Performance of Recycled Asphalt Shingles in Hot Mix Asphalt

Final Report
August 2013

Sponsored through
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16. Abstract

State highway agencies are increasingly intersted in using recycled asphalt shingles (RAS) in hot mix asphalt (HMA) applications, yet many agencies share common questions about the effect of RAS on the performance of HMA. Previous research has allowed for only limited laboratory testing and field surveys. The complexity of RAS materials and lack of past experiences led to the creation of Transportation Pooled Fund (TPF) Program TPF-5(213). The primary goal of this study is to address research needs of state DOT and environmental officials to determine the best practices for the use of recycled asphalt shingles in hot-mix asphalt applications. Agencies participating in the study include Missouri (lead state), California, Colorado, Illinois, Indiana, Iowa, Minnesota, Wisconsin, and the Federal Highway Administration. The agencies conducted demonstration projects that focused on evaluating different aspects (factors) of RAS that include RAS grind size, RAS percentage, RAS source (post-consumer versus post-manufactured), RAS in combination with warm mix asphalt technology, RAS as a fiber replacement for stone matrix asphalt, and RAS in combination with ground tire rubber. Field mixes from each demonstration project were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and recovery with subsequent binder characterization. Pavement condition surveys were then conducted for each project after completion.

The demonstration projects showed that pavements using RAS alone or in combination with other cost saving technologies (e.g., WMA, RAP, GTR, SMA) can be successfully produced and meet state agency quality assurance requirements. The RAS mixes have very promising prospects since laboratory test results indicate good rutting and fatigue cracking resistance with low temperature cracking resistance similar to the mixes without RAS. The pavement condition of the mixes in the field after two years corroborated the laboratory test results. No signs of rutting, wheel path fatigue cracking, or thermal cracking were exhibited in the pavements. However, transverse reflective cracking from the underlying jointed concrete pavement was measured in the Missouri, Colorado, Iowa, Indiana, and Minnesota projects.

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PERFORMANCE OF RECYCLED ASPHALT SHINGLES IN HOT MIX ASPHALT

Final Report
August 2013

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EXECUTIVE SUMMARY

Transportation agencies in the United States have been increasingly using recycled asphalt shingles (RAS) in hot mix asphalt (HMA) applications over the last 25 years. Initial use of RAS started with recycled post-manufactured shingles, but now many agencies are also interested in using post-consumer RAS in asphalt applications. Post-consumer asphalt shingles typically contain 20 to 30 percent asphalt by weight of the shingles, as well as fine angular aggregates, mineral filler, polymers, and cellulosic fibers from the shingle backing. Each year, an estimated 10 million tons of post-consumer shingles are placed in landfills in the United States. Utilization of this waste product presents an opportunity to replace virgin asphalt binder with the RAS binder while taking advantage of the additional fibers which can improve performance. Thus a material that has historically been deemed a solid waste and has been placed in landfills can decrease pavement costs and reduce the burden on ever-decreasing landfill space.

Many agencies share common questions about the effect of post-consumer RAS on the performance of HMA. Previous research has allowed for only limited laboratory testing and field surveys. The complexity of RAS materials and lack of past experiences led to the creation of Transportation Pooled Fund (TPF) Program TPF-5(213). TPF-5(213) is a partnership of several state agencies with the goal of researching the effects of recycled asphalt shingles (RAS) on the performance of HMA applications. Agencies participating in the study include Missouri (lead state), California, Colorado, Iowa, Illinois, Indiana, Minnesota, Wisconsin, and the Federal Highway Administration. The agencies conducted demonstration projects that used HMA with RAS to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements containing post-consumer RAS.

The demonstration projects focused on evaluating different aspects (factors) of RAS that were deemed important for each state to move forward with a RAS specification. RAS factors addressed in the different demonstration projects included the evaluation of the RAS grind size, RAS percentage, RAS source (post-consumer versus post-manufactured), RAS in combination with warm mix asphalt technology, RAS as a fiber replacement for stone matrix asphalt (SMA) pavements, and RAS in combination with ground tire rubber (GTR). Several of the demonstrations projects also included control sections to compare traditionally used mix designs containing either RAP only or no recycled product to mix designs containing RAS.

Field mixes from each demonstration project were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and recovery with subsequent binder characterization. Pavement condition surveys were then conducted for each project after completion.

The demonstration projects showed that pavements using RAS alone or in combination with other cost saving technologies (e.g., WMA, RAP, GTR, SMA) can be successfully produced and meet state agency quality assurance requirements for mix asphalt content, gradation, and volumetrics. These mixes have very promising prospects since laboratory test results indicate good rutting resistance based on the flow number and dynamic modulus tests. The mixes also demonstrated good fatigue cracking resistance in the four-point bending beam apparatus, with the SMA mixes from Illinois (which used 5% RAS and no added fibers) exhibiting the most desirable fatigue characteristics. Fracture properties of the mixes at low temperatures determined by the SCB fracture energy test showed no statistical change in mixes with RAS compared to the mixes without RAS for the Missouri, Minnesota, Indiana, Wisconsin, Illinois and Colorado projects. Based on the SCB results, the addition of RAS materials to HMA is not detrimental to its fracture resistance, and fibers in the RAS could be contributing to the mix performance.

The test results of the extracted binder from these mixes showed that when RAS is used in HMA, the performance grade of the base binder increases on the high and low side. The average results of all the mixes in the study showed that for every 1 percent increase in RAS, the low temperature grade of the base binder increased 1.9°C; and for every 1 percent increase in RAP, the low temperature grade of the base binder increased 0.3°C.

The pavement condition of the mixes in the field after two years corroborated the laboratory test results. No signs of rutting, wheel path fatigue cracking, or thermal cracking was exhibited in the pavements. However, transverse reflective cracking from the underlying jointed concrete pavement was measured in the Missouri, Colorado, Iowa, Indiana, and Minnesota projects. The pavement condition surveys in Missouri revealed the pavement containing coarsely ground RAS exhibited more transverse cracking than the pavement containing finely ground RAS, but the non-RAS pavement exhibited less cracking than both coarse and fine RAS pavements. The non-RAS pavement in Colorado also showed slightly less cracking than the RAS pavement. In contrast, the RAS pavements exhibited the same amount of cracking or less than the non-RAS pavements for the Iowa, Indiana, Illinois, and Wisconsin demonstration projects.

1. INTRODUCTION

Significant interest in modifying hot mix asphalt (HMA) with recycled asphalt shingles (RAS) is growing every year among state highway agencies in the United States. This is driven by the potential to reduce the cost of HMA and the desire for environmental stewardship. Post-consumer asphalt shingles typically have 20 to 30 percent asphalt by weight of the shingles. Utilization of this waste product presents an economic opportunity, particularly when virgin asphalt binder prices are high, by replacing virgin asphalt binder with the RAS binder. Thus a material that has historically been deemed a solid waste and has been placed in landfills has monetary value and can also reduce the burden on ever-decreasing landfill space. Further, recycling asphalt shingles and using them in HMA in lieu of virgin asphalt binder reduces greenhouse gases generated at refineries that produce asphalt binder. RAS is also showing great potential as a material that can be used to replace fibers in stone mastic asphalt (SMA) mixes.

Recycling asphalt shingles is not a new concept as shingle manufacturer scrap has been recycled for use in HMA for more than 25 years. In the last 20 years, recycled post-consumer shingles have also been used in HMA. Most recycled shingles have been used in commercial and/or residential paving projects and are not commonly accepted by state transportation agencies. The recent substantial increase in crude petroleum prices—as well as refining modifications that have resulted in removing asphalt binders from the marketplace—has led to considerable price increases in asphalt binder in the past several years. This substantial increase in the cost of asphalt binder, coupled with the advancement of shingle processing technology, has created the impetus for state transportation agencies to begin using RAS.

Many agencies share common questions about the effect of RAS on the performance of HMA. Previous research has allowed for only limited laboratory testing and field surveys. The complexity of RAS materials and lack of past experiences led to the creation of Transportation Pooled Fund (TPF) Program TPF-5(213). TPF-5(213) is a partnership of several state agencies in the United States with the goal of researching the effects of RAS on the performance of varied asphalt applications. As part of the pooled fund research program, multiple state demonstration projects were conducted to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements containing RAS. The following agencies have participated in TPF-5(213):

- Missouri Department of Transportation (MoDOT) – lead agency
- Iowa Department of Transportation (Iowa DOT)
- Minnesota Department of Transportation (MnDOT)
- Indiana Department of Transportation (INDOT)
- Wisconsin Department of Transportation (WisDOT)
- Colorado Department of Transportation (CDOT)
- Illinois Department of Transportation (IDOT)
- California Department of Transportation (Caltrans)
- Federal Highway Administration (FHWA)

Each state highway agency in the pooled fund study, with the exception of Caltrans and the FHWA, proposed a unique field demonstration project that investigated different aspects of asphalt mixes containing RAS specific to their state needs. The demonstration projects focused on evaluating different aspects (factors) of RAS that were deemed important for their state to move forward with a RAS specification. RAS factors addressed in the different demonstration projects included the evaluation of the RAS grind size, RAS percentage, RAS source (post-consumer versus post-manufactured), RAS in combination with warm mix asphalt technology, RAS as a fiber replacement for stone matrix asphalt (SMA) pavements, and RAS in combination with ground tire rubber (GTR). Several of the demonstration projects also included control sections to compare traditional mix designs containing either RAP only or no recycled product to mix designs containing RAS.

This report presents the results of the laboratory performance tests on the field and laboratory produced mixes for the demonstration projects as well as the results of the pavement condition surveys conducted after the projects were completed. Since the experimental plan for the demonstration projects were tailored and individualized to meet the needs of each particular state agency, the greatest value of this study is in the separate analysis of each project. Therefore, the body of this report presents a summary of all the results obtained in the study, while a more detailed description and evaluation of each State's demonstration project and its results are presented as Appendices A through G.

2. LITERATURE REVIEW

The use of recycled asphalt shingles (RAS) in hot mix asphalt (HMA) has been studied and used in asphalt pavements for the past twenty years. The vast majority of research on RAS has been focused on post-manufacturer asphalt shingles and its use in HMA historically. Over the last seven years, the focus on research has moved to post-consumer asphalt shingles due in part to the limited availability of post-manufacturer asphalt shingles, the rise in asphalt prices and the success in the use of post-manufacturers RAS in HMA pavements. It has been estimated that more than 11 million tons of asphalt shingles are landfilled every year and over sixty percent are post-consumer asphalt shingles. The environmental incentive, a large quantity of asphalt shingles available for landfill diversion and the economic value, replacement of virgin asphalt, aggregate and fibers, has brought this research to the forefront for state environmental and transportation engineers.

Some of the earliest published literature on the use of post-manufacturers' recycled shingles in HMA was done by Emery and MacKay (1991) and although it included other recycled materials it accurately identifies the limiting factors to utilizing RAS in pavement construction today: material variability; collection, storage and processing costs; lack of technical guidance and specifications; environmental constraints; and agency conservatism. Research completed on post-manufacturer recycled shingles has found the material to perform as well or better than HMA mixes not containing post-manufacturers' RAS (Watson et al. 1998; Foo et al. 1999; Reed 1999; Amirkhanian and Vaughan 2001).

Research has shown that the composition of RAS provides both an economical value and mix properties that can enhance the performance of asphalt pavements. However, the continuing challenges in utilizing RAS are found to be in the quality control and quality assurance of the final product along with identifying mix designs that meet the requirements of specifying agencies which includes the volumetric properties of RAS for their inclusion in HMA volumetric properties.

One of the critical components in the research of RAS has been to identify the composition of the post-manufacturer and post-consumer asphalt shingles. Brock (2007) summarizes the composition of post-manufacturers and post-consumer shingles in Table 1 below, which in turn reveals the economic opportunity for virgin asphalt, aggregate and fiber replacement in asphalt pavements.

Table 1. Asphalt shingle composition (Brock 2007)

	Organic		Fiberglass		Old	
	lb/100 ft^2	%	lb/100 ft^2	%	lb/100 ft^2	%
Asphalt	68	30	38	19	72.5	31
Filler	58	26	83	40	58	25
Granules	75	33	79	38	75	32
Mat	0	0	4	2	0	0
Felt	22	10	0	0	27.5	12
Cut-out	(2)	1	(2)	1	0	0
Totals	**221**		**202**		**235**	

Brock also reported on the economic benefits of utilizing post-manufacturer recycled shingles (organic vs. fiberglass) and post-consumer recycled shingles. The summary of Brock's economic analysis is summarized in Table 2.

Table 2. Economic analysis of asphalt recycling use (Brock 2007)

	Organic ($)	**Fiberglass ($)**	**Old ($)**
Asphalt at $400/ton	120.00	76.00	124.00
Filler at $10/ton	2.60	2.80	2.50
Granular at $10/ton	3.33	2.66	3.20
Mat at $10/ton		.14	
Felt at $10/ton	1.00	.07	1.20
Sub-totals	126.93	81.67	130.90
Disposed cost	25.00	25.00	25.00
Sub-totals	151.93	106.67	155.90
Process cost	(10.00)	(10.00)	(12.00)
Net value	141.93	96.67	143.90
HMA savings per ton			
4%	5.68	3.36	5.76
5%	7.10	4.83	7.19
6%	8.32	5.80	8.63

Cochran (2006) determined recycling post-consumer asphalt shingles was economically beneficial and considered the performance, environmental issues, and energy consumption in the life-cycle cost analysis.

With the rise in asphalt prices, state budget cuts, past and recent research results on RAS performance and the opportunity to divert this valuable commodity from landfills many states are now researching or utilizing post-manufactured and/or post-consumer RAS in asphalt applications. For economical, sustainability and performance opportunities RAS is becoming a recycled product that is gaining acceptance by owner/agencies.

Today there are more than 20 states that have specifications, developmental specification or are considering the use of RAS in asphalt applications. Table 3 below summarizes the status of states utilizing RAS which includes that status on specifications utilizing RAS; the percent of RAS and RAS type. (There are also several Canadian Provinces utilizing RAS in HMA (Brock 2007).)

Table 3. State DOT specifications for RAS

State	State Specifications for using RAS[1]
	Post-Manufacturer RAS (M); Post-Consumer RAS (C)
AL	State Specification allowing 5% M or 3% C
GA	State Specification allowing 5% M or C
IA	State Specification allowing 5% M or C
IL	State Specification allowing 5% M or C
IN	State Specification allowing binder replacement of 15% M or C for surface coarse mixes (Maximum 25% binder replacement for mixes less than 9 million ESALs)
KS	State Specification allowing 5% M or C
KY	24% Binder Replacement
MA	State Specification allowing 5% M
MD	State Specification allowing 5% M
MN	State Specification allowing 5% M or C
MO	State Specification allowing 7% M or C
NC	State Specification allowing 5% M or C
NJ	State Specification allowing 5% M
NH	State Specification allowing 0.6% binder replaced with M or C from % of total mix
NY	State Specification allowing 5% M
OH	State Specification allowing 5% M or C
PA	State Specification allowing 5% M or C
SC	State Specification allowing 5% M or C
TX	State Specification allowing 5% M or C
VA	State Specification allowing 5% M or C
WI	State Specification allowing binder replacement of 20% M or C (5% max when used in combination with RAP)

(1) Reflects specifications for RAS utilization without RAP. Each state has additional requirements for RAS used in combination with RAP and different virgin binder requirements. See state DOT construction specifications for details.

The primary environmental issues that have historically arisen associated with post-consumer RAS are the presence of asbestos and polycyclic aromatic hydrocarbons (PAH). Innovative Waste Consulting Services published a report on the environmental issues associated with post-consumer asphalt shingle recycling in 2007 (Townsend et al. 2007). Based upon available data from Florida, Iowa, Maine, Massachusetts, Minnesota, and Missouri, Townsend et al. found approximately 1.5% of samples of more than 27,000 loads contained asbestos above the Environmental Protection Agency (EPA) limit of 1%. With the increase in states allowing for the use of post-consumer RAS along with the growing number of asphalt shingle recycling facilities nationwide, there is a need to update this research.

The incidence of asbestos-containing materials (ACM) being found in shingles today is extremely low. Today there are over 80,000 samples collected from loads of post-consumer asphalt shingles and tested for ACM and the incidence of ACM above the EPA limit of 1% continues to be well below the 1.5% as found in 2007 (Townsend et al. 2007). In addition, there is data that shows that the total asbestos content of asphalt shingles manufactured in 1963 is only 0.02 percent; in 1977, it dropped to 0.00016 percent. Today roofing contractors do not encourage the placement of new shingles over old ones as it reduces the service life of the new shingles if the old shingles are not removed. On the contrary, due to earlier practices of reroofing over worn out roofs with new shingles, there continues to be a very small risk of finding asbestos in post-consumer shingles until about 2016. However, ACM continues to be used in roofing products such as mastic, roofing tar, roof flashing and roofing felts that can create ACM issues as these materials are often removed with shingles and historically landfilled together. Thus it is important that appropriate sorting of materials and ACM testing be done for the wider use of RAS in asphalt applications across the nation to occur. State DOT's continue to address the issue of ACM when they look to utilizing post-consumer asphalt shingles in asphalt applications (Powers 2010). Testing protocols for ACM by National Emission Standards for Air Pollutants (NESHAP) coordinators can vary from state to state and sometimes between local agencies and thus present challenges for shingle recycling operators and State DOTs. Quality control is vital in creating a quality end product and is achieved when all entities directly or indirectly in the recycling of post-consumer RAS work closely together and understand their roles and responsibilities. Communication between agencies has proven to be very effective in implementing quality control protocols that have led to quality end products. The Iowa DOT, Illinois DOT, Illinois Tollway, Minnesota DOT and Texas DOT have worked very closely with their environmental agencies to prepare guidelines on quality control for both environmental and technical protocols for shingle recycling operators and asphalt producers.

Kriech et al. (2002) conducted a laboratory study examining four virgin asphalt roofing samples testing the concentration of 29 different PAHs. The research found the leaching results for all 29 PAHs were below the detection limit of 0.1mg/L specified by the EPA through NESHAP (Kriech et al. 2002). Inspec-Sol, Inc. (2008) conducted a preliminary material and environmental investigation on the use of asphalt shingle aggregate (ASA) on three sections of the Lunenburg County recreational trails. One of the study outcomes was to identify the leached contaminants from the ASA into the soil and groundwater and assess the environmental impacts and define the risk associated with the exposure of trail users with the ASA material. ASA mixes of aggregate/asphalt shingles included three ratios: 25:75, 50:50, and 75:25. The potential for leaching of the ASA was assessed by measuring the changes in chemical concentrations of total petroleum hydrocarbons (TPH), polycyclic aromatic hydrocarbons (PAHs) and metals in the soil beneath the ASA material and in the groundwater. Changes in TPH, PAHs and Metals concentrations were observed in the soil, however, no obvious trends of chemical concentration increases were observed. Changes in groundwater chemical concentrations were not observed. Soils beneath the trail structure were found to have low permeability and therefore posed minimal contamination, if any, to the groundwater. Based on preliminary quantitative risk assessment (PQRA) for human health found one of the three locations (25:75 ASA) with a slightly elevated risk, however the results were based on very conservative assumptions and if a 25:75 mix were to be considered for use in the future they recommended considering conducting a site specific risk assessment (SSRA) using more detailed, site specific information than was used for the PQRA (Inspec-Sol 2008).

Literature associated with performance testing of asphalt pavements containing post-consumer RAS have increased over the last few years. A challenge for most states is to determine and integrate RAS properties into HMA mix design properties that must be taken into consideration when using post-consumer RAS. Monitoring the end product through well-defined specifications is helps ensure an owner/agency is receiving a final quality product that will lead to realizing the benefits of RAS.

As the use of RAS in asphalt applications has increased so have the knowledge base. Earlier research completed by Button et al. (1996) and Abdulshafi et al. (1997) found that a finer grind was going to produce a more consistent and better performing mix. Button et al. (1996) also found that the mixes containing a finer ground post-consumer RAS increased the tensile strength more than a coarser grind. More recent research by McGraw et al. (2010) found that a finer grind size will activate higher percentages of asphalt binder from the RAS and eliminate the likelihood of nails being found in the mix.

Along with grind size, earlier research by Button et al. (1996) found that moisture susceptibility improved in all post-consumer RAS mixtures, however, the RAS mixes were compacted at a temperature of about 14°C higher than the control mixture and it was thought that the higher temperature alone could improve the adhesion of asphalt to aggregate and thus improve resistance to moisture. Further research on the laboratory mix designs adding the shingles at different stages of the mixing (i.e. adding the RAS after the asphalt is mixed with the aggregate as compared to mixing with the RAS with the aggregate prior to heating or prior to the addition of the virgin asphalt) could be beneficial. Pre-blending the shingles with RAP or sand in the field and adding the shingles to the RAP conveyor belt could also be researched to determine if dusting of the shingles occurs or deters asphalt binder mixing. Maupin (2010) reported that the pre-blending of the shingles with the aggregate (#10) was found to differ from the field ratio (50/50) to the lab determined shingle/No.20 ratio 33/67 and 37/65 ratios for the base mix and surface mix projects. However, the contractor adjusted the amount of blended material at the plant to produce a mix with the proper binder content for the mix and was able to meet the target job mix formula values. McGraw et al. (2010) found that the lab RAP/post-Consumer mixtures failed to meet current MnDOT moisture sensitivity tests (modified Lottman), while the RAP/Post Manufacturers had higher values. Increased moisture sensitivity could point to a decrease in durability and with the two results of the two research projects showing conflicting results, further research was suggested by McGraw et al. (2010).

Binder grading has been shown to follow a very consistent pattern at low temperatures among recent research (Maupin 2008; McGraw et al. 2010; Scholz 2010). All studies found that there was only a loss of one binder grade in the mixes with 5% post-consumer only (no RAP), however, at the high temperatures the grade jumps varied along with AC contents and percent binder replacement as shown previously in Table 1.

The most recent research completed by McGraw et al. (2010) and Scholz (2010) found that for the post-consumer RAS only mixtures there was a significant effect on the high temperature (surpassing the critical high temperatures of as-received virgin binders) and a moderate effect on the low temperatures. However, the studies differ in results when incorporating RAP at different percentages. Scholz (2010) found that with the inclusion of RAP at increasing percentages there was no significant shift in the low temperature grades, which was not expected. McGraw et al. (2010) found significant changes in the low temperature grades with increasing RAP percentages. It should be mentioned that the mixes used in the Scholz (2010) study used a finer ground RAP and a coarser graded RAS. Although the grade changes at high temperatures showed improved rutting resistance in most mixes, there is also a concern that the linear rate of stiffness may produce fatigue cracking at the intermediate temperatures. McGraw et al. (2010) also looked at two mix designs using a softer binder (performance grade 52-34 in place of a 58-28 with 25%RAP and 5%RAS). The use of the softer binder with the RAS/RAP mixtures resulted in dropping the PG grade by one grade at both the high and low temperatures. However, the new binder to total binder content ratio did vary from the RAS/RAP with the 58-28 and fell below the AASHTO 70% requirement designated in the MnDOT specification. McGraw et al. (2010) suggested it would be of value to complete additional research in the use of a softer binder with mixes containing RAS to better understand the benefits and outcomes.

McGraw et al. (2010) utilized two different sources of RAP and found there to be little difference in the performance of the mixes containing the different RAP sources, however, Marasteanu et al. (2007) found that when adding post-consumer RAS to RAP mixes with lower performance grades the RAS had little effect on the low temperature results. Thus the variability and/or quality or binder grade of the RAP may have an effect on the final mix and the use of fractionated as compared to RAP could be shown to give more control or consistency/repeatability to the mixes.

Furthermore, Marasteanu et al. (2007) concluded that more research on the benefits of the RAS fibers were needed. Recent field demonstration projects have found RAS to be economically and performance wise very good for stone mastic asphalt mixes (SMA). The Illinois DOT District 1 completed a demonstration project on Illinois Interstate 94 in 2009 utilizing post-consumer RAS in their SMA binder and surface mix. The Illinois DOT found small changes in the utilization of RAS yielded substantial mix savings as the RAS reduced the virgin asphalt content by 1.25% (295 tons), reduced the virgin dust (177 tons) and sand (236 tons) purchases and eliminated the required fiber machine and the addition of fibers (Jones 2010).

