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Evaluation of Reclaimed Asphalt Pavement for Surface Mixtures

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JOINT TRANSPORTATION RESEARCH PROGRAM

INDIANA DEPARTMENT OF TRANSPORTATION
AND PURDUE UNIVERSITY

EVALUATION OF RECLAIMED ASPHALT PAVEMENT FOR SURFACE MIXTURES

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JOINT TRANSPORTATION RESEARCH PROGRAM

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16. Abstract

The Indiana Department of Transportation has successfully used Reclaimed Asphalt Pavement (RAP) for decades because of its economic and environmental benefits. Because of uncertainties regarding the types of aggregates contained in RAP and their resulting frictional properties, however, INDOT has until recently disallowed the use of RAP in asphalt surface mixtures. In addition, the hardened asphalt binder in the RAP could potentially increase the occurrence of thermal cracking. This research was conducted to explore the effects on RAP with poor or unknown aggregate qualities to establish maximum allowable RAP contents to provide adequate friction. The effects of RAP on thermal cracking were then investigated at the potential allowable RAP contents.

Laboratory testing showed that the addition of poor quality RAP materials did impact the frictional properties and cracking resistance of the mixtures, but that lower amounts of RAP had little effect. The frictional performance of the laboratory fabricated and field sampled RAP materials was acceptable at contents of 25% but may be questionable at 40%.

Field friction testing was also conducted on existing roadways with RAP to explore their field frictional performance. Several low volume roadways and one experimental interstate project were tested. The field results showed acceptable performance after 3 to 5 years of low volume traffic at RAP contents of 15-25% and after more than 10 years of interstate traffic with 15% RAP.

The low temperature testing showed an increased susceptibility to thermal cracking as the RAP content increased but the change in critical cracking temperature was relatively small at the 25% RAP level. At 40% RAP without a change in the virgin binder grade, the critical cracking temperature was about 6°C warmer than the control mixture. This finding supports the need for a binder grade change for RAP contents greater than 25%, as indicated in other research and as required by the current INDOT specifications.

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EXECUTIVE SUMMARY

EVALUATION OF RECLAIMED ASPHALT PAVEMENT FOR SURFACE MIXTURES

Introduction

The Indiana Department of Transportation (INDOT) has successfully used Reclaimed Asphalt Pavement (RAP) for decades because of its economic and environmental benefits. However, until recently, INDOT has disallowed the use of RAP in asphalt surface mixtures because of uncertainties regarding the types of aggregates contained in RAP and their resulting frictional properties, as well as the potential for the hardened asphalt binder in the RAP to increase the occurrence of thermal cracking.

This research was conducted to explore the effects of the inclusion of RAP with poor or unknown aggregate qualities in asphalt surface mixtures to establish maximum allowable RAP contents to provide adequate friction. The effects of RAP on thermal cracking were then investigated at the potential allowable RAP contents.

Slabs of asphalt mixtures with 15%, 25% and 40% of a laboratory fabricated RAP made with poor quality aggregate (with respect to friction) were tested to represent a "worst case." The slabs were subjected to polishing to simulate the effects of traffic, and changes in the surface texture and friction were measured periodically. Based on these results, possible threshold RAP contents of 25% and 40% were proposed. These threshold limits were further evaluated by testing slabs made with field-sampled RAP materials from across the state. In addition, low temperature cracking tests were performed on mixtures at the potential RAP threshold limits.

Findings

- The testing showed that the addition of poor quality RAP materials did impact the frictional properties and cracking resistance of the mixtures, but that lower amounts of RAP had little effect. The frictional performance of the laboratory fabricated and field-sampled RAP materials was acceptable at contents of 25% but may be questionable at 40%.
- Field friction testing was also conducted on existing roadways with RAP to explore their field frictional performance. Several low volume roadways and one experimental interstate project were tested. The field results showed acceptable performance after 3 to 5 years of low volume traffic at RAP contents of 15% to 25% and after more than 10 years of interstate traffic with 15% RAP.

Low temperature indirect tensile testing showed an increased susceptibility to thermal cracking as the RAP content increased, but the change in critical cracking temperature was relatively small at the 25% RAP level. At 40% RAP without a change in the virgin binder grade, the critical cracking temperature was about 6 °C warmer than that of the control mixture. This finding supports the need for a binder grade change for RAP contents greater than 25%, as indicated in other research and as required by the current INDOT specifications.

Implementation

The results of this work confirmed the current INDOT specifications regarding changing the virgin binder grade for mixtures with more than 25% RAP and the recent move to allow RAP in surface mixtures. The current specifications allow up to 40% binder replacement for Category 1 and 2 surface mixtures, and up to 25% for Category 3, 4 and 5 mixtures, with limits on the RAP gradation that require the use of the finer RAP fraction (100% passing the 9.5 mm [⅜ in] sieve). The results of this research showed that these specification limits are reasonable. A related research project, *Maximizing the Use of Local Materials in HMA Surfaces* (SPR-3308), is evaluating the effects of various amounts of low frictional quality coarse aggregates on surface friction; based on the results of that project, INDOT may consider relaxing or eliminating the size restrictions on RAP for hot mix asphalt (HMA) surfaces. INDOT could also consider, on a case-by-case basis, proposals from contractors to mill and stockpile high friction aggregates surface courses separately so that higher RAP contents could be used without sacrificing frictional performance. Because of the costs associated with milling lifts separately and maintaining distinct stockpiles, it is recommend that this should be the contractor's proposal when it is feasible and advantageous to both INDOT and the contractor, rather than being a requirement for all projects. At the current time, there is not a high demand for this option.

These findings have already been implemented and future changes can be readily implemented by revising the specifications if warranted by the results of SPR-3308 and approved by the Specifications Committee. No additional costs are associated with the implementation and, in fact, eventually lower materials costs would be expected. Continued monitoring of the performance of RAP mixtures in the field can be implemented through the Pavement Management System and the INDOT Office of Research and Development friction testing program.

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1. INTRODUCTION

Reclaimed asphalt pavement (RAP) has been recycled for many years in the U.S. and elsewhere in the production of hot mix asphalt (HMA). When properly designed and constructed, pavements including recycled asphalt can perform as well as or better than pavements constructed from virgin materials. The use of RAP in pavements is desirable since it offers economic benefits without compromising performance. From the sustainability point of view, recycling reuses the existing aggregates and RAP binder, thus reducing the need for new materials and the energy it takes to produce them. In addition, recycling can reduce transportation costs and expenses associated with landfilling or storage of the milled material. There are additional environmental and societal benefits of reusing existing resources that are difficult to quantify.

While most Departments of Transportation (DOTs), including the Indiana DOT (INDOT), already make extensive use of RAP, there are still some applications where this material has not been used to full advantage. Historically, INDOT and many other DOTs have not allowed the use of RAP in pavement surface courses due to concerns about potentially negative effects on pavement friction. Since it is difficult to know specifically what types of aggregate are present in RAP, their effects on friction are unknown. This is especially a concern in regions with predominantly soft aggregates (e.g., limestone), which can be susceptible to polishing.

Another concern with the use of higher RAP contents in surface mixes is the possibility of increased cracking because of the greater amount of oxidized (hardened) binder from the RAP. Since surface courses are exposed to greater temperature fluctuations and lower temperatures than courses deeper in the pavement, they are potentially susceptible to increased thermal cracking. The presence of brittle binder from the RAP may exacerbate the problem.

Under the current economy, there is an increased interest in using higher amounts of RAP in more applications. As a result, some states are considering expanding and revising their specifications regarding RAP usage. Recently, for example, the Indiana DOT began to allow the use of RAP in surface mixes. The initial allowance for RAP in surface courses permitted the use of 15% RAP in surface courses on roadways with a design traffic level of less than 3,000,000 equivalent standard axle loads (ESALs). In 2010, the specifications were expanded to allow up to 15% by weight of the total mixture for higher traffic categories (over 3,000,000 ESALs). Finally, in the 2012 specifications, the allowable RAP content is expressed in terms of binder replacement (percent of recycled binder as a percentage of total binder in the mix); up to 40% of the total binder can now come from recycled materials (RAP and shingles) for traffic volumes below 3,000,000 ESALs and 15% for traffic volumes greater than 3,000,000.

2. PROBLEM STATEMENT

Research was needed to address two potential problems with using RAP in surface mixtures, where friction resistance is the primary concern. When the aggregates present in the RAP are unknown or when a RAP stockpile contains a variety of coarse aggregates from different projects, the potential effects on friction are impossible to quantify. A secondary concern is the possibility that too much RAP or too hard a RAP could over-stiffen the surface course, making it more susceptible to cracking or raveling. These potential problems needed to be studied so that they can be accounted for and avoided.

3. OBJECTIVES

The overall objective of this project is to determine if INDOT can allow an increase in the use of RAP in mainline surface courses for high volume roadways. This may be possible through one of two approaches:

1. either develop a method to ensure that the aggregates in the RAP meet certain properties and provide adequate frictional resistance; or
2. determine a threshold level of RAP that can be used in mainline surface courses, regardless of the type of aggregate, without detrimental effect on the frictional properties of the surface.

The second approach would be easiest to implement since no additional testing by the contractors or INDOT would be required. INDOT has changed the specifications to permit up to 15% RAP (by binder replacement) in high traffic surface courses (Category 3 and higher); this project will explore that level and higher RAP contents.

Lastly, the effects of higher RAP contents on thermal cracking will be explored at the potential threshold levels.

4. FINDINGS AND RESULTS

This section of the report describes the approach taken to address the objectives of the study then summarizes the results of the laboratory and field testing. More details on the approach and test results are provided in the appendices. A review of the pertinent literature is provided in Appendix A.

4.1 Approach

A method to fabricate slabs of asphalt mixtures then simulate the polishing effects of traffic in the laboratory was developed in another research project *(1)*. Using this method, experimental asphalt mixtures are compacted into wooden molds approximately 500 mm (20 in) square using a "rolling pin" attached to a fork lift. The process is illustrated in Figure 4.1.

After compaction, the surface texture (macrotexture, expressed in terms of mean profile depth [MPD]) of the slabs is measured using a laser-based Circular Track

Figure 4.1 Compacting a slab for friction testing.

Meter (CTM; also referred to as Circular Texture Meter) according to ASTM E2157, *Standard Test Method for Measuring Pavement Macrotexture Properties Using the Circular Track Meter (2)*. The frictional properties are measured using a companion device called a Dynamic Friction Tester (DFT), according to ASTM E1911, *Standard Test Method for Measuring Paved Surface Frictional Properties Using the Dynamic Friction Tester (3)*, yielding a value called DF_{20} (dynamic friction at 20 km/h). The DFT is strongly influenced by the aggregate microtexture. Measurements from these two devices, the MPD and DF_{20}, are used to calculate the International Friction Index (IFI) according to ASTM E1960, *Standard Practice for Calculating International Friction Index of a Pavement Surface (4)*. The slabs are subjected to polishing by use of a Circular Track Polishing Machine (CTPM) that uses three rubber tires to simulate the effects of traffic. The CTPM is stopped periodically to allow testing of the texture and frictional properties of the slabs to assess the change in friction caused by the polishing action.

The Friction Number (FN) measured by INDOT's towed friction trailers can also be expressed in terms of IFI, allowing comparison of the lab and field friction tests. (More details on the polishing, testing and modeling of friction used in this study are provided in Appendix B.)

Since this study was to investigate the effects of RAP on pavement friction, the experimental design (shown in Appendix C) involved fabricating slabs with differing amounts of RAP, ranging from 0% (the control) to 40% by mass of the total mixture. (At the time this study was initiated, INDOT specified the allowable RAP content as the mass of RAP expressed as a percentage of the total mass of the mix. Now that INDOT specifies the RAP content in terms of binder replacement, the binder replacement values have been calculated and are shown along with the material and mix design information in Appendix D.) Both dense graded asphalt (DGA) and stone matrix asphalt (SMA) surfaces were studied.

Since one of the major questions about the use of RAP in surface mixes is the potential impact of poor frictional quality aggregates in the RAP, the main portion of this research involved testing a laboratory-produced RAP with poor frictional quality aggregates to represent the "worst case" scenario. A limestone aggregate that was highly susceptible to polishing was identified in consultation with the INDOT Office of Materials Management. Under INDOT specifications, this aggregate would not be allowed for use on medium to high traffic volume roadways. A mix was produced using this poor frictional quality aggregate and aged in the laboratory to produce the "worst case" RAP. The RAP was then incorporated in DGA and SMA surfaces at up to 40% (by mass of the mix) according to the experimental design. The percentages of virgin aggregates were manipulated to keep the gradations of the various mixtures essentially constant as the RAP content changed. The virgin binder content was also adjusted to keep the design air void content constant at 4%. The effects of changing the RAP content on the pavement frictional properties could then be investigated in the laboratory using the previously described slabs, polishing procedure and testing methods.

After the analysis of the effects of the "worst case" RAP on frictional properties, possible allowable maximum RAP contents (thresholds) were determined to be 25% and 40%. Then mixtures were produced using actual RAPs from stockpiles around the state in order to ascertain if field materials would perform acceptably at the possible threshold RAP content. It was anticipated that the "field RAP" would have frictional properties at least as good as or better than the "worst case scenario" lab-produced RAP. Six RAP stockpiles were sampled and tested to verify the results of testing the worst case RAP.

Lastly, actual pavements incorporating various RAP contents were tested in the field using the towed friction trailer (ASTM E274, *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*) *(5)*. Although INDOT did not allow the use of RAP on high traffic volume locations when this research was initiated, there were sites on lower traffic roadways and one experimental section on I-70 that were tested to give some indication of field performance.

4.2 Laboratory Polishing and Testing of Worst Case RAP

The results of testing of eight slabs with varying percentages of the laboratory-produced worst case scenario RAP showed that both macrotexture (expressed by MPD) and dynamic friction (expressed by DF_{20}) changed during the polishing process. (Details are provided in Appendix E; Figures E.1 and E.3 illustrate the changes in macrotexture for the DGA and SMA slabs, respectively.) It can be observed that the macrotexture of the SMA specimens remained relatively constant during testing while macrotexture of the DGA increased significantly. The greatest rate of

increase was observed during the first part of the polishing, up to 30,000 wheel passes, after which it stabilized. Overall, the macrotexture of DGA nearly doubled. During a previous study *(6)*, a similar phenomenon was also observed in the field; there was a significant macrotexture change for a DGA pavement and a relatively low change for an SMA. It has to be noted, however, that the initial MPD (before initiating polishing) was much higher for the laboratory fabricated specimens than for the field test sections, perhaps indicating a need to improve the specimen compaction method (which will be investigated in future work). It appears the current specimen preparation method may not apply enough compactive effort or the polishing action "scrubs" the slab too aggressively.

Based on the analysis of the macrotexture data, it can be concluded that an increase in the RAP content resulted in a slight increase in the MPD for the SMA specimens, while a greater trend was observed for the DGA mixes. Changes in the macrotexture observed during polishing are, most likely, connected to raveling of the DGA specimens. The CTPM is a relatively aggressive polishing method in which the shearing action of the tires abrades the surface. The SMA appears to be more resistant to the abrasion.

While changes in macrotexture can be important, the greatest influence on the IFI results from changes in the DF_{20}. This parameter measures the wet friction and is highly related to the microtexture of the aggregates exposed at the surface. An aggregate that is susceptible to polishing will exhibit decreases in the DF_{20} as polishing progresses. This, then, results in decreases in the IFI value, F60. The trend of the plot of DF_{20} versus number of wheel passes is typically similar to the plot of F60 versus number of wheel passes.

The F60 value, then, combines the effects of both the macrotexture (MPD) and the microtexture (DF_{20}) of the surface. Figure 4.2 shows the comparison of F60 values for DGA with varying RAP contents, and Figure 4.3 shows the same for the SMA slabs. In general, it can be seen from Figure 4.2 that as the RAP content increases, the F60 value decreases for the DGA mixtures. For the SMA slabs, one replicate slab of the control (0% RAP) was fabricated and tested to look at the repeatability of the process. Figure 4.3 shows that

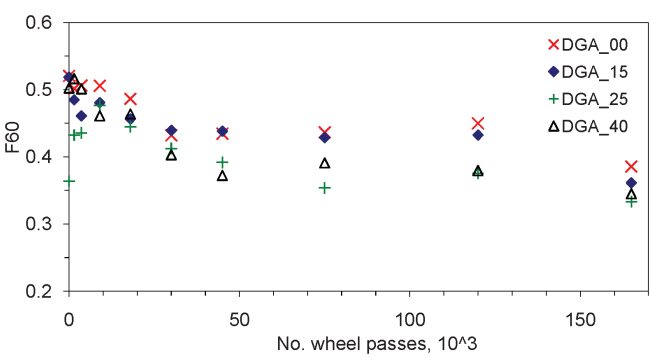

Figure 4.2 Comparison of F60 values for DGA slabs with differing RAP contents.

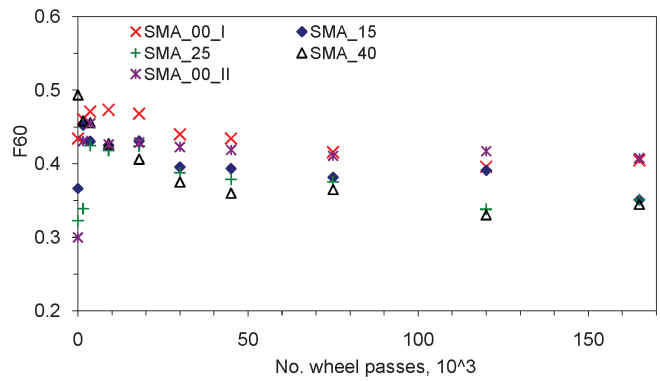

Figure 4.3 Comparison of F60 values for SMA slabs with differing RAP contents.

the two SMA control slabs performed similarly and their terminal polish values were virtually identical. The SMA slabs seem to show little difference between the 15% and 25% RAP slabs throughout most of the polishing and the 40% RAP mix is fairly consistently the lowest F60 value (except very early in the polishing process). Thus, there is also an effect of increasing RAP content for the SMA mixtures, but it appears to be less than for the DGA.

In order to quantify changes in the F60 values taking place during polishing and to evaluate the frictional properties of the mixture, a polishing model developed in previous research *(7)* was used. This model allows for estimation of the terminal friction level (referred to as F60@X1) and the polishing rate (a_4). (NOTE: X_1 represents the number of wheel passes at which the terminal friction level is reached.) The model has a general form shown in Figure 4.4. More details on the model are provided in Appendix B.

In general, a high F60 value at X_1 corresponds to high terminal friction value for the pavement. In addition, the higher (less negative) the a_4 value is, the more resistant the specimen is to polishing. The best frictional performance, therefore, is obtained when a pavement has a higher (less negative) a_4 value and a high F60 value at X_1, indicating that it is resistant to polishing and has a high friction value after polishing.

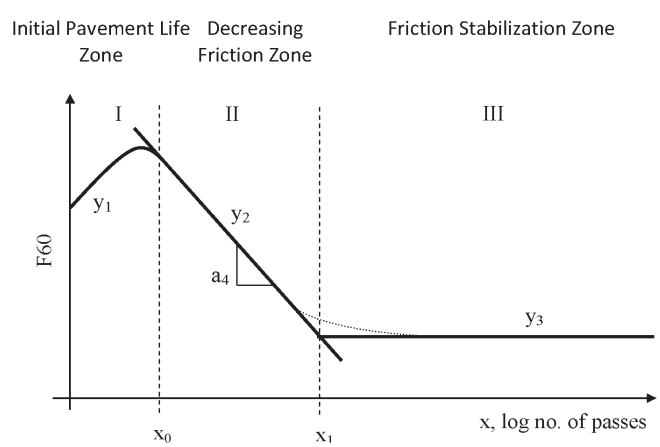

Figure 4.4 Polishing model *(1)*.

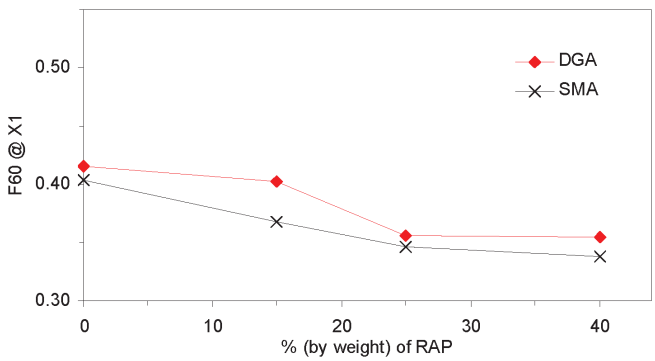

Figure 4.5 Distribution of friction terminal value (F60@x1).

A summary of the distribution of the terminal friction level (F60@X1) and the polishing rate (a_4) parameters is shown in Figures 4.5 and 4.6, respectively.

