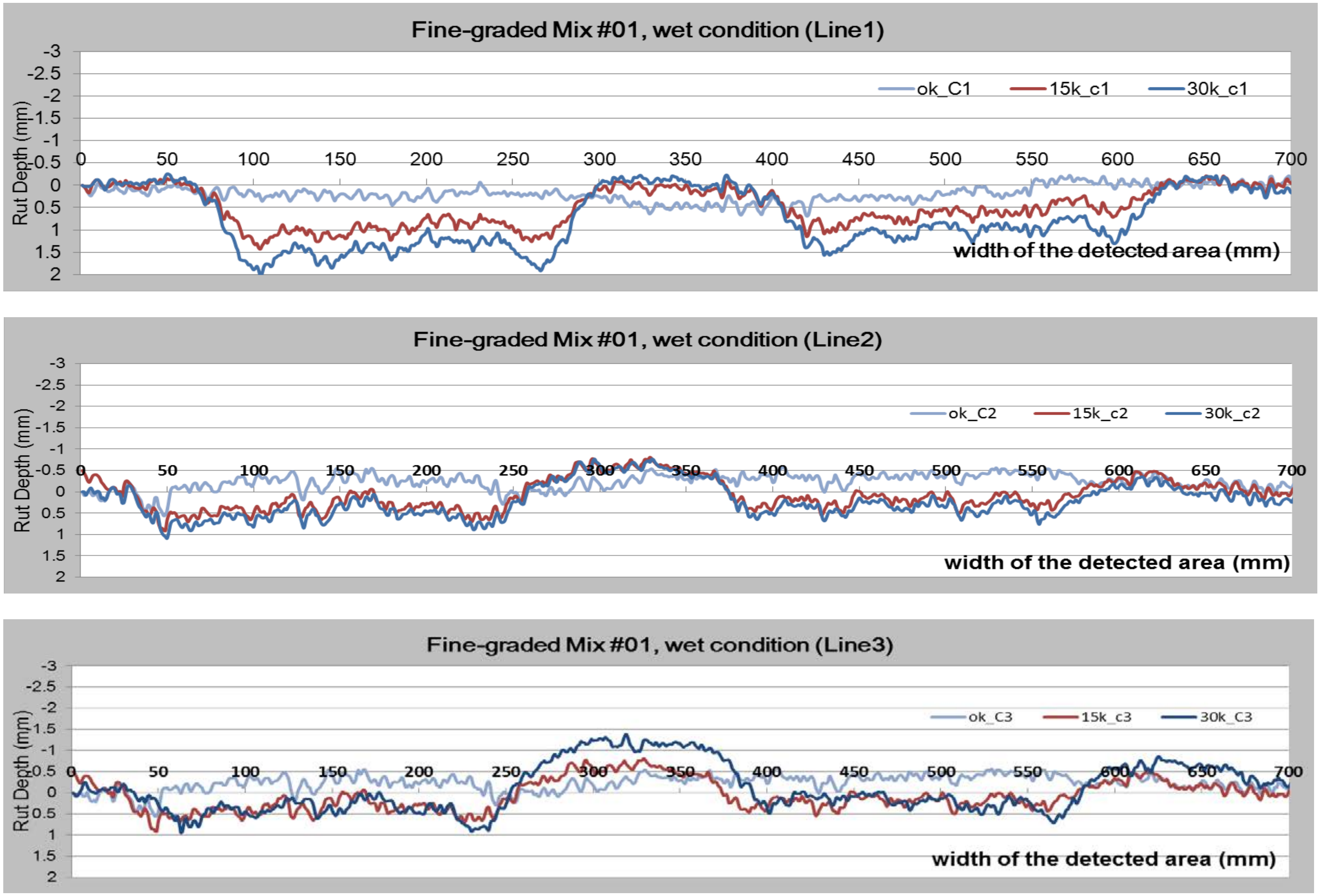



Figure D.13. Rutting Profiles of the fine-graded #01 (wet condition), measured every 15,000 passes (from 0 to 30,000 passes).

# APPENDIX E FIELD PERMEABILITY TESTING

## Introduction

This appendix summarizes water permeability testing conducted on fine and coarse-graded binder course paved on Interstate 57 in District 5 near Pesotum during the summer of 2010 by researchers at the University of Illinois at Urbana-Champaign. IDOT's BMPR falling head permeameter apparatus was used in the testing. Testing was conducted on July 13, July 14, and August 30, 2010, prior to the start of this research project.