The AASHTO 70% new binder to total asphalt binder criterion for RAS/RAP mixtures have been shown to have a strong correlation in laboratory mixtures between virgin binder content and the high/low PG temperatures of the binders, McGraw et al. (2010), Scholz (2010) and Maupin (2010). However, there are instances in Minnesota (McGraw et al. 2010) where field surveys of pavements with higher binder replacements do not seem to influence the cracking (fatigue or low temperature) in comparison to the control mixes. The Texas DOT Special Provision 341-024 (2010) for dense graded mixtures only requires 65% virgin binder contents for surface mixes. The mixes are working well (Lee 2010), however, there is a concern that when the mixes are being prepared in the field that they are assuming that 100% of the RAS binder is effective, which is not true and the effective RAS binder is found to be closer to 80% and that mixes may end up being overly stiff. This is not unique to Texas and also mentioned by McGraw et al. (2010) as seen on the Minnesota Highway 10 project. Integration and holding times of the RAS in the hoppers are important in reaching an optimum effective binder from RAS and RAS/RAP mixtures. The McGraw et al. (2010) provides strong data results to indicate that the AASHTO 70% ratio of new binder to recycled binder content is reasonable.

The recent studies also showed that there are inconsistencies with lab produced mixes of RAS/RAP simulating the production or field mixes (Maupin 2008; McGraw et al. 2010; Scholz 2010; Maupin 2010). This confirms the importance for completing field surveys to be used in comparing lab results.

In addition, there is also a need for field performance assessment of RAS projects to see the results after years of trafficking and how they have performed to non-RAS mixes as this is lacking in previous research.

3. QC/QA PROCEDURES FOR UTILIZING RAS

The quality control and quality assurance (QC/QA) in the utilization of RAS is critical to achieving a quality pavement that meets agency specifications. It is necessary that RAS sources meet the required DOT environmental and transportation standards. Many states require asphalt producers to use RAS sources that are certified by the DOT. State agencies in the pooled fund study used the demonstration projects either as a tool for developing statewide RAS specifications or as an opportunity to test already written preliminary specifications. The research team monitored the QC/QA practices agencies implemented during the demonstration projects to address quality and environmental concerns. This chapter discusses a compilation of those strategies and highlights the most effective procedures regarding the quality control in sourcing of RAS and its integration at asphalt plants.

3.1 Sourcing

3.2.1 Post-Manufactured Shingles

Asphalt roofing manufacturers have waste shingles that are accepted by recycling asphalt shingle facilities. The shingles are delivered on pallets wrapped in plastic or in roll-offs with and without the wrapping. Asphalt contents can vary among different manufacturers, and therefore it can be advantageous to stockpile materials from each source separately to control the asphalt contents of the final product. Documentation of the source and tonnages should be required to be kept on file and available for review by environmental and transportation agencies.

3.2.2 Post-Consumer Shingles

Asphalt shingle recycling facilities should be required to document the source of the post-consumer shingles accepted at their facilities. Recycling facilities should screen in-coming loads to ensure no hazardous materials are accepted and loads do not exceed ten percent by weight of non-shingle material. Similarly to post-manufactured shingles, documentation of the source and tonnages should be required to be kept on file and available for review by environmental and transportation agencies.

Over 60 percent of post-consumer asphalt shingles come from storm damage. Many times these storms can damage newer roofs with recently installed shingles. When loads of post-consumer shingles are delivered to a recycling facility due to storm damage, asphalt contents and percentage of granular material can vary. Newer post-consumer shingles may contain lower asphalt contents and lower binder viscosities compared to older post-consumer shingles, which may have binder that is stiffer due to more aging and higher asphalt contents. Therefore, asphalt shingle recyclers that closely monitor their intake can have better control over stockpiling.

3.2 Asbestos Testing and Analysis

3.2.1 Post-Manufactured Shingles

No testing for asbestos should be necessary for post-manufactured shingles since asphalt shingles manufactured today do not contain asbestos.

3.2.2 Post-Consumer Shingles

Asphalt shingles manufactured in the United States prior to the mid 1980's may have contained asbestos. As a result, asphalt shingle recycling facilities are required to meet NESHAP and Occupational Safety and Health Act (OSHA) requirements. NESHAP requirements state that asbestos-containing roofing materials may not be ground up for recycling. NESHAP defines ACM as any material containing more than 1% asbestos as determined using polarized light microscopy. To ensure that delivered loads of post-consumer shingle scrap do not contain asbestos, many state agencies require the owner of the recycling facility to follow a specified sampling and testing plan. Samples are required to be obtained and tested for ACM using the polarized light method by an accredited laboratory. Typical sampling and testing frequencies require a sample to be obtained every 50 to 100 tons. In the event that a sample is found to contain greater than 1% ACM, the pile is required to be stockpiled separately and disposed of in accordance with state environmental regulations.

3.3 Sorting

3.3.1 Post-Manufactured Shingles

Post-manufactured shingles usually do not have specific sorting protocols since they are delivered on pallets and easily identified as clean of construction debris. However, post-manufactured shingles delivered in roll-offs can include shingle globs, metal or other objects that could damage the industrial grinders used in the processing and are many times screened to limit costly repairs.

3.3.2 Post-Consumer Shingles

Post-consumer shingles are often first sorted by trained personnel to remove all non-shingle material (i.e. paper, metal, plastic, felt paper). Sorting is done by hand over a conveyor belt or on the ground at the time of load dumping and again at the time of grinding (Figure 1). Removing all non-recyclable materials is important to the shingle recyclers as hammers or other large metal objects incur costly repairs to the industrial grinders and loss of time for machines down for repair. Recyclable material such as paper, metals, and plastics can be separated and recovered at a recycling facility. All non-recyclable material can be disposed of at a landfill.

Figure 1. Post-consumer shingle manual sorting

3.4 Processing

Processing of the post-manufactured and post-consumer RAS can be done with an industrial grinder. The industrial grinders utilize water nozzles to control dust and reduce heat build-up during the grinding process. Post-manufactured RAS can be more challenging to grind due to the softer asphalt which can clump together. This may contribute to further heat build-up which requires more water. In the case of nails present in the shingles during the grinding process, grinders can be fitted with pulley magnets and cross-bar magnets which can effectively remove them.

3.4.1 Sizing

State agencies require sizing of the RAS to meet gradation specifications. Sizing varies from 100% passing the 1/2-inch to 1/4-inch screen. If industrial grinders are not able to meet the state specification on the first grind, a screening process can be used to remove the oversized RAS or reprocess the RAS a second time (Figure 2).

States agencies have moved to a finer grind size to increase performance of the pavements. A finer grind size can increase fiber availability, surface area of the RAS binder, and eliminate tabs on the surface mixes.

Figure 2. RAS screening

3.4.2 Deleterious Materials

Minimum requirements for deleterious material contents range from 0.5 percent to 3.0 percent by weight depending on the agencies. Deleterious material includes all non-shingle material. Wood particles and metal shavings can contribute the majority of the weight of deleterious contents in RAS.

3.4.3 Moisture Content

Asphalt shingles can hold up to 20% moisture, and so it is important to keep the use of water during the process to a minimum. A moisture content of seven percent or less is optimum. RAS stockpiles can also absorb moisture from the bottom of the pile so it is important to place piles on a non-permeable surface and/or one with proper drainage to deter standing water. Higher moisture contents can result in clumping, bridging in the bins, or slower production rates. Storage of the RAS under a cover will keep the moisture content under control.

3.4.4 Stockpile Storage

Moisture contents and clumping can be better controlled when stockpiles are covered. Covering the RAS at the asphalt plant can protect it from rain and direct sunlight (Figure 3). To help prevent RAS clumping in a stockpile, some agencies allow HMA producers to blend in a percentage of sand with RAS stockpile.

RAS also has a very limited time in the drum. If the heat from the plant burner is working at removing the moisture, there is little time for the heat to reduce the viscosity of the RAS binder allowing it to separate from the RAS granules. RAS with high moisture contents then ultimately increase the potential for a poor bond between the RAS and virgin components.

Storage of the final post-consumer and post-manufactured RAS piles should be kept separate, and covering them will provide the following benefits:

- Protect the RAS from conglomerating due to direct sunlight;
- Protect the stockpiles from rain to keep the moisture content down;
- Protect the RAS piles from the wind that can cause fines blowing off the stockpile; and
- Protect the RAS piles from cross contamination.

Figure 3. Covered RAS stockpile

3.5 Quality Control for Asphalt Facilities

It is important to know the properties of the RAS prior to use in the HMA. Having consistent asphalt contents and gradations throughout both post-consumer and post-manufactured stockpiles helps ensure the final HMA end product contains the targeted mix volumetric properties and field density. As a result, state agencies require asphalt producers to verify the asphalt content, gradation, deleterious materials content, and moisture content of RAS used in mix designs. Some agencies such as the Illinois DOT require continual testing of RAS properties (i.e., asphalt content, gradation, Gmm) at specified frequencies as it is being stockpiled. If more than 20 percent of stockpile contains RAS with properties that deviate outside the targeted production range, than the stockpile may not be used for DOT projects.

3.5.1 Introduction to Plant

RAS is typically introduced in the HMA plant through a separate recycling bin and can vary depending on plant configurations (e.g., drum plant or batch plant). Many drum plant facilities introduce RAS into the drum recycle collar where RAP is normally introduced. Load cells or weigh belts measure the amount of RAS as it is metered into the collar. Some facilities load RAP over the RAS on the same conveyor belt to eliminate the blowing of RAS fines.

Load cells can give the highest accuracy in weighing the RAS, however, inconsistencies can still be found in the ability to keep the flow of RAS even and consistent on the belt. This includes bridging and clumping in the bins, which creates uneven distribution on the belt and leads to variability in asphalt contents in plant production mixes. One strategy to reduce variability on the belt is to use an auger system that distributes the RAS onto the belt.

3.5.2 Plant Temperatures

Higher temperatures at drum plants can help remove moisture from RAS more quickly and facilitate the blending of RAS binder with virgin binder. However, agencies are becoming increasingly concerned that higher temperatures accelerate RAS aging during construction. Additionally, the cost to the asphalt producer to increase temperatures during mixing can reduce the savings benefits of RAS both environmentally and economically.

4. RESEARCH PLAN

4.1 Demonstration Projects

Each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated different factors of asphalt mixes containing RAS. Table 4 summarizes the factors each state chose to investigate for their field demonstration project.

Table 4. RAS factors evaluated in field demonstration projects

State Agency	RAS Factors
Missouri	• Difference between a fine grind RAS and a coarse grind RAS • Effect of replacing a percentage of RAP with RAS
Iowa	• Effect of different RAS percentages
Minnesota	• Difference between post-manufacturer and post-consumer RAS • Comparison of RAS mixes to traditional RAP mixes
Indiana	• Compatibility of RAS with WMA foaming technology • Difference between asphalt mixtures containing RAP versus RAS
Wisconsin	• Using RAS with Evotherm® as a compaction aid at hot mix temperatures
Colorado	• Replacing a percentage of RAP with RAS
Illinois	• Using RAS in SMA as a replacement for fibers • Performance difference between laboratory and plant produced RAS mixes • Using RAS mixes with a PG70-28 compared to a PG58-28 with 12% GTR • Difference between RAS only mixes and RAP-RAS mixes

The mix designs developed for the field demonstration projects and the project location are summarized in Tables 5 and 6. A full description of the mix design properties and project location is provided in the reports for the individual state agencies in Appendices A through G.

Table 5. Multi-state mix design experimental plan

State Agency	Mix ID	% RAS	% RAP	RAS Source	Treatment
Missouri	15% RAP	0	15	-	RAP only
	Fine RAS	5	10	post-consumer	< 9.5 mm grind size
	Coarse RAS	5	10	post-consumer	< 12.5 mm size
Iowa	0% RAS	0	0	-	No RAS
	4% RAS	4	0	post-consumer	4% RAS
	5% RAS	5	0	post-consumer	5% RAS
	6% RAS	6	0	post-consumer	6% RAS
Minnesota	30% RAP	0	30	-	RAP only
	Post-Cons. RAS	5	0	post-consumer	Post-Consumer RAS
	Post-Manuf. RAS	5	0	post-manufactured	Post-Manufactured RAS
Indiana	HMA-RAP	0	15	-	HMA only using RAP
	HMA-RAS	3	0	post-consumer	HMA only using RAS
	WMA-RAS	3	0	post-consumer	Foaming WMA with RAS
Wisconsin	Evo	3	13	post-consumer	Evotherm®
	No Evo	3	13	post-consumer	No WMA additive
Colorado	RAP Only	0	20	---	RAP only
	RAS/RAP	3	15	post-manufactured	RAP with RAS
Illinois	Dcon 70-28P	0	5	post-consumer	PG 70-28 (Plant mix)
	Dcon 70-28L	0	5	post-consumer	PG 70-28 (Lab mix)
	Dcon 58-28L	0	5	post-consumer	PG 58-28 w/ 12% GTR (Lab mix)
	Curran 70-28P	11	5	post-consumer	PG 70-28 (Plant mix)
	Curran 70-28L	11	5	post-consumer	PG 70-28 (Lab mix)
	Curran 58-28L	11	5	post-consumer	PG 58-28 w/ 12% GTR (Lab mix)

Table 6. Demonstration project summary

State Agency	Mix ID	NMAS	Ndes	PG	Project Description
Missouri	15% RAP	12.5	80	64-22 w/ 10% GTR	1.75” surface course of a 3.75” overlay on concrete pavement (US Route 65)
	Fine RAS	12.5	80	64-22 w/ 10% GTR	
	Coarse RAS	12.5	80	64-22 w/ 10% GTR	
Iowa	0% RAS	12.5	76	58-28	2” surface course placed over concrete on State Highway 10
	4% RAS	12.5	76	58-28	
	5% RAS	12.5	76	58-28	
	6% RAS	12.5	90	58-28	
Minnesota	30% RAP	12.5	90	58-28	Surface course for MnRoads shoulders and mainline transitions on I-94
	Post-Cons. RAS	12.5	90	58-28	
	Post-Manuf. RAS	12.5	90	58-28	
Indiana	HMA-RAP	9.5	100	70-22	1.5” mill and overlay on US Route 6
	HMA-RAS	9.5	100	70-22	
	WMA-RAS	9.5	100	70-22	
Wisconsin	Evo	12.5	75	58-28	2” leveling course on State Highway 144
	No Evo	12.5	75	58-28	
Colorado	RAP Only	12.5	100	64-28	2” mill and overlay on US Route 36
	RAS/RAP	12.5	100	64-28	
Illinois	DCon 70-28P	12.5	80	70-28	2” SMA binder course placed over continuous reinforced concrete on Interstate 80
	DCon 70-28L	12.5	80	70-28	
	DCon 58-28L	12.5	80	58-28 w/ 12% GTR	
	Curran 70-28P	12.5	80	70-28	
	Curran 70-28L	12.5	80	70-28	
	Curran 58-28L	12.5	80	58-28 w/ 12% GTR	

4.1.1 Missouri

The Missouri Department of Transportation (MoDOT) investigated RAS grind size and asphalt mixes with RAS and modified asphalt binder. The objective of this demonstration project was to identify potential economic and performance benefits when incorporating a finer grind size of RAS in HMA using asphalt modified with GTR and transpolyoctenamer rubber (TOR). MoDOT's experimental plan included three mixes, a Control mixture containing 15 percent RAP and no RAS, a Fine RAS mixture containing 10 percent RAP and 5 percent fine RAS, and a Coarse RAS mixture containing 10 percent RAP and 5 percent coarse RAS. Each mixture contained a PG 64-22 binder with 10 percent GTR and 4.5% TOR by weight of the GTR.

4.1.2 Iowa

The Iowa Department of Transportation (Iowa DOT) investigated the effect of different percentages of post-consumer RAS in HMA. The objective of this demonstration project was to evaluate the performance of mixes containing RAS at increasing percentages and compare their performance to an Iowa DOT mix design containing no recycled product: no recycled RAP or RAS. The Iowa DOT demonstration project included a 0 percent RAS mix, 4 percent RAS mix, a 5 percent RAS mix, and a 6 percent RAS mix.

4.1.3 Minnesota

The Minnesota Department of Transportation (MnDOT) selected in-service pavement sections at their MnROAD Cold Weather Road Research Facility pavement test track for their demonstration project. The pavement sections were constructed in 2008 and included shoulder mixes and transition traffic lanes that used post-manufactured and post-consumer RAS. The pavement sections were selected to compare the performance of HMA containing post-manufactured RAS with HMA containing post-consumer RAS and to evaluate their performance to an asphalt mixture using RAP only, no RAS. MnDOT's demonstration project included three mixes: a 30 percent RAP mix with no RAS, a 5 percent RAS mix using post-consumer RAS, and a 5 percent RAS mix using post-manufactured RAS.

4.1.4 Indiana

The Indiana Department of Transportation (INDOT) investigated using RAS in combination with foaming warm mix asphalt (WMA) technology. The objective of this demonstration project was twofold: first, to evaluate the performance of WMA containing RAS, and second, to compare a typical INDOT mix design that contains RAP to a mix design that contains RAS. INDOT designed an experimental plan with three mixes: a mix with 15 percent RAP, a mix with 3 percent RAS, and a mix with 3 percent RAS produced using foaming WMA technology.

4.1.5 Wisconsin

The Wisconsin Department of Transportation (WisDOT) investigated the effect of using Evotherm® warm mix asphalt technology as a compaction aid in HMA containing post-consumer RAS. The objective of this demonstration project was to evaluate the performance of a typical WisDOT mix design containing RAS, with and without Evotherm®, at hot mix production and compaction temperatures during late season construction (November). To accomplish this, WisDOT's experimental plan included two mixes, each containing the same mix design using 3 percent RAS and 13 percent RAP. One mix contained Evotherm® while the other mix did not.

4.1.6 Colorado

The Colorado Department of Transportation (CDOT) investigated the economic and performance benefits when replacing RAP with RAS in HMA. The objective of this demonstration project was to compare a typical CDOT mix design that contains 20 percent RAP to a mix design that contains 15 percent RAP and 3 percent post-manufactured RAS.

4.1.7 Illinois

The Illinois Department of Transportation (IDOT) investigated the economic and performance benefits of replacing fibers and virgin asphalt with RAS in SMA. The objective of this demonstration project was to evaluate the performance of SMA mixtures using post-consumer RAS, RAP, and GTR with different base binders and to investigate the performance differences between laboratory produced SMA-RAS mixes to plant produced SMA-RAS mixes. The mixes for IDOT's demonstration project were collected from two different contractors, Curran Construction and D Construction, Inc. Each mix produced by the contractor was an SMA with a PG 70-28 containing 5 percent RAS with no added fibers. The Curran mix used 11 percent RAP in addition to the 5 percent RAS. Plant produced and laboratory produced samples of the mixes were obtained for performance testing. In addition, laboratory samples of each mix were also produced using a PG 58-28 with 12 percent GTR.

4.2 Laboratory Testing

During each field demonstration project, Iowa State University collected representative samples of each asphalt mixture for laboratory testing. A portion of the samples were sent to the University of Minnesota and the Minnesota Department of Transportation (MnDOT) for Semi-Circular Bend (SCB) testing and binder extraction and recovery, respectively. The laboratory testing plan is presented in Table 7.

The asphalt was recovered from the RAS and asphalt mixtures following AASHTO T164 Method A (Centrifuge Method) by using a blend of toluene and ethanol as the extraction solvent. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. At

Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO R29 "Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder."

Table 7. Laboratory testing plan

	Laboratory Test	Iowa State University	Univ. of Minnesota	Minnesota DOT	Missouri DOT
Processed Shingles	Binder Extraction			X	
	High Temperature PG			X	
	Gradation (Before Extraction)	X			
	Gradation (After Extraction)	X			
Mixture	Binder Extraction			X	
	Binder PG Characterization	X			
	Gradation	X			
	Dynamic Modulus	X			
	Flow Number	X			
	Beam Fatigue	X			
	Semi-Circular Bending (SCB)		X		
	Creep Compliance using BBR				X

Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution, and a washed gradation was conducted after extraction to evaluate the size distribution of the fine aggregates in the RAS product.

Laboratory performance testing was conducted on laboratory compacted samples of loose mix collected in the field during the demonstration projects. In the case of the Illinois demonstration project, performance testing was conducted on both field and laboratory produced mixes. Dynamic modulus tests were conducted to characterize the stiffness of the asphalt mixtures over a wide range of temperatures and frequencies. The flow number test was conducted to evaluate the permanent deformation resistance of the asphalt mixtures. Asphalt mixture durability and resistance to fatigue cracking was evaluated using the four-point bending beam apparatus. The semi-circular bending (SCB) test was conducted to evaluate the low-temperature cracking susceptibility of the asphalt mixtures. As an additional low-temperature test, the asphalt mixture samples were cut into small beams and tested at low temperatures in the bending beam rheometer (BBR).

4.2.1 Dynamic Modulus

The dynamic modulus test was conducted to determine the stress-strain relationship of the asphalt mixtures under continuous sinusoidal loading for a wide range of temperature and frequency conditions. A higher dynamic modulus indicates lower strains will result in a pavement structure when the asphalt mixture is stressed from repeated traffic loading. The mechanistic-empirical pavement design guide (MEPDG) uses |E*| as the stiffness parameter to calculate an asphalt pavements strains and displacements.

The test was conducted by following AASHTO TP62-07. Five replicate test specimens of each asphalt mixture were compacted to 100 mm in diameter and 150 mm in height at 7 ± 0.5 percent air voids. The specimens were directly compacted to their geometry using a Pine gyratory compactor with a compaction mold modified to a 100 mm inner diameter. Specimens were tested by applying a continuous sinusoidal load at 9 different frequencies (0.1, 0.3, 0.5, 1, 3, 5, 10, 20, and 25 Hz) and three different temperatures (4, 21, and 37°C). Sample loading was adjusted to produce strains between 50 and 150 μstrain in the sample.

A UTM-25 servo-hydraulic testing machine from IPC Global, which is capable of applying a load up to 25 kN, was used to test asphalt mixture specimens. The UTM-25 was housed in an environmental chamber capable of controlling the temperature of the test specimens. Three linear variable differential transformers (LVDTs) were mounted between gauge points glued to the test specimens to measure the deformations in the sample. The LVDTs were spaced 120 degrees apart. Dynamic modulus computer software from IPC Global was used to control the load settings and calculation of the dynamic modulus for each test run. This is the same software designed to control the Asphalt Mixture Performance Tester.

4.2.2 Flow Number

The flow number test was conducted to measure the permanent deformation resistance of the asphalt mixtures. A repeated dynamic load was applied to the specimen for up to several thousand load cycles. The flow number was defined as the number of load cycles an asphalt mixture can tolerate until it flows. Cumulative permanent deformation in the sample was plotted versus load cycles. The flow number was reached at the onset of tertiary flow, which was determined at the cycle corresponding to the lowest cumulative percent strain rate.

Tests were conducted following procedures used in NCHRP Report 465. The same specimens used for dynamic modulus test were used for the flow number test since the dynamic modulus test is nondestructive. The specimens were placed in the UTM-25, unconfined, with a testing temperature of 37°C to simulate the climactic conditions that cause pavement to be susceptible to rutting. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 sec followed by 0.9 sec of dwell time. The loading cycles were repeated for a total of 10,000 load cycles or until the specimen reached 5 percent cumulative strain. Three LVDT’s were attached to each sample during the test to measure the cumulative strains.

4.2.3 Four-Point Bending Beam

Four-point beam fatigue testing was conducted according to AASHTO T321, “Determining the Fatigue Life of Compacted Hot-Mix Asphalt (HMA) Subjected to Repeated Flexural Bending.” Samples of field produced asphalt were compacted to 7 ± 0.5 air voids in a linear kneading compactor to obtain a compacted slab with dimensions 380 mm in length, 210 mm in width, and

50 mm in height. Each slab was saw-cut into three beams with dimensions 380 mm in length, 63 mm in width, and 50 mm in height. Two slabs were compacted for each asphalt mixture to produce six beams for testing.

The equipment used to conduct the four-point bending beam test included a digitally controlled, servo-pneumatic closed loop beam fatigue apparatus from IPC Global. A control data and acquisition system (CDAS) was connected to the beam fatigue apparatus which connected to a computer that controlled the load during the test. The beam fatigue apparatus was housed in an environmental chamber maintained at the testing temperature of 20 ± 0.5 °C. Beams were placed in the environmental chamber at least two hours prior to testing to allow them to equilibrate to the testing temperature. The mode of loading used for the test was strain controlled. Haversine wave pulses were applied to the specimen during the test at 10 Hz.

Testing was conducted at varying strain levels to generate a fatigue curve for each asphalt mixture. For each of the six beam specimens prepared for each asphalt mixture, strain levels of 375, 450, 525, 650, 800, and 1000 micro-strains were applied. Testing at these strain levels were repeated for all the mixtures tested except for the two Indiana mixtures containing 3% RAS. Due to a limited amount of material, only 3 three beams of these mixtures were tested at 400, 700, and 1000 micro-strain levels.

During testing of a beam specimen, properties of flexural stiffness, modulus of elasticity, dissipated energy, and phase angle were recorded by the software every 10 cycles. On the 50th cycle, the stiffness of the beam specimen was recorded as the initial stiffness. The beam specimens were tested until failure, which was defined as the cycle corresponding to a 50 percent reduction of the initial beam flexural stiffness.