Changes in the terminal friction level shown in Figure 4.5 suggest that the addition of poor frictional quality RAP indeed influences the friction, as could be expected. The more RAP material that is added, the lower the friction value becomes. This general trend can be observed for both DGA and SMA mixtures. The DGA friction is slightly higher than that of the SMA at all RAP contents, probably because of somewhat higher macrotexture of the slabs. For the SMA mixtures, the changes in the F60@X1 values generally decrease linearly up to 25% RAP then flattens somewhat. For the DGA mixtures the F60 drops more between samples with RAP contents of 15% and 25% than between 0% and 15% or 25% to 40%. It has to be noted, however, that even the lowest F60@X1 values (observed for the specimens with 40% RAP content and equal to about 0.34–0.35) are much higher than the "flag value" determined in another study *(8)*. According to that study, the F60 value should be greater than 0.17–0.20.

Changes in the polishing rate shown in Figure 4.6 suggest that the addition of poor quality RAP did indeed influence the polishing susceptibility, as could be expected. The general trend observed for both DGA and SMA mixtures shows that as more polish susceptible RAP material is added, the combined mixture becomes less resistant to polishing. It can also be noticed that for specimens with 15% "poor" quality (laboratory) RAP the polishing rate was about −0.02 (for both DGA and SMA specimens) and that it was similar to the polishing rate of specimens with no RAP. This suggests that up to 15% RAP has an insignificant influence on the polishing resistance of the mixture, even when the RAP itself is highly polishable. In another study *(1)*, a polishing rate of less than about −0.03 (less negative) was found to be insignificant. In this study, the lowest values are similar for both SMA and DGA mixtures at 40% RAP and are equal to about −0.033. This is above the level at which the polishing rate might be considered significant.

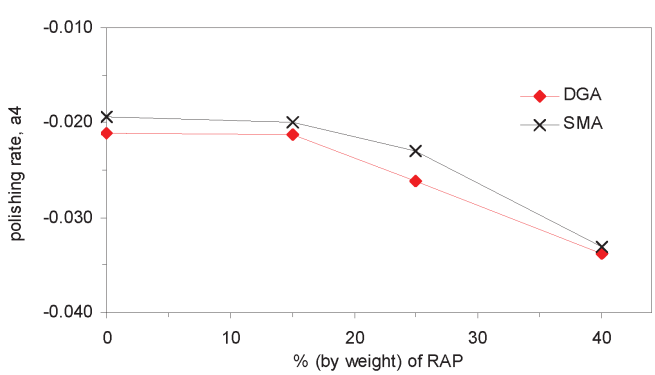

Figure 4.6 Distribution of polishing rate (a_4).

Considering the F60 values in light of the previous research, it appears that although the friction does decrease as the amount of poor quality RAP increases, even a 40% RAP mixture provides a friction level higher than the "flag value" and would be expected to perform acceptably. The polishing rate, a_4, appears to be acceptable at over 25% RAP but at 40% RAP, the rate is beyond the level of significance (i.e., more negative than −0.03). These results suggest a possible threshold RAP level of 25% would be appropriate to ensure adequate terminal friction and an acceptable rate of polishing. The threshold could possibly be somewhat higher, especially for lower traffic volume or lower speed roadways, so the remainder of the laboratory testing (both friction and mechanical) will focus on RAP contents of 25% and 40%.

These findings also seem to support INDOT's recent change to allow up to 40% RAP for lower volume surfaces (Category 1 and 2) and 15% RAP for higher volume surfaces (keeping in mind that these limits are in terms of binder replacement, not simply mass of RAP in the mix). Based on friction considerations alone, it might be possible to increase the RAP content for higher volume surfaces to 20% or 25% RAP; however, this would need to be verified. Mechanical properties, especially the resistance to thermal cracking, should also be considered; some results of low temperature cracking tests are presented later in this report.

4.3 Laboratory Polishing and Testing of Actual Field RAPs

Since the results of testing the worst case RAP seemed to indicate an acceptable threshold level of 25% RAP—or possibly more—the actual RAP materials collected from six RAP stockpiles around the state were used to fabricate slabs with 25% and 40% RAP (by mass of the mix). These slabs were then polished and tested as described for the laboratory-produced RAP mixtures to verify if the proposed threshold would hold true for actual RAP materials. Two of the RAPs were randomly selected to be incorporated in SMA mixes and the other four were used in DGA mixes. A priority was placed on testing more DGA mixes since INDOT is currently using more DGA than SMA surfaces. Besides, the frictional performance of the DGA and

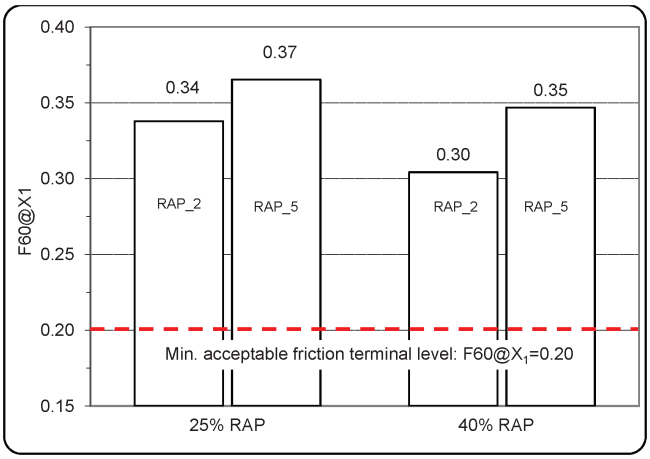

Figure 4.7 Terminal friction value for field-sampled and laboratory-produced RAP SMA mixes.

SMA mixtures in terms of F60 and a_4 had not been widely different, as shown in Figures 4.5 and 4.6.

Figure 4.7 shows the terminal friction levels determined for the SMA mixtures with RAP sources 2 and 5. Both RAP sources provide terminal friction levels that exceed the friction flag value. Both mixes also exhibit acceptable polishing rates (as shown in Appendix E). The terminal friction level drops when more RAP is added to the mix (and the polishing rate worsens). This suggests that the 40% RAP mixes could polish faster and to a lower terminal friction level than mixes with 25% RAP. Although the friction level appears to be adequate for these two RAP sources, it may be prudent to restrict the use of very high RAP content mixes to lower volume surfaces if the RAP aggregate type or qualities are unknown.

INDOT is currently using more DGA mixes than SMA mixes because of the higher cost of SMAs. Therefore the results for the DGA mixes may be more pertinent. Figure 4.8 shows the terminal friction levels for the DGA mixes incorporating four field-sampled RAP sources compared to the friction flag value. The terminal friction values for the DGA slabs with 25% RAP are above 0.23, but those for some of the mixes with 40% RAP are at or very near the flag value.

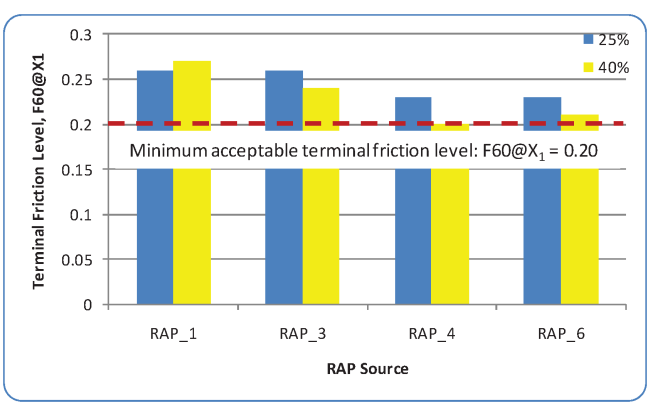

Figure 4.8 Terminal friction value for field-sampled and laboratory-produced RAP DGA mixes.

If these mixes are looked at in terms of binder replacement instead of by mass of the mix, the RAP content is lower for most RAP sources. These mixes have binder replacement values in the range of 19% to 23% instead of 25%, and 27% to 38% instead of 40%. The exception is RAP 1, which has slightly higher binder replacement values of 26% and 42%, respectively. Therefore, a limit of 40% RAP by binder replacement would likely be too high to ensure good frictional properties with unknown RAP aggregates.

Based on the friction polishing and testing, then, it appears 25% RAP by binder replacement would be the upper limit for a threshold value of RAP in surface mixes for medium or higher traffic. This suggests that, for some RAP sources, 25% may be somewhat high. Other data needs to be considered in addition to the frictional performance, such as thermal cracking resistance, to set an acceptable threshold level. Another consideration is the merit in progressing in steps and accumulating information on field performance to refine the specifications in the future. From that point of view, allowing 20% RAP by binder replacement would be a reasonable first step pending additional field performance history, especially for high volume roadways.

This research is based on the assumption that the frictional properties of the RAP aggregate are unknown or mixed. There may be cases where it is advantageous to control the milling and stockpiling operations so that the properties of the RAP are known. In that case, INDOT could consider allowing the use of greater percentages of RAP. For example, the Illinois DOT allows the use of higher percentages of RAP if the contractor mills and stockpiles surface mixes separately from other pavement layers. In Indiana, if a contractor mills a surface containing steel slag, for example, it would be reasonable to allow the use of higher percentages of that material in the surface from a friction standpoint (as long as mechanical performance is acceptable). This could be considered on a case-by-case basis when the contractor sees an advantage and approaches the department with a proposal.

4.4 Low Temperature Testing

To further explore whether higher RAP contents could be permitted without detriment to the performance of the surfaces, mixtures with the potential threshold levels of RAP were prepared and tested for low temperature cracking resistance. The selected mixtures were tested for low temperature creep and stiffness according to AASHTO T322, *Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt Using the Indirect Tensile Test Device (9)*. Using this data, the critical cracking temperatures for pavements constructed from these mixtures could be estimated; the critical cracking temperature is where the thermal stresses accumulating

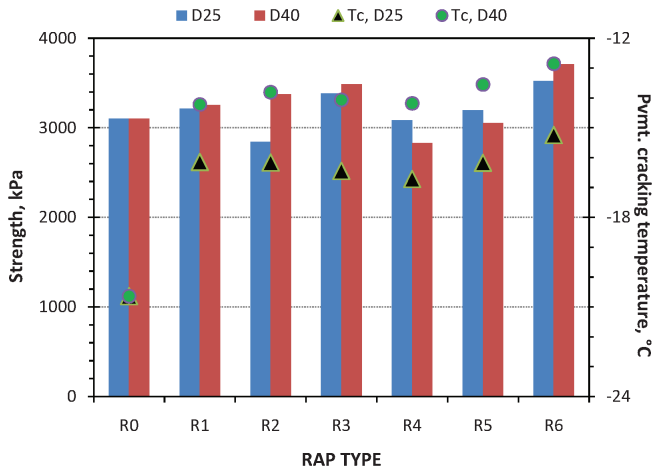

Figure 4.9 Average mixture strength of actual RAP sources and critical cracking temperatures.

in a pavement exceed the strength of the material of which is it composed. The details of the test results and analysis are provided in Appendix F and are summarized here.

The test results showed that, in general, the strengths of the mixtures with and without RAP were similar; in some cases the RAP mixes had higher strengths and in others lower strength than the control, as shown in Figure 4.9. On the other hand, the stiffness of the RAP mixes was always higher than the stiffness of the control mix, as shown in Figure 4.10. In addition, the 40% RAP mix was almost always (five of six cases) stiffer than the 25% RAP mix; in the sixth case, the values were nearly the same. Because the stiffnesses of the RAP mixes were higher with no significant difference in the strength, the estimated critical cracking temperatures of the RAP mixes were warmer (less negative) than those of the control. (A stiffer mix can be thought of as being more brittle and will be more likely to crack unless the mix strength also increases.)

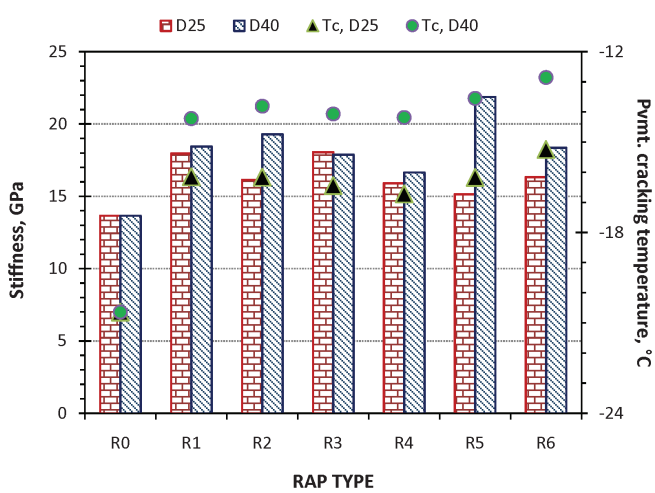

Figure 4.10 Average mixture stiffness of actual RAP sources and critical cracking temperatures.

The critical cracking temperatures were warmer than the control by around 4 °C for the 25% RAP mixes and around 6 °C for the 40% RAP mixes. All of these mixtures, however, were made with PG64-22, that is, without adjusting the virgin binder grade for the higher amount of hardened RAP binder. Had those mixes been made with a softer virgin binder grade, it is likely that the critical cracking temperatures of the 40% RAP mixes and the control would have been comparable. Previous research on plant produced RAP mixes from Indiana and Michigan shows that changing the binder grade may not be necessary at 25% RAP but would be advisable at 40% RAP.

The results of the testing in this study, then, suggest that mixes with 25% RAP may have a slightly increased chance of exhibiting more thermal cracking than a virgin mix. A mix with 40% RAP would have an even greater chance of cracking if the virgin binder grade is not adjusted to compensate for the stiffness of the RAP binder. The current INDOT specifications, however, do require a binder grade change when the RAP content is greater than 25%, based in part on the previous research findings. The results here tend to support the INDOT specifications. Field performance monitoring of some high RAP surfaces would help to determine if this possibility of increased cracking is observed in the field.

4.5 Field Friction Testing

Eight existing field sections where INDOT had allowed the use of RAP in surface mixes were identified and tested as a part of this research effort. The as-constructed information was obtained from construction records and in situ friction tests were conducted.

The eight different road sections were on various categories of roadways, including interstate highways, state and U.S. roads. The specific roads, RAP contents, Reference Posts (RPs) of the chosen test section and the year constructed are shown in Table 4.1. Within each contract length, a one-mile section was chosen for CTM and DFT testing. These sections were selected to avoid major towns and junctions with other roadways; in addition, straight segments with no superelevation were chosen to provide convenient and safe test sites. (CTM and DFT testing require the operator to be exposed on the roadway, so safety was a concern.) The I-70 sections are two test sections from the national Long Term Pavement Performance (LTPP) SPS-9A project; one is a control section with no RAP and a PG 64-28 binder and the other includes 15% RAP with the same binder. These are the oldest sections and have the highest traffic levels. Details on the projects, mixes and friction testing results are provided in Appendix G.

As shown in Table 4.1, two of these sites have been in place since 1997 on I-70; the others were placed in 2005 and 2006. These sections can offer good insights into the friction levels provided by RAP surfaces.

TABLE 4.1
Field Sites Tested

Road	Location	Contract	RAP, %	PG Grade	Year Completed
SR-38	Richmond-Hagerstown	RS-27534	15%	70-22	2005
US-35	Richmond	RS-27998	25%	58-28	2006
SR-103	New Castle	RS-28000	25%	58-28	2006
I-70 East	East of Indianapolis SPS-9A	R-22923	0%	64-28	1997
I-70 East	East of Indianapolis SPS-9A	R-22923	15%	64-28	1997
SR-47	South of Crawfordsville	RS-28319	15%	64-22	2006
SR-32	Fountain Co.	RS-28324	15%	64-22	2006
US-136	Fountain Co.	RS-28317	15%	64-22	2006

TABLE 4.2
Summary of 2007 Frictional Properties of Tested Sections

Road	Years in Service (2007)	Estimated Cumulative Traffic, NVA, 10^6	MPD, mm	DF_{20}	F60 (from CTM/DFT)	Towed Trailer Data	
						Smooth FNS40	Rib FNR40
SR-38	2	3.4	0.40	0.52	0.25	47	54
US-35	1	2.2	0.30	0.50	0.22	33	54
SR-103	1	3.1	0.33	0.55	0.24	45	57
I-70 (0%)	10	152.5	*	*	*	22	*
I-70 (15%)	10	152.5	*	*	*	28	*
SR-47	1	1.1	0.37	0.61	0.27	37	58
SR-32	1	0.4	0.35	0.69	0.29	58	62
US-136	1	0.7	0.38	0.65	0.29	45	58

*CTM and DFT were not performed due to restrictions on traffic control on Indianapolis interstates.

TABLE 4.3
2007 Special Friction Testing and 2008–2010 Inventory Testing Results

Road	2007 Data FNS40	2008 FNS40	2009 FNS40	2010 FNS40
SR-38	47	—	—	40.7
US-35	33	31.1	—	—
SR-103	45	—	—	—
I-70 (0%)	22	22.5	28.4	22.5
I-70 (15%)	28	38.8	44.0	38.8
SR-47	37	—	44.8	—
SR-32	58	65.3	—	—
US-136	45	—	—	42.8

— No inventory testing performed under three-year cycle.

In 2007, special friction testing was performed by the INDOT Office of Research and Development (ORD). Testing was also performed using the CTM and DFT. In addition, routine inventory testing results were obtained from the ORD on these sections during other years. The 2007 data is shown in Table 4.2, and the inventory data from 2008 through 2010 is shown in Table 4.3.

With the exception of the I-70 control section, all of the surfaces with RAP are performing well to date. Even the mixes with 25% RAP by mass of the mix (US-35 and SR-103) are performing well based on the test results to date. Inventory data for these sections should be monitored in the future to continue to evaluate the performance of these sections.

The I-70 control section with no RAP may be approaching the friction flag value. Surprisingly, the I-70 section with 15% RAP has been out-performing the control section from a frictional point of view since at least 2007. The nature of the RAP material used there, however, is not known. Nonetheless, this data does show that mixes with 15% RAP can perform acceptably for over ten years under heavy traffic.

This actual field data supports the current INDOT practice of allowing the use of RAP in surface courses up to 15% RAP. The friction inventory data should be monitored for these sections to ascertain the long term performance of these higher RAP mixtures.

5. CONCLUSIONS

The results of this research lead to the following conclusions regarding the expected frictional and

cracking performance of SMA and DGA mixtures with RAP:

- Polishing and testing 9.5 mm mixtures that incorporated RAP produced in the laboratory to have poor frictional properties suggested that the addition of small quantities of RAP would have little effect on the surface friction. The addition of greater amounts of RAP did influence both the terminal friction level and polishing rate.
- Based on testing the poor quality, laboratory-produced RAP, possible upper limits for the allowable RAP content of 25% to 40% appeared reasonable for frictional considerations.
- Mixtures produced with six RAPs sampled from random stockpiles around the state were polished and tested at the possible threshold levels. The aggregate types in the RAPs sampled were not determined. This testing showed that the SMA mixes produced with two of the RAP sources provided adequate friction in the laboratory at both 25% and 40%, but the performance at 25% was better. For the DGA slabs, however, the terminal friction level of the 40% RAP mixes was at or near the friction flag value established in earlier research. The 25% RAP DGA mixes had terminal friction levels higher than the flag value.
- Low temperature cracking performance of surface mixtures is also important and may be affected by a high RAP content, therefore this property was also explored at the 25 and 40% RAP contents with the Indirect Tensile Test (IDT). This testing and analysis revealed that the tensile strengths of DGA mixes with and without RAP were not significantly different, but the stiffnesses of the mixes did vary. The mixtures with field-sampled RAP were stiffer than the control and the 40% RAP mixes typically were stiffer than the 25% RAP mixes.
- Because of the increased stiffness of the RAP mixes, their critical cracking temperatures were somewhat higher than that of the control mix suggesting that these mixes might be more susceptible to thermal cracking. The 40% RAP mixes had warmer cracking temperatures than the 25% RAP mixtures and would be expected to experience more or earlier cracking. The cracking temperatures would improve if a softer virgin binder grade was incorporated in the 40% RAP mix as required by current INDOT specifications.
- Field friction testing of eight existing surfaces in Indiana, with two on the I-70 SPS-9 site being as old as 13 years, support the current INDOT specifications.
- Based on all of the laboratory friction and cracking testing and considering the field performance of the existing RAP surfaces, the current INDOT specifications allowing 25% RAP (by binder replacement) in Category 3, 4 and 5 surfaces, with restrictions on the maximum size of the RAP, appear reasonable. An on-going research project, SPR-3308, *Maximizing the Use of Local Materials in HMA Surfaces (10)*, may provide results that suggest the size restrictions can be relaxed or eliminated in the future.
- INDOT could also consider, on a case-by-case basis, allowing exceptions where the contractor proposes milling surfaces with known high quality aggregates, such as steel slag, separately so that the high quality aggregates can be incorporated in new surfaces at higher RAP contents. If the aggregate frictional characteristics are acceptable, higher RAP contents should be satisfactory for Category 5 roads. The decision to mill and stockpile layers separately has cost implications, so this should be the contractor's choice based on specific project and plant situations. There are also logistical issues involved in identifying and testing stockpiles, so implementing this option must be agreeable to INDOT.
- Monitoring of the field sections evaluated here and other pavements constructed with RAP in the surfaces should continue to evaluate the changes in friction level and observe cracking performance. Based on field observations, additional changes in the specifications may be warranted.
- Future research implementation work should be conducted to refine the laboratory friction polishing and testing. Improvements are warranted in terms of verifying the calibration of the equipment and compacting the test slabs. The test method has proven to be a valuable tool but could provide more meaningful and reliable results with these refinements.