## Background

Deterioration within pavements is caused by several factors. Two of these factors are water and air penetrating into the pavement, resulting in an increased potential for water damage, oxidation, and cracking. Traditional coarse-graded Superpave mixture gradations generally have interconnected voids, which allows water to penetrate the pavement structure resulting in stripping (the breaking of the adhesive bond between the aggregate surface and asphalt binder). As a result, one of the expectations of introducing fine-graded mixtures is to reduce the size and amount of interconnected voids to decrease the rate of water infiltration. As permeability is reduced, the life of a pavement would be expected to be longer (i.e., fine versus coarse-graded mixtures).

Permeability is the flow rate of water through a material. The permeability of HMA pavements is affected by several factors. The permeability is dependent upon the size of air voids within a pavement (Hudson and Davis, 1965). Furthermore, different gradations and particle shapes can result in a different rate of permeability (Ford and Mc Williams, 1988).

In order to estimate water permeability in a pavement, the coefficient of permeability (k) in the Darcy's law equation, the fundamental theory of permeability for soils, was used as shown in the following equation. However, several assumptions are made in order for this equation to be applicable: (1) a homogenous material; (2) steady state flow conditions; (3) laminar flow; (4) incompressible fluid; (5) saturated material; and (6) one-dimensional flow.

$$k = \frac{aL}{At} In(\frac{h_1}{h_2})$$

where:

k = coefficient of Permeability;

a = area of stand pipe;

L= length of sample;

A = cross-sectional area of sample;

t = time over which head is allowed to fall;

$h_1$ = water head at beginning of test; and

$h_{2 =}$ water head at end of test.

## Field Permeability Test

Much of the previous permeability testing reported in the literature has been completed using cores cut from the field and tested in the laboratory. Permeability measured by this method results in one-dimensional flow in correspondence with Darcy's law. The measurement of field permeability is much more difficult to attain, as water will not flow in one direction when introduced locally in a pavement during a field permeability test. In addition, it is difficult to control the degree of saturation and precise flow conditions in the field. Research as reported by Allen (1999) has shown that a device and method for in-place permeability testing can be used to accurately measure permeability through non-destructive, in-situ testing. Allen's field permeameter was shown to provide similar trends as the laboratory permeameter, and was also shown to be repeatable and easy to use.

Based on an extensive literature review, a falling head permeability test is the most appropriate test for an in-place permeability measurement. This test method provides an estimate of the water permeability of an HMA pavement. The test is based upon the assumption that the pavement thickness is equal to the immediate underlying HMA lift thickness; the area of the tested sample is equal to the area of the permeameter from which water is allowed to penetrate the HMA pavement; one-dimensional flow exists; laminar flow of the water is maintained; and that Darcy's law is valid. Figure E.1 shows IDOT's falling head permeability equipment which was used in this project.

Figure E.1. A falling head permeability test.

Figure E.2 presents a cross-sectional schematic of a location where the test was completed, for the purpose of illustrating the transverse offsets selected. Five different locations at each station were tested; 2 in (50.8 mm), 6 in (152.4 mm), 3 ft (0.92 m), 6 ft (1.84 m), and 11 ft (3.35 m) from the centerline of the pavement.

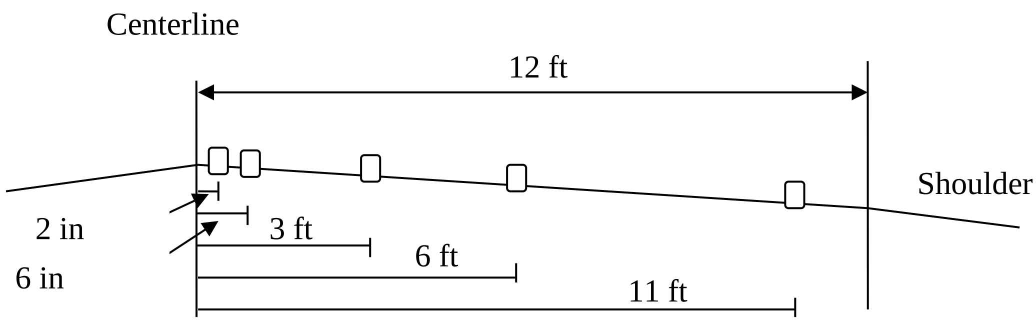



Figure E.2. Transverse measurement locations.