4.2.4 Semi-Circular Bending

To evaluate the low temperature fracture properties of the mixes, 150 mm diameter specimens containing 7 ± 0.5 percent air voids were compacted in Iowa State University's laboratory and delivered to the University of Minnesota for SCB testing. SCB tests were conducted by following the procedure in "Investigation of Low Temperature Cracking in Asphalt" (Marasteanu et al., 2007). Testing was conducted at four different low temperatures: PG low temperature, PG low temperature +4°C, PG low temperature +10°C, and PG low temperature +16°C. Triplicate specimens were tested at each temperature.

All tests were performed inside an environmental chamber, and liquid nitrogen was used to obtain the required low temperature. The temperature was controlled by the environmental chamber temperature controller and verified using an independent platinum resistive-thermal-device (RTD) thermometer. The load line displacement (LLD) was measured on both faces of the test specimens using a vertically mounted Epsilon extensometer with 38 mm gage length and ±1 mm range. One end was mounted on a button that was permanently fixed on a specially made frame, and the other end was attached to a metal button glued to the sample. The average LLD measurement was used for each specimen. The crack mouth opening displacement (CMOD) was

recorded by an Epsilon clip gage with 10 mm gage length and a +2.5 and -1.0 mm range. The clip gage was attached at the bottom of the specimen. A constant CMOD rate of 0.0005mm/s was used and the load and load line displacement (P-u), as well as the load versus LLD curves were plotted. A contact load with a maximum load of 0.3 kN was applied before the actual loading to ensure uniform contact between the loading plate and the specimen. The testing was stopped when the load dropped to 0.5 kN in the post peak region. The load and load line displacement data were used to calculate the fracture toughness and fracture energy (G_f).

4.2.5 Low Temperature Creep Compliance using the Bending Beam Rheometer

An additional low temperature performance test was conducted using the BBR by following the test method proposed in the report "Development of a Simple Test to Determine the Low Temperature Creep Compliance of Asphalt Mixtures" (Marasteanu et al., 2009). This performance test uses the BBR to apply a creep load to a thin beam of an asphalt mixture cut from a compacted laboratory specimen. The advantage of this test method is that the creep stiffness and creep compliance of the asphalt mixture can be directly tested at low temperature in a relatively quick and convenient process.

Asphalt mixture test specimens of 150mm in diameter and 115mm in height were compacted at Iowa State University's laboratory in a gyratory compactor. One gyratory sample was compacted for each mixture type in the pooled fund study. The specimens contained 7 ± 1% air voids and were delivered to the Missouri Department of Transportation's (MoDOT) central materials laboratory in Jefferson City, MO where they were cut and tested. MoDOT cut each gyratory sample horizontally into thirds for a bottom, middle, and top slice. From each slice, MoDOT cut five thin beams with dimensions 6.35 ± 0.05mm thick, 12.70 ± 0.05mm wide, by 127 ± 2.0mm long for a total of 15 beams. The 15 beams were randomly selected to be in one of three temperature treatment groups: PG low temperature +4°C, PG low temperature +10°C, and PG low temperature +16°C. Each treatment group contained five beams. The beams were tested in the BBR using the same procedure beams of asphalt are tested in the BBR following AASHTO T313-08. For testing the beams of asphalt mixtures, however, the 980 ± 50 mN load used in AASHTO T313-08 was increased to 4413 ± 50 mN. The duration of the test was 240 seconds with creep stiffness measurements recorded at 8, 15, 30, 60, 120, and 240 seconds.

4.3 Pavement Condition Surveys

After pavement construction for the demonstration project, field evaluations were conducted on the pavement test section following every winter season after construction to assess the field performance of the pavement concerning cracking, rutting, and raveling.

5. LABORATORY TEST RESULTS AND ANALYSIS

5.1 Mix Design Properties

The first objective of the laboratory portion of the study was to evaluate the mix designs and their individual material components. Properties of the RAS used in the mix designs for the demonstration projects are presented in Tables 8 and 9. All the state agencies for the demonstration projects specified a 1/2" minus RAS grind size. In the case of the Missouri demonstration project, a 3/8" minus grind was compared to a 1/2" minus grind.

The asphalt contents of the post-manufactured RAS sources (Minnesota and Colorado) range from 14.6 to 18.1 percent asphalt. This is lower than the asphalt content measured in the post-consumer RAS sources which range in asphalt content from 20.5 to 36.7 percent asphalt. RAS from post-consumer shingles will contain a larger percentage of asphalt because older shingles were made with a cellulose-fiber paper-backing which absorbs more asphalt than currently used fiberglass-mat backing shingles. Also, as shingles age on a roof, the loss of aggregate granules increases the percentage of asphalt in the shingle. The larger range in asphalt contents of post-consumer shingles highlights the variability of different post-consumer shingle sources and the importance of keeping shingles from different sources separate during recycling operations.

Table 8. Asphalt content, performance grade, and gradation of RAS before extraction

Sieve Size (US)	MO		IA	MN		IN	WI	CO	IL
	coarse grind	fine grind							
	post-cons. RAS	post-cons. RAS	post-cons. RAS	post-manuf. RAS	post-cons. RAS	post-cons. RAS	post-cons. RAS	post-manuf. RAS	post-cons. RAS
3/4"	100	100	100	100	100	100	100	100	100
1/2"	98	100	97	100	100	100	100	99	100
3/8"	94	99	95	95	99	97	99	95	100
#4	75	82	84	70	85	74	83	70	91
#8	62	67	67	56	73	62	70	55	74
#16	42	43	44	32	49	38	47	31	48
#30	22	21	22	12	24	18	24	13	24
#50	12	12	10	4	10	9	11	6	11
#100	5	5	3	1	3	4	3	2	3
#200	1.2	0.9	0.6	0.4	0.5	0.7	0.6	0.3	0.5
% AC(1)	21.7	25.0	21.7	14.6	20.5	26.8	35.4	18.1	36.7
High PG	137.3	146.1	124.1	109.1	122.5	134.2	124.1	111.2	129.7

(1)Results from MnDOT's chemical extraction

The high temperature performance grade (PG) of the extracted RAS binders is also reported in Table 8. MnDOT tested all the RAS sources for their high temperature PG using the DSR. The high temperature PG of the RAS binders is higher than traditional paving grade binders. This is expected since the binder in roofing shingles is produced with an air-blowing process which oxidizes the asphalt.

The high temperature PG of the post-consumer RAS binder ranges from 122.2°C to 146.1°C. These temperatures are noticeably higher than the post-manufactured RAS binder which ranges from 109.1°C to 111.2°C. The post-consumer RAS binders are stiffer because they come from in-service roofing shingles that have experienced at least several years of aging. Post-manufactured RAS comes from waste produced during shingle manufacturing.

The gradation of the RAS aggregate granules is presented in Table 9. The aggregate particle size distributions have the characteristics of finely crushed sand with approximately 20 to 25 percent passing the #200 sieve. This shows that in addition to replacing virgin asphalt, RAS can also reduce the amount of fine aggregate and dust in a mix design.

Table 9. Aggregate gradation of RAS after extraction

Sieve Size (US)	MO		IA	MN		IN	WI	CO	IL
	Coarse RAS	Fine RAS		post-manuf. RAS	post-cons. RAS				
3/4"	100	100	100	100	100	100	100	100	100
1/2"	97	99	99	100	100	100	100	100	100
3/8"	96	99	98	100	100	99	99	100	100
#4	90	94	95	99	100	90	99	95	97
#8	85	91	90	97	99	87	89	93	91
#16	67	73	72	80	85	69	71	74	74
#30	46	53	51	58	65	47	47	54	52
#50	39	46	40	40	49	40	39	46	44
#100	31	37	30	28	35	34	31	35	36
#200	21.9	26.1	21.3	22.0	24.1	26.5	23.0	26.4	27.8

The proportions of virgin asphalt and recycled asphalt in each of the asphalt mixtures are presented in Table 10. The percent binder replacements for all the mixes range from 12.9 to 35.0 percent. These values are based on the laboratory mix designs and/or job mix formula (JMF). However, since most of the mixes tested in this study were field samples, the exact material proportions will slightly vary from the mix designs. The asphalt content measured from the extraction of the field mix samples are shown in the far right column in Table 10.

Table 10. Mix design asphalt contents(1)

State Agency	Mix ID	% RAS	% RAP	%AC(1) RAS	%AC in RAP	% Virgin AC	% Binder Replacement	Total % AC	%AC in production sample(2)
Missouri	15% RAP	0	15	-	4.5	4.0	14.9	4.7	4.3
	Fine RAS	5	10	22.1	4.5	3.7	30.2	5.3	4.8
	Coarse RAS	5	10	22.1	4.5	3.7	30.2	5.3	4.8
Iowa	0% RAS	0	0	-	-	5.5	0	5.5	5.3
	4% RAS	4	0	20.5	-	4.6	16.3	5.5	5.5
	5% RAS	5	0	20.5	-	4.4	19.4	5.4	5.8
	6% RAS	6	0	20.5	-	4.2	22.8	5.4	5.3
Minnesota	30% RAP	0	30	-	5.9	3.5	33.3	5.3	5.5
	Post-Cons. RAS	5	0	26.0	-	3.7	26.0	5.0	3.9
	Post-Manuf. RAS	5	0	18.0	-	3.9	18.8	4.8	4.8
Indiana	HMA-RAP	0	15	-	7.3	4.6	19.3	5.7	5.6
	HMA-RAS	3	0	26.8	-	5.4	12.9	6.2	6.0
	WMA-RAS	3	0	26.8	-	5.4	12.9	6.2	6.0
Wisconsin	Evo	3	13	30.1	3.8	3.2	30.4	4.6	4.7
	No Evo	3	13	30.1	3.8	3.2	30.4	4.6	4.8
Colorado	RAP Only	0	20	-	4.5	4.2	17.6	5.1	4.5
	RAS/RAP	3	15	18.1	4.5	4.0	23.1	5.2	4.9
Illinois	DCon 70-28P	5	0	26.0	-	4.9	21.0	6.2	6.0
	DCon 70-28L	5	0	26.0	-	4.9	21.0	6.2	6.2
	DCon 58-28L	5	0	26.0	-	4.9	21.0	6.2	5.6
	Curran 70-28P	5	11	26.0	7.1	3.9	35.0	6.0	5.6
	Curran 70-28L	5	11	26.0	7.1	3.9	35.0	6.0	6.3
	Curran 58-28L	5	11	26.0	7.1	3.9	35.0	6.0	5.7

(1)Values reported in the mix design which slightly vary from the values measured by MnDOT as presented in Table 8.
(2)Results from MnDOT's chemical extraction of production sample

5.2 Binder Characterization

The performance grade of the binder extracted from the field samples and the asphalt binder used during production is presented in Table 11. When RAS and/or RAP is added to the mix designs of each state demonstration project, the binder performance grade increases on the high and low side as expected. While the increase on the high PG side will stiffen the asphalt mixture to help reduce permanent deformation, the increase on the low PG side could increase the low temperature cracking potential of the mixture.

To compensate for the increased low temperature stiffness due to the addition of RAS and/or RAP materials, it is common practice to use a softer virgin binder with a lower PG. However, RAS and RAP have different performance grades and asphalt contents. Knowing which virgin binder to use or the amount of recycled product to add to the virgin binder is necessary to achieve a desired final PG. Since blending charts could theoretically be used to estimate these values, an attempt was made to develop a “rule-of-thumb” of how RAS and/or RAP binder will change the low temperature grade of an asphalt mixture.

Table 11. Mix design performance grade

State Agency	Mix ID	% RAS	% RAP	PG High Temp, °C	PG Inter. Temp, °C	PG Low Temp, °C	PG
Missouri	Asphalt Binder	-	-	70.3	24.1	-22.8	70-22
	15% RAP	0	15	75.0	26.3	-16.8	76-16
	Fine RAS	5	10	90.1	28.7	-8.7	94-4
	Coarse RAS	5	10	88.3	28.3	-4.9	94-4
Iowa	Asphalt Binder	-	-	61.1	17.9	-28.2	58-28
	0% RAS	0	0	73.0	23.7	-19.7	72-16
	4% RAS	4	0	75.8	21.3	-19.1	72-16
	5% RAS	5	0	81.3	22.1	-16.8	76-16
	6% RAS	6	0	86.1	24.4	-14.7	86-10
Minnesota	Asphalt Binder	-	-	-	-	-	58-28
	30% RAP	0	30	68.8	20.6	-22.7	64-22
	Post-Cons. RAS	5	0	71.1	19.7	-21.2	70-16
	Post-Manuf. RAS	5	0	71.3	18.5	-21.7	70-16
Indiana	Asphalt Binder	-	-	72.2	25.3	-24.2	70-22
	HMA-RAP	0	15	75.6	26.2	-20.1	70-16
	HMA-RAS	3	0	77.6	26.2	-14.2	76-10
	WMA-RAS	3	0	78.8	26.3	-15.1	76-10
Wisconsin	Asphalt Binder	-	-	60.7	18.0	-29.1	58-28
	Evo	3	13	68.5	18.7	-24.0	64-22
	No Evo	3	13	69.5	20.3	-22.5	64-22
Colorado	Asphalt Binder	-	-	66.4	12.4	-34.8	64-34
	RAP Only	0	20	67.6	18.7	-27.5	64-22
	RAS/RAP	3	15	71.9	19.7	-21.1	64-16
Illinois	Asphalt Binder	-	-	73.2	15.5	-29.9	70-28
	DCon 70-28P	0	5	72.8	21.0	-24.3	70-22
	DCon 70-28L	0	5	72.7	19.1	-23.7	70-22
	DCon 58-28L-GTR	0	5	77.2	18.5	-21.3	76-16
	Asphalt Binder	-	-	73.2	15.5	-29.2	70-28
	Curran 70-28P	11	5	82.8	26.8	-18.1	82-16
	Curran 70-28L	11	5	84.4	25.7	-14.5	82-10
	Curran 58-28L-GTR	11	5	81.8	23.5	-17.7	76-16

The average results of all the mixes show that for every 1 percent increase in RAS, the low temperature grade will increase 1.9°C; and for every 1 percent increase in RAP, the low temperature grade will increase 0.3°C. Therefore, as a rule of thumb, 3 percent RAS or 20 percent RAP would be the amount of recycled material needed for no more than one low temperature grade bump (6°C).

The wide range of asphalt contents in the RAS materials used in this study (from 14.6 percent to 36.7 percent) demonstrates the importance of evaluating the effects of RAS binder based on the percent binder replaced in the mix, rather than the percentage of RAS. When considering all the pooled fund study results, the average RAS asphalt content was 24.5 percent and the average optimum asphalt content of the mixtures was 5.5 percent. Using these values and the results above, for every 1 percent increase in binder replacement with RAS, the low temperature grade will increase 0.43 percent. For every 1 percent increase in binder replacement with RAP, the low temperature grade will increase 0.3 percent. Therefore, to cap the increase in the low temperature performance grade by one grade bump (6°C), either a maximum of 14 percent binder replacement with RAS binder could be used or a maximum of 20 percent binder replacement with RAP binder could be used.

Of course, the above analysis is only based on the average results when using all the data from the demonstration projects. It is important to also consider the large differences in material properties, sources, and factors in the experimental design for each state's demonstration project. Some demonstration projects used post-consumer RAS while others used post-manufactured RAS. Also, some demonstration projects used polymers and/or recycled tire rubber to modify the virgin binder which may have confounding effects when blended with recycled binders. Therefore, the variety of demonstration projects shows the necessity to further evaluate the projects on a case-by-case basis. The state summaries in Appendices A through G, discuss binder results of each project in greater detail.

5.3 Dynamic Modulus

The dynamic modulus test data was analyzed at selected temperatures and frequencies to determine which RAS materials and other mix treatments affect the mean dynamic modulus values. These results are evaluated on a per-project basis in Appendices A through G. The test data was also used to construct master curves, where dynamic modulus data from frequency sweeps were shifted to obtain one smooth curve that plots the dynamic modulus over a wide frequency range at a designated reference temperature.

The following sigmoidal function was used to model the mater curves.

$$\text{Log}|E^*| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma(\log f_r)}}$$

where:

fr = reduced frequency at the reference temperature;
δ = minimum value of E^*;
$\delta + \alpha$ = maximum value of E^*; and
β, γ = parameters describing the shape of the sigmoidal function.

The following second-order polynomial equation was used to calculate the shift factors for each frequency sweep at a fixed temperature.

$$\log f_r = \log f + a_1(T_R - T) + a_2(T_R - T)^2$$

where:

fr = reduced frequency at the reference temperature;
f = loading frequency at the test temperature;
a_1, a_2 = the fitting coefficients;
T_R = the reference temperature, °C; and
T = the test temperature, °C.

The reference temperature was selected as 21°C. Fitting parameters were determined using numerical optimization with the "Solver" function in Microsoft Excel.

The dynamic modulus master curves are presented in Figures 4 through 11. For the Missouri, Iowa, and Minnesota demonstration projects, the dynamic modulus of the mixes increases as RAS is incorporated to the mix designs. For the Indiana project, the 3 percent RAS mixes had comparable dynamic modulus values to the 15 percent RAP mixes. For the Colorado project, the 20 percent RAP was a stiffer mix than the 15 percent RAP and 3 percent RAS mix. When Evotherm® was added to the Wisconsin mix, no statistical change in the dynamic modulus was detected. In the case of the Illinois demonstration project, it showed the laboratory mixes were stiffer than the plant mixes. Additionally, the mixes using a PG 58-28 with 12 percent GTR maintained similar dynamic modulus values as the mixes using a polymer modified PG 70-28. The dynamic modulus results of each project are further analyzed in the state summaries.