6. RECOMMENDATIONS FOR IMPLEMENTATION

This research generally supports the current INDOT specifications allowing the use of RAP in asphalt surface mixtures. The current specifications allow up to 15% RAP by binder replacement on Category 3 and higher roadways and 40% on lower volume roadways. The results suggest, however, that INDOT consider increasing the allowable RAP content on Category 3 and 4 roadways to 20%. The frictional and cracking performance of existing and new surfaces incorporating RAP should be monitored to determine if further increases in the allowable RAP content are feasible. INDOT should also consider allowing contractors to use higher amounts of RAP, up to at least 25%, for specific cases where the contractors offer to mill high quality surfaces, such as steel slag mixtures, separately so that the aggregate frictional properties are known. A change in PG binder grade may be required to mitigate low temperature cracking.

Reclaimed Asphalt Pavement has been proven to be an environmentally friendly and economical commodity leading to reduced material costs while maintaining pavement quality and performance. Implementation of the findings of this research will allow INDOT to maximize the benefits of using this valuable resource to further reduce paving costs, reduce the environmental costs of disposal of old pavement materials and provide a safe and durable pavement for the traveling public.

REFERENCES

1. Kowalski, K. J., R. S. McDaniel, and J. Olek. Development of a Laboratory Procedure to Evaluate the Influence of Aggregate Type and Mixture Proportions on the Frictional Characteristics of Flexible Pavements. *Journal of the Association of Asphalt Paving Technologists*, Vol. 77, 2008, pp. 35–70.
2. ASTM Standard E2157. *Standard Test Method for Measuring Pavement Macrotexture Properties Using*

the Circular Track Meter. ASTM International, West Conshohocken, Pennsylvania.
3. ASTM Standard E1911. *Standard Test Method for Measuring Paved Surface Frictional Properties Using the Dynamic Friction Tester*. ASTM International, West Conshohocken, Pennsylvania.
4. ASTM Standard E1960. *Standard Practice for Calculating International Friction Index of a Pavement Surface*. ASTM International, West Conshohocken, Pennsylvania.
5. ASTM Standard E274. *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*. ASTM International, West Conshohocken, Pennsylvania.
6. Kowalski, K. J., R. S. McDaniel, A. Shah, and J. Olek. Long Term Monitoring of the Noise and Frictional Properties of PFC, SMA and DGA Pavements. In *Proceedings of the* 88th *Transportation Research Board Annual Meeting*. CD-ROM. Washington, D.C., January 11–15, 2009.
7. Kowalski, K. J., R. S. McDaniel, and J. Olek. *Identification of Laboratory Technique to Optimize Superpave HMA Surface Friction Characteristics* Publication FHWA/IN/JTRP-2010/6. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2010. doi: 10.5703/1288284314265.
8. Kowalski, K. J., R. S. McDaniel, J. Olek, A. Shah, and S. Li. Development of the International Friction Index Flag Value. Paper No. 705, presented at the Tenth International Conference on Application of Advanced Technologies in Transportation, Athens, Greece, May 27–31, 2008.
9. AASHTO T322. *Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt Using the Indirect Tensile Test Device*. 2007. American Association of State Highway and Transportation Officials.
10. McDaniel, R. S., and A. Shah. *Maximizing the Use of Local Materials in HMA Surfaces*. Publication FHWA/IN/JTRP-2012/07. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2012. doi: 10.5703/1288284314667.

APPENDIX A

LITERATURE REVIEW

A.1 RECLAIMED ASPHALT PAVEMENT (RAP)

Since the 1970s, RAP has been used in millions of tons of HMA. McDaniel et al. *(1)* investigated the physical properties (rutting resistance and low temperature cracking resistance) of RAP mixes with up to 50% RAP from three states in the Midwest, including Indiana. That research indicated that mixtures with up to 25% RAP would be expected to perform well in terms of rutting and low temperature cracking, while mixes with 40% to 50% RAP could be more problematic in terms of meeting the Superpave mix design requirements, primarily because of the presence of high amounts of fine material in the RAP.

A.1.1 Processing and Storing RAP

The National Asphalt Pavement Association has a publication entitled *Recycling Hot Mix Asphalt Pavements (2)* that discusses processing and handling RAP at the plant and during construction. Raw RAP is typically generated by two methods, milling from surface layers or removing from full-depth HMA layers. These materials are processed by crushing, sieving and stockpiling. By crushing or screening the raw RAP, the material is mixed and oversized materials are removed. Storing processed RAP under a covered roof is recommended to avoid excessive moistures and reduce the fuel consumptions. The Asphalt Institute *(3)* also recommends that the height of RAP stockpiles should be limited to a maximum of 3 meters (10 ft) to help prevent agglomeration or sticking together of the RAP particles.

Solaimanian and Kennedy *(4)* showed that high variability in RAP material greatly affects the variability of the asphalt content and gradation of the production mixture, especially at higher percentages of RAP. Kandhal et al. *(5)* found that Voids in Total Mix (VTM) is affected mostly by asphalt content, the percent passing No. 200 sieve, and the relative proportions of coarse and fine aggregates. VTM can be increased by reducing the asphalt content, the percent passing 0.075 mm, or both. Stroup-Gardiner and Wagner *(6)* reported that there were some concerns about RAP stockpiles with widely variable gradations as well as high percentages of dust (minus 0.075 mm); thus limiting its use in Superpave mixtures. Screening the RAP allowed up to 40% of the coarse RAP fraction to be used.

Mayes et al. *(7)* pointed out that dusts, minus 0.075 mm, can be reduced by proper screening processes. These screening processes also can reduce the amount of aged binders on the fine aggregates or dusts. Therefore, the processed RAP actually can be a consistent product. Similarly, Nady *(8)* showed that the variability of RAP can be controlled and may not be as bad as might be expected. He compared a milled RAP pile from Iowa DOT (IDOT) projects with other random RAP piles. The comparison revealed the consistency and uniformity of both source of RAP. Therefore, even if the RAP is from random pavements, with proper processing, it may still provide high-quality aggregate and acceptable control of the gradation. He concluded that minimum changes in IDOT gradation requirements over time (meaning the RAP had similar gradations to currently used mixtures), fairly uniform aggregate production over time, and the processing of RAP helped prevent significant variation in the gradations of mixes containing RAP.

A.1.2 Review of RAP Specifications for Surface Mixtures

A review of national specifications was done early in this project to identify specifications for RAP. Some states, such as Alabama, Florida, Illinois, Iowa, Missouri, South Carolina, and Wyoming, ask contractors to process and label the RAP stockpiles as a sort of "Certified RAP." Furthermore, agencies such as Illinois, Iowa, New York State, North Carolina, South Carolina, Utah, and Vermont, require contractors to process and stockpile RAP separately for different purposes.

In the past, a 1997 review of recycling practices *(9)* showed that about 18% of states do not allow the use of RAP in surface courses. However, this was before the use of Superpave specifications became widespread. A 2008 review of RAP specifications revealed forty-three states and Puerto Rico (or 84%) allow the use of RAP. Among them, twenty states including Puerto Rico (or 47%) specifically indicate an upper limit of RAP for surface mixtures.

However, fourteen states (or 33%), including Alabama, Florida, Georgia, Illinois, Kentucky, Louisiana, Massachusetts, Missouri, New Mexico, Oklahoma, Oregon, South Carolina, and Washington, did not allow the use of RAP either for all or for some surface mixtures such as OGFC, SMA, and Superpave mixtures. Until recently, in the state of Indiana, RAP could only be used for surface mixtures on shoulders and relatively low traffic volume roads.

A.1.3 Recommended Use of RAP in the Superpave Mix Design System

Usage of RAP decreased as states began to adopt the Superpave mix design system. In 1997, the FHWA Superpave Mixtures Expert Task Group (FHWA Mix ETG) provided guidelines for including RAP in Superpave mixture design procedures. These guidelines were based on existing practices and experiences with the use of RAP in Marshall, Hveem, and other types of mix design procedures. The guidelines established tiers of different RAP contents:

Tier 1. RAP content ≤15% by mass of total mixture: Treat the RAP as anther stockpile of aggregates, and select the same asphalt binder grade, based on climate and traffic, that would be used for a standard mix design using only virgin materials.
Tier 2. RAP content 16% to 25% by mass of total mixture: Use the next softer grade of asphalt binder than would be selected for use in a virgin mix design.
Tier 3. RAP content □25% by mass of total mixture: Select asphalt binder grade by recovering and testing the asphalt from the RAP mix and using appropriate blending charts to obtain the desired binder properties for high and low temperature requirements.

NCHRP project 9-12 *(10)* later concluded that for low RAP contents, 10% to 20%, it is not necessary to test the properties of extracted RAP binders, because there is not enough of the old, hardened RAP binders present to change the total binder properties. At higher RAP contents, however, the RAP binder will have a noticeable effect, and it must be accounted for by using a softer grade of binder. For intermediate ranges of RAP, the virgin binder grade can simply be dropped one grade. For higher percentages of RAP, the RAP binder must be tested to develop blending charts. The findings of NCHRP 9-12 validated FHWA/ETG guidelines regarding the three tiers of RAP usage; however, there was some data that would support alternate break points for the tiers based on the low temperature grade of the RAP binder. AASHTO M323 *(11)* adopted the FHWA Mix ETG binder selection guidelines for Superpave volumetric mix design, shown as Table A.1, because they were supported by the NCHRP research and there was too little data to support the alternate break points.

TABLE A.1
Binder Selection Guidelines for RAP Mixtures

Recommended Virgin Asphalt Binder Grade	RAP Percentage
No change in binder selection	<15%
Select virgin binder one grade softer than normal (e.g., select a PG 5828 if a PG 64-22 would normally be used)	15–25%
Follow recommendations from blending charts	>25%

From AASHTO M323. *Standard Specification for Superpave Volumetric Mix Design.* American Association of State Highway and Transportation Officials.

A.1.4 Long-Term Oven Aging

Bell et al. *(12)* summarized the work to validate the short-term and long-term oven aging techniques developed under SHRP to simulate aging during the construction process and during field service. Field and laboratory-produced samples of the original construction materials were selected and produced in order to distinguish the aging processes and behaviors. The results suggested that:

- Short-Term Oven Aging: 4 hours at 135 °C is representative of the type of aging that occurs during mixing and placement.
- Long-Term Oven Aging:
  - 2 days at 85 °C is representative of approximately 5 years of aging.
  - 3 days at 85 °C is representative of approximately 7 years of aging for a dry-freeze zone and 15 years for a wet–no freeze zone. However, there were no specifications developed for wet-freeze and dry–no freeze zones in this study. (Indiana is in a wet-freeze zone.)

AASHTO R30 *(13)* suggests three types of mixture conditioning for Superpave volumetric mixture design, short-term, and long-term. The recommendation for short-term oven conditioning is similar to the results of Bell's study, though the long-term conditioning processes are slightly different. R30 requires aging time and temperature as described below:

- 2 hours ± 5 minutes at compaction temperature is designed to allow for binder absorption during the mix design.
- 4 hours ± 5 minutes at 135 °C is representative of the effects of plant-mixing and construction on the mixture.
- 5 days ± 0.5 h (or 120 hours) at 85 ± 3 °C is representative of seven to ten years of aging in the field.

A.2 SKID RESISTANCE OF PAVEMENTS

Frictional properties of surface mixtures are significantly related to highway safety. A well maintained surface course provides an adequate level of friction to operate vehicles safely. According to National Transportation Safety Board and FHWA reports *(14)*, approximately 13.5% of fatal accidents and 25% of all accidents occur on wet-pavements.

A.2.1 Physics of Friction

The classic theory of friction force is as known "Coulomb Friction," expressed as Equation A.1 *(15)*:

$$F_f = \mu \times N \tag{A.1}$$

where: F_f = the maximum possible force exerted by friction;
μ = the coefficient of friction;
N = the normal force to the contact surface.

The modern understanding of the friction force between a tire and pavement *(16,17)* is that the rubber materials (or tire) govern the friction force while molecular-kinetic thermal processes occur and molecular chains are created against the contact surface, which is the pavement in this case. There are two separate mechanisms involved, hysteresis and adhesion, as expressed in Equation A.2:

$$F_\mu = F_a + F_h \tag{A.2}$$

where: F_μ = friction force;
F_a = adhesion force involved by the interface shear strength and contact area;
F_h = hysteresis force generated from losses of rubber materials damping.

A detailed description of the mechanisms of adhesion and hysteresis force shows that the first component, the adhesion force, is produced by the outermost atoms of the rubber molecules are in direct contact with the outer molecules of the surface *(18)*. Rubber is a polymer, and its molecular structure resembles strings of spaghetti. The surface is crystalline most of the time, with the atoms close together. But when there is a speed difference between the rubber and contact surface, the "strings" in the rubber will be stretched. Some molecular bonds will break, and new ones will be formed. This process repeats itself as one surface moves over the other. Obviously, breaking and stretching molecular bonds takes energy and produces a force. That is the adhesion force. It reaches its maximum when the speed difference between the two surfaces is somewhere between 0.03 and 0.06 meters per second.

The second component, hysteresis, exists because rubber is being deformed. As the tire carcass is being distorted, the rubber gets compressed in some areas, and it gets stretched in other areas. For stretching to be possible, the atoms must move alongside each other. This is an irreversible process because of friction. The friction will make the tire heat up. Again, all this takes energy, and thus gives a force. That force is the hysteresis force, which is very similar to the adhesion force, only its size is determined by the internal friction in the rubber, which is also called the damping loss.

As the weight on the tire and the amount of slip vary, the proportions of the two components change. If the pavement is wet and rough, the hysteresis component will be dominant over adhesion. The water film on the pavement acts as a lubricant, decreasing the adhesion force. The roughness of the surface will cause the tires to continuously deform, which increases the hysteresis force. In contrast, if the pavement is dry and smooth, adhesion will be the dominant force because the rubber can bond to the pavement surface; hysteresis is reduced because the tires do not deform as much on a smooth surface.

Additionally, there is an alternate approach to describe the friction force. Similar to the classic friction force, Kennedy et al. *(19)* referred to the classic frictional force as the horizontal frictional force, F_h, and the normal force as vertical force F_v. Thus, Equation A.2 can be converted to Equation A.3) as follows.

$$F_h = \mu \times F_v \tag{A.3}$$

where: μ = the coefficient of friction.

A.2.2 Factors Affecting Friction

There are a number of factors which influence the frictional properties of HMA pavements. The most important factor is

whether the surface conditions are wet or dry. Pavements under dry conditions will most likely provide appropriate skid resistance. Hence research is mostly focused on the skid resistance under wet conditions when pavements can be extremely slippery. The factors affecting friction are discussed below:

Traffic wear. Shankar *(20)* applied statistical and economic methods to analyze accident frequencies and concluded that higher Annual Average Daily Traffic (AADT) may cause reduced frictional resistance and increase the possibility of fatal accidents. Shupe *(21)* also pointed out that an accumulation of oil, worn rubber and dust particles on the pavement has a significant effect on the friction characteristics.

Water film. When water is present on the road surface, it can reduce the adhesion force of tires; the hysteresis force may also be reduced by the presence of water, but only minutely. Nevertheless, the friction force is reduced by the reduction of adhesion force. Shupe *(21)* indicated that tires can have good interaction with the pavement through a 0.01 in water film but the friction force will greatly diminish if the water film continues developing to greater depths. Kulakowski *(22)* conducted research both in the laboratory and in situ to investigate the effect of water film thickness on tire and pavement friction. The results showed that at 64 km/h (40 mph) as little 0.05 mm (0.002 in) of water can reduce dry surface friction by 20% to 30%. Up to 0.025 mm (0.001 in) of water, the friction can be decreased 75% in the field.

Tire effect. Kennedy *(19)* reported that the adhesion force of rubber tires may increase and the hysteresis force may decrease when the temperature increases. However, the combined effects of both components will lead to lower skid resistance measurements as temperature increases. Shupe *(21)* also indicated that proper tire pressure is necessary to penetrate the water film and maintain adequate friction.

Seasonal variance. Several literatures *(19,21,22,23)* indicated that during dry periods, frictional resistance is dominated by microtexture. But when the road is wet, the pavement macrotexture has a greater effect. For example, Kennedy *(19)* indicated that road surfaces in England are wet only 15% of the summer months (May to September). Under these dry conditions, polishing predominates and causes low skid resistance values. On the other hand, though the roads are wet 60% of the winter, the frictional measurements tend to be as much as 25% higher. Shupe *(21)* gave an explanation of this phenomenon. During the dry period, tires on the highway polish individual exposed pieces of aggregate and produce fine dust. Those dust particles can act as additional lubrication, a so-called "ball-bearing" effect, and may result in a slippery pavement condition. During the wet period, rainfall may wash the dust from the pavement. Therefore, the pavement becomes relatively coarser and increases the skid resistance.

Aggregate properties. Carbonate rocks are the major source of mineral aggregates in the Midwest. Goodwin *(24)* and Aughenbaugh and Lounsbury *(25)* reported there is a belt of Silurian rocks from metropolitan Chicago area to northwestern and east-central Indiana, where limestone ($CaCO_3$) and dolomitic limestone ($Ca(Mg)CO_3$) are quarried.

Shupe *(21)* concluded that some limestone aggregates consisting of pure calcium carbonate should not be used for high volume roads because of their tendency to polish. Other types of carbonate aggregates composed of dolomitic limestone would be expected to provide adequate skid resistance. He also indicated that the best method of predicting the polishing characteristics of an aggregate in a specific mixture is to duplicate the mixture in the laboratory, subject it to an accelerated polishing procedure, and evaluate the change in frictional resistance.

Aughenbaugh and Lounsbury *(25)* investigated the carbonate aggregates in northern part of Indiana. They sampled aggregates from 28 sites and analyzed them by petrographic analysis methods. They reported that aggregates from eastern Indiana had higher Los Angeles abrasion losses and absorption. Another finding was that the variation in the calcium-magnesium ratio had no apparent effect on abrasion losses or absorption test, except as they affected the texture.

West et al. *(23,26)* investigated the friction resistance of aggregates in Indiana. Aggregate coupons were made for the British wheel test (ASTM D3319) *(27)* and British pendulum test (ASTM E303) *(28)*. Results indicated that dolomites blended with slag could provide high friction resistance for high traffic volume roads. Crushed gravel and some specific limestones were also proven acceptable for friction if the aggregate properties could meet standard requirements. Furthermore, for gravels, the frictional resistance correlated well with the freeze and thawing loss test (AASHTO T103) *(29)*, absorption test (ASTM C127) *(30)*, and percentage of crushed gravel and metamorphic rocks; for carbonate aggregates, acid insoluble residue test (ASTM D3042) *(31)* is the most influential factor for limestone; while the absorption test and elemental magnesium (M_g) content test (ASTM C602) *(32)* are the most important evaluation methods for dolomite. However, although a minimum 10.3% elemental Mg content is advised, dolomite with less 10.3% could be also regarded as a potential aggregate for surface courses if the properties of absorption and soundness loss (ASTM C88) *(33)* pass other specifications.

As reported in *NCHRP Synthesis 291*, Henry *(34)* conducted a worldwide survey regarding pavement friction. One of the survey responses about evaluation methods for aggregate polishing revealed that the Los Angles Abrasion test (AASHTO T96) *(35)* is the most commonly used method. The British Wheel test is second, most commonly in Europe. Additionally, Quebec and Slovakia included the Mean Texture Depth (MTD, measured by sand patch test) with British pendulum test for mixture evaluations. In Japan, instead of MTD, the Dynamic Friction Tester (DFT) is used in addition to the British pendulum test to evaluate the friction properties of laboratory mixtures.

Rogers et al. *(36)* concluded that the friction performance is determined by a proper mix design and the use of satisfactory aggregates. They reached similar conclusions as Shupe *(19)* that calcium carbonate rocks are categorized as softer (Mohs hardness between 3 and 3.5) and give significantly lower values in an aggregate friction resistance test. Rogers also suggested and compared several testing methods to estimate wear-resistance (indicating macrotexture) and polish resistance (indicating microtexture). He suggested the Aggregate Abrasion Value test (AAV) (BS 812), LA abrasion test, and Micro-Deval abrasion (AASHTO T327) *(37)* are good indicators of aggregate wearing resistance; while the Polished Stone Value test (PSV) (BS 812) is a suitable tool to evaluate polish resistance. They also found that good AAV value coincides with a low LA abrasion weight loss. However, an aggregate with high LA abrasion loss might still retain good resistance to abrasion. It was implied that LA abrasion is not a reliable test. Results from Micro-Deval tests generally agree with the AAV. However, AAV is more time consuming and expensive compared to the Micro-Deval test. Cooley *(38)* and Prowell *(39)* also have similar conclusions that results from the LA abrasion and Micro-Deval tests might give opposite answers about the frictional resistance of aggregates.