Two different types of HMA mixtures were produced and constructed on this project: coarse- and fine-graded binder course mixes. Several consecutive stations in both coarse and fine-graded pavement sections were tested to obtain the permeability data. Stations were spaced approximately 0.5 miles (0.8 km) miles apart.

## Results

Results of the field permeability testing are presented in Table E.1, while averaged results are presented in Figure E.3.

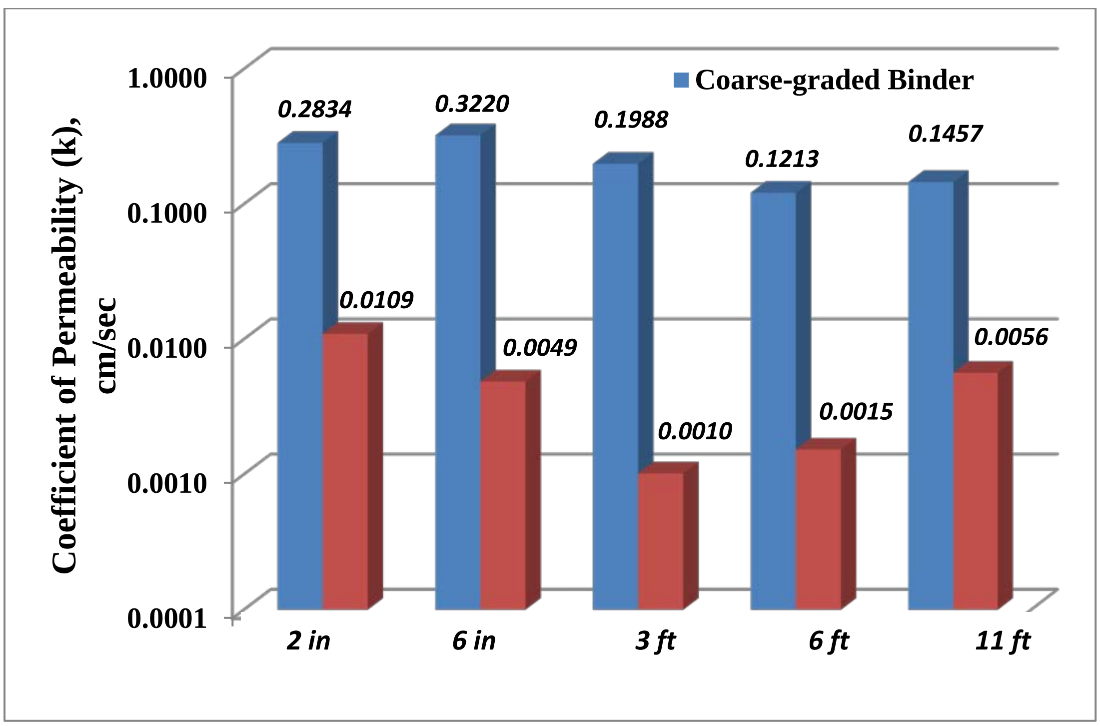



Figure E.3. Chart showing average permeability at all test locations.

Based upon the results presented in Figure E.3, the permeability values of both coarse and fine-graded mixtures were found to be significantly different. ***Average permeability values of the coarse-graded pavement were approximately 25 times higher than that of the fine-graded pavement at each location.*** This indicates that the fine-graded pavement will be much more resistant to water infiltration as compared to the coarse-graded mix.