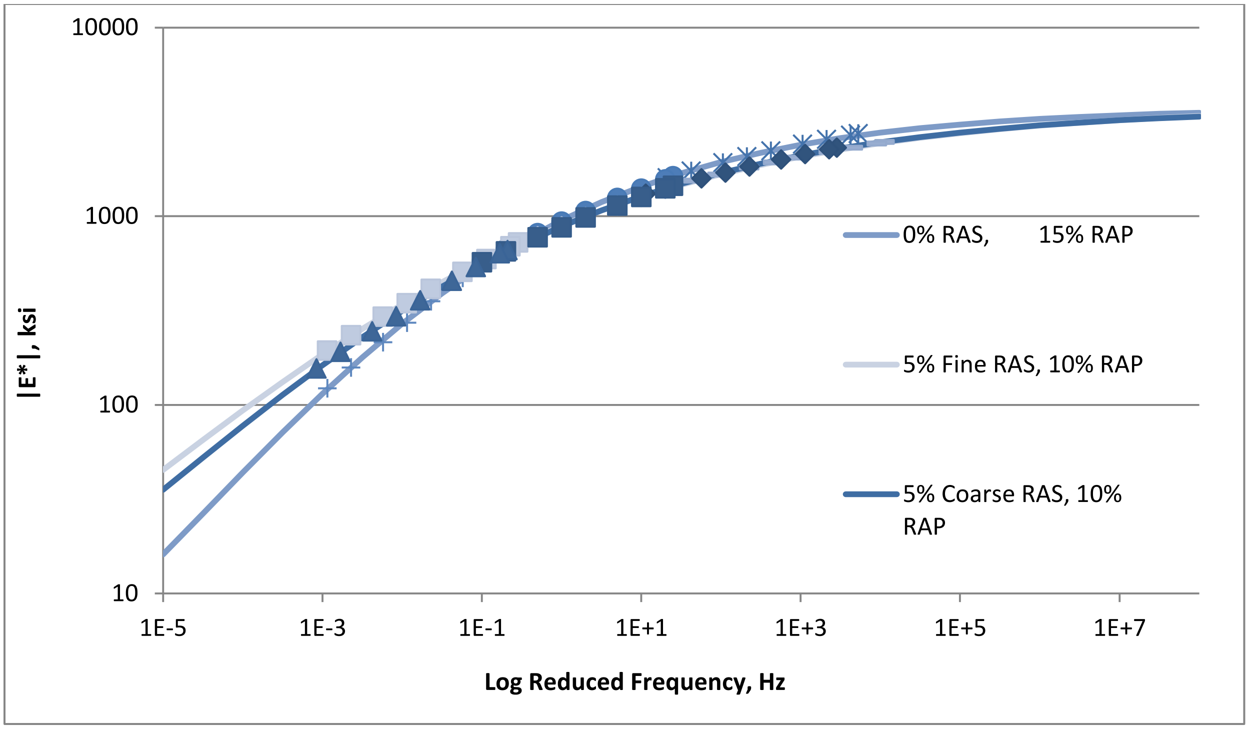


Figure 4. Missouri demonstration project dynamic modulus results

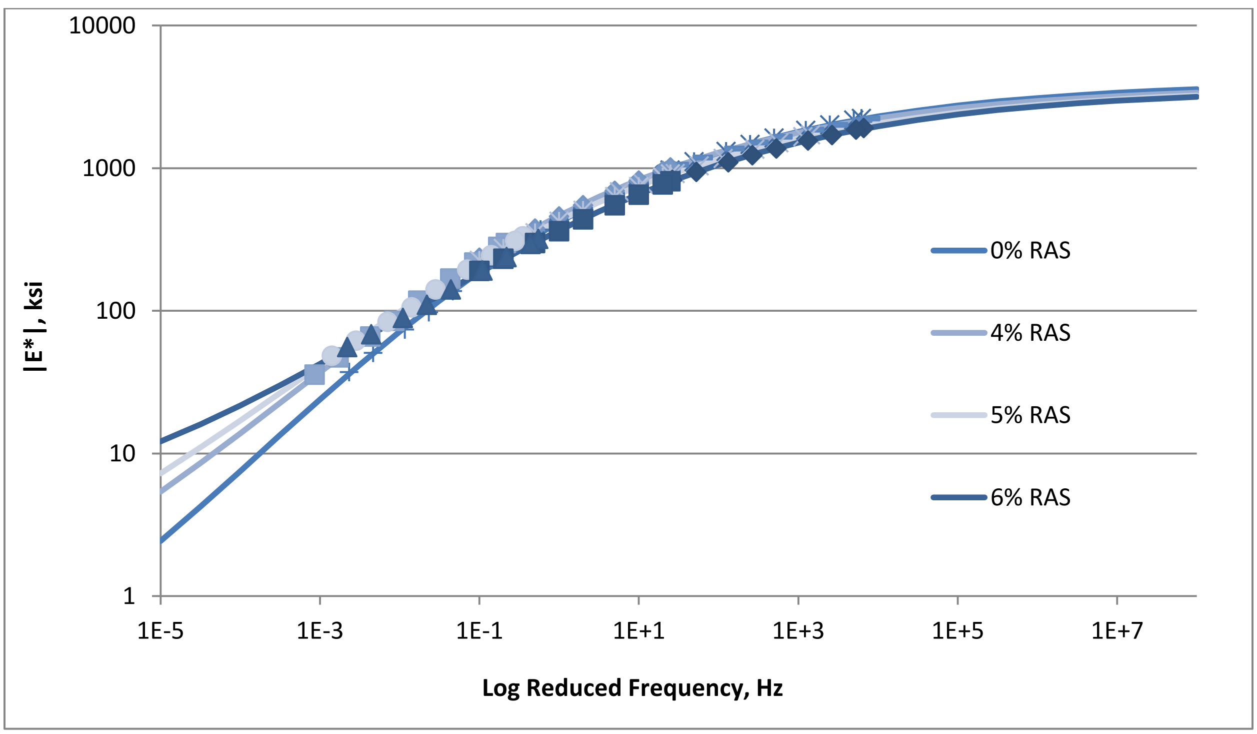


Figure 5. Iowa demonstration project dynamic modulus results

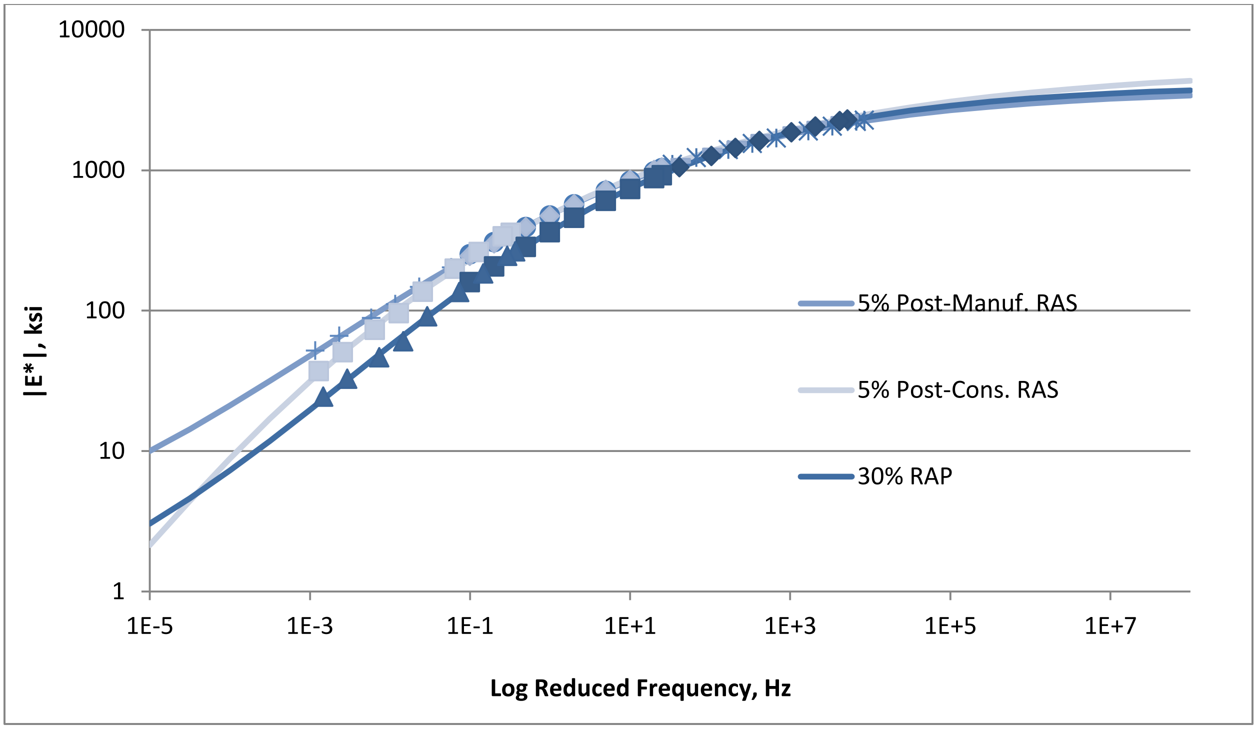


Figure 6. Minnesota demonstration project dynamic modulus results

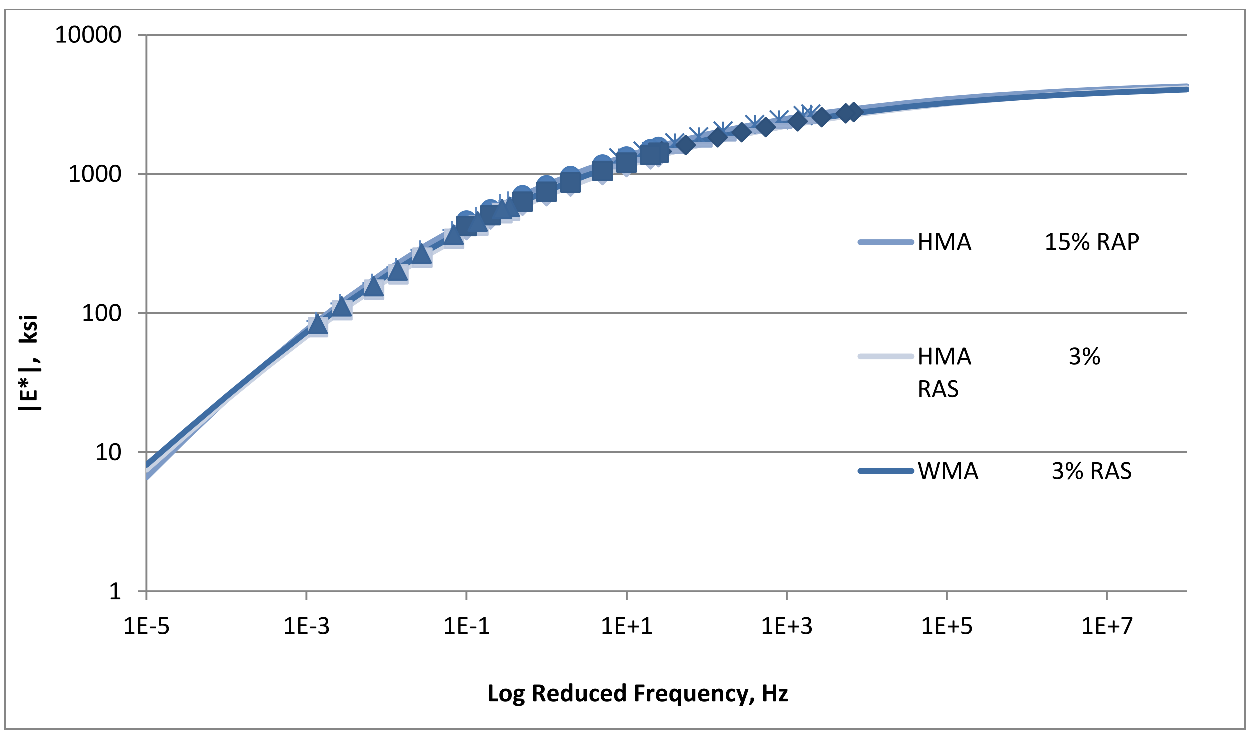


Figure 7. Indiana demonstration project dynamic modulus results

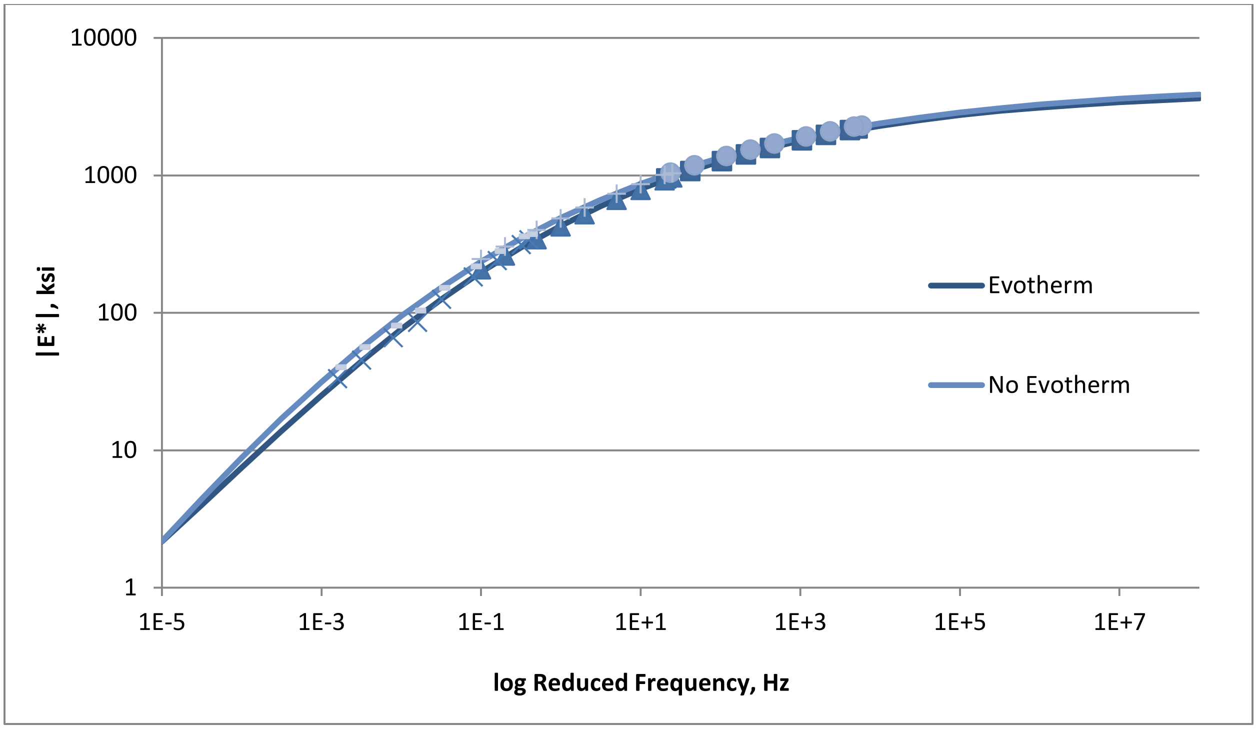


Figure 8. Wisconsin demonstration project dynamic modulus results

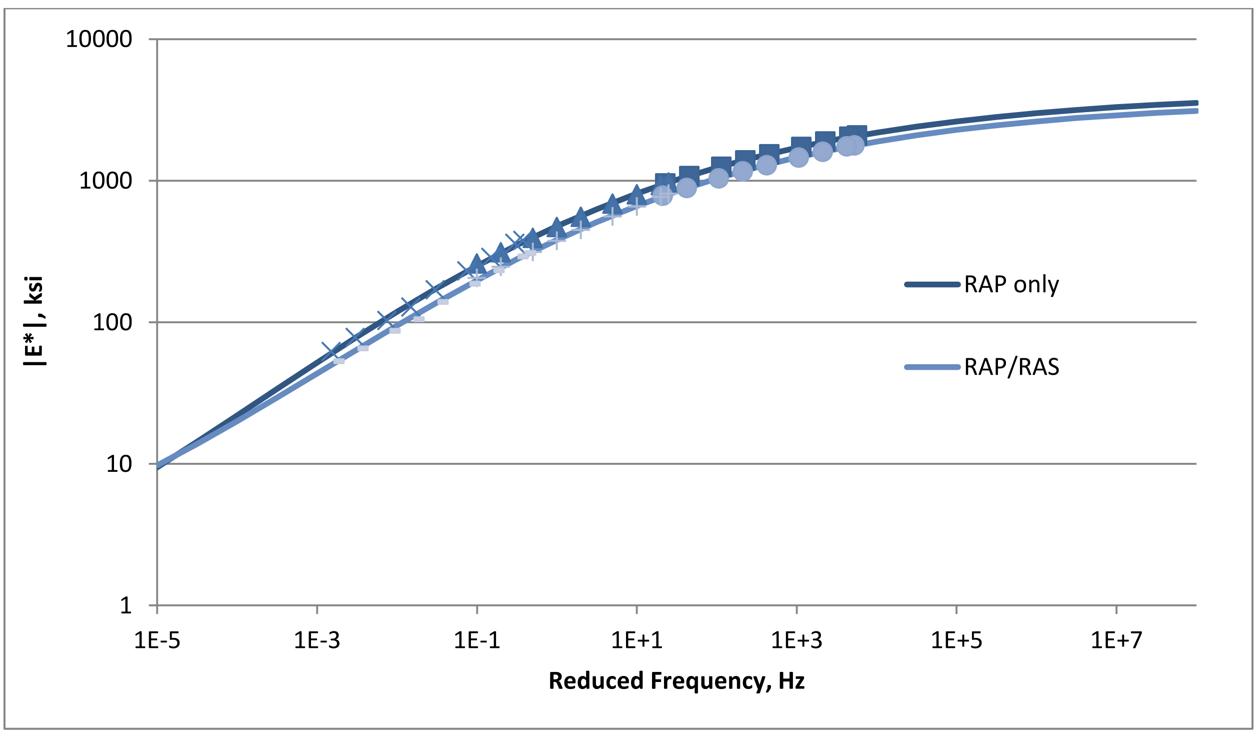


Figure 9. Colorado demonstration project dynamic modulus results

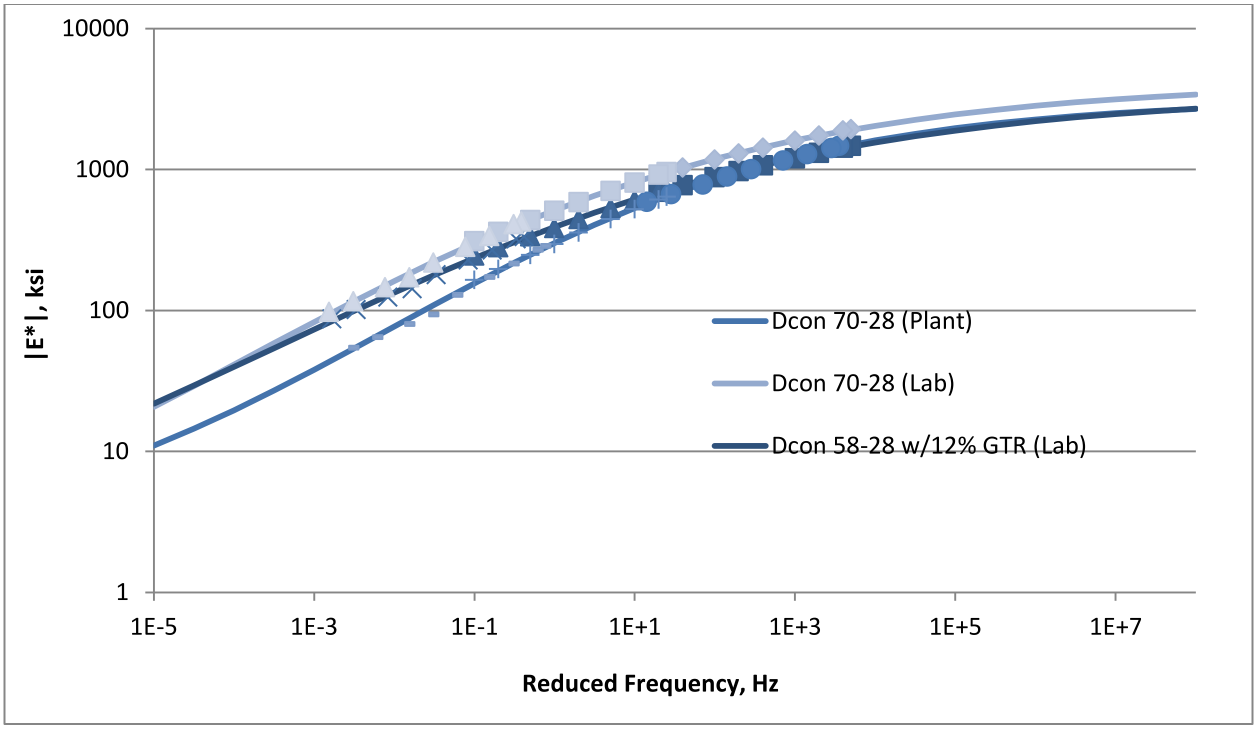


Figure 10. Illinois demonstration project by D Construction dynamic modulus results

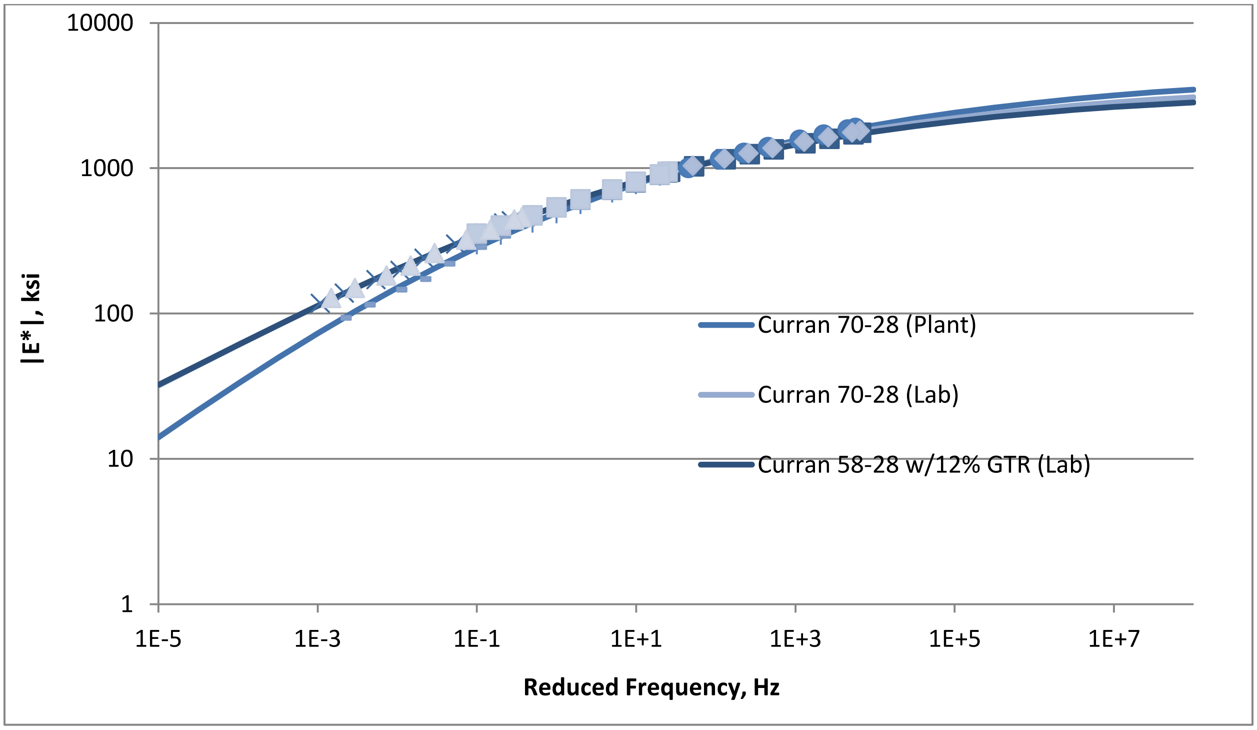


Figure 11. Illinois demonstration project by Curran dynamic modulus results

5.4 Flow Number

The mean flow numbers, as presented in Table 12, demonstrate that higher amounts of RAS and/or RAP will increase the flow number, and thus the rutting resistance, of the asphalt mixture. For example, as RAS is increased in the mix design for the Iowa project, the flow number increases. Likewise, when 11 percent RAP is added to the Illinois SMA mixture, the flow numbers also increase.

Mixes with larger flow numbers also have relatively higher binder performance grades. The Iowa, Minnesota, and Wisconsin mixes, which have a PG 58-28 binder, possess the lowest flow numbers of the pooled fund study. In contrast, the Missouri, Illinois, and Indiana mixes possess the highest flow numbers. Each of their binder grades either have a high temperature PG of 70 or use GTR to stiffen the virgin binder. Therefore, not only the amount of recycled product (i.e., RAS and RAP), but also the performance grade of the base binder has a large effect on the rutting resistance of the mixes.

Table 12. Flow number results

State Agency	Mix ID	% RAS	% RAP	PG	Flow Number	Standard Deviation
Missouri	15% RAP	0	15	64-22 w/ 10% GTR	>10000	-
	Fine RAS	5	10	64-22 w/ 10% GTR	>10000	-
	Coarse RAS	5	10	64-22 w/ 10% GTR	>10000	-
Iowa	0% RAS	0	0	58-28	711	305.2
	4% RAS	4	0	58-28	2425	1044.1
	5% RAS	5	0	58-28	6092	796.8
	6% RAS	6	0	58-28	5899	397.7
Minnesota	30% RAP	0	30	58-28	767	425.8
	Post-Cons. RAS	5	0	58-28	2497	412.6
	Post-Manuf. RAS	5	0	58-28	1705	347.6
Indiana	HMA-RAP	0	15	70-22	6578	884.9
	HMA-RAS	3	0	70-22	9865	176.9
	WMA-RAS	3	0	70-22	9986	20.4
Wisconsin	Evo	3	13	58-28	3902	2265.6
	No Evo	3	13	58-28	2462	1113.7
Colorado	RAP Only	0	20	64-28	8033	2379.4
	RAS/RAP	3	15	64-28	7687	3919.9
Illinois	DCon 70-28P	5	0	70-28	7923	2522.4
	DCon 70-28L	5	0	70-28	>10000	-
	DCon 58-28L	5	0	58-28 w/ 12% GTR	8737	2035.1
	Curran 70-28P	5	11	70-28	>10000	-
	Curran 70-28L	5	11	70-28	>10000	-
	Curran 58-28L	5	11	58-28 w/ 12% GTR	>10000	-

5.5 Four-Point Bending Beam

A phenomenological approach for fatigue analysis was selected as the chosen methodology to evaluate the fatigue life properties of the mixtures. The phenomenological approach relates the tensile strain at the bottom of an asphalt pavement layer to the number of load repetitions to failure (Ghuzlan et al. 2006). In this approach, fatigue life is plotted versus stress or strain on a log-log scale.

Since strain-controlled was used as the mode of loading, a log-log regression was performed between strain and the number of cycles to failure (N_f), (Figure 12). The relationship between strain and N_f can be modeled using the power law relationship as presented in the following equation.

$$N_f = K1\left(\frac{1}{\varepsilon_o}\right)^{K2}$$

where:

N_f = cycles to failure;
εo = flexural strain;
K1 = regression constant; and
K2 = regression constant.

The fatigue model can be calibrated to relate laboratory to field conditions by applying a shift factor, the hypothesis being that laboratory fatigue tests can simulate field conditions. Because of the challenging nature of duplicating field conditions in a laboratory, no universal shift factor has been measured. Rather, shift factors have ranged between 4 and 100 (NCHRP 2010).

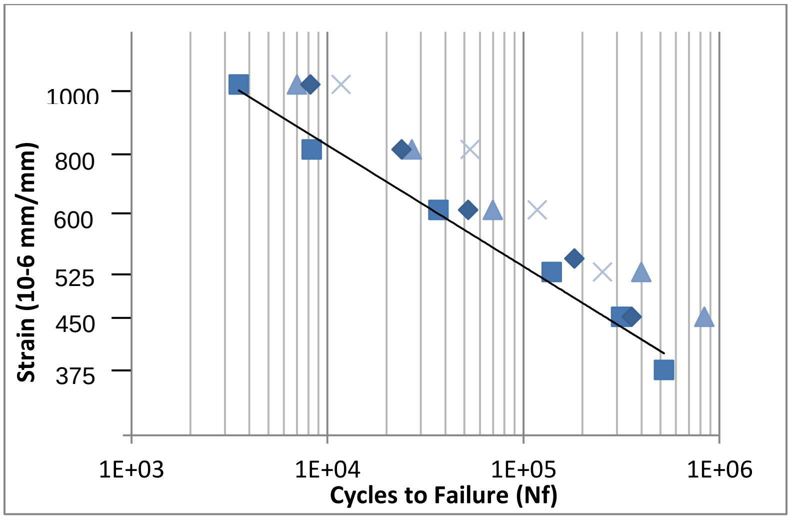


Figure 12. Sample fatigue curve

Pavements that have a higher resistance to tensile strains that develop at the bottom of an asphalt layer due to repeated traffic will have a greater resistance to fatigue cracking. Therefore, fatigue curves of several asphalt mixtures can be used to rank the mixtures resistance to fatigue cracking.

However, the results must take into consideration the mode of loading. Research from the Strategic Highway Research Program (SHRP) A003-A project (Tangella et al. 1990) showed that materials that are more flexible (lower stiffness) perform better in constant strain. The constant strain mode of loading best represents the performance of thin pavements (less than 4 inches) while the constant stress mode of loading best represents the performance of thick pavements (greater than 6 inches). Materials that are stiffer may not perform as well under constant strain in the laboratory, but when used in thick pavements, lower tensile strains will develop under field loading. Therefore, when fatigue testing is done in a constant strain mode of loading, fatigue evaluations should be made in the context of the pavement structure.

If tensile strains are low enough in a pavement structure, the pavement has the ability to heal and therefore no damage cumulates over an indefinite number of load cycles. The level of this strain is referred to as the fatigue endurance limit (FEL). Identifying the fatigue endurance limit in a laboratory is somewhat elusive because it is impossible to test a sample to an infinite number of cycles. The researchers under NCHRP Report 646 (2010) developed a practical definition of FEL as the strain level at which a sample could withstand 50 million load cycles. If a shift factor of 10 was applied to the test results, it would be estimated that the pavement could withstand 500 million loading cycles which represents 40 years of traffic.

Because it can take up to 50 days of testing to see if a sample reaches 50 million cycles, the NCHRP Report 646 researchers developed a procedure to estimate the FEL of asphalt mixture from a fatigue curve. They found that the lower 95% prediction limit at 50 million load cycles from a regression analysis of fatigue data corresponded reasonably close to the FEL. This technique uses the following equation to estimate the fatigue life.

$$\text{Lower Prediction Limit} = \hat{y}_o - t_\alpha s\sqrt{1+\frac{1}{n}+\frac{(x_o-\bar{x})^2}{S_{xx}}}$$

where:
yo = the one-sided lower 95% prediction interval at the micro-strain level corresponding to 50,000,000 cycles;
tα = value of t distribution for n-2 degrees of freedom for a significance level of 0.05;
s = standard error of the regression analysis;
n = number of samples;
Sxx = sum of squares of the x values;
xo = log 50,000,000; and
$\bar{x}$ = average of the fatigue life results.

The K1 and K2 coefficients, R^2 value, and predicted endurance limit for all the mixes are presented in Table 13. With exception of some of the Illinois SMA mixes, all fatigue curves have an R^2 value above 0.9. All the mixes, with or without RAS, performed well with respect to fatigue cracking since all the K2 coefficients are above 4. The SMA mixes from Illinois, in particular, have the greatest endurance limits and thus possess the highest fatigue cracking resistance in a strain-controlled environment. In the case of the Iowa, Missouri, Minnesota, and Colorado demonstration projects, the RAS mixes exhibited slightly better fatigue lives than the non-RAS mixes. These results demonstrate that mixes containing RAS can possess similar or better fatigue properties to mixes without RAS.