The Missouri DOT (MoDOT) requires both LA abrasion and Micro-Deval tests specifically for RAP aggregate examinations. RAP material with a Micro-Deval loss of more than 20% will not be accepted in order to ensure the aggregate quality and prevent moisture damage. If both virgin and extracted aggregates are possibly coming from the same source, the loss of the extracted aggregate from the RAP should not be 5% more than of loss of the virgin aggregate.

Liang *(40)* conducted research about blending high and low skid aggregates. The acid insoluble residue test revealed that the higher the acid insoluble residue, the higher the skid resistance. The Sodium Sulfate Soundness test also indicated that lower soundness loss implied better frictional properties. In that study, a 50/50 blend of high and low skid resistance aggregate met the frictional requirements. But he suggested that blending 60/40 of high and low skid resistance aggregate might be more acceptable in general.

In addition, Dames *(41)* observed that frictional resistance depends not only on the mineralogical properties of aggregate but also on the grain size and distribution, or the microtexture. Kandhal and Parker *(42)* noted that measurements of microtexture on coarse aggregates may not be an efficient means of evaluating the friction resistance. Doty *(43)* reported on a

comparison between friction and surface texture, as measured by the sand patch test and outflow meter. There was a general trend of higher friction with increasing texture depth for a variety of surface types including open and dense graded asphalt, sealed surfaces, and polished and grooved PCC. Surface texture alone, however, did not yield a strong enough relationship to establish a minimum texture depth criterion for use as a specification limit.

Mixture type. In previous research *(44)* conducted by the North Central Superpave Center (NCSC), friction properties of conventional dense-graded HMA, SMA and Porous Friction Course (PFC) were investigated and evaluated in the field. The PFC was composed of 90% steel slag with 10% sand; the SMA consisted of 80% steel slag, 10% stone sand (from a different source than the PFC sand) and 10% mineral filler; the HMA was made of the same source of steel slag blended 50/50 percent with coarse dolomite. This research revealed that the PFC provided the highest friction value, followed by the SMA. Both the PFC and SMA had substantially higher friction values than the conventional HMA even though they were tested before opening the road to traffic. The friction values for the PFC and SMA would be expected to increase after traffic wears away the binder film coating on aggregate particles.

Macro- and microtexture. Yager et al. *(45)* investigated the role of pavement macrotexture in draining airport runways. They note that macrotexture is very important, but it alone could not define the frictional properties of the pavement. Kulakowski et al. *(22)* emphasized the importance of macrotexture by reporting that a thin layer of water on the surface could lead to a significant reduction in friction on the order of 20% to 30% of the dry friction.

Forster *(46)* reported a correlation between skid resistance, as indicated by British Portable Tester numbers (BPN) measured by British Pendulum Tester (*microtexture*), and the texture properties measured by the Sand Patch test (*macrotexture*). An image analysis system was adopted to understand and determine optimal macro and microtexture parameters. He concluded that the overall texture had a significant influence on skid resistance measurements.

Today, it is generally agreed that the pavement friction property depends on both macro- and microtexture. An international standard for road surface texture terminology has been established by the Technical Committee on Surface Characteristics of the World Road Association's "Permanent International Association of Road Congress" (PIARC) *(47)*, as follows:

Megatexture: Wavelength = 50 mm to 500 mm (2 to 20 in)
Macrotexture: Wavelength = 0.5 mm to 50 mm (0.02 to 2 in)
Microtexture: Wavelength = 1 μm to 0.5 mm (0.0004 to 0.02 in)

If both macro- and microtexture are maintained at high levels, they can provide enough resistance to prevent wet accidents. Kennedy *(19)* indicated that microtexture dominates at speeds up to 50 kph (31 mph). For wet pavement friction, macrotexture helps to provide drainage channels for water to escape, and microtexture breaks the last thin film of water coating the aggregate particles to allow aggregate-tire contact *(48)*.

A.2.3 Methods for Measuring Friction

Locked wheel device. Wet pavement friction measurements can be obtained by using the ASTM E274 *(49)* towed friction trailer. The ASTM towed friction trailer allows two types of tires for friction evaluations including the Standard Rib Tire for Pavement Skid-Resistance Test (ASTM E501) *(50)* and Standard Smooth Tire for Pavement Skid-Resistance (ASTM E524) *(51)*. The Indiana Department of Transportation (INDOT) routinely uses the blank or smooth test tire on the trailer, shown as Figure A.1. A locked tire with 24 psi (165 kPa) of pressure sliding on a wetted surface, under a constant speed and load, is used to measure the steady-state friction force. When the towed trailer reaches the standard test speed of 40 mph (64 km/h), the brake is locked after the watering system provides a water film of 0.02 in (0.5 mm). The

Figure A.1 ASTM E-274 towed trailer of INDOT.

friction data is reported as the Skid Number or Friction Number (SN40).

Several studies have shown that the friction measured with the smooth tire is related to both the macrotexture and microtexture of the pavement *(52,53)*. However, Henry *(34)* reported that most states preferred the rib-tire instead of the smooth tire. The possible reasons could be that the frictional value measured with the smooth tire is much lower than the ribbed tire and there are difficulties comparing with historical data if the tire is changed from previous practice.

Measurement of macrotexture. The traditional method for macrotexture measurement is the sand patch test (ASTM E965) *(54)*. The method consists of spreading a fixed volume of dry Ottawa sand or glass spheres over the surface and working them into the surface texture in a circular pattern. The sand is spread until it is flush with the tops of any surface asperities. The area covered by the sand and the known volume of sand allow calculation of the average texture depth, called the Mean Texture Depth (MTD). The method and equipment are simple, but significant variability (poor repeatability) in the measurements has been reported. In addition, only an average texture depth can be obtained. No further analysis of the nature of that texture depth can be accomplished.

The Circular Texture Meter (CTM), shown in Figure A.2, is an advanced way to measure pavement macrotexture. The Mean Profile Depth (MPD) of a pavement surface can be measured with the CTM. Prowell et al. *(52)* observed that the CTM produced comparable macrotexture results to the sand patch method on the National Center for Asphalt Technology (NCAT) Test Track. However, the CTM is easier for the technician to operate and has less operator error than the sand patch method. The CTM, described as ASTM E2157, uses a Charge Coupled Device (CCD)

Figure A.2 Circular Texture Meter (CTM).

laser displacement sensor to measure the surface profile. The laser sensor is mounted on an arm that rotates around a central point at a fixed distance above the pavement and measures the change in elevation of points on the surface. The laser spot size is 70 mm and the vertical resolution is 3 mm. Each test takes about 40 to 45 seconds *(53,55)*. The CTM profile can be analyzed to determine more about the nature of the texture. One advantage of this method is that eight separate arcs of the circle can be analyzed.

Measurement of microtexture. Microtexture, on the other hand, can be measured in the field or the laboratory using the device such as the British Pendulum Tester or the Dynamic Friction Tester (DFT). The British pendulum has been used for many years; however, it yields more variable results and requires more skilled personnel than the DFT.

As shown in Figure A.3, the DFT is a portable device that allows direct measurement of the surface friction of a variety of surfaces, including pavements. Described in ASTM E1911, the DFT consists of a horizontal spinning disk fitted with three spring-loaded rubber sliders that contact the paved surface. The standard sliders are made of the same type of rubber used in friction test tires, though other materials are available for other applications. The disk rotates at tangential velocities up to 80 kph (55 mph). Water flows over the surface being tested, so wet friction is measured as done with the towed friction trailer. The rotating disk is then dropped onto the wet surface and the friction is continuously measured as the disk slows. This continuous measurement allows determination of the speed dependency of the surface friction *(53,55)*. The DFT is relatively small, approximately 511 mm (20.1 in) square and weighing about 11 kg. The tested area is a circular path with a diameter of about 284 mm (11.2 in). A small tank is used to provide water and a personal computer is used for control of the test and data acquisition.

A.2.4 Calculations of International Friction Index (IFI)

Henry et al. *(56)* found that International Friction Index (IFI) can be determined by combining the measurements from the DFT and CTM. IFI was developed in Europe to harmonize friction measurements made in various countries and measured by any of number of different devices. The IFI allows these various measurements to be reported in common measurement terms.

There are three steps to determine the IFI:

1. The speed constant (S_p) is a function of the pavement macrotexture and can be defined by following equation:

$$S_p = a + b \cdot TX$$

where TX is the pavement macrotexture and a and b are constants depending on how the macrotexture is measured.

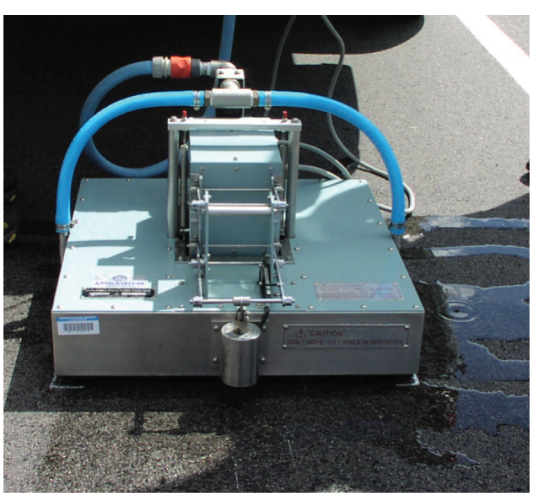

Figure A.3 Dynamic Friction Tester (DFT) in use.

2. The friction number $FR60$ is the adjusted value at a slip speed of 60 km/h converted by FRS, the friction measurement reported by friction measurement device at slip speed S:

$$FR60 = FRS \cdot e^{\frac{S-60}{Sp}}$$

3. Friction number (F_{60}) is defined as

$$F_{60} = A + B \cdot FRS \cdot e^{\frac{S-60}{Sp}}$$

where, A and B are constants based on specific friction measurement device.

For the CTM and DFT, MPD (macrotexture) is used to determine the S_p as:

$$S_p = 14.2 + 89.7 \cdot MPD$$

Additionally, DFT20, which means the friction measurement (microtexture) conducted by DFT at slip speed 20 km/h, is recommended for predicting the F_{60} with the highest correlation between friction measurements of BPN and DFT20. Therefore, the friction number (F_{60}) can be obtained by:

$$F_{60} = 0.081 + 0.732 \times DFT_{20} \times \exp(-40/S_p)$$

REFERENCES

1. McDaniel, R. S., H. Soleymani, and A. Shah. *Use of Reclaimed Asphalt Pavement (RAP) Under Superpave Specifications: A Regional Pooled Fund Project*. Publication FHWA/IN/JTRP-2002/6. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2002. doi: 10.5703/1288284313465.
2. National Asphalt Pavement Association. *Recycling Hot Mix Asphalt Pavements*. Information Series 123 (IS-123). National Asphalt Pavement Association, Lanham, Maryland, 2007.
3. Asphalt Institute. *Asphalt Hot-Mix Recycling (MS-20)*. Asphalt Institute, College Park, Maryland, 1986.
4. Solaimanian, M. T., and T. W. Kennedy. Production Variability Analysis of Hot-Mix Asphalt Concrete Containing Reclaimed Asphalt Pavement. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1543, Transportation Research Board of the National Academies, Washington, D.C., 1996, pp. 89–96.
5. Kandhal, P. S., K. Y. Foo, and J. A. D'Angelo. Control of Volumetric Properties of Hot-Mix Asphalt by Field Management. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1543, Transportation Research Board of the National Academies, Washington, D.C., 1996.
6. Stroup-Gardiner, M., and C. Wagner. Use of Reclaimed Asphalt Pavement in Superpave Hot Mix Asphalt Applications. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1681. Transportation Research Board of the National Academies, Washington, D.C., 1999, pp. 1–9.
7. Mayes, G., M. McCain, and T. Murphy. RAP Merges with Superpave Asphalt. *Asphalt*, Spring/Summer, 1998, Vol. 12, No. 2, pp. 24–26.
8. Nady, R. M. The Quality of Random RAP: Separating Fact from Supposition. *HMAT Magazine*, Vol. 2, No. 2, 1997, pp. 14–17.
9. Kandhal, P. S. Recycling of Asphalt Pavements—An Overview. *Journal of the Association of Asphalt Paving Technologists*, Vol. 66, 1997, 686–696.

10. McDaniel, R. S., and R. M. Anderson. *Recommended Use of Reclaimed Asphalt Pavement in Superpave Mix Design Method: Technician's Manual*. NCHRP Report No. 452, NCHRP Project D9-12. Transportation Research Board of the National Research Council. National Academy Press, Washington, D.C., 1997.
11. AASHTO M323. *Standard Specification for Superpave Volumetric Mix Design*. American Association of State Highway and Transportation Officials.
12. Bell, C. A., M. J. Fellin, and A. Weider. Field Validation of Laboratory Aging Procedures for Asphalt Aggregate Mixtures.*Journal of the Association of Asphalt Paving Technologists*, Vol. 63, 1994, pp. 45–80.
13. AASHTO R30. *Standard Practice for Mixture Conditioning of Hot-Mix Asphalt (HMA)*. American Association of State Highway and Transportation Officials.
14. Kuemmel, D. A., R. C. Sontag, J. A. Crovetti, Y. Becker, J. R. Jaeckel, and A. Satanovsky. *Noise and Texture on PCC Pavements-Results of a Multi-State Study*. Publication WI/SPR-08-99, WisDOT Highway Research Study No. 93-05, SPR No. 0092-45-91. Wisconsin Department of Transportation, 2000.
15. Tipler, P. *Physics for Scientists and Engineers: Vol. 1*, 4th ed. W. H. Freeman and Company, New York, 1998.
16. Kummer, H. W. United Theory of Rubber and Tire Friction. *Engineering Research Bulletin B-94*. The Pennsylvania State University, State College, Pennsylvania, 1966.
17. French, T. *Tire Technology*. Adam Hilger, Bristol, United Kingdom, 1989.
18. Heremans, B. Chapter 1: Tires. In *R/C Car Handling*. 2001. http://home.scarlet.be/~be067749/58/. Accessed Feb. 10, 2007.
19. Kennedy, C. K., A. E. Young, and I. C. Butler. Measurement of Skidding Resistance and Surface Texture and the Use of Results in the United Kingdom. In *ASTM STP 1031: Surface Characteristics of Roadways: International Research and Technologies*. Meyer, W. E., and J. Reichert, Eds. American Society for Testing and Materials, Philadelphia, 1990, pp. 87–102.
20. Shankar, V. N., F. L. Mannering, and W. Barfield. Effect of Roadway Geometrics and Environmental Factors on Rural Freeway Accident Frequencies. *Accident Analysis and Prevention*, Vol. 27, No. 3, 1995, pp. 371–389, doi: 10.1016/0001-4575(94)00078-Z.
21. Shupe, J. W. Section 20: Pavement Slipperiness. In *Highway Engineering Handbook*, Woods, K. B., D. S. Berry, and W. H. Goetz, Eds. McGraw-Hill Book Co. Ltd., Columbus, Ohio, 1960, pp. 1–27.
22. Kulakowski, B. T., and D. W. Harwood. Effect of Water-Film Thickness on Tire-Pavement Friction. In *ASTM STP 1031: Surface Characteristics of Roadways: International Research and Technologies*. Meyer, W. E., and J. Reichert, Eds. American Society for Testing and Materials, Philadelphia, 1990, pp. 50–60.
23. West, T. R., J. C. Choi, D. W. Bruner, H. J. Park, and K. H. Cho. Evaluation of Dolomite and Related Aggregates Used in Bituminous Overlays for Indiana Pavements. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1757. Transportation Research Board of the National Academies, Washington, D.C., 2001, pp. 137–147.
24. Goodwin, J. H. Geology of Carbonate Aggregate Resources of Illinois. In *Illinois Mineral Notes 87*. Department of Energy and Natural Resources, Illinois State Geological Survey, Champaign, Illinois, 1983.
25. Aughenbaugh, N. B., and R. W. Lounsbury. Petrographic Analysis of Northern Indiana Carbonate Aggregates. *The Ohio Journal of Science*, Vol. 66, No. 2, 1966, pp. 179–187.
26. West, T. R., and K. H. Cho. *Development of a Procedure to Identify Aggregate for Bituminous Surfaces in Indiana*. Publication FHWA/IN/JTRP-2000/28. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2000. doi: 10.5703/1288284313186.
27. ASTM D3319. *Standard Practice for Accelerated Polishing of Aggregates Using the British Wheel*. ASTM International, West Conshohocken, Pennsylvania.
28. ASTM E303. *Standard Test Method for Measuring Surface Frictional Properties Using the British Pendulum Tester*. ASTM International, West Conshohocken, Pennsylvania.
29. AASHTO T103. *Standard Method of Test for Soundness of Aggregates by Freezing and Thawing*. American Association of State Highway and Transportation Officials.
30. ASTM C127. *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate*. ASTM International, West Conshohocken, Pennsylvania.
31. ASTM D3042. *Standard Test Method for Insoluble Residue in Carbonate Aggregates*. ASTM International, West Conshohocken, Pennsylvania.
32. ASTM C602. *Standard Specification for Agricultural Liming Materials*. ASTM International, West Conshohocken, Pennsylvania.
33. ASTM C88. *Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate*. ASTM International, West Conshohocken, Pennsylvania.
34. Henry, J. J. *Evaluation of Pavement Friction Characteristics*. NCHRP Synthesis No. 291. Transportation Research Board of the National Research Council. National Academy Press, Washington, D.C., 2000.
35. AASHTO T96. *Standard Method of Test for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*. American Association of State Highway and Transportation Officials.
36. Rogers, C., B. Gorman, and B. Lane. Skid-Resistant Aggregates in Ontario. In *10th Annual Symposium Proceedings, International Center for Aggregates Research*. Baltimore, 2002.
37. AASHTO T327. *Standard Method of Test for Resistance of Coarse Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus*. American Association of State Highway and Transportation Officials.
38. Cooley, L. A. Jr., M. S. Huner, and R. S. James. *Micro-Deval Testing of Aggregates in the Southeast*. NCAT Report 02–09. National Center for Asphalt Technology, Auburn University, Auburn, Alabama, 2002.
39. Prowell, B. D., J. Zhang, and E. R. Brown. *Aggregate Properties and the Performance of Superpave-Designed Hot Mix Asphalt*. National Cooperative Highway Research Program Report No. 539. Transportation Research Board of the National Academies, Washington, D.C., 2005.
40. Liang, R. *Blending Proportions of High Skid and Low Skid Aggregate*. Publication FHWA/OH-2003/014. Ohio Department of Transportation, Akron, Ohio, 2003.
41. Dames, J. The Influence of Polishing Resistance of Sand on Skid Resistance of Asphalt Concrete. In *ASTM STP 1031: Surface Characteristics of Roadways: International Research and Technologies*, Meyer, W. E., and J. Reichert, Eds. American Society for Testing and Materials, Philadelphia, 1990, pp. 14–29.

42. Kandhal, P. S., and F. Parker, Jr. *Aggregate Tests Related to Asphalt Concrete Performance in Pavements*. NCHRP Report 405. Transportation Research Board of the National Research Council. National Academy Press, Washington, D.C., 1998.
43. Doty, R. N. Study of the Sand Patch and Outflow Meter Methods of Pavement Surface Texture Measurements. In *ASTM STP 583: Surface Texture Versus Skidding: Measurements, Frictional Aspects, and Safety Features of Tire-Pavement Interaction*. American Society for Testing and Materials, Philadelphia, 1975, pp. 42–60.
44. McDaniel, R. S. , W. D. Thornton, and J. G. Dominguez. *Field Evaluation of Porous Asphalt Pavement*. Publication Final Report SQDH 2004-3. North Central Superpave Center, Purdue University, West Lafayette, Indiana, 2004.
45. Yager, T. J., and F. Buhlmann. Macrotexture and Drainage Measurements on a Variety of Concrete and Asphalt Surfaces. In *ASTM STP 763: Pavement Surface Characteristics and Materials*. Hayden, C. M., Ed. American Society for Testing and Materials, Philadelphia, 1982, pp. 16–30.
46. Forster, S. W. Pavement Microtexture and Its Relation to Skid Resistance. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1215. Transportation Research Board, Washington, D.C., 1989, pp. 151–164.
47. World Road Association (PIARC). *Report of the Committee on Surface Characteristics*. XVIII World Road Congress, Brussels, Belgium, 1987.
48. Deway, G. R., A. C. Robords, B. T. Armour, and S. R. Looman. Aggregate Wear and Pavement Friction. In *TRB Annual Meeting CD-ROM*. Transportation Research Board, Washington D.C., 2001.
49. ASTM Standard E274. *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*. ASTM International, West Conshohocken, Pennsylvania.
50. ASTM Standard E501. *Standard Specification for Standard Rib Tire for Pavement Skid-Resistance Tests*. ASTM International, West Conshohocken, Pennsylvania.
51. ASTM Standard E524. *Standard Specification for Standard Smooth Tire for Pavement Skid-Resistance Tests*. ASTM International, West Conshohocken, Pennsylvania.
52. Prowell, B. D., and D. I. Hanson. Evaluation of Circular Texture Meter for Measuring Surface Texture of Pavements. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1929. Transportation Research Board, Washington, D.C., pp. 88–96.
53. Nippo Sangyo Co., Ltd. *Product Guide, Circular Texture Meter, ASTM E-2157-01*. http://www.nippou.com/en/products/ct.html. Accessed March 23, 2006.
54. ASTM Standard E965. *Standard Test Method for Measuring Pavement Macrotexture Depth Using a Volumetric Technique*. ASTM International, West Conshohocken, Pennsylvania.
55. Saito, K., T. Horiguchi, A. Kasahara, H. Abe, and J. J. Henry. Development of Portable Tester for Measuring Skid Resistance and Its Speed Dependency on Pavement Surfaces. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1536. Transportation Research Board of the National Academies, Washington, D.C., 1996, pp. 45–51.
56. Henry, J. J., H. Abe, S. Kameyama, A. Tamai, A. Kasahara, and K. Saito. Determination of the International Friction Index (IFI) Using the Circular Texture Meter (CTM) and the Dynamic Friction Tester (DFT). Publication No. 109.01.06.B-2000. Permanent International Association of Road Congresses (PIARC), Nantes, France, 2000.