Table E.2. Results of Field Permeability Testing for Each Station on I-57

| Type of Mix | | STATION | Coefficient of Permeability (k) - cm/s | | | | | STD | Mean | CoV. | Note |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Distance from Center Line (ft) | | | | | | | | |
| | | | 2" | 6" | 3' | 6' | 11' | | | | |
| coarse-graded binder | 1 | 121+00 | 0.1917 | 0.1709 | 0.1372 | 0.1158 | 0.1670 | 0.0299 | 0.1565 | 19.1 | Northbound |
| | 2 | 125+00 | 0.1146 | 0.1740 | 0.0729 | 0.0193 | 0.1198 | 0.0577 | 0.1001 | 57.6 | |
| | 3 | 130+00 | 0.1353 | 0.2878 | 0.0595 | 0.0727 | 0.0658 | 0.0963 | 0.1242 | 77.6 | |
| | 4 | 135+40 | 0.1420 | 0.1320 | 0.0350 | 0.0040 | 0.1297 | 0.0641 | 0.0885 | 72.4 | |
| | 5 | 140+00 | 0.3020 | 0.5189 | 0.1595 | 0.1494 | 0.1297 | 0.1642 | 0.2519 | 65.2 | |
| | 6 | 160+00 | 0.2307 | 0.1507 | 0.4964 | 0.1316 | 0.1094 | 0.1591 | 0.2237 | 71.1 | |
| | 7 | 165+00 | 0.3381 | 0.3138 | 0.1748 | 0.1646 | 0.2711 | 0.0794 | 0.2525 | 31.4 | |
| | 8 | 170+00 | 0.5119 | 0.4646 | 0.2720 | 0.1460 | 0.1698 | 0.1678 | 0.3129 | 53.6 | |
| | 9 | 197+40 | 0.4926 | 0.4707 | 0.4730 | 0.1981 | 0.2950 | 0.1320 | 0.3859 | 34.2 | |
| | 10 | 210+00 | 0.2490 | 0.3365 | 0.2166 | 0.1647 | 0.2454 | 0.0625 | 0.2424 | 25.8 | |
| | 11 | 238+40 | 0.1501 | 0.2500 | 0.2356 | 0.1219 | 0.0685 | 0.0768 | 0.1652 | 46.5 | |
| | 12 | 255+80 | 0.3181 | 0.7652 | 0.2714 | 0.3228 | 0.1031 | 0.2455 | 0.3561 | 68.9 | |
| | 13 | 265+70 | 0.5514 | 0.2482 | 0.1537 | 0.0530 | 0.1047 | 0.1976 | 0.2222 | 88.9 | |
| | 14 | 311+00 | 0.2408 | 0.2247 | 0.0257 | 0.0346 | 0.0604 | 0.1064 | 0.1172 | 90.7 | |
| Average | | | 0.2834 | 0.3220 | 0.1988 | 0.1213 | 0.1457 | | | | |
| Fine-graded binder | 1 | 757+50 | 0.0015 | 0.0016 | 0.0011 | 0.0018 | 0.0017 | 0.0003 | 0.0015 | 17.4 | Southbound |
| | 2 | 730+00 | 0.0016 | 0.0010 | 0.0015 | 0.0018 | 0.0012 | 0.0003 | 0.0014 | 23.2 | |
| | 3 | 685+00 | 0.0484 | 0.0206 | 0.0021 | 0.0019 | 0.0244 | 0.0192 | 0.0195 | 98.5 | |
| | 4 | 675+50 | 0.0020 | 0.0006 | 0.0003 | 0.0019 | 0.0008 | 0.0008 | 0.0011 | 69.9 | |
| | 5 | 652+00 | 0.0012 | 0.0004 | 0.0001 | 0.0002 | 0.0002 | 0.0005 | 0.0004 | 106.0 | |
| Average | | | 0.0109 | 0.0049 | 0.0010 | 0.0015 | 0.0056 | | | | |