Figure 13 plots the K1 coefficient versus the K2 coefficient on a log-log scale and shows good correlation between these two coefficients. All the mixes follow the trend line which helps confirm the fatigue results are valid, and that mixes containing RAS are not found to have “out of the ordinary” fatigue properties.

Table 13. Beam fatigue results

State Agency	Mix ID	% RAS	% RAP	K1	K2	R^2	Endurance Limit (Micro-strain)
Missouri	15% RAP	0	15	5.15E-17	6.40	0.968	139
	Fine RAS	5	10	7.25E-19	6.91	0.992	145
	Coarse RAS	5	10	2.07E-20	7.37	0.968	159
Iowa	0% RAS	0	0	1.43E-13	5.45	0.987	144
	4% RAS	4	0	6.75E-14	5.68	0.987	182
	5% RAS	5	0	1.97E-12	5.27	0.982	175
	6% RAS	6	0	7.07E-14	5.65	0.967	162
Minnesota	30% RAP	0	30	6.66E-11	4.51	0.982	89
	Post-Cons. RAS	5	0	2.22E-09	4.19	0.996	123
	Post-Manuf. RAS	5	0	9.19E-12	4.90	0.994	131
Indiana	HMA-RAP	0	15	7.04E-12	4.87	0.993	114
	HMA-RAS	3	0	1.41E-11	4.77	0.970	118
	WMA-RAS	3	0	1.17E-11	4.81	0.985	110
Wisconsin	Evo	3	13	1.70E-11	4.74	0.976	74
	No Evo	3	13	3.75E-10	4.32	0.984	53
Colorado	RAP Only	0	20	2.34E-13	5.69	0.907	195
	RAS/RAP	3	15	9.22E-14	5.89	0.907	244
Illinois	DCon 70-28P	5	0	5.97E-16	6.51	0.946	195
	DCon 70-28L	5	0	2.92E-11	5.07	0.907	138
	DCon 58-28L	5	0	2.15E-11	4.86	0.593	152
	Curran 70-28P	5	11	2.61E-13	5.64	0.985	208
	Curran 70-28L	5	11	5.26E-27	9.95	0.996	359
	Curran 58-28L	5	11	8.29E-20	7.56	0.735	204

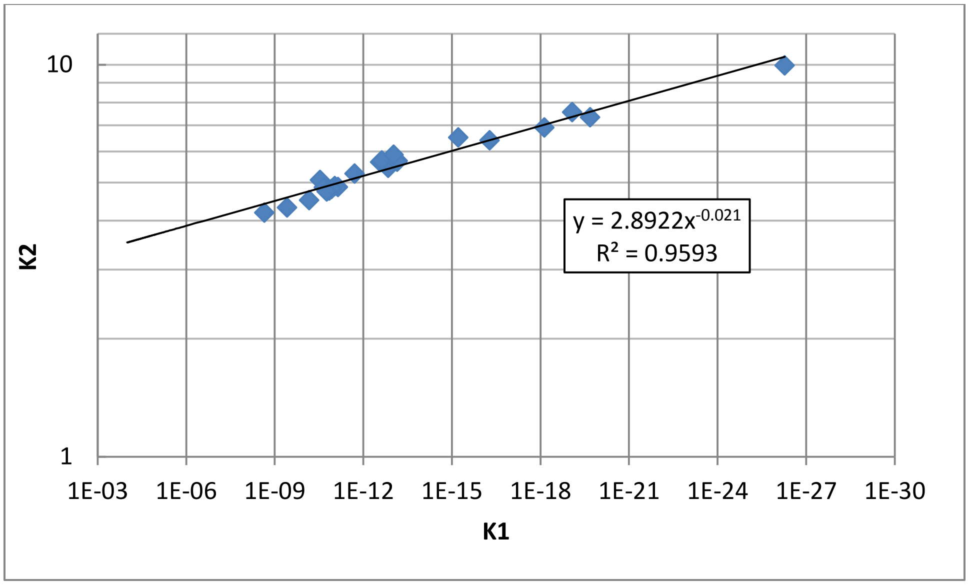


Figure 13. K1 versus K2 coefficients

5.6 Semi-Circular Bending

The SCB test results were analyzed to evaluate the effect of the various RAS treatments on the fracture energy. MacAnova statistical software package was utilized to perform a statistical analysis. ANOVA was used to examine the differences among the mean response values of the different treatment groups. The significance of the differences was tested at 0.05 level of error. The analysis of variance was conducted with the assumption that the errors in the data are independently normal with constant variance.

For each demonstration project, the Gf group means of each RAS treatment levels was compared using a pair-wise comparison to rank the RAS treatment levels with regard to fracture energy. The outcome is reported in Table 14. For each demonstration project, letter A indicates the best performing group of mixtures; letter B the second best, and so on. Groups with the same letter are not statistically different, whereas mixtures with different letters are statistically different.

Table 14. Ranking of mixes by Gf mean value for each demonstration project

State Agency	Mix ID	% RAS	% RAP	Rank	Group mean G_f[J/m^2]	test temp. °C
Missouri	15% RAP	0	15	A	428	-6, -12,-18,-22
	Fine RAS	5	10	A	427	
	Coarse RAS	5	10	A	378	
Iowa	4% RAS	4	0	A	674	-12, -18, -24, -28
	6% RAS	6	0	A/B	659	
	5% RAS	5	0	B/C	558	
	0% RAS	0	0	C	531	
Minnesota	30% RAP	0	30	A	741	-12, -18, -24, -28
	Post-Cons. RAS	5	0	A	777	
	Post-Manuf. RAS	5	0	A	768	
Indiana	HMA-RAP	0	15	A	551	-6, -12,-18,-22
	HMA-RAS	3	0	A	502	
	WMA-RAS	3	0	A	500	
Wisconsin	Evo	3	13	A	329	-12, -18, -24, -28
	No Evo	3	13	A	364	
Colorado	RAP Only	0	20	A	350	-12, -18, -24, -28
	RAS/RAP	3	15	A	318	
Illinois	DCon 70-28P	5	0	A	482	-12, -18, -24, -28
	DCon 70-28L	5	0	A	432	
	DCon 58-28L	5	0	A	430	
	Curran 70-28P	5	11	A	337	
	Curran 70-28L	5	11	A	369	
	Curran 58-28L	5	11	A	385	

For the Iowa mixes, the mixture with 4% RAS has the highest fracture energy and the mixture with 0% RAS the lowest fracture energy. The differences between the 4% RAS and 0% RAS are statistically significant.
For the Missouri mixes, when 5% RAS with a fine grind was replaced with 5% RAP, the fracture energy did not change. However, the mixture with a coarse grind RAS did decrease the fracture energy, but the difference was not statistically significant.

For the Minnesota mixes, the fracture energy results suggest the mixture containing 5% post-consumer RAS performed the best, followed by the mixture containing 5% post-manufactured RAS, then the mixture containing 30% RAP. From the ANOVA analysis, there were no statistical differences between the different mix types, indicating no reduced effect in cracking performance in pavements with RAS.

For the Indiana mixes, the mixture with RAS and foaming warm mix technology performed as well as the mixture with RAP only. There were no statistical differences between the different mix types.

For the Wisconsin mixes, when Evotherm® was added to the HMA as a compaction aid, the fracture energy did not change. While the Evotherm® mixture did have a lower fracture energy than the non-Evotherm® mixture, the difference was not statistically significant.

For the Colorado mixes, when 5 percent RAP was replaced with 3 percent RAS in the HMA, the fracture energy did not statistically change.

For the Illinois mixes, the fracture energy results for the D Construction mixes show there are no statistical differences between the three mix types. Likewise, fracture energy results for the Curran mixes also show there are no statistical differences between the three mix types. Using a PG 58-28 (w/ GTR) in place of a polymer modified PG 70-28 did not affect the fracture energy of the SMA. Additionally, the PG 70-28 SMA mixes produced in the field had a similar low temperature fracture energy as the PG 70-28 SMA mixes produced in the laboratory. Although the D Construction SMA mixes have higher fracture energies than the Curran SMA mixes, the difference between the group means between these two mix types was not statistically significant at the 95 percent confidence level. The p-value was 0.0674. Therefore, adding 11 percent RAP to the SMA mix design did not change its fracture energy.

5.7 Creep Compliance using the BBR

The data obtained from the creep compliance test using the BBR was first used to compare the stiffness of the HMA beams to the stiffness of the beams of extracted binder. The average stiffness of the three HMA beams at each temperature was compared to the average stiffness of the two beams of extracted binder at the same temperature. Stiffness measured at 60 seconds was used for the comparison. The results show little correlation between binder stiffness and HMA stiffness at the same temperature for this type of analysis (Figure 14). The data was also used to compare the stiffness of the HMA measured in the BBR versus the fracture energy measured ten degrees higher in the SCB test. The comparison between the two test results, shown in Figure 15 also shows little correlation between the test results.

The lack of correlation in these two plots may be due to the large amounts of recycled products (i.e., RAP, RAP, GTR, polymers, fibers in the RAS, etc.) in combination with the small geometry of the test samples. Testing HMA beams in the BBR is a newly developed test procedure, and as more sophisticated techniques are developed to analyze the results for field performance models, these results may become more useful. At this time, further analysis of the test data is recommended.

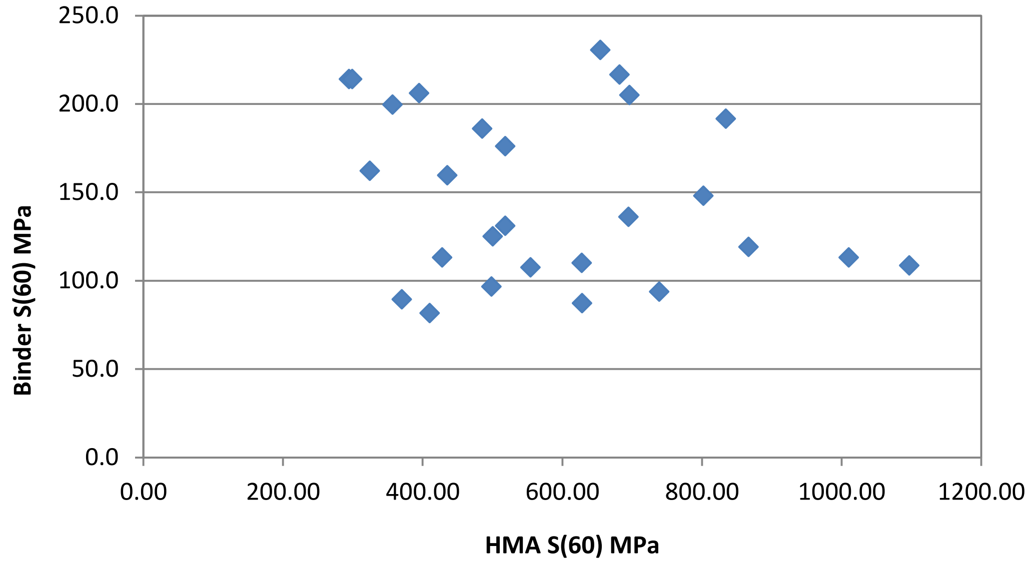


Figure 14. BBR Mix S(60) versus Binder S(60)

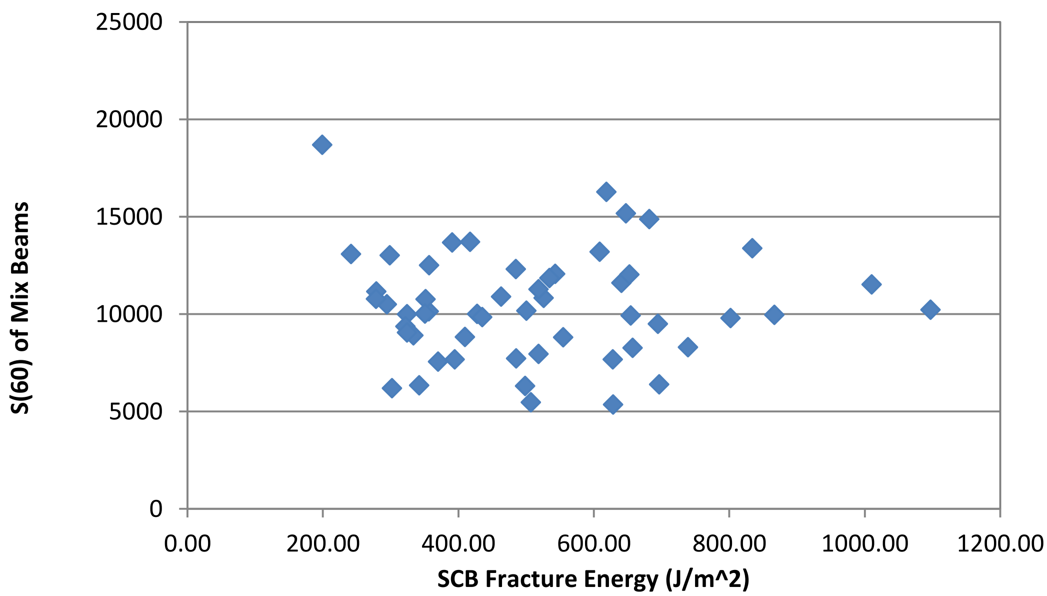


Figure 15. BBR Mix S(60) versus SCB fracture energy

6. PAVEMENT CONDITION SURVEYS

Pavement condition surveys were conducted by the project team following each demonstration project. For the Missouri, Iowa, Wisconsin, Colorado, and Illinois projects, the research team was able to conduct a survey in the summer or fall after paving to evaluate the new pavements containing RAS. The goal of the study was to conduct a pavement survey after every winter season for several years to evaluate how the pavement performed in low temperature climates. The number of pavement surveys was dependent on the timing of the project. Three post-winter surveys were completed for Minnesota and Indiana; two post-winter surveys were completed for Missouri and Iowa; and one post-winter survey was completed for Colorado, Illinois and Wisconsin (Table 15). Additional surveys may be helpful to better identify the trend in pavement performance over the course of several years.

The surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* by FHWA. For each demonstration project, thee 500-foot sections were randomly selected for each mix type paved. The surveys were conducted in these locations.

No measureable pavement deformation was found during any of the surveys, thus the pavements performed well with respect to the rutting. Some minor popouts and raveling was observed in a few of the pavement sections, but not enough to report for any definitive conclusions with regard to pavement performance. In most cases it was determined to be the result of a construction defect or snow plows. The clearest and most telling distress regarding pavement performance for all the projects was transverse cracking. This cracking was most likely reflective cracking since all the pavements with transverse cracks were asphalt overlays placed over jointed concrete pavement. The severity level and linear length of the transverse cracks was measured in each section. It is reported in linear feet per 500 feet of one traffic lane width in Table 14. The research team does not recommend comparing the amount of cracking between the different demonstration projects since of the mixes served as overlays for a concrete pavement rehabilitation while others served as either a binder/leveling course or surface course for new construction.

An overview of Table 15 does show that on some projects (i.e. Missouri) the RAS pavements exhibited more cracking than the non-RAS pavements. However, on other projects (i.e. Iowa, Indiana) some of the RAS pavements exhibited the same amount of cracking or less than the non-RAS pavements. A more detailed analysis for the pavement surveys is reported in the state summaries.

Table 15. Pavement transverse cracking

State Agency	Mix ID	Transverse cracking (feet per 500 feet of 1 traffic lane)				
		After construction	1 winter after construction	2 winters after construction	3 winters after construction	4 winters after construction
Missouri	15% RAP	0	30	46	-	-
	Fine RAS	0	52	97	-	-
	Coarse RAS	0	41	139	-	-
Iowa	0% RAS	0	144	156	-	-
	4% RAS	0	137	142	-	-
	5% RAS	0	148	153	-	-
	6% RAS	0	146	147	-	-
Minnesota	30% RAP(1)	-	-	-	0	0
	Post-Cons. RAS(2)	-	-	-	143	173
	Post-Manuf. RAS(3)	-	-	-	150	199
Indiana	HMA-RAP	-	4	158	191	-
	HMA-RAS	-	35	162	172	-
	WMA-RAS	-	47	264	277	-
Wisconsin	Evo	0	0	-	-	-
	No Evo	0	0	-	-	-
Colorado	RAP Only	0	0	-	-	-
	RAS/RAP	0	25	-	-	-
Illinois	Dcon 70-28P	0	0	-	-	-
	Dcon 70-28L	0	0	-	-	-
	Dcon 58-28L	0	0	-	-	-
	Curran 70-28P	0	0	-	-	-
	Curran 70-28L	0	0	-	-	-
	Curran 58-28L	0	0	-	-	-

(1)Cell 20 shoulder
(2)East mainline transition
(3)West mainline transition

7. CONCLUSIONS AND RECOMMENDATIONS

This report presents the results of Transportation Pooled Fund (TPF)-5(213), a collaboration of seven state transportation agencies in the United States with the goal of researching the effects of RAS on the performance of asphalt applications. As part of TPF-5(213), each state highway agency proposed a unique field demonstration project that investigated different aspects of asphalt mixes containing RAS specific to their state needs. The objective of these projects was to provide adequate laboratory and field test results to answer design, performance, and environmental questions about asphalt pavements with RAS. The demonstration projects focused on evaluating different aspects (factors) of RAS that were deemed important for each state to move forward with a RAS specification. RAS factors addressed in the different demonstration projects included the evaluation of the RAS grind size, RAS percentage, RAS source (post-consumer versus post-manufactured), RAS in combination with warm mix asphalt technology, RAS as a fiber replacement for stone matrix asphalt (SMA) pavements, and RAS in combination with ground tire rubber. Several of the demonstrations projects also included control sections to compare traditionally used mix designs containing either RAP only or no recycled product to mix designs containing RAS.

Field mixes from each demonstration project were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and recovery with subsequent binder characterization. Pavement condition surveys were then conducted for each project after completion. The results of the study are summarized below:

- Observations from the demonstration projects show that RAS pavements can be successfully produced and meet state agency quality assurance requirements for mix asphalt content, gradation, and volumetrics. This includes the SMA mixes produced in Illinois which used 5% RAS in lieu of fibers; the RAS mixes produced in Indiana and Wisconsin that used foaming and Evotherm® WMA technologies, respectively; and the RAS mixes produced in Missouri which used RAS, RAP, GTR, and transpolyoctenamer rubber.
- When RAS is used in HMA, the shingle binder blends with the base binder which increases the performance grade of the base binder on the high and low side. The average results of all the mixes in the study show that for every 1 percent increase in RAS, the low temperature grade of the base binder will increase 1.9°C; and for every 1 percent increase in RAP, the low temperature grade of the base binder will increase 0.3°C. Therefore, as a rule of thumb, 3 percent RAS or 20 percent RAP would be the maximum amount of recycled material allowed without requiring a low temperature grade bump (6°C) in the base binder. This corresponds to a 14 percent binder replacement when using RAS and a 20 percent binder replacement when using RAP, when considering the average asphalt content values for all the mix designs. However, this should only be used as a starting point of estimating how RAS will affect HMA binder since the PG of the asphalt blends did vary among the different projects. When estimating how RAS will affect an HMA binder, agencies should consider the RAS source (post-manufactured versus post-consumer) and whether a modifier is used in the base asphalt.

- The flow number and dynamic modulus results from the demonstration project mixes show that using RAS or a combination of RAS/RAP in HMA improves its rutting resistance. The pavement condition surveys confirmed the high rutting resistance of the mixes as there was no measurable amount of wheel path deformation in the pavements.
- All the mixes, with or without RAS, performed well with respect to fatigue cracking in the four-point bending beam test. The K2 coefficients ranged from 4.19 to 9.95 and the estimated fatigue endurance limits ranged from 53 to 359 micro-strain. The SMA mixes from Illinois which used 5% RAS exhibited the most desirable fatigue characteristics. In the case of the Indiana demonstration project, the RAS mixes performed the same as the RAP mix; and in the case of the Iowa, Missouri, Minnesota, and Colorado demonstration projects, the RAS mixes exhibited slightly better fatigue lives than the non-RAS mixes. Fibers in the RAS could be contributing to the improved mix performance. Based on the four-point bending beam results, HMA with RAS should perform as well as HMA without RAS with respect to fatigue performance.
- The SCB test results were evaluated by comparing the low temperature fracture energy group means of the mixtures for each demonstration project. There were no statistical differences at the 95 percent confidence level among the mix fracture energies for every project except Iowa. For the Iowa mixes, the 0% RAS mix had a statistically lower fracture energy than the 4% RAS mix which suggests that RAS can improve the fracture resistance of HMA. With regards to the Missouri, Minnesota, Indiana, Wisconsin, Illinois, and Colorado demonstration projects, the lack of statistical differences in fracture energy indicates that the mixes with RAS have the same fracture resistance as the mixes without RAS. Based on the SCB results, the addition of RAS materials to HMA is not detrimental to its fracture resistance.
- The pavement condition surveys in Missouri revealed the pavement containing coarsely ground RAS exhibited more transverse cracking than the pavement containing finely ground RAS. In both the Missouri and Colorado demonstrations projects, the RAS pavements exhibited slightly more cracking than the non-RAS pavements. In contrast, the RAS pavements exhibited the same amount of cracking or less than the non-RAS pavements for the Iowa, and Indiana demonstration projects. In the Indiana project, more cracking was observed for the RAS mix produced with foaming WMA technology than the RAS mix produced without foaming. In the Minnesota project, slightly more cracking was also observed in the mix using post-manufactured RAS compared to the mix using post-consumer RAS. However, when taking into consideration the variability of the existing pavement condition beneath the asphalt overlays and the small difference in crack length among the different mix types for some projects, definitive conclusions solely based on the surveys should be reserved.
- Since the demonstration projects were conducted in different locations with different climates, materials sources, and factors in the experimental designs, a separate report was written for each demonstration project that gives a detailed description of its construction and provides additional evaluation of the laboratory test data and pavement condition surveys. The reports are attached in Appendices A through G.

REFERENCES

Abdulshafi, O.; Kedzierski, B.; Fitch, M.; and Muhktar, H. "Evaluation of the Benefits of Adding Waste Fiberglass Roofing Shingle to Hot-Mix Asphalt", Report Number FHWA/OH-97/006m, Ohio Department of Transportation, July 1997.

Amirkhanian, S.N.; and Vaughan, K.M., "Field Evaluation of Use of Waste Shingles in Asphalt Mixtures," Report FHWA-SC-01-04, 35 pages, 2001.

Brock, Ben, "Economics of RAS in HMA", Presentation at the 3rd Asphalt Shingle Recycling Forum, Chicago, Illinois, November 1-2, 2007.

Button, J.; Williams, D.; and Scherocman, J. "Roofing Shingles and Toner in Asphalt Pavements", Texas Transportation Institute, Research Report 1344-2F, July 1996.

Cochran, Kimberly, "Construction and Demolition Debris Recycling: Methods, Markets, and Policy," Ph.D. Dissertation, University of Florida, Gainesville, Florida, 2006.

Emery, J. and MacKay, M., Use of Wastes and Byproducts as Pavement Construction Materials, Proceedings of the Transportation Association of Canada Annual Conference, Winnipeg, Manitoba, Canada, Volume 1, September 15-16, 1991.

Foo, K.Y.; Hanson, D.I.; and Lyn, T.A. "Evaluation of Roofing Shingles in Hot Mix Asphalt," Journal of Materials in Civil Engineering, Vol. 11, No. 1, p. 15-20, American Society of Civil Engineers, 1999.

Ghuzlan, K.A., Carpenter, S.H. 2006. "Fatigue damage analysis in asphalt concrete mixtures using the dissipated energy approach", Canadian Journal of Civil Engineering, Volume 33, pp890-901

Inspec-Sol, Inc., "Materials Assessment, Environmental Monitoring and Preliminary Human Health Risk Assessment Screening Results, Asphalt Shingle Aggregate Trail Project", prepared for the Municipality for the District of Lunenburg, Lunenburg County, Nova Scotia, February 2008.

Janisch, D.; and Turgeon, C. "Minnesota's Experience with Scrap Shingles in Bituminous Pavements", Report Number MN/PR-96/34, Minnesota Department of Transportation, 1996.

Jones, S. "Recycled Asphalt Shingles District 1 Experiences", presented at the Asphalt Shingle Recycling 101 Workshop, Illinois DOT District 1, May 5, 2010.

Kriech, A.J.; Kurek, J.T.; Osborn, L.V.; Wissel, H.L.; and Sweeney, B.J. "Determination of Polycyclic Aromatic Compounds in Asphalt and Corresponding Leachate Water," Polycyclic Aromatic Compounds, 22(3-4), 517-536, 2002.