APPENDIX B

FRICTION TESTING AND MODEL

In order to determine the frictional properties of the various mixtures, a test procedure developed in another study, *Identification of Laboratory Technique to Optimize Superpave HMA Surface Friction Characteristics* (JTRP Report No. FHWA/IN/JTRP-2010/6) *(1)*, was utilized. This procedure is briefly described here.

First, slabs are fabricated from the mixture to be tested. Laboratory-produced HMAs are reheated to the compaction temperature. Based on the volume of the mold and the specific gravity of the mix, the approximate weight of mix that would yield 7% to 8% air voids (V_a) is determined. That amount of mix is then placed in a square wooden mold (500 mm [20 in] by 500 mm [20 in] and 38 mm [1.5 in] deep) and compacted using a large "rolling pin" mounted on a fork lift. Once compacted, the slabs are allowed to cool thoroughly.

Following compaction, the slabs are subjected to polishing and their frictional properties are periodically measured. Polishing is performed using a device called a Circular Track Polishing Machine (CTPM), shown in Figure B.1. This device consists of three rubber tires attached to a rotating plate. The wheels travel over the same footprint as that of the devices used to measure friction and texture (described below). The polishing wheels travel at approximately 47 rotations per minutes (RPMs). Since each revolution rotates three tires over the same track on the surface, there are about 141 cumulative wheel passes per minute. Water is sprayed on the slab surface to help remove the debris generated during polishing. During polishing, a total load of 0.65 kN is applied through the tires to the surface.

Before polishing is initiated and periodically during polishing, the surface texture and friction of the slabs are measured. The surface texture is measured using a laser-based Circular Track Meter (CTM), following ASTM E2157 *(2)*. The texture is reported in terms of the Mean Profile Depth (MPD) and measured in millimeters. Then, the friction of the surface is measured using a Dynamic Friction Tester (DFT), following ASTM E1911 *(3)*. In the DFT device, three rubber sliders attached to the disk are accelerated to tangential velocities of up 90 km/h (56 mph) and then dropped onto the surface. The torque generated as the disk slows provides an indication of the friction at various speeds. The main value of interest here is the DFT number at 20 km/h (12 mph), designated DF20. The previously determined MPD value can be combined with the DF20 value and used to calculate the International Friction Index (IFI) following ASTM E1960 *(4)*. The IFI consists of two parameters: the calibrated wet friction at 60 km/h (F60) and the speed constant of wet pavement friction (S_p).

Figure B.1 Circular Track Polishing Machine (CPTM).

The polisher is stopped periodically during testing so the measurement of friction and texture can be performed. In this study, this was done after the following cumulative numbers (in thousands) of wheel passes: 1.5, 3.6, 9, 18, 30, 45, 75, 120 and 165.

Typically, for asphalt mixtures the initial friction tends to be low because of the presence of binder film coating the aggregate particles. After the binder film is worn off by traffic, the friction increases rapidly. Continued wheel passes tend to cause a decrease in the friction level, and sometimes changes in the texture, as the aggregate particles undergo polishing and sometimes are dislodged (ravel). Eventually, the friction tends to level off at the so-called terminal friction value. This occurs when embedded aggregates at the surface are polished as much as they will polish and further wheel passes do not cause additional loss of friction. This general trend in friction is observed both in the field and in the lab. Past research work has shown that terminal friction can usually be obtained in the CTPM after fewer than 165,000 wheel passes (55,000 CTPM revolutions), even for mixtures with high friction aggregates like steel slag.

In addition to the MPD, the DF20 parameter is also determined after each increment of polishing cycles. These two parameters are used to calculate the calibrated wet friction (F60) values (following the ASTM E1960) *(4)*, as shown below:

$$F60 = 0.81 + 0.732 DF_{20} e^{\frac{-40}{Sp}} \quad (1)$$

$$S_p = 14.2 + 89.7 MPD \quad (2)$$

where: DF_{20} = wet friction number measured at the speed of 20 km/h
MPD = mean profile depth (mm).

When using Equation 1 with the typical range of MPD values (0.3 mm to 1.7 mm) and DF_{20} values (0.3 to 0.7), it can be noted that the F60 parameter is highly influenced by the DF_{20}. The trend of the plot of DF_{20} versus number of wheel passes is typically similar to the plot of F60 versus number of wheel passes. An example of the typical changes in the DF_{20} values taking place during polishing is shown in Figure B.2 for the SMA mixture with 0% RAP content.

In order to quantify changes in the F60 values taking place during polishing and to evaluate the frictional properties of the mixture, a polishing model developed in previous research (JTRP Report 2010/06) *(1)* is used in this study. This model allows for estimation of the terminal friction level (referred to as a F60@X1) and the polishing rate (a_4). (NOTE: X1 represents the number of wheel passes at which the terminal friction level is reached.)

The model has a general form shown in Figure B.3 and in Equations 3, 4 and 5 *(9)*.

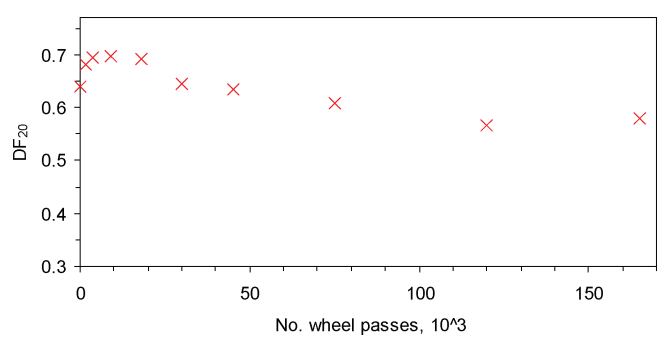

Figure B.2 Dynamic friction (DF_{20}) data for SMA mixture with 0% RAP.

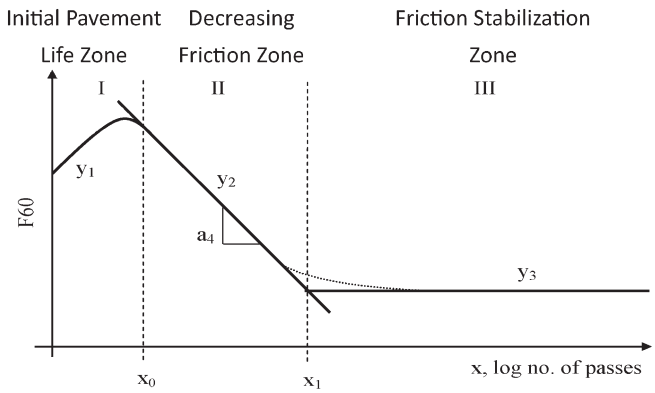

Figure B.3 Polishing model. (Modified from Kowalski, K. J., R. S.McDaniel, and J. Olek. Identification of Laboratory Technique to Optimize Superpave HMA Surface Friction Characteristics. Publication FHWA/IN/JTRP-2010/6. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2010. doi: 10.5703/1288284314265.)

$$y_1(\mathrm{x}) = a_0 + a_1 x + a_2 x^2 + a_3 x^3 \qquad 0 < x \le x_0 \tag{3}$$

$$y_2(x) = a_4 \ln(x + 1 - x_0) + a_5 \qquad x_0 < x \le x_1 \tag{4}$$

$$y_3 = a_4 \ln(x_1 + 1 - x_0) + a_5 = const. \qquad x > x_1 \tag{5}$$

The parameters a_0 to a_5 can be found by minimizing the sum of square errors (SSE), assuming that the minimum SSE would result in the model that best fits the measured data:

$$SSE = \sum (data\ predicted\ by\ the\ model - measured\ data)^2 \tag{6}$$

The model generated for the tested mixes yielded relatively high coefficient of determination (R^2) values with an average of 0.87. Of the eight mixes with laboratory-produced poor quality RAP tested, one mix had an R^2 value equal to 0.73, another one was 0.79 and the rest of them had R^2 values greater than 0.8. In general, the higher (less negative) the a_4 value is, the more resistant the specimen is to polishing. A high F60 value corresponds to high terminal friction value for the pavement.

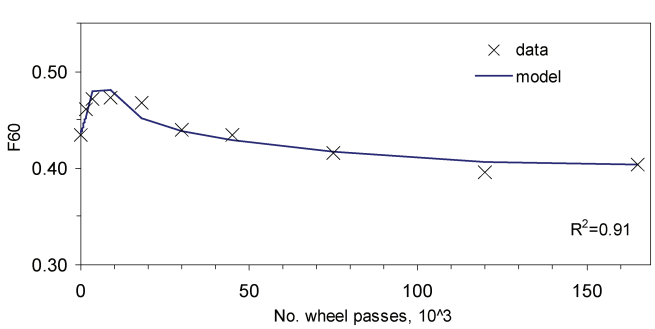

Figure B.4 Calibrated wet friction (F60) data and model predicted F60 values for SMA mixture with 0% RAP.

An example of the typical changes in the F60 values taking place during polishing is shown in Figure B.4 for the SMA mixture with 0% RAP content. On the same figure, the model fitting the data with $R^2 = 0.91$ is also shown. It can be observed that after about 130,000 wheel passes the changes in the F60 value were relatively small, suggesting that the specimen had reached its terminal friction level.

RECALIBRATION OF THE DFT

During the course of this study, problems developed with the DFT and service was required. After servicing, the device was recalibrated by the DFT technician. When the DFT was returned to the NCSC and testing resumed, a marked difference in the DFT readings was noted. Unfortunately, despite being asked to take readings on slabs before and after servicing without applying additional polishing passes, the technician assisting at the time did not do so. Consequently, another way to relate the readings before and after servicing was required.

In support of other studies, periodic testing of the INDOT Test Track was performed with the CTM and DFT to allow correlation of those devices with the towed friction trailer. CTM and DFT readings were taken on the same day that the towed friction trailer calibration was checked. While these values show seasonal variation from one set of readings to another, since the CTM/DFT readings were taken on the same day as the towed friction trailer, these differences can be ignored. Readings were taken on the asphalt section, the tined concrete and the slick concrete to allow comparison over a range of friction levels. In addition, tests were conducted with both the rib and smooth tires on the towed friction trailer.

In order to relate the DFT readings taken before and after servicing, then, the CTM and the serviced DFT were used to test the track in August 2011, and these readings were compared to the towed friction trailer data. This comparison showed that the DFT values changed by a differing amount depending on the level of friction. On the slick concrete section, which provides very low friction, the change in DFT value was around 0.11. On the tined concrete, which provides a high level of friction, the change was about 0.40. On the asphalt section, which provides an intermediate level of friction, the change in DFT values was also intermediate—around 0.20. The DFT readings after servicing were lower than before servicing.

All of the slabs with the lab fabricated worst case RAP were tested before servicing of the DFT.

The SMA slabs with actual RAP samples were also tested before the repair. On the DGA slabs with actual RAPs, the DFT problems were noted after the initial readings had been recorded, so these (the subsequent readings) are the only values that need to be corrected.

Since the friction flag value was established in earlier research by correlation of the towed trailer to the DFT/CTM before problems

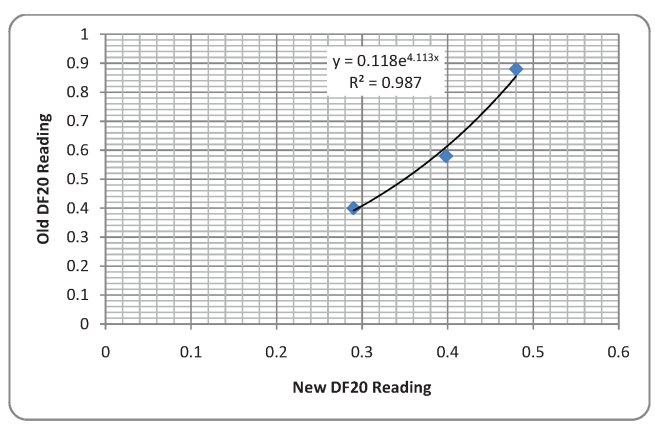

Figure B.5 Shift factor to correct DFT readings taken after repair of equipment.

developed with the DFT, it was determined that the readings in the present study taken after servicing should be "corrected" to the readings before servicing. So, the DFT readings taken in the present study after recalibration will be corrected by a shift factor that will increase them to be comparable with the readings taken before servicing. Figure B.5 shows the pre-servicing DFT readings versus the post-servicing DFT readings. The best fitting trend line (giving the highest R-squared value) is an exponential line. Therefore, an exponential shift factor corresponding to the measured DFT value will be used to give a "corrected" DFT value similar to those measured before the repair.

The newly calibrated equipment is very likely giving correct readings now but the flag value to which we compare the readings was developed before the recalibration of the equipment. The "corrected" readings compare well to previous measurements, giving some confidence that the adjustment is reasonable. Future research should be proposed to refine the laboratory friction testing and polishing protocol. Topics to be addressed in that research could include equipment calibration, reevaluation of the flag values, improved slab compaction procedures and improvements to the polishing procedures (such as looking at different downward forces to reduce the tendency to cause raveling of the surfaces).

REFERENCES

1. Kowalski, K. J., R. S. McDaniel, and J. Olek. *Identification of Laboratory Technique to Optimize Superpave HMA Surface Friction Characteristics*. Publication FHWA/IN/JTRP-2010/6. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2010. doi: 10.5703/1288284314265.
2. ASTM Standard E2157. *Standard Test Method for Measuring Pavement Macrotexture Properties Using the Circular Track Meter*. ASTM International, West Conshohocken, Pennsylvania.
3. ASTM Standard E1911. *Standard Test Method for Measuring Paved Surface Frictional Properties Using the Dynamic Friction Tester*. ASTM International, West Conshohocken, Pennsylvania.
4. ASTM Standard E1960. *Standard Practice for Calculating International Friction Index of a Pavement Surface*. ASTM International, West Conshohocken, Pennsylvania.

APPENDIX C

EXPERIMENTAL DESIGN

This appendix outlines the experimental design for the research project, including the design to evaluate the frictional properties of the laboratory-produced RAP and the verification of proposed threshold RAP contents using six RAPs sampled from hot mix plants around the state of Indiana.

EXPERIMENTAL MATRIX FOR LABORATORY-PRODUCED "WORST CASE" RAP

Slabs were compacted, polished and tested for frictional properties (using the DFT and CTM). RAP contents ranged from 0% (control) to 40% in both DGA and SMA mix types, as shown below. The laboratory-produced poor frictional quality RAP was used.

TABLE C.1
"Worst Case" RAP Contents Used in Experimental DGA and SMA Mixtures

RAP Content	0%	15%	25%	40%
DGA Mix	×	×	×	×
SMA Mix	×	×	×	×

EXPERIMENTAL MATRIX FOR ACTUAL FIELD-SAMPLED RAPS

To verify the threshold values suggested by testing the laboratory-produced RAP, slabs were fabricated using six different actual RAP materials sampled from six hot mix plants around the state at 25% and 40% RAP, as illustrated below. Slabs were polished and tested for frictional properties. In addition, Superpave gyratory compacted specimens were fabricated and tested for their low temperature cracking resistance.

TABLE C.2
RAP Sources, Contents and Mixture Types Used in Verification Testing

RAP Source	25% RAP	40% RAP	Used in
(1) Central Indiana	×	×	DGA
(2) Northwestern Indiana	×	×	SMA
(3) West Central Indiana	×	×	DGA
(4) West Central Indiana	×	×	DGA
(5) Southern Indiana	×	×	SMA
(6) Northeastern Indiana	×	×	DGA

APPENDIX D

MATERIALS AND MIX DESIGNS

The compositions of the actual RAP samples, the laboratory fabricated RAP, and the DGA and SMA mixtures are described below.

RAP SAMPLE COMPOSITION

RAP samples collected from various RAP stockpiles around Indiana were extracted following ASTM D2172, test method B (AASHTO designation T164) *(1)* to determine the aggregate gradation and binder content. The RAP stockpiles consisted of material milled off of various state and local contracts, which was then crushed and screened through a 19 mm sieve. The RAP aggregate gradations, determined according to AASHTO T30 *(2)*, are shown in Figure D.1. Analysis of the data in Figure D.1 indicates that the gradations were fairly similar and consistent, even though the RAP samples were collected from HMA plants widely dispersed across the state of Indiana. The greatest variation in the gradation occurred on the 2.36 mm (No. 8) sieve, where the maximum difference between the various RAPs was equal to 16%. It can also be noticed that gradation of RAP_1 is slightly different (finer) than the gradations for other five RAPs. The binder content for RAP_1 is equal to 6.6% and is also a little higher than the binder contents for the other samples (which were between 4.7% and 5.9% with an average of 5.5%).

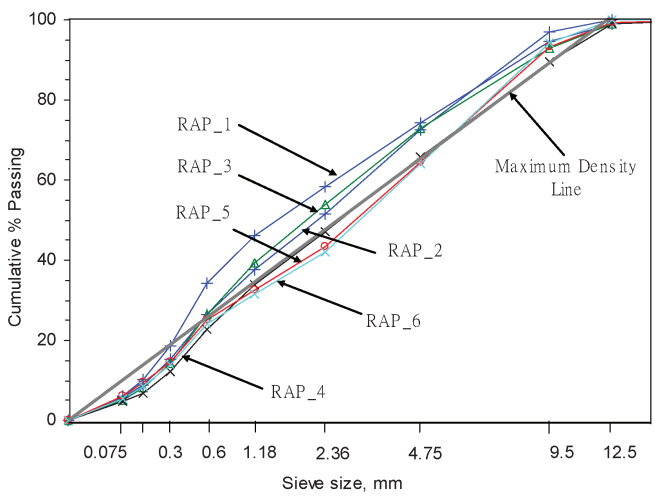

Figure D.1 Gradation of six field-sampled RAP sources.

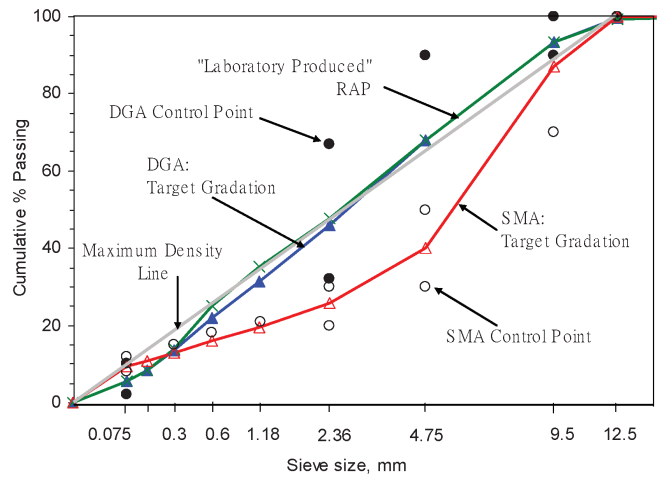

Figure D.2 Gradations of SMA and DGA mixtures and of laboratory-produced RAP.

SMA AND DGA MIX DESIGNS

Four SMA and four DGA mixes, each with four levels of RAP content (0%, 15%, 25% and 40%), were designed and tested in this study. The mixes were designed based on several example Indiana DMFs for 9.5 mm Nominal Maximum Aggregate Size (NMAS) mixes. The target gradations of the SMA and DGA blends are shown in Figure D.2. The laboratory-produced RAP was blended with steel slag (SS) for the SMA mixes and with air cooled blast furnace (ACBF) slag for the DGA mixes. (It should be noted that the ACBF slag is not currently allowed for SMA surface mixes in Indiana.) In addition, a mineral filler (lime) as well as cellulose fibers (in the amount of 0.3% by weight) were also added to the SMA.

A PG 64-22 binder was used to produce all mixes. It is important to note that the binder grade was not changed for the entire range for RAP percentages added (i.e., no "grade bumping" was performed). Also, PG 64-22 binder is not the typical grade used for SMA mixtures in Indiana; a (nominally) softer binder was used here in order to reduce the compactive effort needed to fabricate the test specimens for friction measurements.