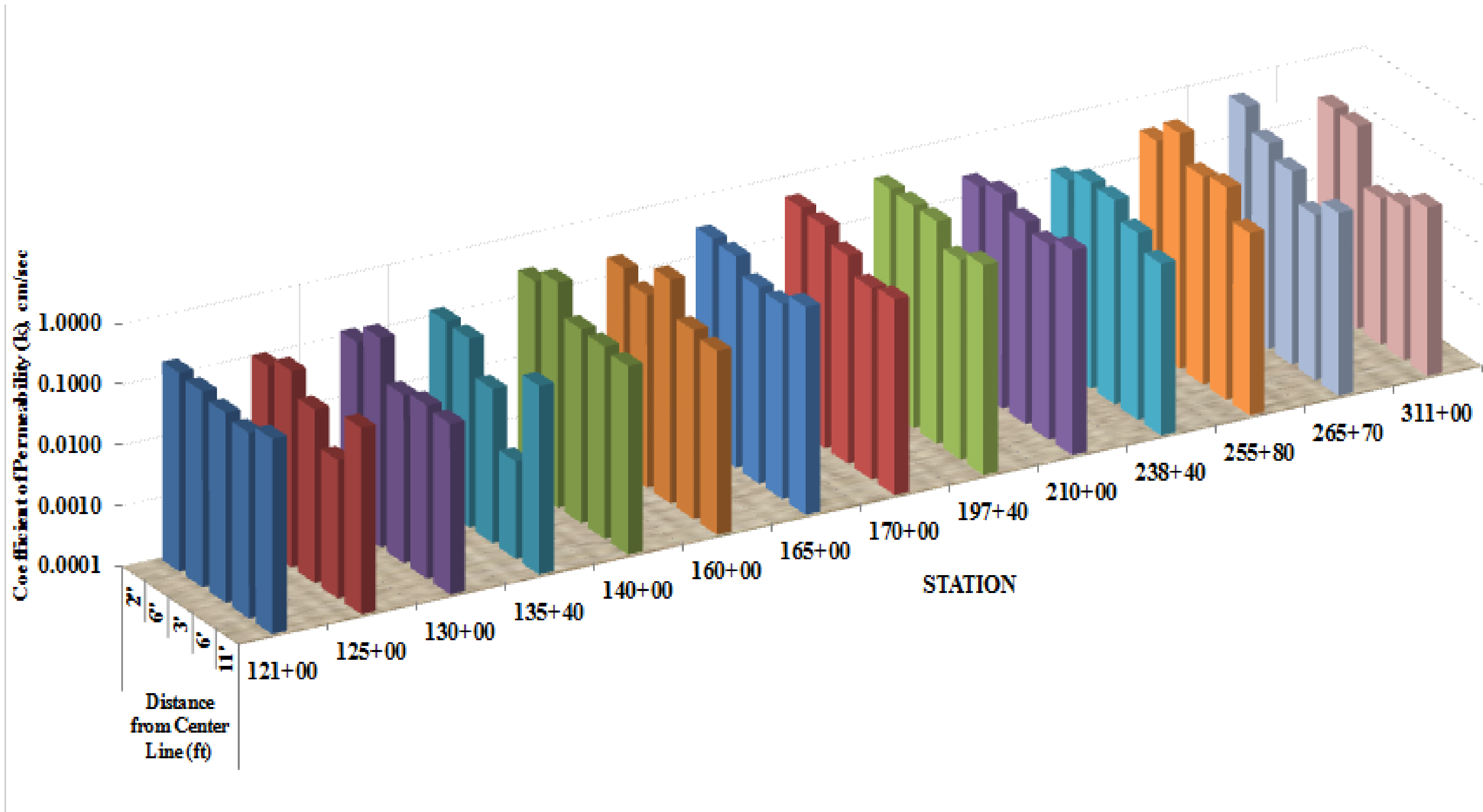



Figure E.4. Permeability: Coarse-graded mix.