Marasteanu, M.; Zofka, A.; McGraw, J.; Krivit, D.; Schroer, J.; and Olson, R. "Recycled Asphalt Shingles in Hot Mix Asphalt", Journal of the Association of Asphalt Paving Technologists, Volume 76, pp 235-274, 2007.

Maupin, G.W. Jr. "Use of Manufactured Waste Shingles in a Hot-Mix Asphalt field Project in Virginia", Report Number VTRC 08-R11, Virginia Transportation Research Council and the Virginia Department of Transportation, March 2008.

Maupin, G.W. Jr. "Investigation of the Use of Tear-Off Shingles in Asphalt Concrete", Report Number FHWA/VTRC 10-R23, Virginia Transportation Research Council, the Virginia Department of Transportation and Federal Highway Administration, May 2010.

McGraw, Jim; Johnson, Eddie; Johnson, Greg; Dai, Shongtao; Linell, Dave; and Watson, Mark. "Incorporation of Recycled Asphalt Shingles in Hot-Mixed Asphalt Pavement Mixtures", Report Number MN/RC 2010-08, Minnesota Department of Transportation Office of Materials and Road Research and the Minnesota Department of Transportation Research Services Section, February 2010.

NCHRP Report 646. 2010. "Validating the Fatigue Endurance Limit for Hot Mix Asphalt", Transportation Research Board, National Highway Research Council, Washington D.C.

Owens Corning Corporation, "Asphalt Roofing Shingles into Energy Project", Prepared for the Department of Energy, Award No. DE-FG36-06GO86009, April 2008.

Private communication, Dave Powers, Ohio Department of Transportation, 2010.

Private communication, Robert E. Lee, Texas Department of Transportation, 2010.

Reed, A.B. "Reclaimed Manufacturer Asphalt Roofing Shingles in Asphalt Mixtures," Report Number PA-99-005+91-71, Pennsylvania Department of Transportation, 46 pages, 1999.

Rubino, Blake; Ashlock, Jeramy; and Williams, R. Christopher. "Effects of Recycled Asphalt Shingles on Mechanical Properties of Loess", Iowa State University, August 2009.

Scholz, Todd. "Preliminary Investigation of RAP and RAS in HMAC", Report Number OR-RD-10-12, Kiewitt Center for Infrastructure and Transportation, Oregon State University and the Oregon Department of Transportation Research Section, February 2010.

Tangella, S.C.S.R., Craus, J., Deacon, J.A., Monismith, C.L., 1990. "Summary Report on Fatigue Response of Asphalt Mixtures", Prepared for Strategic Highway Research Program Project A-003-A

Texas Department of Transportation, Texas Special Provision 341-024, Austin, Texas, 2010.

Townsend, T.; Powell, J.; and Xu, C. "Environmental Issues Associated with Asphalt Shingle Recycling." Prepared for the Construction Materials Recycling Association and the US EPA Innovations Workgroup, October 19, 2007, Gainesville, Florida.

Watson, D.E.; Johnson, A.; and Sharma, H.R. "Georgia's Experience with Recycled Roofing Shingles in Asphaltic Concrete," Transportation Research Record 1638, p. 129-133, Transportation Research Board, Washington, D.C., 1998.

APPENDIX A. REPORT FOR THE MISSOURI DEPARTMENT OF TRANSPORTATION SPONSORED DEMONSTRATION PROJECT

A1. Introduction

This report presents a summary of the results obtained from the field demonstration project sponsored by the Missouri Department of Transportation (MoDOT) as part of Transportation Pooled Fund (TPF) 5-213 *Performance of Recycled Asphalt Shingles in Hot Mix Asphalt*. TPF 5-213 is a partnership of several state agencies with the goal of researching the effects of recycled asphalt shingles (RAS) on the performance of asphalt applications. As part of the Pooled Fund research program, multiple field demonstration projects were conducted to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements that include RAS.

Each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated different aspects of asphalt mixes containing RAS. The field demonstration project sponsored by the Missouri Department of Transportation (MoDOT) investigated two RAS factors: RAS grind size (*high priority*) and asphalt mixes with RAS and modified asphalt binder (*moderate priority*). The objective of this demonstration project was to identify potential economic and performance benefits when incorporating a finer grind size of RAS in HMA using asphalt modified with ground tire rubber (GTR) and transpolyoctenamer rubber (TOR).

A2. Experimental Plan

To evaluate the RAS factors of grind size and compatibility of RAS with modified asphalt binder, MoDOT designed an experimental plan to address the following questions:

- Is there a difference in pavement performance between fine and coarse RAS grinds?
- Does replacing five percent recycled asphalt pavement (RAP) with five percent RAS affect pavement performance?
- Can asphalt modified with GTR and TOR be used in conjunction with RAS?

The experimental plan is presented in Table A2.1. The plan was implemented during the demonstration project by producing three asphalt mixtures: a Control mixture, a Fine RAS mixture, and a Coarse RAS mixture.

Table A2.1. Experimental plan

Mix ID	% RAS	% RAP	RAS Source	RAS Grind Size
Control	0	15	-	-
Fine RAS	5	10	Post-Consumer	< 9.5 mm
Coarse RAS	5	10	Post-Consumer	< 12.5 mm

Since the use of RAP in hot mix asphalt (HMA) is becoming a standard practice when producing HMA, MoDOT also wanted to consider the combined effects of RAP and RAS. Therefore the experimental plan utilized mixtures with RAP. Additionally, MoDOT uses GTR and TOR as asphalt modifiers to grade bump their liquid asphalt. In the experimental plan, each asphalt mixture contained a virgin PG 64-22 blended with 10% GTR by weight of asphalt binder and 4.5% TOR by weight of GTR to achieve an equivalent PG 70-22 liquid asphalt grade.

During production of the asphalt mixtures, Iowa State University collected samples of each mixture for laboratory testing. A portion of the samples were sent to the University of Minnesota and the Minnesota Department of Transportation (MNDOT) for Semi-Circular Bend (SCB) testing and binder extraction and recovery, respectively. The laboratory testing plan is presented in Table A2.2.

Table A2.2. Laboratory testing plan

	Laboratory Test	Iowa State University	University of Minnesota	Minnesota DOT
Processed Shingles	Binder Extraction			X
	High Temperature PG			X
	Gradation (Before Extraction)	X		
	Gradation (After Extraction)	X		
Mixture	Binder Extraction			X
	Binder PG Characterization	X		
	Gradation	X		
	Dynamic Modulus	X		
	Flow Number	X		
	Beam Fatigue	X		
	Semi-Circular Bending (SCB)		X	

The asphalt was recovered from the RAS and asphalt mixtures following AASHTO T164 Method A (Centrifuge Method) by using a blend of toluene and ethanol as the extraction solvent. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. At Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO R29 “Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder.”

Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution, and a washed gradation was conducted after extraction to evaluate the size distribution of the fine aggregates in the RAS product.

Performance testing was completed on the field produced asphalt mixtures at low, intermediate, and high temperatures. Dynamic Modulus and Flow Number were conducted at high temperatures to evaluate the modulus and rutting resistance of asphalt mixtures. The durability of the mixtures at intermediate temperatures was evaluated using the dynamic modulus and four-point beam fatigue test. The SCB test conducted at the University of Minnesota evaluated the fracture properties of the mixtures at low temperatures.

After construction of the pavement for the demonstration project, field evaluations were conducted on each pavement test section one and two years after paving to assess the field performance of the pavement concerning cracking, rutting, and raveling.

A3. Project Location

The location for the demonstration project was US Route 65 south of Springfield, Missouri starting in Green County and ending in Christian County. US Route 65 is a divided four-lane highway. The test sections were placed on the two northbound (NB) lanes starting at the bridge just south of the Highway F in Ozark, Missouri (South End Bridge) and ending at the Lake Springfield Bridge (Lake Springfield Bridge) in Springfield, Missouri for a total length of 8.8 miles. The project limits are identified below in Figure A3.1.

Figure A3.1. Project location

A4. Project Description

The demonstration project was conducted by Journagan Construction Company (Journagan Construction) in May and June of 2010. A 3.75 inch HMA overlay was placed over an existing jointed concrete pavement in two lifts. The base course was a 2 inch lift of HMA containing 20 percent RAP and a nominal maximum aggregate size (NMAS) of ¾ inch. The surface course included one of the three test sections: a Control section with 15% RAP only (0% RAS), a test section with 5% Fine RAS and 10% RAP, and a test section with 5% Coarse RAS and 10% RAP. The pavement cross section is shown in Figure A4.1.

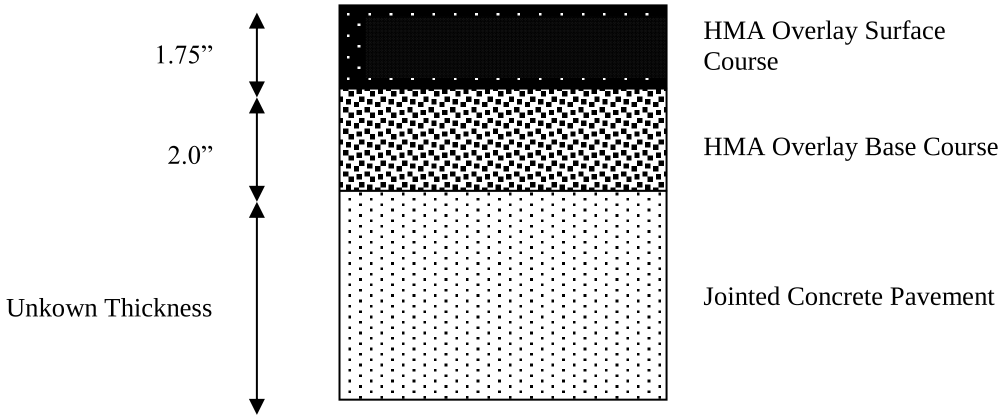


Figure A4.1. Pavement cross-section

Journagan Construction paved the surface course test sections in four parts. On May 21st and 22nd, the Control section was paved on the NB passing lane beginning at the south end of the project and continuing north approximately 3.2 miles. On May 24th through May 26th, the Coarse RAS section was paved on the NB passing lane starting at the 3.2-mile project mark and continuing 5.6 miles to the north end of the project. On May 27th and 28th, the Fine RAS section was paved on the NB driving lane beginning at the project start and continuing for approximately 3.1 miles. On June 10th through June 12th, the driving lane of the Coarse RAS section was paved starting at the 3.1-mile project mark and ending at the Lake Springfield Bridge for a total of approximately 5.7 miles. A plan view of the test section on US Route 65 is shown in Figure A4.2. A detailed description of the test section locations is provided in Appendix A.

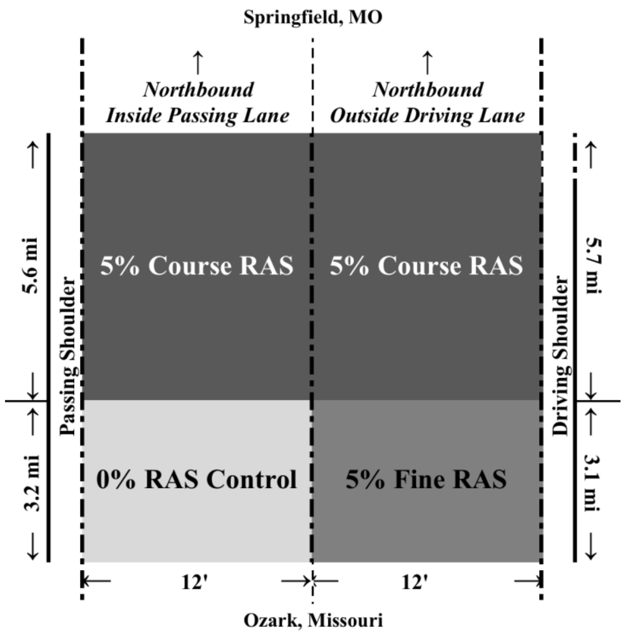


Figure A4.2. Plan view of US Route 65 project test sections

Paving was completed at night to reduce delays due to the high volume of traffic. Wet spring weather conditions created delays and extended the project for several weeks. Weather conditions during the paving were ambient temperatures ranging from 78-90 degrees Fahrenheit and partly cloudy with moderate to high humidity.

The asphalt plant for the project was located in Ozark, Missouri at the site of Journagan Construction's Ozark Quarry. The longest haul distance from the plant to the project was approximately 10 miles. The asphalt plant is single drum counter-flow plant with a capacity to produce 3,000 tons of HMA per day. It was modified to incorporate RAS by adding a separate loading bin. The shingles passed over a vibrating screen prior to being placed on the conveyor belt and added in the recycled product column on the drum (Figure A4.3).

Figure A4.3. Plant RAP/RAS bins, screen, and conveyor belt entry into drum

A total of approximately 11,000 tons of HMA was placed for the surface course test sections (this total does not include shoulder tonnages). The test sections included a total of approximately 446 tons of RAS and 1,183 tons of RAP. Tonnages for the RAS, RAP, and total HMA for each test section are summarized in Table A4.1 below.

Table A4.1. Project tonnages

Material	Control Section (Tons)	Fine RAS Section (Tons)	Coarse RAS Section (Tons)
RAS	---	94	352
RAP	291	188	704
Total HMA	1,943	1,882	7,044

A5. Shingle Processing

For the RAS materials used in the mix designs, Journagan Construction collected and stockpiled clean loads of post-consumer shingles from local roofing and hauling companies. Clean loads are defined as loads containing less than 10% non-shingle material, and loads that do not meet this criterion are turned away. Missouri’s National Emission Standards for Hazardous Air Pollutants (NESHAP) requires that all shingles come from residential buildings with four or fewer dwellings. Journagan is not required to conduct testing for asbestos containing materials (ACM), however, local authorities do collect random samples as needed to verify that the shingles do not contain over 1% ACM.

Journagan Construction hired a mobile grinding service to complete the grinding of their stockpiled shingles. The shingles were ground using an industrial Peterson grinder to produce a fine grind RAS and a Bandit grinder to produce a coarse grind RAS. The fine RAS contained 100% passing the ⅜ inch minus (9.5 mm) screen, and the coarse RAS contained 100% passing the ½” minus (12.5 mm) screen. During the grinding process, the shingles were passed over magnets to remove all ferrous metals and water nozzles were used to control heat build-up and dust. The final post-processed RAS stockpiles were uncovered and open to prevailing weather conditions. Pictures of the RAS final products are presented in Figures A5.1 and A5.2.

Figure A5.1. Fine RAS

Figure A5.2. Coarse RAS

The gradation test results of the RAS products before extraction are presented in Table A5.1. The Fine RAS is finer than the Coarse RAS on the coarser sieve sizes, however, both products have similar gradations on the finer sieve sizes.

Table A5.1. RAS gradation before extraction

Sieve Size (US)	Sieve Size (mm)	Coarse RAS	Fine RAS
3/4"	19	100	100
1/2"	12.5	98	100
3/8"	9.5	94	99
#4	4.75	75	82
#8	2.36	62	67
#16	1.18	42	43
#30	0.6	22	21
#50	0.3	12	12
#100	0.15	5	5
#200	0.075	1.2	0.9

A6. Asphalt Mix Design and Prodution Results

Two HMA mix designs were prepared by Journagan Construction for the demonstration project. The first mix design contained 15% RAP and 0% RAS. The second mix design contained 10% RAP and 5% fine RAS. To produce the asphalt mixture that contained coarse RAS, Journagan Construction used the mix design containing fine RAS and replaced the fine RAS with coarse RAS during production. The mix design gradations obtained from laboratory testing of the sampled asphalt mixtures are presented in Figure 6.1. As shown in the figure, the asphalt mixtures had similar aggregate structures with gradations passing below the restricted zone. The aggregates supplied for the asphalt mix design came from the Burlington and Reeds Spring formations in the Journagan Ozark Quarry.

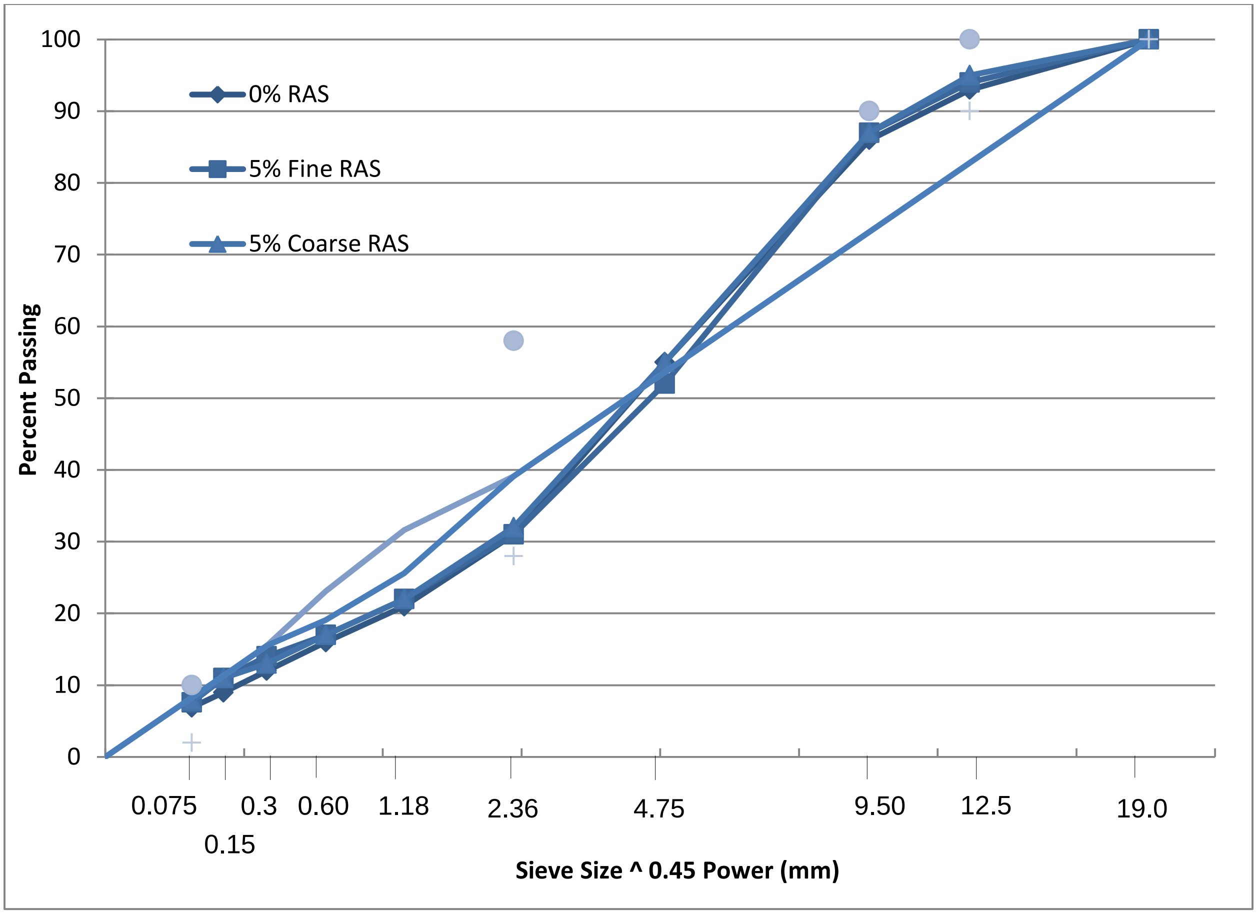


Figure A6.1. Asphalt mix design gradations

The asphalt content and gradation after extraction for the recycled materials used in the mixed designs are presented in Table A6.1. While both products came from the same source of shingles, there were some differences between the Fine RAS and the Coarse RAS. The Fine RAS sample contained a little more asphalt (25.0%) than the Coarse RAS sample (21.7%), possibly due to differences in the stockpiled material or granule loss in the Fine RAS during grinding and handling. In contrast to the high asphalt content of the RAS products, the RAP used for the mixtures contains 4.5% asphalt. The RAP used for the mix designs came from millings on MoDOT projects.

Table A6.1. RAS and RAP properties after extraction

Sieve Size (US)	Sieve Size (mm)	Coarse RAS	Fine RAS	RAP
3/4"	19	100	100	100
1/2"	12.5	97	99	97
3/8"	9.5	96	99	92
#4	4.75	90	94	72
#8	2.36	85	91	55
#16	1.18	67	73	44
#30	0.6	46	53	35
#50	0.3	39	46	28
#100	0.15	31	37	21
#200	0.075	21.9	26.1	14.8
% Asphalt Content		21.7	25.0	4.5

The asphalt contained in the mixtures is presented in Table A6.2. The mix designs show that replacing 5% RAP with RAS increased the percent binder replacement of the mixtures from 14.9% to 30.2%. The increase in binder replacement decreased the virgin asphalt content 0.3% to 4.0%. Replacing 5% RAP with RAS in the mixture increased the asphalt demand by 0.6%, changing the optimum asphalt content from 4.7% to 5.3%. The higher optimum asphalt content is likely the result of a 0.2% increase in absorption (Table A6.2) and 1.3% increase in VMA (Table A6.3). RAS contains high angularity aggregate granules that change the aggregate packing of the mixture, thus increasing VMA.

Table A6.2. Mixture asphalt demand properties

Mix Property	Control	Fine RAS	Coarse RAS
% RAS	0	5	5
% RAP	15	10	10
% Total AC	4.7	5.3	5.3
% Virgin AC	4.0	3.7	3.7
% Binder Replacement	14.9	30.2	30.2
% Effective Asphalt	4.2	4.6	4.6
% Asphalt Absoprtion	0.5	0.7	0.7

The asphalt mix design volumetric properties are presented in Table A6.3. The Coarse RAS volumetric targets were the same as the Fine RAS volumetric targets since only a mix design for the Fine RAS mixture was prepared. The designs were dense-graded Superpave bituminous mixtures, following MoDOT's specification SP190CLG for the project. The mix designs met MoDOT's design traffic level C, which correspond to 3M < 30M equivalent single axle loads (ESAL's) over a 20-year design period. The target voids for all mixes were 4%. The bituminous mixtures include a PG 64-22 virgin asphalt binder terminally blended with 10% GTR by weight of asphalt binder that was blended with 4.5% TOR by weight of GTR at the asphalt plant to achieve a PG 70-22 liquid graded asphalt.

Table A6.3. Mixture design properties

Mix Property	Control	Fine RAS	Coarse RAS
Design Gyrations	80	80	80
NMAS	½”	½”	½”
Virgin PG Grade	64-22	64-22	64-22
% GTR(1)	10	10	10
% TOR(2)	4.5	4.5	4.5
% Voids	4.0	4.0	4.0
% VMA	14.3	15.6	15.6
% VBE	10.3	11.6	11.6
% VFA	72	74	74
-#200/Pbe	1.3	1.5	1.5

(1) GTR by weight of asphalt binder
(2) TOR by weight of GTR

Production control results by Journagan Construction are presented in Table A6.4. Laboratory test results are based on the first tests conducted during the production of the three mixes. The laboratory results show the Control mixture was on target with mix design target values. For the RAS mixtures, there does not appear to be a large difference between the Fine and Coarse RAS initial production test results. While the asphalt percentage was on target, the air voids and VMA were slightly lower than the laboratory design values. However, the results were still within production tolerance, and the VMA still exceeded the minimum 14.0 required for the 1/2” NMAS mixture. The pavement density results obtained from field cores show the contractor was able to successfully compact the Control and RAS mixtures.

Table A6.4. Mixture and construction quality control results

Mix Property	Control 5/21/2010		Fine RAS 5/27/2010		Coarse RAS 6/10/2010	
	JMF	QC Results	JMF	QC Results	JMF	QC Results
% Total AC(1)	4.7	4.6	5.3	5.3	5.3	5.3
% Voids(1)	4.0	3.9	4.0	3.2	4.0	3.1
% VMA(1)	14.3	14.0	15.6	14.4	15.6	14.7
Mainline Density(2)	92.0%	92.2%	92.0%	92.9%	92.0%	92.7%
Joint Density(2)	90.0%	90.9%	90.0%	NA	90.0%	90.9%

(1) First quality control test result during production
(2) Average of core density results

A7. Laboratory Test Results

Binder Testing

Performance Grade (PG) testing of the extracted binders was conducted to obtain their high, low, and intermediate PG temperatures as shown in Table A7.1. The high temperature performance grades of the RAS binders at 137.3°C and 146.1°C are higher than traditional paving grade binders. This is expected since the binder in roofing shingles is produced with an air-blowing process which oxidizes the asphalt. Additionally, the RAS used in the mix designs is from post-consumer shingles, so the binder in the RAS has experienced at least several years of aging.

Because the RAS mixtures are heated to high temperatures and placed in a centrifuge at high speeds during the recovery process, the RAS and virgin asphalt should be fully blended. The addition of the RAS materials raised the low and high performance grade of the virgin binder. The continuous PG for the control mixture was 75.0-16.8, while the Fine RAS mixture was 90.1-8.7 and the Coarse RAS mixture was 88.3-4.9. Both RAS mixtures contained similar performance grades indicating the gradation of the ground shingles does not change the properties of the blended binder.