The design binder content for each mix was that which provided 4% air voids (V_a) in the mix compacted in the Superpave Gyratory Compactor (SGC). The compaction effort used (N_{design}) was equal to 100 gyrations for the SMA and 125 for the DGA mixtures. This compactive effort corresponds to an anticipated high traffic level (>30 million ESALs). The design process was conducted following AASHTO M325 *(3)* and AASHTO M46 *(4)* for the SMA mixes and AASHTO M323 *(5)* and AASHTO R35 *(6)* for the DGA mixes. Details of the mix design are shown in Table D.1. It should be noted that the binder content shown in Table D.1 includes the binder from the RAP.

TABLE D.1
Mixture Type, Material and Volumetric Data

HMA Type	DGA				SMA			
RAP Content, % (by Weight)	0	15	25	40	0	15	25	40
RAP Content, % (by Volume)	0	15	25	40	0	19	31	48
Main Aggregate Type	Air Cooled Blast Furnace Slag				Steel Slag			
Main Aggregate Content, %	100	85	75	60	93	78	68	56
Mineral Filler, %	0	0	0	0	7	7	7	4
Binder Content, P_b, %	6.5	6.4	6.3	6.2	5.8	5.8	5.7	5.7
Bulk Spec. Grav., G_{sb}	2.59	2.56	2.56	2.55	3.44	3.25	3.14	3.00
Max. Theor. Spec. Grav., G_{mm}	2.45	2.45	2.45	2.45	3.07	2.99	2.92	2.77

TABLE D.2
Binder Replacement in Mixtures with Laboratory-Produced RAP

Mix Type	Parameter	0% RAP	15% RAP	25% RAP	40% RAP
DGA	Total Binder Content	6.5%	6.4%	6.3%	6.2%
	Binder Replacement	0%	12.9%	21.8%	35.5%
SMA	Total Binder Content	5.8%	5.8%	5.7%	5.7%
	Binder Replacement	0%	14.2%	24.1%	38.6%

TABLE D.3
Binder Replacement in Mixtures with Field-Sampled RAP

				Binder Replacement, %	
RAP Source	RAP Binder Content	Mix Type	Total Binder	At 25% RAP	At 40% RAP
1	6.6%	DGA	6.3%	26.2	41.9
2	4.7%	SMA	6.2%	18.9	30.3
3	5.9%	DGA	6.3%	23.4	37.5
4	5.8%	DGA	6.3%	23.0	26.8
5	5.7%	SMA	6.2%	23.0	36.8
6	5.2%	DGA	6.3%	20.6	33.0

The percent binder replacement was also calculated to allow comparison to INDOT current specifications for RAP usage. The binder replacement was calculated according to the formula:

$$\text{Binder Replacement} = \frac{A \times B}{E} \times 100\%$$

where: A = RAP binder content, %
B = RAP % in mixture
E = Total binder content in mix

NOTE: No shingles were included in any of these mixtures, so the equation is simplified from that in the 2012 INDOT Specifications.

Table D.2 summarizes the binder replacement values for the mixtures with laboratory-produced RAP, and Table D.3 shows the binder replacements for the mixtures made with 25% and 40% field-sampled RAPs.

Table D.2 shows that the RAP content in terms of binder replacement is somewhat below the RAP content by mass of mix because the laboratory-produced RAP had a binder content of 5.5%, which was below the total design binder content of each mix. The binder replacement values for the SMA were closer to the RAP content by mass because the design binder contents of the SMA mixes were lower than for the DGA.

Table D.3 shows that the binder replacement percentages are usually less than the RAP content by mass of mix for the mixtures produced with actual field-sampled RAP sources, except with RAP source 1, which had a high binder content of 6.6%.

With the laboratory-produced RAP and with the field-sampled RAPs, there are mixtures that exceed the current allowable binder replacement value of 15% for high volume roadways. The mix produced with 40% of RAP source 1 also exceeds the binder replacement value of 40% for lower traffic roadways.

REFERENCES

1. AASHTO T164. *Standard Method of Test for Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA)*. Association of State Highway and Transportation Officials.
2. AASHTO T30. *Standard Method of Test for Mechanical Analysis of Extracted Aggregate*. Association of State Highway and Transportation Officials.
3. AASHTO M325. *Standard Specification for Stone Matrix Asphalt (SMA)*. Association of State Highway and Transportation Officials.
4. AASHTO M46. *Standard Specification for Asphalt Plank*. Association of State Highway and Transportation Officials.
5. AASHTO M323. *Standard Specification for Superpave Volumetric Mix Design*. Association of State Highway and Transportation Officials.
6. AASHTO R35. *Standard Practice for Superpave Volumetric Design for Hot-Mix Asphalt (HMA)*. Association of State Highway and Transportation Officials.

APPENDIX E

LABORATORY FRICTION TESTING RESULTS

LABORATORY-PRODUCED "WORST CASE" RAP

Tables E.1 through E.3 summarize the results of polishing and testing the macrotexture and friction of the DGA and SMA slabs (as described in Appendix B) made with the laboratory fabricated "worst case" RAP. Figures E.1 and E.2 illustrate the comparisons of the MPD and F60 values for the DGA slabs with differing RAP contents, while Figures E.3 and E.4 do the same for the SMA slabs; these will likely be of interest to the most readers.

It can be seen from Table E.1 that the MPD of the DGA slabs is quite high. Field measured MPD values on DGA are typically in the range of 0.30 mm to 0.70 mm. The MPDs of the SMA slabs are also somewhat high, but not as high as the DGA. Field measured MPDs on SMA are typically in the range of 1.00 mm to 1.30 mm. This points to a potential issue with the slab compaction in the lab, as will be discussed more later in the section on testing actual field-sampled RAPs.

The data shown in Table E.2 for DGA slabs and Table E.3 for the SMA slabs does indicate that the friction level (indicated by DF20 and F60) tends to go down when the laboratory-produced RAP is added to the mix. This data is plotted in the figures below. The increase in the texture depth, especially for the DGA slabs, should also be noted. While there is a slight increase in texture depth on the SMA slabs, the increase for the DGA slabs amounts to doubling or almost tripling the texture depth. This again points to potential inadequate slab compaction or raveling of the slab surfaces, especially with the DGA. Raveling has been noted visually on some slabs. The NCSC is conferring with NCAT about the possibility of reducing the downward pressure on the polishing device to reduce the scrubbing action that may cause excessive raveling. The NCSC is also talking to researchers at the Texas Transportation Institute about using a different type of tire on the polisher. These improvements are in the preliminary stages now and require more research.

ACTUAL FIELD RAP

The results of testing lab fabricated slabs of asphalt mixtures with varying percentages of actual field-sampled RAP materials are shown below. The SMA results will be presented first, followed by the DGA results. Based on the testing of the lab-fabricated RAP, the field RAPs were tested at 25% and 40% by mass of the mix. (As shown in Appendix D, the binder replacement values ranged from 18.9% to 26.2% and from 26.8% to 41.9%, respectively.)

SMA WITH 25% AND 40% FIELD-SAMPLED RAP

RAP sources 2 and 5 were randomly selected to be used in SMA mixtures. A summary of the DF20, MPD and F60 values for these

TABLE E.1
Summary of DF20, MPD and F60 Values for DGA and SMA Slabs with Laboratory-Produced RAP

Mix	RAP	DF20			MPD, mm			F60		
	Content	min	max	diff	min	max	diff	min	max	diff
DGA	0	0.51	0.89	0.38	0.99	2.21	1.22	0.39	0.52	0.13
	15	0.47	0.91	0.44	0.91	2.09	1.17	0.36	0.52	0.16
	25	0.44	0.72	0.28	0.67	1.76	1.09	0.33	0.48	0.15
	40	0.45	0.83	0.38	1.06	1.96	0.90	0.34	0.52	0.17
SMA	0_I	0.57	0.70	0.13	1.41	1.55	0.13	0.40	0.47	0.08
	0_II	0.42	0.70	0.28	1.17	1.33	0.16	0.30	0.45	0.15
	Ave_0	0.49	0.70	0.20	1.29	1.44	0.14	0.35	0.46	0.12
	15	0.48	0.66	0.19	1.49	1.61	0.12	0.35	0.45	0.10
	25	0.42	0.60	0.18	1.64	1.77	0.13	0.32	0.42	0.10
	40	0.42	0.73	0.31	1.53	1.88	0.35	0.33	0.49	0.16

NOTE: Two SMA control slabs with 0% RAP were tested; they are signified by SMA 0_I and SMA 0_II. The average of the readings on these two slabs is also shown.

TABLE E.2
DF20, MPD and F60 vs. Wheel Passes for DGA Slabs

No. Revolutions	No. Wheel Passes	DF20				MPD				F60			
		DGA 0	DGA 15	DGA 25	DGA 40	DGA 0	DGA 15	DGA 25	DGA 40	DGA 0	DGA 15	DGA 25	DGA 40
0	0	0.89	0.91	0.66	0.83	0.99	0.91	0.67	1.06	0.52	0.52	0.36	0.50
500	1500	0.77	0.73	0.72	0.80	1.39	1.43	0.96	1.37	0.50	0.48	0.43	0.52
1200	3600	0.74	0.69	0.68	0.74	1.70	1.38	1.19	1.61	0.51	0.46	0.44	0.50
3000	9000	0.71	0.68	0.70	0.65	2.09	1.86	1.56	1.77	0.51	0.48	0.48	0.46
6000	18000	0.68	0.63	0.64	0.65	2.01	2.03	1.66	1.96	0.49	0.46	0.44	0.46
10000	30000	0.58	0.60	0.58	0.55	2.21	2.07	1.65	1.90	0.43	0.44	0.41	0.40
15000	40000	0.60	0.60	0.54	0.50	1.94	2.05	1.68	1.76	0.43	0.44	0.39	0.37
25000	75000	0.59	0.59	0.47	0.54	2.15	1.91	1.76	1.68	0.44	0.43	0.35	0.39
40000	120000	0.61	0.59	0.51	0.52	2.14	1.99	1.68	1.69	0.45	0.43	0.38	0.38
55000	165000	0.51	0.47	0.44	0.45	2.14	2.09	1.75	1.86	0.39	0.36	0.34	0.34

TABLE E.3
DF20, MPD and F60 vs. Wheel Passes for SMA Slabs

No. Revolutions	No. Wheel Passes	DF20				MPD				F60			
		SMA 0*	SMA 15	SMA 25	SMA 40	SMA 0*	SMA 15	SMA 25	SMA 40	SMA 0*	SMA 15	SMA 25	SMA 40
0	0	0.53	0.51	0.42	0.73	1.30	1.49	1.65	1.53	0.36	0.37	0.32	0.49
500	1500	0.67	0.66	0.45	0.66	1.35	1.51	1.73	1.64	0.44	0.45	0.34	0.46
1200	3600	0.70	0.62	0.60	0.65	1.40	1.53	1.65	1.68	0.46	0.43	0.42	0.46
3000	9000	0.68	0.61	0.59	0.60	1.36	1.56	1.69	1.71	0.45	0.42	0.42	0.43
6000	18000	0.68	0.62	0.59	0.56	1.34	1.58	1.77	1.71	0.45	0.43	0.42	0.41
10000	30000	0.65	0.55	0.53	0.50	1.33	1.61	1.77	1.81	0.43	0.40	0.39	0.37
15000	40000	0.64	0.55	0.51	0.48	1.36	1.61	1.76	1.78	0.42	0.39	0.38	0.36
25000	75000	0.62	0.53	0.51	0.49	1.34	1.60	1.72	1.78	0.42	0.38	0.38	0.36
40000	120000	0.60	0.55	0.45	0.42	1.40	1.52	1.64	1.88	0.41	0.39	0.34	0.33
55000	165000	0.60	0.48	0.47	0.45	1.38	1.60	1.66	1.81	0.40	0.35	0.35	0.34

*Average of two slabs' readings.

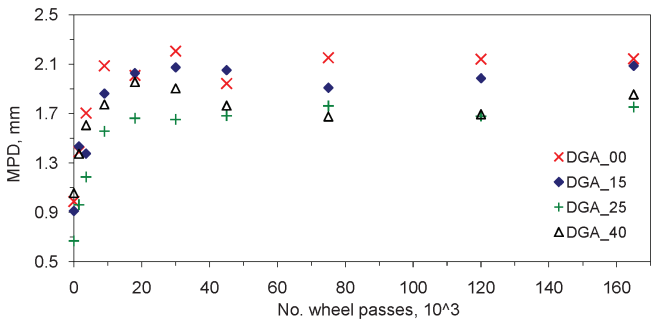

Figure E.1 Comparison of MPD for DGA with different RAP contents.

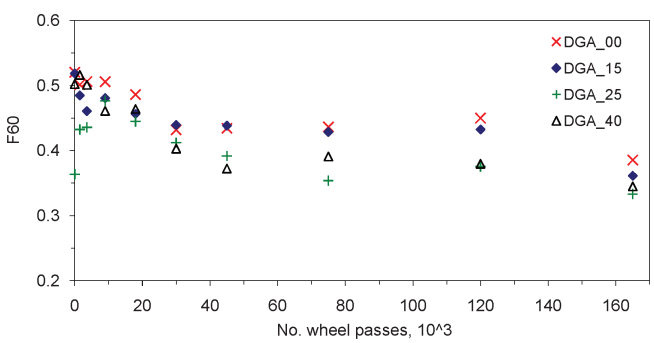

Figure E.2 Comparison of F60 for DGA with different RAP contents.

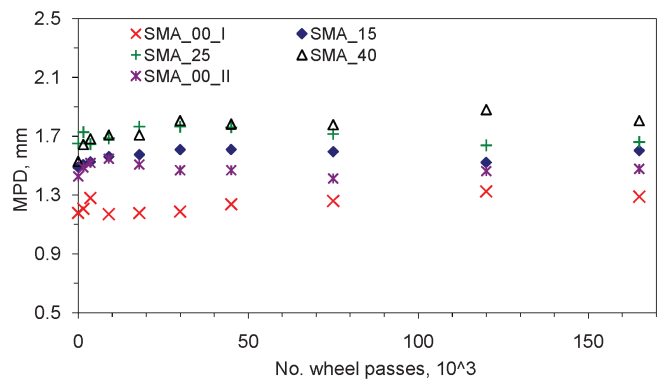

Figure E.3 Comparison of MPD for SMA with different RAP contents.

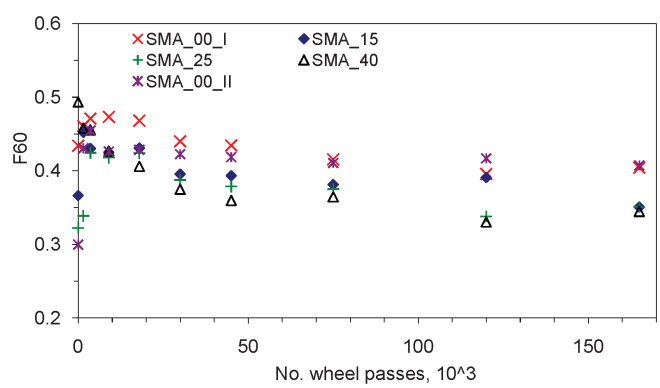

Figure E.4 Comparison of F60 for SMA with different RAP contents.

mixes is shown in Table E.4. The results for the SMA with lab fabricated RAP are also shown for comparison.

Table E.4 shows that that the slabs with field RAP were quite similar in terms of MPD; in addition, these MPD values are more in the range typically observed in SMAs on actual roadways. The slabs with laboratory-produced RAP had higher MPD values than the field RAP slabs and higher than observed in the field, but they were not excessively high. In investigating the reason for the more realistic MPD values for the field RAP slabs, it was discovered that the compaction process for the slabs had been modified between the time the laboratory-produced RAP mix slabs and the field RAP slabs were produced. When the latter slabs were made, additional weights (a slab of granite and four buckets of sand) were placed on top of the fork lift arms to increase the downward pressure). This additional weight does appear to have improved the compaction process to make it more representative of field compaction.

In light of the change in the compaction process, it is best to compare the DF20 and F60 values for the field RAP slabs to the friction flag value rather than to the lab RAP slabs. The increased macrotexture of the lab RAP slabs would be expected to increase the friction (DF20 and F60 values), making the lab RAP appear better than it might actually be in the field. The friction flag value was developed based on comparison of the CTM/DFT to the

TABLE E.4
Summary of DF20, MPD and F60 Values for SMA Slabs with Actual RAP

RAP	RAP	DF20			MPD, mm			F60		
Source	Content	min	max	diff	min	max	diff	min	max	diff
RAP 2	25	0.48	0.69	0.21	1.07	1.35	0.28	0.33	0.46	0.13
	40	0.42	0.62	0.20	1.10	1.24	0.14	0.30	0.41	0.11
RAP 5	25	0.38	0.63	0.25	1.12	1.43	0.31	0.28	0.42	0.14
	40	0.46	0.70	0.24	1.19	1.55	0.36	0.34	0.45	0.11
Lab	25	0.42	0.60	0.18	1.64	1.77	0.13	0.32	0.42	0.10
	40	0.42	0.73	0.31	1.53	1.88	0.35	0.33	0.49	0.16

NOTE: Laboratory-produced RAP shown for comparison.

TABLE E.5
DF20, MPD and F60 Values vs. Wheel Passes for Field-Sampled RAPs in SMA

		DF20				MPD				F60			
No. Revolutions	No. Wheel Passes	25% RAP 2	40% RAP 2	25% RAP 5	40% RAP 5	25% RAP 2	40% RAP 2	25% RAP 5	40% RAP 5	25% RAP 2	40% RAP 2	25% RAP 5	40% RAP 5
0	0	–	–	0.38	–	–	–	1.18	–	–	–	0.28	–
500	1500	0.69	0.63	0.60	0.70	1.33	1.13	1.12	1.28	0.46	0.40	0.39	0.45
1200	3600	0.58	0.62	0.63	0.57	1.22	1.23	1.32	1.19	0.39	0.41	0.42	0.38
3000	9000	0.60	0.57	0.59	0.60	1.07	1.22	1.31	1.29	0.39	0.38	0.40	0.40
6000	18000	0.58	0.53	0.56	0.56	1.24	1.23	1.21	1.45	0.39	0.36	0.38	0.39
10000	30000	0.55	0.51	0.54	0.55	1.29	1.10	1.26	1.41	0.38	0.34	0.37	0.38
15000	40000	0.50	0.48	0.58	0.51	1.16	1.25	1.29	1.49	0.34	0.33	0.39	0.36
25000	75000	0.50	0.48	0.56	0.51	1.27	1.22	1.43	1.55	0.35	0.33	0.39	0.36
40000	120000	0.50	0.44	0.51	0.48	1.20	1.17	1.37	1.41	0.34	0.31	0.36	0.34
55000	165000	0.48	0.42	0.52	0.46	1.23	1.24	1.26	1.45	0.33	0.30	0.36	0.34

towed friction trailer on actual pavements, and the texture depth of the field RAP slabs is reasonably comparable to actual pavements. Nonetheless, the lab RAP slab results are shown here for comparison, though it should be remembered that had a lower texture been achieved, the friction level would likely have been somewhat lower.

Table E.5 shows the DF20, MPD and F60 values vs. wheel passes for the field-sampled RAP sources in SMA mixes. Figures E.5 through E.7 show the plots of DF20, MPD and F60 graphically.

Figure E.8 shows the terminal friction levels determined for the SMA mixtures with RAP sources 2 and 5 compared to the lab-produced, poor quality RAP. Figure E.9 shows the polishing rate for the same two field-sampled RAPs in comparison to the lab RAP. These figures seem to show that the lab RAP performs better that RAP 5 in terms of terminal friction level but worse in terms of polishing rate. Because of the increased macrotexture of the lab-RAP slabs, however, the F60 value may be artificially high. This is supported by the fact that the lab RAP has a much worse polishing slope than the field RAPs, suggesting that the aggregate in the lab RAP is indeed a poor frictional performer.

Comparison of the SMA mixes made with field-sampled RAP at 25% and 40% shows that the terminal friction value drops when the RAP content is increased to 40% and the polishing rate becomes more negative. This suggests that the 40% RAP mixes would likely not provide as high a friction level in the field as the 25% RAP mixes. Although the terminal friction level and the polishing rate for the 40% RAP mixes are still in the acceptable ranges, it may be prudent to restrict the very high RAP contents to low volume surfaces until more field performance data is available

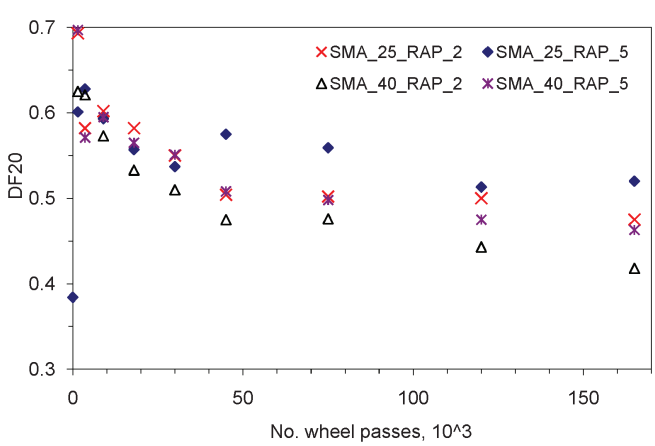

Figure E.5 DF20 values for RAP sources 2 and 5 in SMA.