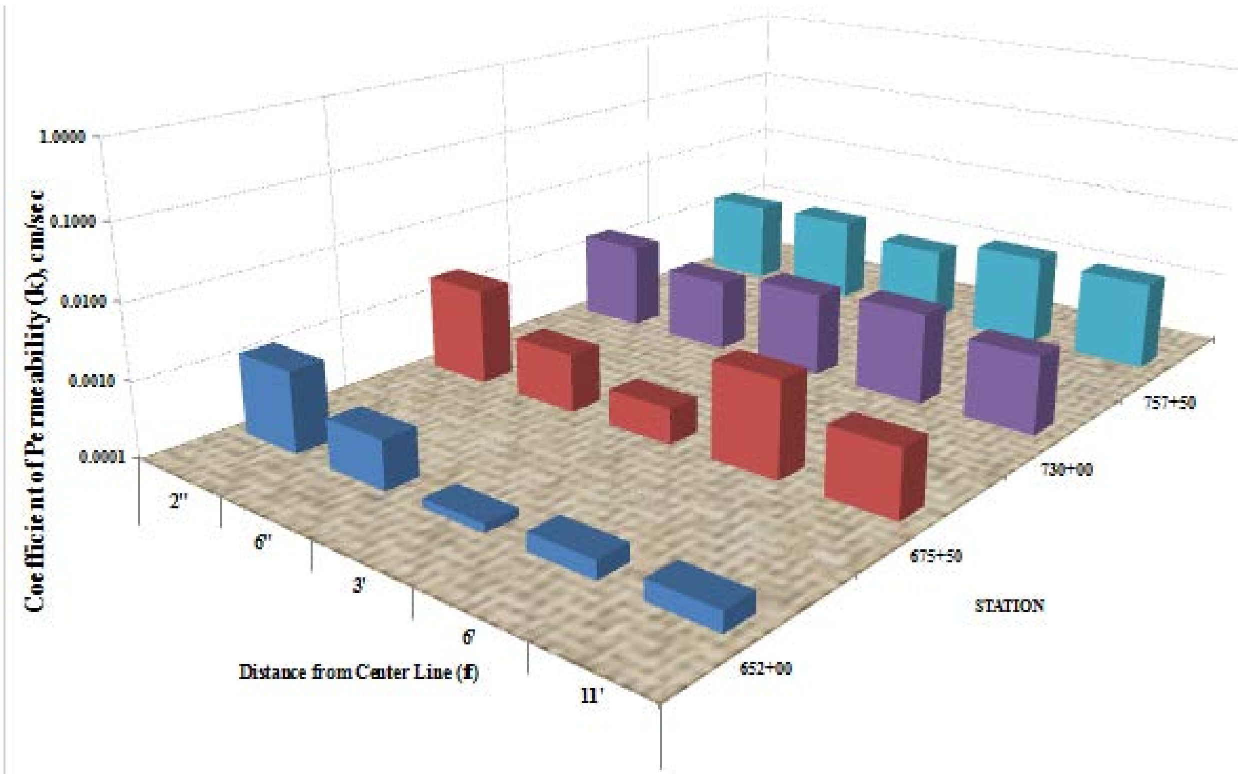



Figure E.5. Permeability: Fine-graded mix (excluding test location located adjacent to overpass).

Figure E.4 and E.5 present permeability values obtained at every test station for both coarse and fine-graded mixtures, respectively. Testing locations closest to the edge of the pavement generally had higher permeability values than those in the middle of the lane. This is not surprising, as it is well known that the middle of the paving lane typically receives greater passes and better confinement and therefore more compaction effort as compared to the pavement edges. Note that the confinement characteristics of the pavement varied across the project. In some cases, both lane edges were confined (inlay), while in other locations, only the left edge of the pavement was confined. Confinement was found to have a significant effect on permeability, and was a significant factor in permeability variation across the project.

**Summary**

This brief section presented field permeability testing conducted on fine and coarse-graded binder course overlay lifts placed on I-57 in IDOT District 5 during the summer of 2010. Based upon the test results collected, there is a significant difference in the water permeability of fine- and coarse-graded mixtures used on this project. The fine-graded mixture had a much lower permeability than the coarse-graded mixture (25 times lower, on average). Although promising, more research is needed to fully evaluate the feasibility of fine-graded binder and surface course mixtures for use on Illinois highways.

## References

Hudson, S.B. and Davis, “Relationship of Aggregate Voidage to Gradation.” Association of Asphalt Paving Technologists, Volumes 34, 1965.

Ford, M.C. and C.E. McWilliams, “Asphalt Mix Permeability,” University of Arkansas, Fayetteville, Arkansas, 1988.

Olson, R.E. and D.E. Daniel, “Measurement of the Hydraulic Conductivity of Fine-Grained Soils,” Permeability and Groundwater Containment Transport, ASTM STP 746, T.F. Zimmie and C.O. Riggs, Eds., American Society for Testing and Materials, 1981.

Allen, L. “Permeability of Superpave Mixtures: Evaluation of Field Permeameters” NCAT Report No.99-1, February, 1999.

A Draft Specification, “Field Estimation of Water Permeability of Compacted Asphalt Paving Mixtures Field Permeameter.”

# APPENDIX F PROPOSED CHANGES TO HOT MIX ASPHALT MIXTURE DESIGN COMPOSITION AND VOLUMETRIC REQUIREMENTS TO ACCOMMODATE FINE-GRADED BINDER COURSE

**Proposed Changes to the Standard Specifications for Road and Bridge Construction**

Revise table and second paragraph in Article 406.05(c) of the Standard Specifications to read.

| “Leveling Binder | |
|---|---|
| Nominal, Compacted, Leveling Binder Thickness, in (mm) | Mixture Composition |
| ≤ 1 1/4 (32) | IL-9.5 or IL-9.5L |
| 1 1/4 to 2 (32 to 50) | IL-9.5, or IL-9.5L |

Density requirements of Article 406.07(c) shall apply for leveling binder, machine method, when the nominal, compacted thickness is 1 1/4 in (32 mm) or greater for IL-9.5 and IL-9.5L mixtures.”