Table A7.1. Performance grade of extracted binders

Material Identification	High PG Temp, °C	Intermediate PG Temp, °C	Low PG Temp, °C	Performance Grade
PG 70-22	70.3	24.1	-22.8	70-22
Fine RAS	137.3	-	-	-
Coarse RAS	146.1	-	-	-
Control Mixture	75.0	26.3	-16.8	76-16
Fine RAS Mixture	90.1	28.7	-8.7	94-4
Coarse RAS Mixture	88.3	28.3	-4.9	94-4

Dynamic Modulus

The dynamic modulus (|E*|) is a key material property that determines the stress-strain relationship of an asphalt mixture under continuous sinusoidal loading. A higher dynamic modulus indicates lower strains will result in a pavement structure when the asphalt mixture is stressed from repeated traffic loading. The mechanistic-empirical pavement design guide (MEPDG) uses |E*| as the stiffness parameter to calculate an asphalt pavements strains and displacements.

The test was conducted following AASHTO TP62 using five replicate samples of 150 mm in height and 100 mm in diameter. Each sample was compacted to 7 ± 0.5% air voids. Samples were tested by applying a continuous sinusoidal load at 9 different frequencies (0.1, 0.3, 0.5, 1, 3, 5, 10, 20, and 25 Hz) and three different temperatures (4, 21, and 37°C). Sample loading was adjusted to produce strains between 50 and 150 µstrain in the sample.

Master curves were constructed at a reference temperature of 21°C and plotted on a log-log scale for a general comparison as presented in Figure A7.1. At high temperatures, the addition of the RAS binder increases the overall stiffness of the mixture since the RAS mixtures have a higher dynamic modulus than the Control mixture. Fibers in the RAS could also be providing a reinforcing effect that augments the mixture’s modulus at high temperatures. Higher dynamic modulus values in the RAS mixtures indicate replacing 5% RAP with RAS in these mixtures will improve their field rutting performance.

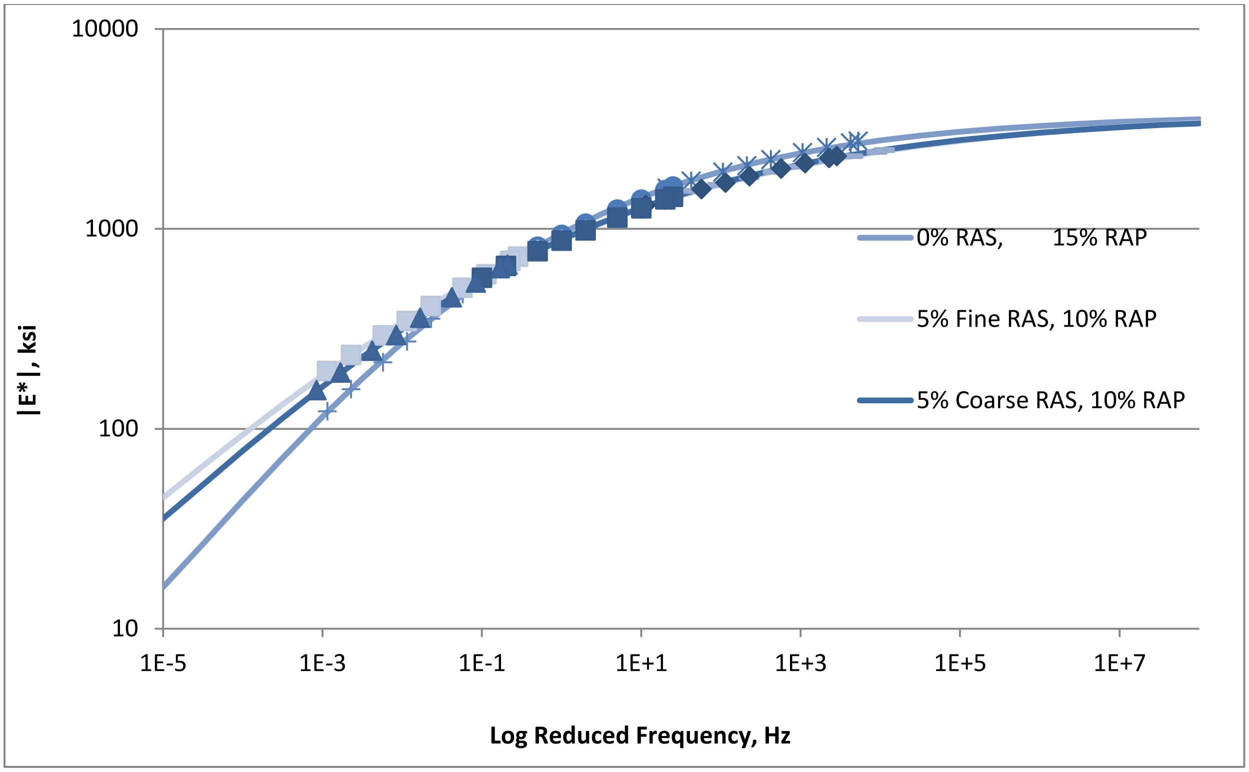


Figure A7.1. Comparison of master curves for MoDOT mixes

The plot in Figure A7.2 presents the mean dynamic modulus at 21°C and 37°C at specific frequencies for a more direct comparison. At 21°C and 5 Hz, the Control mixture dynamic modulus is statistically higher at a 95% confidence level than the two RAS mixture's dynamic modulus. Low modulus values at this temperature are considered desirable in thin asphalt pavements (less than 4") for fatigue cracking resistance. Mixtures with lower stiffness and can deform more easily without building up large stresses. Lower modulus values in the RAS mixtures at this temperature, however, are counter intuitive because of the inclusion of a stiffer binder. A review of the mix designs shows the RAS mixtures have a higher binder content and a higher volume of effective binder which could be reducing the overall stiffness of the RAS mixtures at intermediate temperatures. The data also shows that using either a coarse RAS grind or a fine RAS grind did not appear to have a large effect on the modulus of the mixtures.

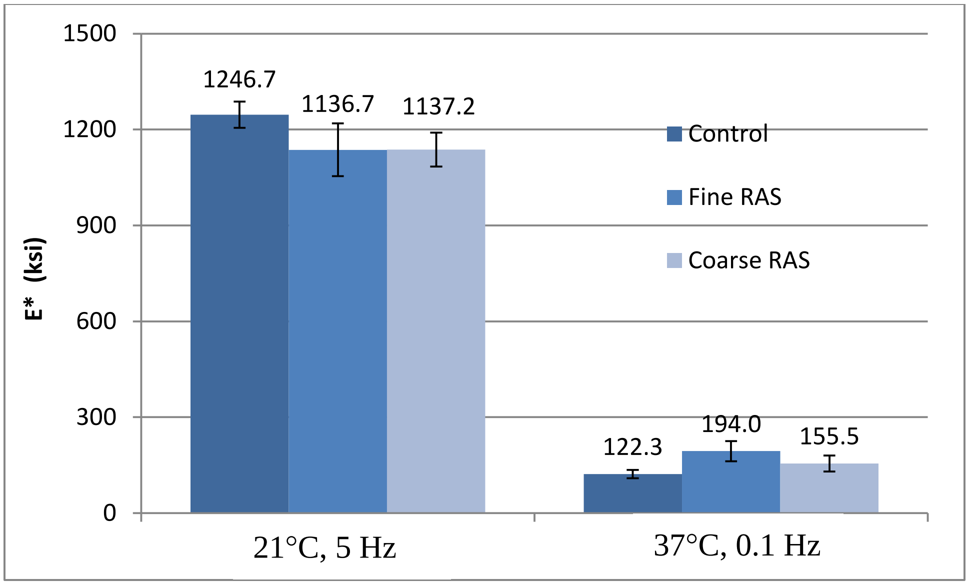


Figure A7.2. Dynamic modulus comparison at 21°C, 5 Hz and 37°C, 0.1 Hz

Flow Number

The flow number test measures the permanent deformation resistance of asphalt mixtures by applying a repeated dynamic load to a sample for up to several thousand load cycles. The flow number is defined as the number of load cycles an asphalt mixture can tolerate until it flows. Cumulative permanent deformation in the sample is plotted versus load cycles. The flow number is reached at the onset of tertiary flow.

Tests were conducted following procedures used in NCHRP Report 465. Samples used in dynamic modulus test were used for the flow number test since the dynamic modulus test is nondestructive. The samples were placed in a hydraulically loaded universal testing machine, unconfined, with a testing temperature of 37°C to simulate the climactic conditions that cause pavement to be susceptible to rutting. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 sec followed by 0.9 sec of dwell time. The loading cycle was repeated for a total of 10,000 load cycles. Three LVDT's were attached to each sample during the test to measure the cumulative strains.

Tertiary flow was not reached in any of the samples after 10,000 load cycles; therefore, all three mixtures should be very resistant to permanent deformation. Yet, the mixtures were still compared in terms of percent accumulative strain after 10,000 load cycles. Test results are presented in Figure A7.3. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. Since the error bars of the RAS mixtures do not overlap with the error bars of the Control mixture, the RAS mixtures performed statistically better than the Control mixture at a 5 percent Type I error level, with the Fine RAS mixture showing better resistance to cyclical loading than the Coarse RAS mixture.

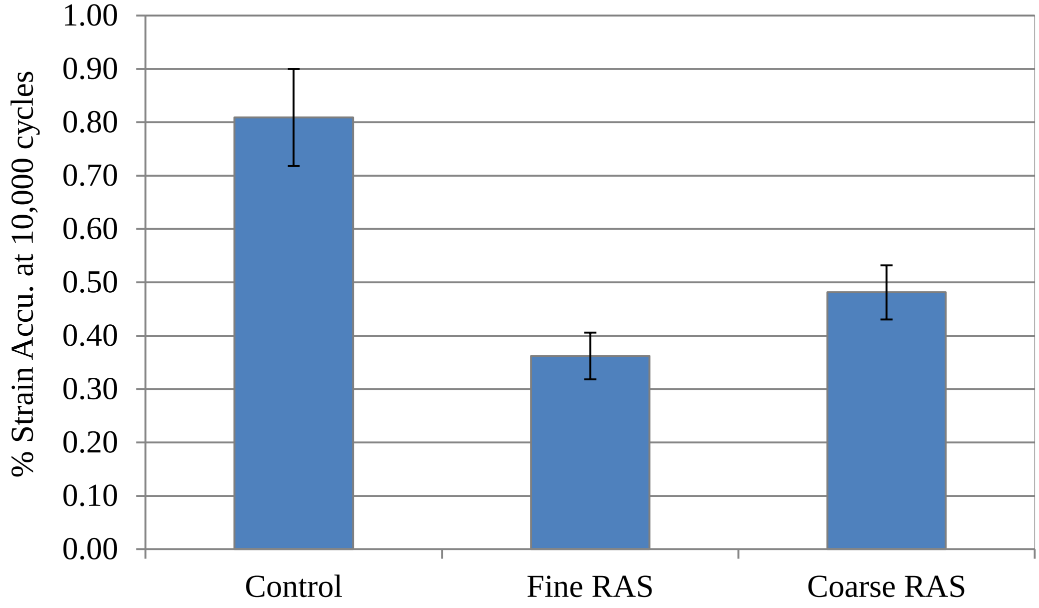


Figure A7.3. Flow number test results

Beam Fatigue

Fatigue cracking is the major cracking distress in asphalt pavements caused by repeated heavy traffic loads. Pavements that have a higher resistance to tensile strains that develop at the bottom of an asphalt layer due to repeated traffic will have a greater resistance to fatigue cracking. The four-point beam fatigue test following AASHTO T321 was conducted to evaluate the load associated cracking resistance of the mixtures.

The mixes were compacted in a linear kneading compactor to create slabs with 7% air voids. The slabs were saw-cut into beams with dimensions 15 inches in length, 2.5 inches in width, and 2 inches in height. The beams were tested in a strain controlled mode of loading at 20°C with haversine wave pulses applied to the beam at 10 Hz. Six beams were tested, each at a different strain level (375, 450, 525, 650, 800, and 1000 μstrain), until the flexural stiffness of the beam was reduced to 50% of the initial stiffness. A log-log regression was performed between strain and the number of cycles to failure (N_f). The relationship between strain and N_f can be modeled using the power law relationship as presented in Equation 1.

$$N_f = K1\left(\frac{1}{\varepsilon_o}\right)^{K2} \tag{1}$$

where N_f = cycles to failure; ε_o = flexural strain; and K1 and K2 = regression constants.

The fatigue curves from beam fatigue test results are presented in Figure A7.4 with the fatigue model coefficients in Table A7.4. At higher levels of strain, the Control mixture has a longer fatigue life than both RAS mixtures. However, at lower levels of strain, the trend of the fatigue curves show the RAS mixtures performing better than the Control mixtures. When comparing the fine and coarse RAS mixtures, the Fine RAS mixture has improved fatigue performance versus the Coarse RAS mixture.

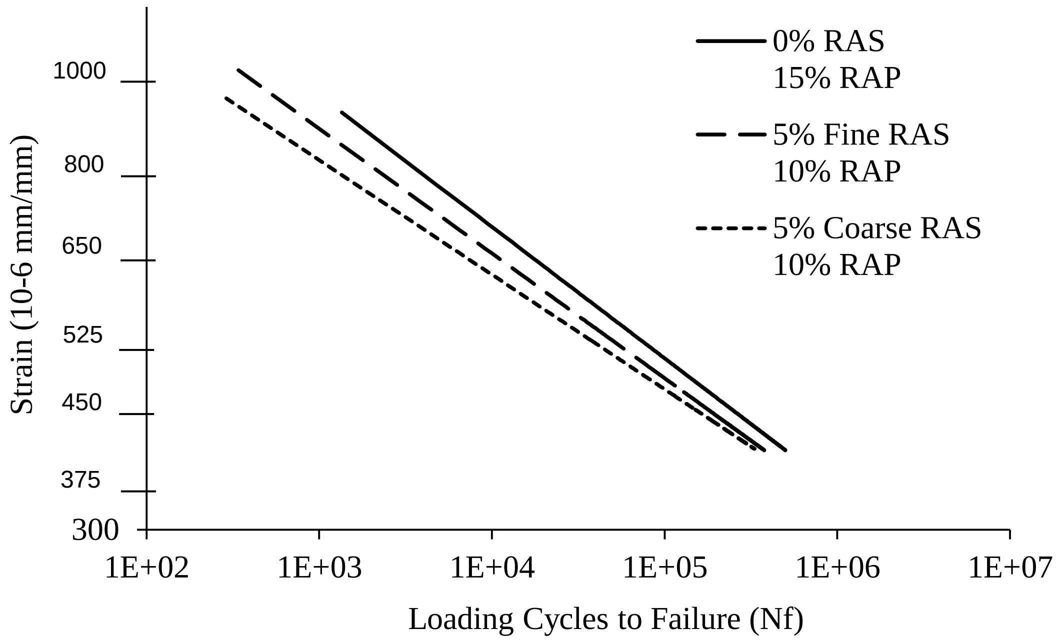


Figure A7.4. ε-N fatigue curves

The fatigue endurance limit (FEL) of each mixture was also predicted. If tensile strains are low enough in a pavement structure, the pavement has the ability to heal and therefore no damage cumulates over an indefinite number of load cycles. The level of this strain is referred to as the FEL. The FEL of each mixture was estimated using the lower 95% prediction limit at 50 million load cycles as proposed in NCHRP Report 646 and shown in Equation 2.

$$\text{Lower Prediction Limit} = \hat{y}_o - t_\alpha s\sqrt{1+\frac{1}{n}+\frac{(x_o-\bar{x})^2}{S_{xx}}} \qquad (2)$$

where:
y_o = the one-sided lower 95% prediction interval at the micro-strain level corresponding to 50,000,000 cycles;
t_α = value of t distribution for n-2 degrees of freedom for a significance level of 0.05;
s = standard error of the regression analysis;
n = number of samples;
S_{xx} = sum of squares of the x values;
x_o = log 50,000,000; and

$\bar{x}$ = average of the fatigue life results.

The FEL estimates are presented in Table A7.4. All three Missouri mixes exhibit similar long-term endurance limits. The RAS mixtures have the highest, and thus most desirable, endurance limits, indicating that RAS may improve the FEL in the Missouri mixes.

Table A7.4. Beam fatigue results

Mix ID	% Binder Replacement	K1	K2	R^2	Endurance Limit (Micro-strain)
0% RAS – 15% RAP	19.1	5.15E-17	-6.40	0.968	139
5% Fine RAS – 10% RAP	30.2	7.25E-19	-6.91	0.992	145
5% Coarse RAS – 10% RAP	30.2	2.07E-20	-7.37	0.968	159

Semi-Circular Bending

The low temperature fracture properties of the mixtures were obtained from SCB tests by following the procedure in “Investigation of Low Temperature Cracking in Asphalt” (Marasteanu et al., 2007). Testing was conducted at four different low temperatures: -6°C, -12°C, -18°C, and -22°C.

All tests were performed inside an environmental chamber, and liquid nitrogen was used to obtain the required low temperature. The temperature was controlled by the environmental chamber temperature controller and verified using an independent platinum resistive-thermal-device (RTD) thermometer. The load line displacement (LLD) was measured on both faces of the test specimens using a vertically mounted Epsilon extensometer with 38 mm gage length and ±1 mm range. One end was mounted on a button that was permanently fixed on a specially made frame, and the other end was attached to a metal button glued to the sample. The average LLD measurement was used for each specimen. The crack mouth opening displacement (CMOD) was recorded by an Epsilon clip gage with 10 mm gage length and a +2.5 and -1.0 mm range. The clip gage was attached at the bottom of the specimen. A constant CMOD rate of 0.0005mm/s was used and the load and load line displacement (P-u), as well as the load versus LLD curves were plotted. A contact load with a maximum load of 0.3 kN was applied before the actual loading to ensure uniform contact between the loading plate and the specimen. The testing was stopped when the load dropped to 0.5 kN in the post peak region. The load and load line displacement data were used to calculate the fracture toughness and fracture energy (G_f).

The fracture energy (G_f) parameter is presented in Figure A7.5. The laboratory test results were analyzed to evaluate the effect of the various RAS treatments on the fracture energy. MacAnova statistical software package was utilized to perform a statistical analysis. The analysis of variance (ANOVA) was used to examine the differences among the mean response values of the different treatment groups. The significance of the differences was tested at 0.05 level of error. The analysis of variance was conducted with the assumption that the errors in the data are independently normal with constant variance.

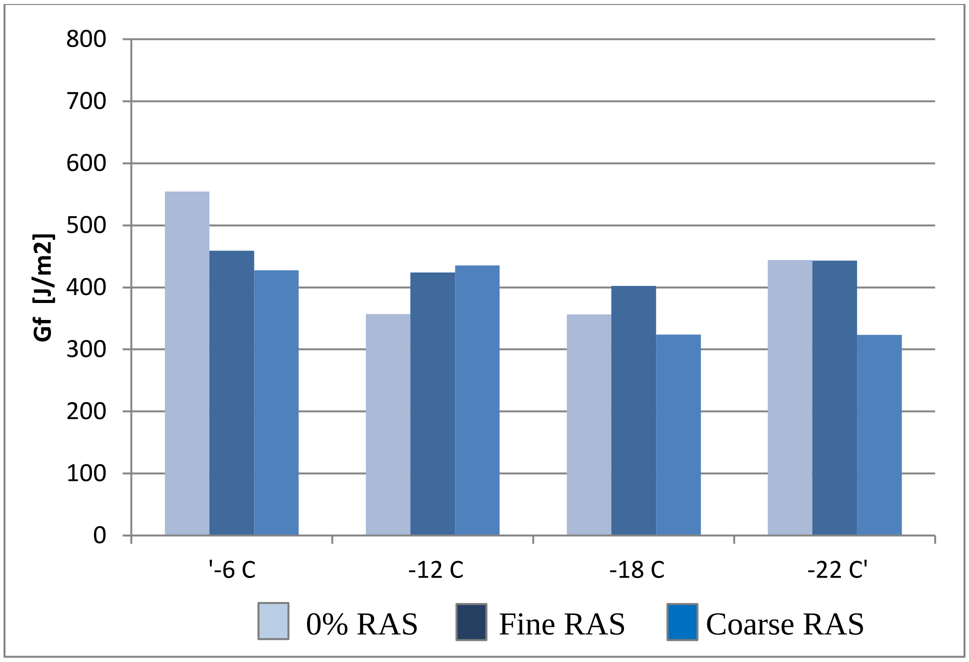


Figure A7.5. Missouri mixture fracture energy (G_f)

The Gf group means of each RAS treatment levels was compared using a pair-wise comparison to rank the RAS treatment levels with regard to fracture energy. The outcome is reported in Table A7.5, in which statistically similar RAS treatments are grouped together. Letter A indicates the best performing group of mixtures; letter B the second best, and so on. Groups with the same letter are not statistically different, whereas mixtures with different letters are statistically different.

For the Missouri mixes, when 5% RAP was replaced with RAS, the fracture energy did not change. While the mixture with a coarse grind RAS did decrease the fracture energy from 427 to 378 **J/m^2,** the difference was not statistically significant.

Table A7.5. Ranking of mixes by G_f mean value for -6, -12, -18, and -22°C temperatures

Rank	Treatment	Group mean G_f [J/m^2]
A	Control	428
A	Fine RAS	427
A	Coarse RAS	378

A8. Field Evaluations

Pavement condition surveys for the Missouri DOT demonstration project were completed in December 2010, May 2011 and March 2012. Three 500-foot sections were randomly selected in each of the test sections. The 500-foot surveys for the Control section were in the passing lane only, the 500-foot surveys for the Fine RAS section were in the driving lane only, and the 500-foot surveys for the Coarse RAS sections were in both the passing and driving lanes. Due to traffic control concerns, the Coarse RAS survey sections were limited to the first three miles of the pavement section. For the Coarse RAS sections, two 500-foot surveys were completed in the passing lane and one in the driving lane. The surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* by FHWA.

The condition surveys found a progression of transverse cracking over the two years within the three sections. It is highly likely the cracking found is reflective, due to the condition of the concrete prior to paving. In 2009 prior to the demonstration project, the Missouri DOT completed a pre-condition survey on the jointed concrete pavement. High severity distresses along the pavement were patched, which included areas of patching within the test sections. Additionally, the surface lift was placed on top of a 2-inch HMA base that included 20% RAP, which could also play a role in the different sections due to the variability of HMA placement in the field, weather conditions and workmanship.

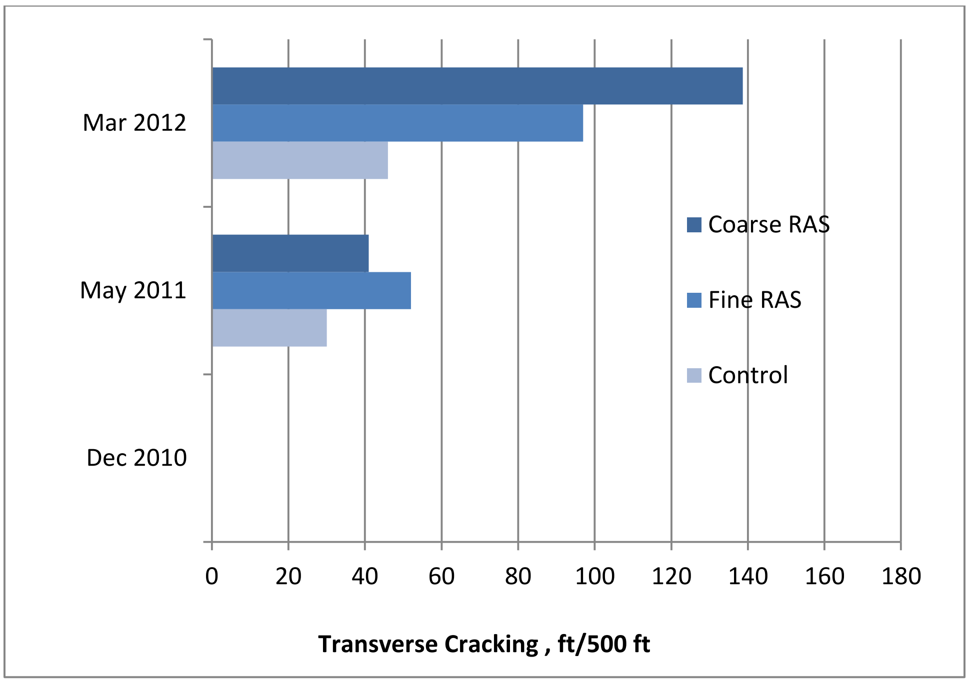


Figure A8.1. Missouri pavement evaluation

After one year, the Fine RAS sections contained the least amount of linear length of transverse cracking per 500 feet as shown in Figure A8.1. After two years, both RAS sections contained a greater amount of transverse cracking than the Control section.