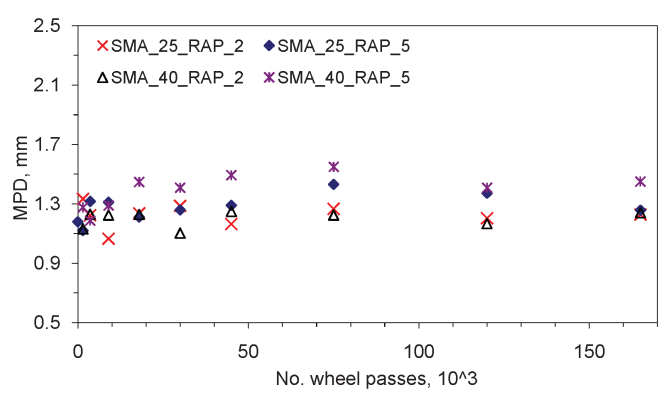

Figure E.6 MPD values for RAP sources 2 and 5 in SMA.

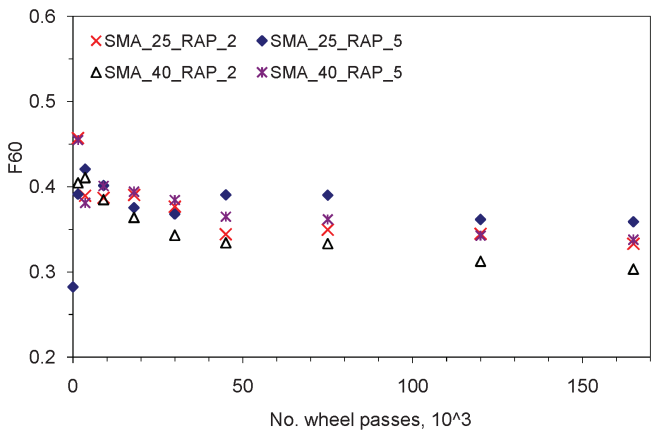

Figure E.7 F60 values for RAP sources 2 and 5 in SMA.

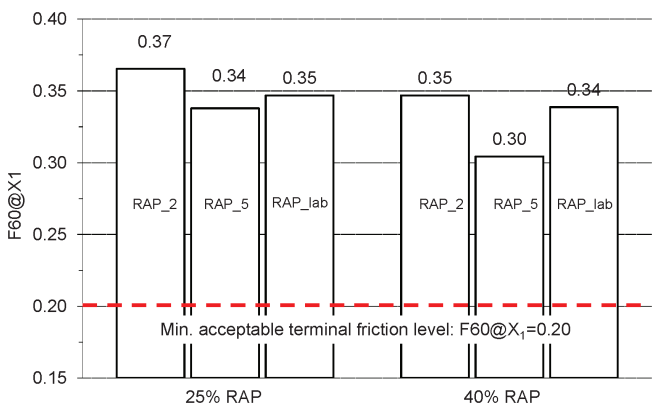

Figure E.8 Terminal friction value for field-sampled and laboratory-produced RAP mixes.

to verify if the field performance is acceptable. A limit of 25% RAP would be more conservative for medium to higher traffic roadways. Since few SMA surfaces are being constructed currently, the results for the DGA surfaces may be more pertinent.

DGA WITH 25% AND 40% FIELD-SAMPLED RAP

RAP sources 1, 3, 4 and 6 were randomly selected to be used in DGA slabs. A summary of the DF20, MPD and F60 values for

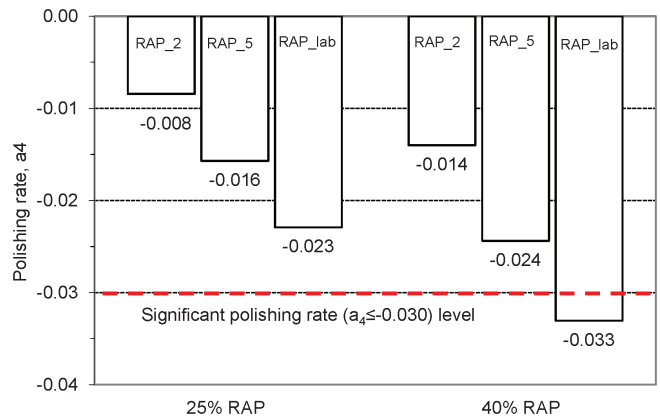

Figure E.9 Polishing rate for field-sampled and laboratory-produced RAP mixes.

these mixes is shown in Table E.6, which shows that there are some substantial changes in the MPD for some of the DGA slabs. Table E.7, which shows the DF20, MPD and F60 values vs. wheel passes for the DGA slabs produced with field-sampled RAPs, also shows that the minimum MPD readings occur within the first 1500 or occasionally the first 3600 wheel passes. The maximum readings occur later after some raveling of the slabs has occurred. Figures E.10 through E.15 show these results graphically.

Comparison of Table E.6 with Table E.4 shows that the initial MPD readings are substantially lower for the DGA slabs than for the SMA slabs, as expected. The maximum MPD readings for the DGA slabs start to approach the texture of the SMA slabs because of this raveling. Comparison of Table E.6 with Table E.4 also shows that the DF20 readings for the DGA slabs are lower than for the SMA initially but are fairly comparable later. The minimum F60 values for the DGA slabs are lower than for the SMA slabs and the maximum F60 values are similar or slightly lower for the DGA.

The slabs with laboratory-produced RAP have higher MPD values, as was observed with the laboratory-produced SMA slabs for the same reason—the change in the compaction process. The MPDs of these slabs are much higher than observed on similar pavement types on the field. Therefore, it is again advisable to compare the friction values of the DGA slabs with actual RAPs to the friction flag value rather than to the lab RAP slabs.

The terminal friction numbers for the DGA slabs with field-sampled RAP are above 0.23 for the 25% RAP mixes, but are only 0.20 and above for the 40% RAP mixes. The friction flag value corresponds to about 0.20, so the mixes with 40% RAP are approaching that level. In addition, some of the field RAP sources (specifically RAP 4 and 6 and perhaps 3) may not have reached terminal friction yet—there appears to be a downward trend in

TABLE E.6
Summary of DF20, MPD and F60 Values for DGA Slabs with Actual RAP

RAP	RAP	DF20			MPD, mm			F60		
Source	Content	min	max	diff	min	max	diff	min	max	diff
RAP 1	25	0.35	0.78	0.42	0.46	1.19	0.73	0.26	0.44	0.18
	40	0.32	0.58	0.26	0.84	1.27	0.43	0.25	0.37	0.12
RAP 3	25	0.36	0.76	0.40	0.35	0.91	0.56	0.25	0.40	0.15
	40	0.30	0.77	0.47	0.68	1.35	0.67	0.24	0.41	0.17
RAP 4	25	0.30	0.68	0.38	0.53	1.04	0.51	0.23	0.37	0.14
	40	0.24	0.68	0.44	0.72	1.03	0.31	0.20	0.38	0.18
RAP 6	25	0.29	0.69	0.40	0.46	0.86	0.40	0.22	0.37	0.15
	40	0.34	0.71	0.37	0.47	0.61	0.14	0.21	0.34	0.13
Lab	25	0.44	0.72	0.28	0.67	1.76	1.09	0.33	0.48	0.15
	40	0.45	0.83	0.38	1.06	1.96	0.90	0.34	0.52	0.17

TABLE E.7
DF20, MPD and F60 Values vs. Wheel Passes for Field-Sampled RAPs in DGA

		DF20				MPD				F60			
No. Revolutions	No. Wheel Passes	25% RAP 1	40% RAP 1	25% RAP 3	40% RAP 3	25% RAP 1	40% RAP 1	25% RAP 3	40% RAP 3	25% RAP 1	40% RAP 1	25% RAP 3	40% RAP 3
0	0	0.78	0.55	0.62	0.56	0.46	1.01	0.35	0.68	0.36	0.35	0.27	0.32
500	1500	0.76	0.53	0.76	0.77	0.84	0.84	0.64	0.68	0.44	0.33	0.40	0.41
1200	3600	0.70	0.56	0.66	0.69	0.88	0.87	0.70	0.81	0.41	0.35	0.37	0.40
3000	9000	0.56	0.58	0.54	0.57	0.98	1.00	0.74	1.00	0.36	0.37	0.32	0.36
6000	18000	0.56	0.56	0.58	0.54	1.08	1.07	0.76	1.22	0.37	0.37	0.34	0.37
10000	30000	0.38	0.40	0.57	0.41	1.05	1.11	0.89	1.26	0.28	0.29	0.35	0.30
15000	40000	0.36	0.32	0.52	0.35	1.09	1.16	0.89	1.28	0.27	0.25	0.33	0.27
25000	75000	0.42	0.35	0.42	0.41	1.19	1.27	0.89	1.30	0.30	0.27	0.28	0.30
40000	120000	0.38	0.35	0.36	0.35	1.06	1.22	0.91	1.31	0.27	0.27	0.25	0.27
55000	165000	0.35	0.33	0.38	0.30	1.19	1.16	0.84	1.35	0.26	0.27	0.26	0.24
		25% RAP 4	40% RAP 4	25% RAP 6	40% RAP 6	25% RAP 4	40% RAP 4	25% RAP 6	40% RAP 6	25% RAP 4	40% RAP 4	25% RAP 6	40% RAP 6
0	0	0.62	0.52	0.58	0.45	0.53	0.72	0.46	0.47	0.32	0.31	0.29	0.24
500	1500	0.68	0.68	0.69	0.70	0.66	0.74	0.59	0.50	0.37	0.38	0.36	0.34
1200	3600	0.57	0.58	0.61	0.71	0.67	0.74	0.70	0.49	0.32	0.34	0.35	0.34
3000	9000	0.48	0.54	0.64	0.61	0.85	0.82	0.75	0.55	0.30	0.33	0.37	0.32
6000	18000	0.56	0.48	0.60	0.61	1.01	0.92	0.83	0.53	0.36	0.32	0.36	0.32
10000	30000	0.39	0.48	0.41	0.46	1.04	0.99	0.84	0.52	0.28	0.32	0.27	0.25
15000	40000	0.33	0.39	0.29	0.46	0.91	1.01	0.86	0.53	0.24	0.28	0.22	0.26
25000	75000	0.33	0.46	0.34	0.43	0.99	1.03	0.85	0.53	0.25	0.32	0.24	0.25
40000	120000	0.37	0.40	0.33	0.39	1.02	0.93	0.83	0.61	0.27	0.27	0.24	0.24
55000	165000	0.30	0.24	0.32	0.34	0.96	0.98	0.76	0.52	0.23	0.20	0.23	0.21

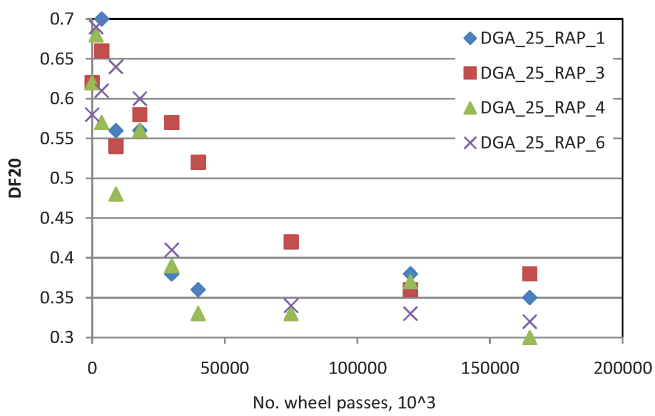

Figure E.10 DF20 values vs. wheel passes for DGA slabs with 25% field-sampled RAPs.

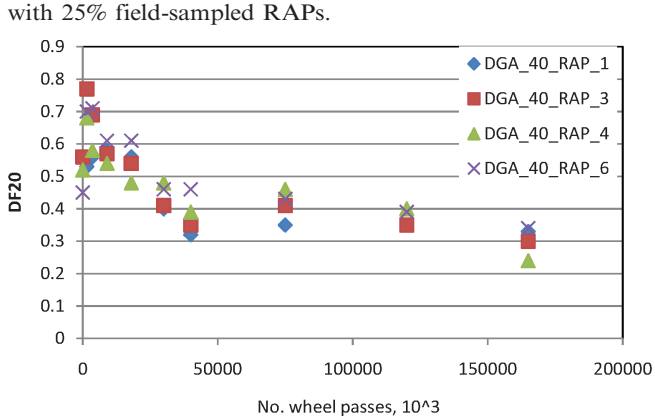

Figure E.11 DF20 values vs. wheel passes for DGA slabs with 40% field-sampled RAPs.

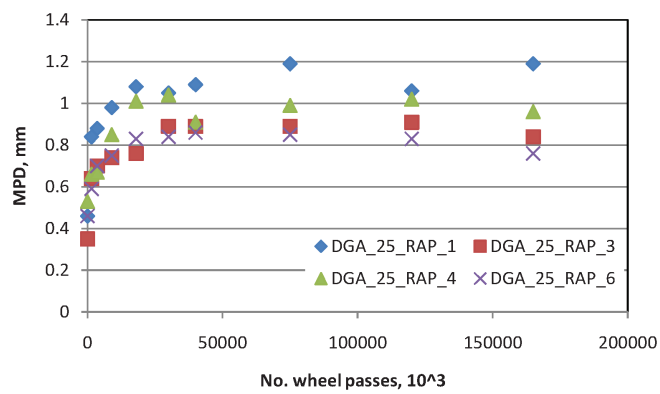

Figure E.12 MPD vs. wheel passes for DGA slabs with 25% field-sampled RAPs.

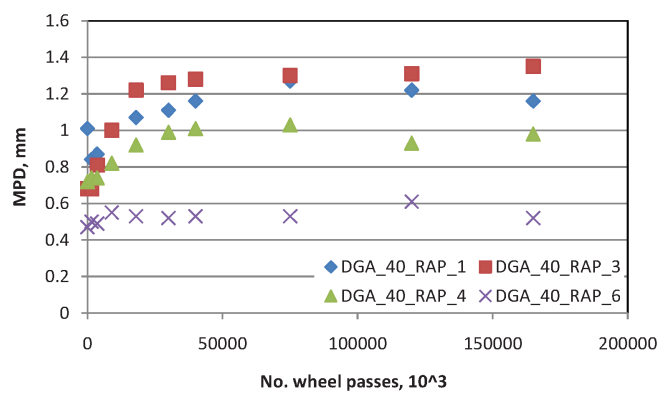

Figure E.13 MPD vs. wheel passes for DGA slabs with 40% field-sampled RAPs.

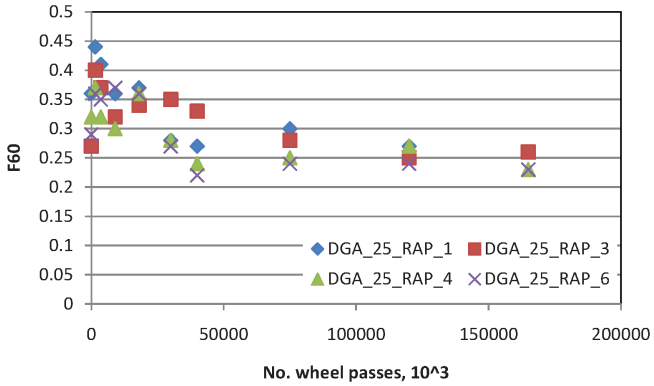

Figure E.14 F60 values vs. wheel passes for DGA slabs with 25% field-sampled RAPs.

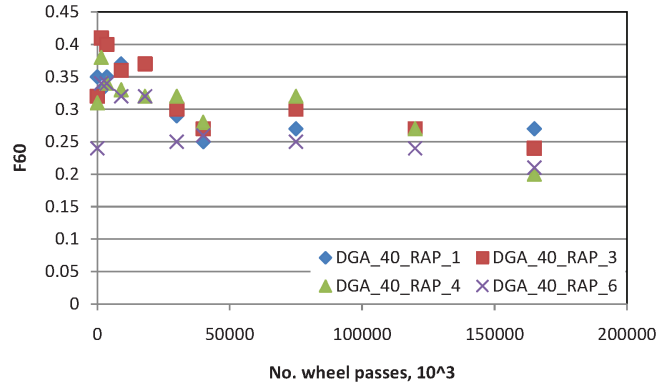

Figure E.15 F60 values vs. wheel passes for DGA slabs with 40% field-sampled RAPs.

Figure E.15 at 165,000 wheel passes. This would bring their friction levels below the flag value. This data, then, also supports the conclusion that 40% RAP may be too high if the frictional properties of the aggregate in the RAP are unknown. Since INDOT now specifies the RAP content in terms of binder replacement, and since the RAP content by binder replacement is slightly lower than the percent by mass of the mix in most cases, a 40% limit by binder replacement would allow even higher amounts of RAP aggregate to be included in mixtures. This again argues against a limit of 40%.

Based on the friction polishing and testing, then, it appears 25% RAP by binder replacement would be the upper limit for a threshold value of RAP in surface mixes for medium or higher traffic. The DGA with 40% of RAP 4 had a binder replacement value of 26.8% and had a terminal friction level of 0.20. This suggests that, for some RAP sources, 25% may be somewhat high. Other data needs to be considered in addition to the frictional performance, such as thermal cracking resistance, to set an acceptable threshold level. Another consideration is the merit in progressing in steps and accumulating information on field performance. From that point of view, allowing 20% RAP by binder replacement would be a reasonable first step pending additional field performance history, especially for high volume roadways.

This research is based on the assumption that the frictional properties of the RAP aggregate are unknown or mixed. There may be cases where it is advantageous to control the milling and stockpiling operations so that the properties of the RAP are known. In that case, INDOT could consider allowing the use of greater percentages of RAP. For example, the Illinois DOT allows the use of higher percentages of RAP if the contractor mills and stockpiles surface mixes separately from other pavement layers. In Indiana, if a contractor mills a surface containing steel slag, for example, it would be reasonable to allow the use of higher percentages of that material in the surface from a friction standpoint (as long as mechanical performance is acceptable). This could be considered on a case-by-case basis when the contractor sees an advantage and approaches the department with a proposal.

APPENDIX F

LOW TEMPERATURE TESTING RESULTS

LABORATORY-PRODUCED "WORST CASE" RAP

Based on the analysis of the compositions of RAP across the state, an "average" RAP gradation was proposed and fabricated in the laboratory. The gradation of the laboratory-produced RAP is shown in Figure D.2. (RAP source 1 was excluded because its gradation and binder content were different from the other sources.) This laboratory-produced RAP contained 5.5% of PG 64-22 binder and limestone that polishes substantially when exposed to traffic. This limestone was reported to be one of the lowest quality aggregates available in Indiana in terms of pavement friction, so it can be assumed that the RAP produced with this aggregate may be considered as representing the "worst case scenario."

The limestone aggregate was delivered to the laboratory, oven dried at 105 °C (221 °F) and cooled to room temperature prior to being sieved and sorted into individual size fractions. The aggregate was then batched to produce the desired blends. Prior to mixing, the batched aggregate blends (and the binder) were heated to a mixing temperature of 150 °C (302 °F). The mixing was performed in a five-gallon, "bucket type" laboratory mixer, which was first primed with a "butter" mixture in order to avoid binder loss during preparation of the test specimens. Next, the mix was conditioned for 2 hours at the compaction temperature (145 °C or 293 °F) according to AASHTO R30 *(1)*. After conditioning, the mixture was left in an 85 °C (185 °F) oven for 120 hours, to simulate the aging that occurs over the service life of a pavement. After this exposure, the mixture was cooled and re-mixed in the laboratory mixer to separate it into particles smaller than 12.5 mm. The RAP was then stored in closed buckets until the start of the specimen preparation process.

TESTING RESULTS

Low temperature testing was conducted according to AASHTO T322, *Standard Method of Test for Determining the Creep Compliance and Strength of Hot Mix Asphalt Using the Indirect Tensile Test Device* *(2)*. Three samples each of dense graded mixes containing 25% and 40% of the six actual, field-sampled RAPs were prepared with PG64-22 binder. The samples were then tested for their creep compliance at 0, −10 and −20 °C. Then the strength of each mix was determined by loading the samples at −10 °C until they fractured. The critical cracking temperature was then determined by using the LTStress spreadsheet developed by Dr. Dan Christensen *(3)* to estimate when the thermal stresses that would develop in a pavement (calculated based on the mixture stiffness) would exceed the strength of the mixture. The results of this testing are shown in Table F.1 and Figures F.1 and F.2.

Table F.1 shows that the average critical cracking temperatures of the RAP mixes are lower than that of the control mix by approximately 4 °C at a 25% RAP addition level. The critical cracking temperatures for the 40% RAP mixes are lower than those of the 25% RAP mixes by about 2 to 3 °C. So, the critical cracking temperatures of the 40% RAP mixes were 6 to 8 °C warmer than that of the control mix with no RAP—or about one binder grade warmer. These samples, though, were prepared with PG64-22 binder; in other words, no binder grade adjustment was made for the higher RAP contents. Had an adjustment been made, the critical cracking temperature would have been lower (more negative).