Revise Article 406.14(b) of the Standard Specifications to read.

“(b) If the HMA placed during the initial test strip (1) is determined to be unacceptable to remain in place by the Engineer, and (2) was not produced within 2.0 to 6.0 percent air voids or within the individual control limits of the JMF, the mixture and test strip will not be paid for and the mixture shall be removed at the Contractor’s expense. An additional test strip and mixture will be paid for in full, if produced within 2.0 to 6.0 percent air voids and within the individual control limits of the JMF.”

Revise Article 406.14(c) of the Standard Specifications to read.

“(c) If the HMA placed during the initial test strip (1) is determined to be unacceptable to remain in place by the Engineer, and (2) was produced within 2.0 to 6.0 percent air voids and within the individual control limits of the JMF, the mixture shall be removed. Removal will be paid in accordance to Article 109.04 of the Standard Specifications. This initial mixture and test strip will be paid for at the contract unit prices. The additional mixture will be paid for at the contract unit price, and any additional test strips will be paid for at one half the unit price of each test strip.”

Revise Article 1003.03(c) of the Standard Specifications to read.

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

For mixture IL-19.0, Ndesign = 90 the fine aggregate fraction shall consist of at least 67 percent manufactured sand meeting FA 20 gradation. For mixture IL-19.0, Ndesign = 50 or 70 the fine aggregate fraction shall consist of at least 50 percent manufactured sand meeting FA 20 gradation. The manufactured sand shall be stone sand, slag sand, steel slag sand, or combinations thereof."

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

Revise table in Article 1030.01 of the Standard Specifications to read.

| | |
|---|---|
| "High ESAL | IL-25.0 binder; IL-19.0 binder; IL-12.5; IL-9.5 surface |
| Low ESAL | IL-19.0L binder; IL-9.5L surface |
| All Other | Stabilized Subbase (HMA), HMA Shoulders" |

Revise Article 1030.04(a)(1) of the Standard Specifications to read.

"(1) High ESAL Mixtures. The job mix formula (JMF) shall fall within the following limits.

| High ESAL, MIXTURE COMPOSITION (% PASSING) [1/] | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Sieve | IL-25.0 mm | | IL-19.0 mm | | IL-12.5 mm | | IL-9.5 mm | | IL-4.75 mm | |
| Size | min | max | min | max | min | max | min | max | min | max |
| 1 1/2 in (37.5 mm) | | 100 | | | | | | | | |
| 1 in (25 mm) | 90 | 100 | | 100 | | | | | | |
| 3/4 in (19 mm) | | 90 | 90 | 100 | | 100 | | | | |
| 1/2 in (12.5 mm) | 45 | 75 | 75 | 89 | 90 | 100 | | 100 | | 100 |
| 3/8 in (9.5 mm) | | | | | | 89 | 90 | 100 | | 100 |
| #4 (4.75 mm) | 24 | 42 [2/] | 40 | 60[/] | 28 | 65 | 32 | 69 | 90 | 100 |
| #8 (2.36 mm) | 16 | 31 | 26 | 42 | 28 | 48 [2/] | 32 | 52 [2/] | 70 | 90 |
| #16 (1.18 mm) | 10 | 22 | 15 | 30 | 10 | 32 | 10 | 32 | 50 | 65 |
| #50 (300 μm) | 4 | 12 | 6 | 15 | 4 | 15 | 4 | 15 | 15 | 30 |
| #100 (150 μm) | 3 | 9 | 4 | 9 | 3 | 10 | 3 | 10 | 10 | 18 |
| #200 (75 μm) | 3 | 6 | 3 | 6 | 4 | 6 | 4 | 6 | 7 | 9 |
| Ratio Dust/Asphalt Binder | | 1.0 | | 1.0 | | 1.0 | | 1.0 | | 1.0 [3/] |

1/ Based on percent of total aggregate weight.

2/ The mixture composition shall not exceed 44 percent passing the #8 (2.36 mm) sieve for surface courses with Ndesign = 90.

3/ Additional minus No. 200 (0.075 mm) material required by the mix design shall be mineral filler, unless otherwise approved by the Engineer."

Delete Article 1030.04(a)(4) of the Standard Specifications.

Revise the table in Article 1030.04(b)(1) of the Standard Specifications to read.