Figures F.1 and F.2 show these results graphically. Figure F.1 shows the average mixture strength and Figure F.2 shows the

TABLE F.1
IDT Strength and Stiffness and Critical Cracking Temperature

Mixture	Strength kPa	Stiffness GPa	T_{crit} °C
Control	3102	13.7	−20.6
25% RAP_1	3211	18.0	−16.1
25% RAP_2	2842	16.2	−16.2
25% RAP_3	3384	18.1	−16.4
25% RAP_4	3085	15.9	−16.7
25% RAP_5	3195	15.1	−16.2
25% RAP_6	3521	16.3	−15.2
40% RAP_1	3254	18.5	−14.2
40% RAP_2	2972	19.3	−13.8
40% RAP_3	3488	17.9	−14.1
40% RAP_4	2831	16.7	−14.2
40% RAP_5	3051	21.9	−13.5
40% RAP_6	3710	18.4	−12.9

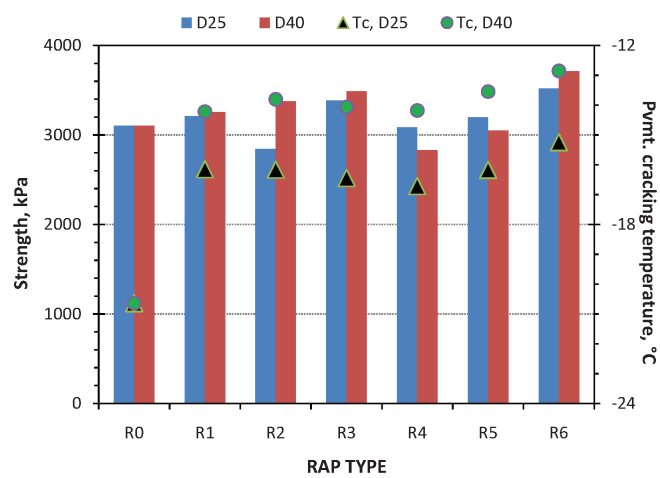

Figure F.1 Average mixture strength of actual RAP sources and critical cracking temperatures.

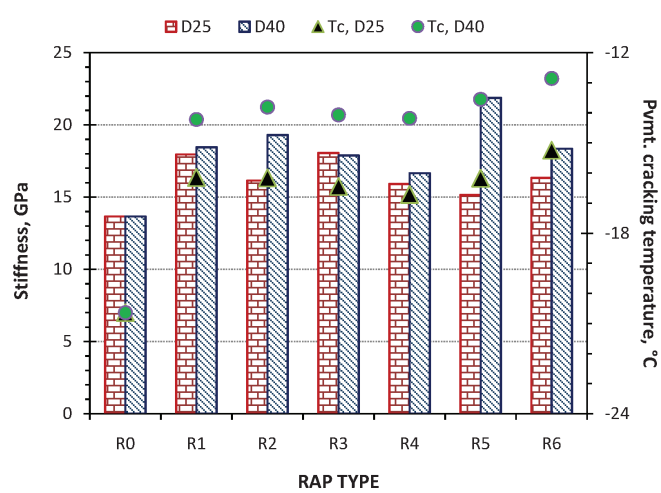

Figure F.2 Average mixture stiffness of actual RAP sources and critical cracking temperatures.

average mixture stiffness. The critical cracking temperatures are also shown in each figure.

Figure F.1 shows that the mixes with RAP have strengths that are fairly comparable to the control mix; some have somewhat higher strengths and others somewhat lower. In fact, statistical analysis shows that the strengths of the control and 25% RAP mixes are not significantly different. (Three replicates of each mix were tested for strength.) The *p*-value, which indicates the likelihood that a more extreme outcome could have been observed if the sample means were the same, is 0.0776 for the 25% RAP mixes. This high value indicates that the observed difference is consistent with the means being equal.

For the 40% RAP mixes, however, the *p*-value for the mixture strength is only 0.0071. Such a small *p*-value indicates that it is unlikely a greater difference could have been observed if the means were equal. Since the strengths of the control and 40% RAP mixes were significantly different, a Bonferroni comparison of means test was conducted to attempt to identify which samples were comparable and which were different. This comparison yielded two sample groupings. In one group, the mixes with RAP from sources 1, 2, 3 and 6 were found to be comparable to the control. In the other group, mixes with RAP from sources 1, 2, 3, 4 and 5 were found to be comparable to the control. When groups overlap to such a great extent, it is not possible to clearly identify which test results are significantly different from the others. So, while we can say that there is a statistically significant difference between the test results for the 40% RAP mixtures, we cannot say definitively that the control mix is significantly different from the 40% RAP mixes, since the control mix appears in each of the two groupings and there is substantial overlap between the groups.

The stiffnesses of the mixes, as shown in Figure F.2, are always higher for the RAP mixes. Some of the 25% RAP mixes are only slightly stiffer than the control. The 40% RAP mixes are generally stiffer than the companion 25% RAP mixes—in some cases substantially stiffer. Statistical analysis of this data was not performed because of the small sample size. (One sample was tested for creep compliance at each test temperature.)

The critical cracking temperature is affected by both the mixture strength and the stiffness. A stiffer mix will be more likely to crack than one with lower stiffness if their strengths are similar. On the other hand, a mix with high tensile strength will be unlikely to crack even if it is stiff. Examination of Figure F.2 shows that the critical cracking temperatures tend to follow the same trends as the stiffnesses. If a softer binder grade had been used with the 40% RPA mixes, the stiffness would have decreased, as seen in previous research, and the critical cracking temperature would have been lower (more negative) as well.

These results support INDOT's current specifications which allow the use of the design asphalt binder grade for RAP contents (in terms of binder replacement) of up to 25% and require using one grade lower for RAP contents up to 40%. (The binder replacement percentage is lower than the percentage by mass of mix for all the RAP sources except for Source 1, where it is slightly higher, as indicated in Appendix D.) A one grade change in the virgin binder would represent about a 6 °C change in the critical cracking temperature, based upon other research *(4)*.

REFERENCES

1. AASHTO R30. *Standard Practice for Mixture Conditioning of Hot-Mix Asphalt (HMA)*. American Association of State Highway and Transportation Officials.
2. AASHTO T322. *Standard Method of Test for Determining the Creep Compliance and Strength of Hot-Mix Asphalt Using the Indirect Tensile Test Device*. 2007. American Association of State Highway and Transportation Officials.
3. Christensen, D. "*LTSTRESS,*" English units version, May 1997.
4. McDaniel, R. S., H. Soleymani, R. M. Anderson, P. Turner, and R. Peterson. *Recommended Use of Reclaimed Asphalt Pavement in the Superpave System*. National Cooperative Highway Research Program Web Document 30. National Cooperative Highway Research Program, Washington, D.C.

APPENDIX G

FIELD FRICTION TEST RESULTS

INTRODUCTION

Eight existing field sections where INDOT had allowed the use of RAP in surface mixes were identified. The as-constructed information was obtained from construction records and in situ friction tests were conducted.

The eight different road sections were on various categories of roadways, including interstate highways, state and U.S. roads. The specific roads, RAP contents, Reference Posts (RPs) of the chosen test section and the year constructed are shown in Table G.1. Within each contract length, a one-mile section was chosen for CTM and DFT testing. These sections were selected to avoid major towns and junctions with other roadways; in addition, straight segments with no superelevation were chosen to provide convenient and safe test sites. (CTM and DFT testing require the operator to be exposed on the roadway, so safety was a concern.) The I-70 sections are two test sections from the SPS-9A project; one is a control section with no RAP and a PG 64-28 binder and the other includes 15% RAP with the same binder. These are the oldest sections and have the highest traffic levels.

Aside from the SPS-9A sections, none of the other sections were experimental, so the construction was not specially monitored; routine construction testing and inspection was performed by INDOT. Construction records were used to identify the materials and volumetrics of the asphalt surface courses. That data is shown in Table G.2. The gradations and fineness moduli of the mixes are shown in Table G.3. To differentiate the I-70 sections, the control with no RAP will be labeled I-70 (0%) and the 15% RAP section will be labeled I-70 (15%).

TEST EQUIPMENT AND PROCEDURE

For the field part of the study the ASTM E274 *(1)* locked wheel friction trailers were used. Special friction testing was done on the sections in 2007 and inventory data was collected in 2008–2010 when the sections were tested during routine inventory testing (every three years on non-interstate routes). The selected mile-long test segments were also tested with the CTM and DFT in 2007.

During a typical measurement, the friction trailer (shown in Figure G.1) is towed at a constant speed over the tested pavement. When the test is initiated, water is sprayed ahead of the tire so the wet pavement friction can be determined. The wheel is fully locked, and the resulting torque is recorded. Based on the measured torque (converted to the horizontal force) and dynamic vertical load on the test wheel, the wet coefficient of friction between the test tire and pavement surface can be calculated. The

TABLE G.1
Locations of Pavements with RAP in Surface Courses

					RP (Contract)		RP (Tested)		
Road	Location	Contract	RAP %	PG Grade	From	To	From	To	Year Completed
SR-38	Richmond-Hagerstown	RS-27534	15%	70-22	105.33	116.43	113.00	114.00	2005
US-35	Richmond	RS-27998	25%	58-28	10.44	23.15	18.00	19.00	2006
SR-103	New Castle	RS-28000	25%	58-28	10.76	14.09	13.00	14.00	2006
I-70 East	SPS-9A	R-22923	0%	64-28	100.74	100.84	100.74	100.84	1997
I-70 East	SPS-9A	R-22923	15%	64-28	101.00	101.09	101.00	101.09	1997
SR-47	South of Crawfordsville	RS-28319	15%	64-22	0.00	7.95	3.00	4.00	2006
SR-32	Fountain Co.	RS-28324	15%	64-22	17.73	25.97	19.00	20.00	2006
US-136	Fountain Co.	RS-28317	15%	64-22	8.41	16.34	10.00	11.00	2006

TABLE G.2
Material Type, Quantity and Volumetric Data for Field Test Sections

Roadway	SR-38	US-35	SR-103	I-70 (0%)	I-70 (15%)	SR-47	SR-32	US-136
Natural Sand, %	9	11	25			20	20	20
Manufactured Sand (Limestone), %	23	18						
Manufactured Sand (Dolomite), %				40	31	11	11	11
Manufactured Sand (Gravel), %						12	12	12
Limestone, %	53	46	34					
Dolomite, %				30	27			
Gravel, %			16					
Crushed Gravel, %						42	42	42
Blast Furnace Slag, %				30	27			
RAP, %	15	25	25	0	15	15	15	15
Total, %	100	100	100	100	100	100	100	100
G_{sb}	2.70	2.70	2.66	2.57	2.58	2.68	2.68	2.68
G_{mm}	2.53	2.52	2.49	2.45	2.49	2.49	2.49	2.49
Binder Type	PG 70-22	PG 58-28	PG 58-28	PG 62-28	PG 62-28	PG 64-22	PG 64-22	PG 64-22
Binder Content (%)	5.0	5.0	5.2	6.8	6.7	5.4	5.4	5.4

Modified from McDaniel, R. S., H. Soleymani, and A. Shah. *Use of Reclaimed Asphalt Pavement (RAP) Under Superpave Specifications: A Regional Pooled Fund Project*. Publication FHWA/IN/JTRP-2002/6. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2002. doi: 10.5703/1288284313465.

TABLE G.3
Fineness Moduli and Aggregate Gradation for Field Test Sections

Roadway	SR-38	US-35	SR-103	I-70 (0%)	I-70 (15%)	SR-47	SR-32	US-136
NMAS, mm	9.5	9.5	9.5	9.5	12.5	9.5	9.5	9.5
FM	4.30	4.31	4.19	4.51	4.58	4.30	4.30	4.30
Sieve size, mm				Percent passing				
25	100	100	100	100	100	100	100	100
19	100	100	100	100	100	100	100	100
12.5	100	100	100	100	100	100	100	100
9.5	93	93	98	91	89	93	93	93
4.75	60	59	73	50	50	62	62	62
2.36	48	47	44	41	39	45	45	45
1.16	31	31	30	29	27	32	32	32
0.6	21	20	20	19	18	19	19	19
0.3	12	12	10	12	11	12	12	12
0.15	6.5	7.3	6.8	7.5	6.7	7.4	7.4	7.4
0.075	4.5	5.2	4.9	5.0	4.1	5.5	5.5	5.5

Modified from McDaniel, R. S., H. Soleymani, and A. Shah. *Use of Reclaimed Asphalt Pavement (RAP) Under Superpave Specifications: A Regional Pooled Fund Project*. Publication FHWA/IN/JTRP-2002/6. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2002. doi: 10.5703/1288284313465.

friction number (FN) is then reported as the coefficient of friction multiplied by 100.

As shown in Figure G.2, the friction trailer used in Indiana is typically equipped with two types of tires: ASTM E501 (2006) *(2)* rib tire (on the right side) and ASTM E524 (2006) *(3)* smooth tire (on the left side).

Following the recommendations of the ASTM E274 (2006) *(1)* specification, the test speed and type of tire (rib [R] and smooth [S]) are stated when the friction is reported. The typical reporting format used in Indiana is FNS40 to indicate a test at 40 mph with a smooth tire. During tests for this research, five measurements (as required by the ASTM E274 [2006]a *(1)* specification) were conducted.

During tests with the CTM and DFT devices, machines were positioned in the left (L) and right (R) wheel paths of the driving (right) lane and in the center of the lane (C) for comparison purposes. Five sets of tests (L, C and R) were conducted, resulting in observations at 15 locations. Using both smooth and rib tires, towed friction trailer tests were conducted at a speed of 40 mph.

Figure G.1 ASTM E274 towed friction trailer: (a) oblique and (b) side views. (Courtesy of Kowalski, K. J. "Influence of Mixture Composition on the Noise and Frictional Characteristics of Flexible Pavements." PhD diss., Purdue University, West Lafayette, Indiana, 2007.)

TESTING SCHEDULE, WEATHER AND TRAFFIC CONDITIONS

The pavement sections tested in this study were constructed between 1997 and 2006. All tests were conducted with ambient temperatures significantly above the freezing temperature of water (above 10 °C).

Information about the traffic on field sections is shown in Table G.4. Based on the Average Annual Daily Traffic (AADT) information, the number of vehicle axles passes (NVA) on the test section (per month) was calculated. During the calculation, several simplifications were applied. It was assumed that an equal

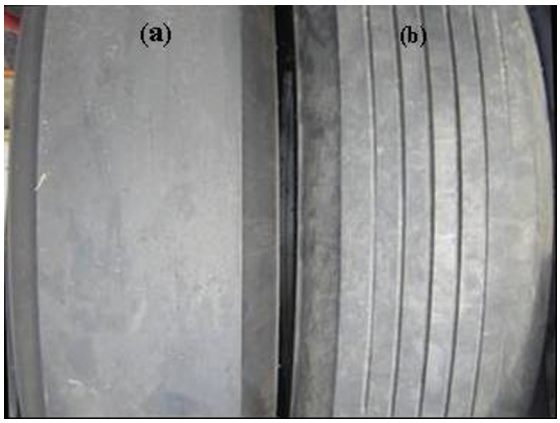

Figure G.2 Tires used for the friction measurements: (a) smooth and (b) rib. (Modified from McDaniel, R. S., H. Soleymani, and A. Shah. *Use of Reclaimed Asphalt Pavement [RAP] Under Superpave Specifications: A Regional Pooled Fund Project*. Publication FHWA/IN/JTRP-2002/6. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2002. doi: 10.5703/1288284313465.)

TABLE G.4
Summary of the 2007 Frictional Properties of the Tested Sections

Road	Years in Service at Test Time	Cumulative Traffic, NVA, 10^6	MPD, mm	DF_{20}	F60 (from CTM/DFT)	FNS40	FNR40
SR-38	2	3.4	0.40	0.52	0.25	47	54
US-35	1	2.2	0.30	0.50	0.22	33	54
SR-103	1	3.1	0.33	0.55	0.24	45	57
I-70 (0%)	10	152.5	*	*	*	22	*
I-70 (15%)	10	152.5	*	*	*	28	*
SR-47	1	1.1	0.37	0.61	0.27	37	58
SR-32	1	0.4	0.35	0.69	0.29	58	62
US-136	1	0.7	0.38	0.65	0.29	45	58

*CTM and DFT were not performed due to restrictions on traffic control on Indianapolis interstates. (Modified from McDaniel, R. S., H. Soleymani, and A. Shah. *Use of Reclaimed Asphalt Pavement [RAP] Under Superpave Specifications: A Regional Pooled Fund Project*. Publication FHWA/IN/JTRP-2002/6. Joint Transportation Research Program, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana, 2002. doi: 10.5703/1288284313465.)

TABLE G.5
2007 Special Friction Testing and 2008–2010 Inventory Testing Results

Road	2007 Data FNS40	2008 FNS40	2009 FNS40	2010 FNS40
SR-38	47	—	—	40.7
US-35	33	31.1	—	—
SR-103	45	—	—	—
I-70 (0%)	22	22.5*	28.4	22.5*
I-70 (15%)	28	38.8*	44.0	38.8*
SR-47	37	—	44.8	—
SR-32	58	65.3	—	—
US-136	45	—	—	42.8

— No inventory testing performed.

*The similarity in values in 2008 and 2010 was noted and looked suspect. These values were verified and are correct; these average values are identical although the individual readings comprising the averages were different.

number of vehicles travelled in both directions (AADT was divided by 2) and that 55% of the vehicles were using the driving lane on interstate highways (I) and U.S. highways. In the case of state roads (SR), there was only one lane in each direction, so the 55% "lane dividing" factor was not used. For simplification, vehicles were divided into two categories: trucks and cars. It was assumed that the average truck has 4.5 axles and the average car has 2 axles. Results were multiplied by 30, which is the average number of days per month. No traffic growth adjustment factors were employed. This simplified equation for NVA for interstate and state highways had the following form:

$$\mathrm{NVA} = (\mathrm{AADT})\cdot 0.5\cdot 0.55\cdot[\%\mathrm{Trucks}\cdot 4.5 + (100\% - \%\mathrm{Trucks})\cdot 2]30 \quad (1)$$

This calculation was performed as part of a doctoral dissertation *(4)* in order to relate the friction number to the cumulative amount of traffic on the roadway at the time of CTM, DFT and special friction testing. The cumulative traffic so determined is shown in Table G.4, along with the field friction test results from 2007. It can be noticed that the mean profile depth (MPD) values are quite consistent and in the range of 0.33 to 0.40 mm while the dynamic friction (DF_{20}) values are in the range of 0.50 to 0.69. Based on the texture and friction tests (MPD and DF_{20}), the calibrated wet friction at 60 km/h (F60) value was also calculated. The F60 values were between 0.22 and 0.29. The results of friction tests using ASTM E274 *(1)* friction trailer were between 22 and 58 (for tests at 40 mph with smooth tire, FNS40) and between 54 and 62 when tested with rib tire (FNR40).

These results show that the friction levels on the SPS-9A sections on I-70 (HM-4 control and HM-5 with 15% RAP) were the lowest. These are also the oldest sections and have much higher traffic. After ten years in service, however, the sections are still providing friction levels above the INDOT "flag value" of 20 with the smooth tire. Interestingly, the section with 15% RAP is providing somewhat higher friction than the control section with no RAP under the same traffic. All of the other sections are providing higher levels of friction, well above the flag value.

To gain additional information about the performance of these roadway sections with RAP, the friction inventory data from 2008 through 2010 was searched. The average friction values measured on these projects are shown in Table G.5.

Inventory data is typically collected every three years on the non-interstate roadways and every year on the interstates. The data in Table G.5 shows that overall the sections with RAP are still performing well. The sections on I-70, especially the control section, are lower; these experimental SPS-9A sections are starting to deteriorate. On the non-interstate projects, the friction numbers appear to have dropped somewhat, which would be expected. The 2008 result on SR-32 is higher than in 2007. This roadway was constructed in 2006, so may not have been very old when tested in 2007; the increase in friction number in 2008 may reflect the wearing away of the asphalt binder film on the aggregate.

In any case, with the exception of the I-70 control section, all of the surfaces with RAP are performing well to date. Inventory data for these sections should be monitored in the future to continue to evaluate the performance of these sections.

Even on the I-70 sections, under heavy traffic and after 13 years in service, the 15% mix is performing as well as or better than the control section with no RAP and both sections are providing a level of friction above the flag value.

REFERENCES

1. ASTM Standard E274. *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*. ASTM International, West Conshohocken, Pennsylvania.
2. ASTM Standard E501. *Standard Specification for Standard Rib Tire for Pavement Skid-Resistance Tests*. ASTM International, West Conshohocken, Pennsylvania.
3. ASTM Standard E524. *Standard Specification for Standard Smooth Tire for Pavement Skid-Resistance Tests*. ASTM International, West Conshohocken, Pennsylvania.
4. Kowalski, K. J. "Influence of Mixture Composition on the Noise and Frictional Characteristics of Flexible Pavements." PhD diss., Purdue University, West Lafayette, Indiana, 2007.