<table>
<tr><th colspan="7">“VOLUMETRIC REQUIREMENTS<br>High ESAL</th></tr>
<tr><th></th><th colspan="5">Voids in Mineral Aggregate<br>(VMA),<br>% minimum</th><th rowspan="2">Voids Filled with Asphalt Binder<br>(VFA),<br>%</th></tr>
<tr><th>Ndesign</th><th>IL-25.0</th><th>IL-19.0</th><th>IL-12.5</th><th>IL-9.5</th><th>IL-4.75 1/</th></tr>
<tr><td>50</td><td rowspan="4">12.0</td><td rowspan="4">13.5</td><td rowspan="4">14.0</td><td rowspan="4">15.0</td><td>18.5</td><td>65–78 2/</td></tr>
<tr><td>70</td><td rowspan="3"></td><td rowspan="3">65–75</td></tr>
<tr><td>90</td></tr>
<tr><td></td></tr>
</table>

1/ Maximum Draindown for IL-4.75 shall be 0.3%
2/ VFA for IL-4.75 shall be 76-83%”

Revise the table in Article 1030.04(b)(1) of the Standard Specifications to read.

| “VOLUMETRIC REQUIREMENTS Low ESAL | | | | |
|---|---|---|---|---|
| Mixture Compositio n | Design Compactiv e Effort | Design Air Voids Target % | VMA (Voids in Mineral Aggregate) , % min. | VFA (Voids Filled with Asphalt Binder), % |
| IL-9.5L | $N_{DES}$ =30 | 4.0 | 15.0 | 65-78 |
| IL-19.0L | $N_{DES}$ =30 | 4.0 | 13.5 | N/A” |

Delete Article 1030.04(b)(4) of the Standard Specifications.

Revise the Control Limits Table in Article 1030.05(d)(4) of the Standard Specifications to read.

| "CONTROL LIMITS | | | | | |
|---|---|---|---|---|---|
| Parameter | High ESAL Low ESAL<br><br>Individual Test | High ESAL Low ESAL<br><br>Moving Avg. of 4 | All Other<br><br>Individual Test | IL-4.75<br><br>Individual Test | IL-4.75<br><br>Moving Avg. of 4 |
| % Passing: [1/] | | | | | |
| 1/2 in (12.5 mm) | ± 6 % | ± 4 % | ± 15 % | | |
| No. 4 (4.75 mm) | ± 5 % | ± 4 % | ± 10 % | | |
| No. 8 (2.36 mm) | ± 5 % | ± 3 % | | | |
| No. 16 (1.18 mm) | | | | ± 4 % | ± 3 % |
| No. 30 (600 μm) | ± 4 % | ± 2.5 % | | | |
| Total Dust Content No. 200 (75 μm) | ± 1.5 % | ± 1.0 % | ± 2.5 % | ± 1.5 % | ± 1.0 % |
| Asphalt Binder Content | ± 0.3 % | ± 0.2 % | ± 0.5 % | ± 0.3 % | ± 0.2 % |
| Voids | ± 1.2 % | ± 1.0 % | ± 1.2 % | ± 1.2 % | ± 1.0 % |
| VMA | –0.7 % [2/] | –0.5 % [2/] | | –0.7 % [2/] | –0.5 % [2/] |

1/ Based on washed ignition oven
2/ Allowable limit below minimum design VMA requirement"

Revise the Density Control Limits Table in Article 1030.05(d)(4) of the Standard Specifications to read.

| "DENSITY CONTROL LIMITS | | |
|---|---|---|
| Mixture Composition | Parameter | Individual Test |
| IL-4.75 | Ndesign = 50 | 93.0–97.4 % [1/] |
| IL-9.5, IL-12.5 | Ndesign = 90 | 92.0–96.0 % |
| IL-9.5,IL-9.5L, IL-12.5 | Ndesign < 90 | 92.5–97.4 % |
| IL-19.0, IL-25.0 | Ndesign = 90 | 93.0–96.0 % |
| IL-19.0, IL-19.0L, IL-25.0 | Ndesign < 90 | 93.0–97.4 % |
| All Other | Ndesign = 30 | 93.0 [2/]–97.4 % |

1/ Density shall be determined by cores or by correlated, approved thin lift nuclear gauge.
2/ 92.0 % when placed as first lift on an unimproved subgrade."