While measuring the length of transverse cracking in the pavements, the severity level of the cracks was also measured. Following the guidelines of the Distress Identification Manual, transverse cracks were categorized into three levels: low severity (crack widths ≤ 0.25 in), moderate severity (crack widths 0.25 in ≥ 0.75 in), and high severity (crack widths > 0.75 in).

It may be possible the greater amount of cracking in the Fine RAS section compared to the Control section was due to its placement in the driving lane which will experience heavier traffic loads. Since the Coarse RAS sections were placed in both the driving and passing lanes, the amount of cracking in each lane can be compared to check if there are any performance differences between the two lanes. Figure A8.3 shows the average amount of transverse cracking in the Coarse RAS sections in the passing and driving lanes. There does not appear to be a large difference between the two lanes; therefore, the current difference in crack length between the Control and Fine RAS sections does not appear to be influenced by the lane location.

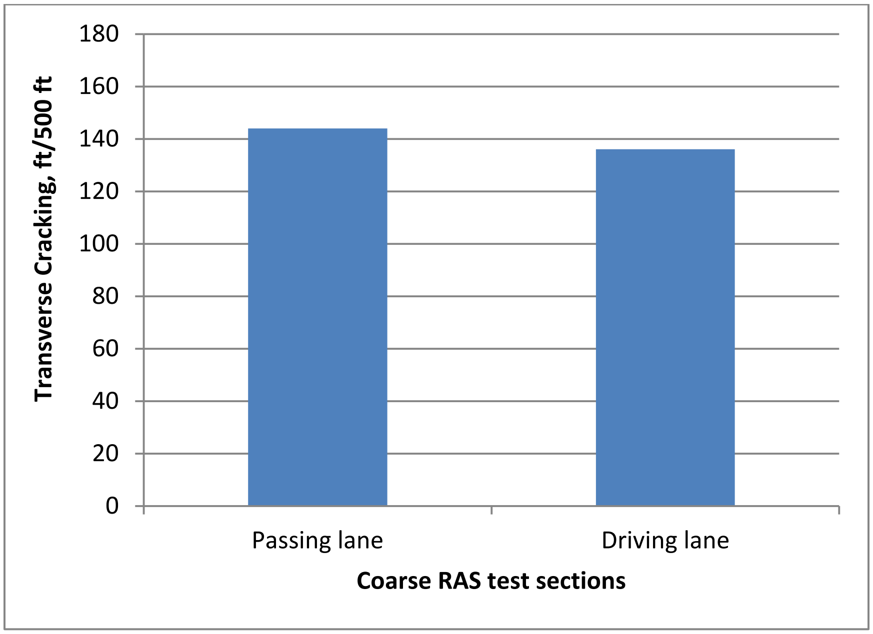


Figure A8.2. Transverse cracking in the coarse RAS test sections (March 2012)

Although the RAS pavement sections contained more transverse cracking than the Control sections after two years, most of the cracks in the RAS sections were of low severity while the Control sections had a greater percentage of cracks with a moderate to high severity. As shown in Figure A8.3, 11.5% of the transverse cracks measured in the Coarse RAS sections have a moderate or greater severity level, 4.1% of the transverse cracks measured in Fine RAS sections have a moderate or greater severity level, and 34.8% of the transverse cracks measured in Control sections have a moderate or greater severity level. Whether the low severity cracks in the RAS sections will expand into moderate of high severity cracks remains to be seen. Meanwhile, the current pavement survey data suggests that replacing 5% RAP with RAS may help prevent low severity cracks from expanding into a higher level of severity. The addition of fibers from the RAS could help prevent existing cracks from expanding.

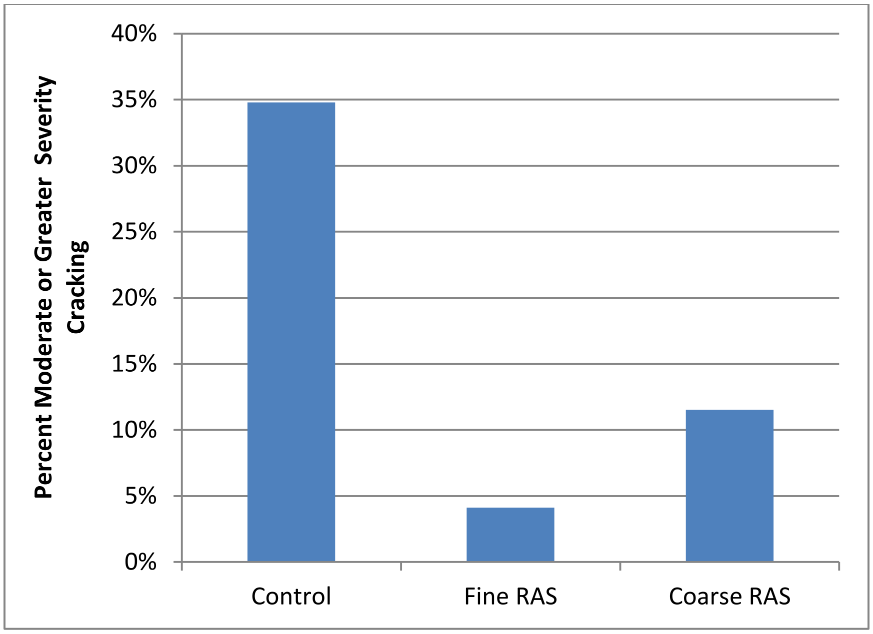


Figure A8.3. Percent of transverse cracks with moderate severity or greater (March 2012)

Examples of the transverse cracks (TC) measured in the pavement test sections are presented in Figures A8.4 and A8.5. In May 2011, it was also noted that several of the transverse cracks found in the Fine RAS material appeared to have originated in the 0% RAS lane (passing lane) and propagated into the Fine RAS lane (driving lane). In March 2012, a 20-foot longitudinal crack was documented in the Coarse RAS section and 20-square foot of raveling was documented in the Fine RAS section. Power-Point Presentations of the distress surveys by 500-foot sections are available for viewing on the TPF-5(213) website.

Figure A8.4. Low severity TC (Fine RAS)

Figure A8.5. Medium severity TC (Control)

A9. Conclusions

A Missouri DOT demonstration project was conducted as part of Transportation Pooled Fund 5-213 to evaluate the effects of replacing 5% RAP with RAS in asphalt mix designs that contain RAP and GTR. Three RAS mix designs were evaluated, a control mixture containing 15% RAP and no RAS, a mixture that replaced 5% for the RAP with a coarse grind RAS (100% passing the 1/2" sieve), and another mixture that replaced 5% of the RAP with a fine grind RAS (100% passing the 3/8" sieve). Field mixes of each pavement were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and recovery with subsequent binder characterization. The results of the study are summarized below:

- Observations from the demonstration project show that the contractor successfully produced and constructed the RAS pavements while meeting MoDOT's quality assurance requirements. However, the pavement with the finer grind RAS visually appeared to be a more homogenous mixture than the coarser grind RAS. The likelihood of RAS tabs protruding the pavement was reduced with a finer grind RAS.
- Mix designs with RAS and asphalt binder modified with 10% GTR by weight of asphalt binder and 4.5% TOR by weight of GTR were successfully designed and produced to meet MoDOT specifications. Laboratory performance tests indicated the mixtures have excellent rutting resistance which was enhanced by the RAS and GTR. The combination of GTR and RAS did not affect the fatigue and low temperature cracking properties of the mixtures since the mixture with GTR alone exhibited a similar FEL and fracture energy as the mixtures with GTR and RAS.
- The performance grade of the total binder in the asphalt mixtures was raised in both low and high temperature grades with the addition of RAS. The continuous PG for the control mixture was 75.0-16.8, while the Fine RAS mixture was 90.1-8.7 and the Coarse RAS mixture was 88.3-4.9.
- Adding RAS to the mix designs increased the dynamic modulus at high temperatures (the Coarse RAS by 36% and the Fine RAS by 59%) for improved rutting resistance. This was likely due to the stiffer RAS binder and fibers contained in the RAS. At intermediate temperatures, the dynamic modulus of both RAS mixtures decreased 9%, which can improve the fatigue cracking resistance of the mixtures when in thin lift pavements. The reduction in stiffness may be the result of higher binder contents in the RAS mixtures.
- In the flow number test, all three mixtures did not reach tertiary flow and their percent strain accumulation was measured at the end of 10,000 load cycles. The Control mixture accumulated 0.81% strain. As found in the dynamic modulus test, adding RAS improved the permanent deformation resistance of the mixtures, with the Fine RAS mixture exhibiting a lower amount of strain accumulation (0.36%) compared to the Coarse RAS mixture (0.48%).
- The four-point bending beam results showed that for thin lift pavements, the Control mixture had greater fatigue resistance at higher strain levels while the RAS improved the mixtures' estimated fatigue endurance limit (from 139 to 159 µstrain).

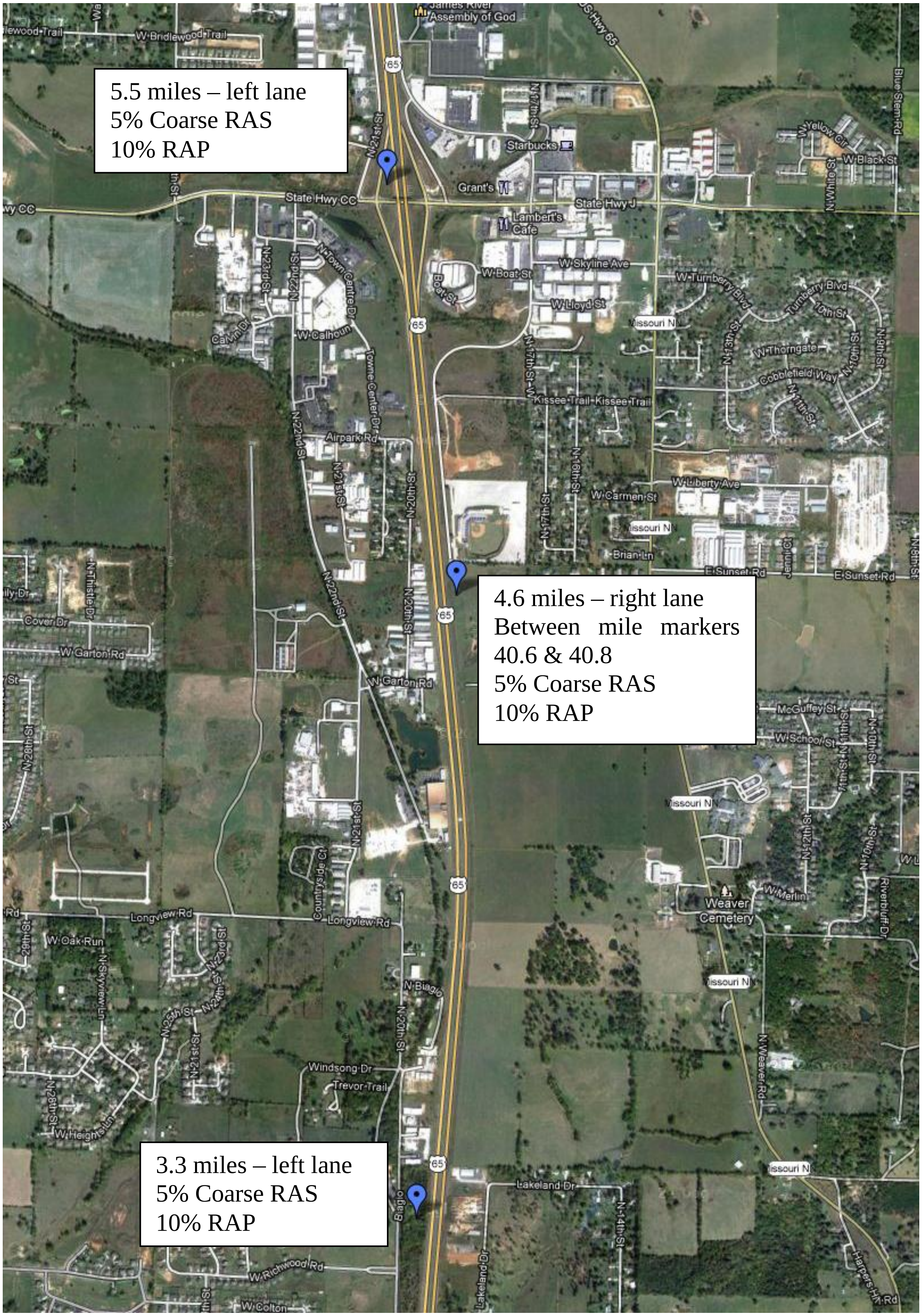
- The SCB test was performed to measure the low temperature cracking susceptibility of the mixtures by measuring their fracture energy at -6°C, -12°C, -18°C, and -22°C. Statistical analyses of the results show that replacing 5% RAP with RAS did not change the low temperature fracture energy of the mixtures.
- Field condition surveys conducted one and two years after the demonstration project revealed that all three pavement sections are susceptible to reflective cracking. The distress found in the three test sections is attributed to the distress of the concrete pavement below. Areas of concrete patches constructed prior to the demonstration project reflected through the base and surface course pavement overlays to produce low to high severity transverse cracking. The RAS pavement sections displayed a greater amount of transverse cracking than the Control pavement sections. However, the Control pavement sections exhibited the greatest percentage of transverse cracking higher than a low severity level (34.8%). In contrast, the Fine RAS pavement section exhibited the least percentage of transverse cracking higher than a low severity level (4.1%).

A10. MoDOT Demonstration Project Acknowledgments

The researchers gratefully acknowledge the support of Joe Schroer at the Missouri DOT. The research work was sponsored by Federal Highway Administration (FHWA) TPF-5(213) and the Transportation Pooled Fund (TPF) partners: Missouri (lead agency), California, Colorado, Illinois, Indiana, Iowa, Minnesota, and Wisconsin DOTs.

A11. **Pavement Survey Locations**


2.2 miles – left lane
0% RAS
15% RAP
2.0 miles – right lane
5% Fine RAS
10% RAP
1.8 miles – left lane
0% RAS
15% RAP


5.5 miles – left lane
5% Coarse RAS
10% RAP
4.6 miles – right lane
Between mile markers
40.6 & 40.8
5% Coarse RAS
10% RAP
3.3 miles – left lane
5% Coarse RAS
10% RAP
State Hwy CC
State Hwy J
Longview Rd
Weaver Cemetery

APPENDIX B. REPORT FOR THE IOWA DEPARTMENT OF TRANSPORTATION SPONSORED DEMONSTRATION PROJECT

B1. Introduction

This report presents a summary of the results obtained from the field demonstration project sponsored by the Iowa Department of Transportation (Iowa DOT) as part of Transportation Pooled Fund (TPF) 5-213 *Performance of Recycled Asphalt Shingles in Hot Mix Asphalt*. TPF 5-213 is a partnership of several state agencies with the goal of researching the effects of recycled asphalt shingles (RAS) on the performance of asphalt applications. As part of the pooled fund research program, multiple field demonstration projects were conducted to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements that include RAS.

Each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated different aspects of asphalt mixes containing RAS. The field demonstration project sponsored by Iowa DOT investigated the effect of different percentages of post-consumer RAS in hot mix asphalt (HMA). The objective of this demonstration project was to evaluate the performance of mixes containing RAS and compare their performance to an Iowa DOT mix design containing no recycled product: no recycled asphalt pavement or RAS.

B2. Experimental Plan

To evaluate the performance of HMA with RAS at different percentages, Iowa DOT designed an experimental plan to address the following questions:

- Is there an added value to the performance of the mix when adding RAS, or will the RAS compromise mix performance?
- At what RAS percentage does the mix perform best? How will the RAS mix compare to a virgin mix?

The experimental plan is presented in Table B2.1. The plan was implemented during the demonstration project by producing four asphalt mixtures: a mixture with 4% RAS, a mixture with 5% RAS, a mixture with 6% RAS, and a control mixture with 0% RAS.

Table B2.1. Experimental plan

Mix ID	% RAS	RAS Source
0% RAS	0	-
4% RAS	4	post-consumer
5% RAS	5	post-consumer
6% RAS	6	post-consumer

During production of the asphalt mixtures, Iowa State University collected samples of each mixture for laboratory testing. A portion of the samples were sent to the University of Minnesota and the Minnesota Department of Transportation (MnDOT) for Semi-Circular Bend (SCB) testing and binder extraction and recovery, respectively. The laboratory testing plan is presented in Table B2.2.

The asphalt was recovered from the RAS and asphalt mixtures following AASHTO T164 Method A (Centrifuge Method) by using a blend of toluene and ethanol as the extraction solvent. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. At Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO R29 "Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder."

Table B2.2. Laboratory testing plan

	Laboratory Test	Iowa State University	University of Minnesota	Minnesota DOT
Processed Shingles	Binder Extraction			X
	High Temperature PG			X
	Gradation (Before Extraction)	X		
	Gradation (After Extraction)	X		
Mixture	Binder Extraction			X
	Binder PG Characterization	X		
	Gradation	X		
	Dynamic Modulus	X		
	Flow Number	X		
	Beam Fatigue	X		
	Semi-Circular Bending (SCB)		X	

Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution, and a washed gradation was conducted after extraction to evaluate the size distribution of the fine aggregates in the RAS product.

Performance testing was completed on the field produced asphalt mixtures at low, intermediate, and high temperatures. Dynamic Modulus and Flow Number were conducted at high temperatures to evaluate the modulus and rutting resistance of asphalt mixtures. The durability of the mixtures at intermediate temperatures was evaluated using the dynamic modulus and four-point beam fatigue test. The SCB test conducted at the University of Minnesota evaluated the fracture properties of the mixtures at low temperatures.

After construction of the pavement for the demonstration project, field evaluations were conducted on each pavement test section one and two years after paving to assess the field performance of the pavement concerning cracking, rutting, and raveling.

B3. Project Location

The field demonstration project was completed on Highway 10 just west of Paullina, Iowa in Sioux County located in the northwest corner of the state. The test sections were placed on the eastbound (EB) and westbound (WB) lanes of Highway 10. The project started at the east end of Paullina, IA and continued west 16.25 miles, passing through the Granville, IA and ended at the intersection Highway 10 and 450th Street in Alton, IA. The project limits are identified below in Figure B3.1.

Figure B3.1. Project location

B4. Project Description

The demonstration project, Iowa DOT number STP-10 1(70)-2c-84, was a resurfacing of the existing jointed concrete with a two-inch HMA surface course. A cross-section is shown in Figure B4.1.

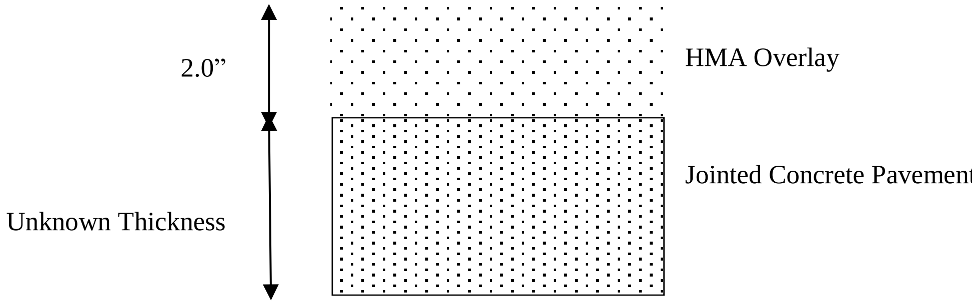


Figure B4.1. Pavement cross-section

Tri-State Paving (Tri-State) paved the surface course test sections in June/July 2010. Starting at the east end (Paullina) of the project, the 5% RAS test sections are approximately 3.4 miles in the EB lane and 3.8 miles in the WB lane; the 4% RAS test sections are approximately 3.6 miles in EB lane and 4.1 miles in the WB lane; the 6% RAS test sections are approximately 6.3 miles in the EB lane and 3.0 miles in the WB lane; and the 0% RAS sections are approximately 3.75 miles in the EB lane and 5.35 miles in the WB lane. A plan view of the test sections on Highway 10 is shown in Figure B4.2.

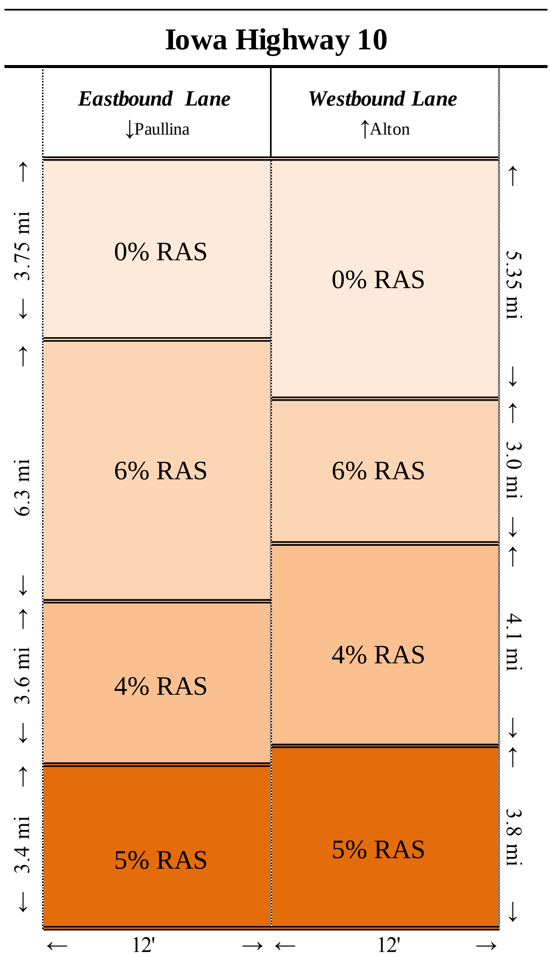


Figure B4.2. Plan view of Highway 10 project test sections

Wet spring weather conditions created some delays and extended the project for several days. Weather conditions during the paving were ambient temperatures ranging from 71-91 degrees Fahrenheit with sunny to cloudy skies and moderate to high humidity. Paving was completed during day hours and traffic during paving was limited to one lane and controlled by flaggers.

Tri-State used a portable plant to produce the HMA. The plant was located approximately three miles west of Paullina on County Road 48. The haul distance from the plant to the furthest project point was 20 miles. The plant was a double barrel drum plant with a capacity to produce up to 500 tons of HMA per hour (Figure B4.3). RAS was the only recycled product used and placed in the bin normally used for recycled asphalt pavement (RAP) (Figures B4.4).

Figure B4.3. Portable plant

Figure B4.4. Adding RAS in bin

A conveyor belt carried the RAS to a vibrating screen (grizzly) to remove any clumps that may have occurred in the stockpiles during the holding time from delivery to plant usage. (Figure B4.5). A second conveyor belt delivered the RAS to the RAP collar where it was incorporated into the double drum (Figure B4.6). Mix temperatures ranged from 297 to 315°F

Figure B4.5. RAS screening

Figure B4.6. Adding RAS to drum

Approximately 30,951 tons of HMA and 1,097 tons on RAS was placed for the demonstration project. Tonnages of RAS and HMA for each test section are summarized below in Table B4.1.

Table B4.1. Project tonnages

Material	0% RAS (Tons)	4% RAS (Tons)	5% RAS (Tons)	6% RAS (Tons)
RAS	---	333	406	358
Total HMA	8,653	7,668	8,149	6,481

B5. Shingle Processing

The RAS was supplied to Tri-State by Dem-Con Companies, LLC (Dem-con) in Shakopee, MN, an Iowa DOT approved supplier of RAS. Dem-con collects and sorts loads of post-consumer shingles onsite. Asbestos testing of each load is conducted using the polarized light microscopy method to verify the shingles do not contain greater than one percent of asbestos containing materials (ACM). Dem-con completes certification forms to verify that shingles from buildings not regulated by the National Emission Standards for Hazardous Air Pollutants (NESHAP) do not contain ACM >1%.

Dem-con uses an industrial Rotochopper grinder, along with an additional screening process to produce a final product with 100% passing the 1/2” sieve and 95% passing the 3/8” sieve. The RAS sample obtained by Iowa State for laboratory testing contained 97% passing the 1/2” sieve. The product passes over two magnets to remove all ferrous material. The industrial grinder utilizes water nozzles to control heat build-up and dust during the grinding process. The final post-processed RAS stockpiles were uncovered and open to all weather conditions. The RAS product is pictured in Figure B5.1, and the gradation test results of the RAS before and after extraction are presented in Table B5.1. Asphalt extracted from the RAS by MNDOT was measured to be 21.7%

Figure B5.1. Post-consumer RAS

Table B5.1. RAS gradations (percent passing)

Sieve Size (US)	Sieve Size (mm)	RAS (Before Extraction)	RAS (After Extraction)
3/4"	19	100	100
1/2"	12.5	97	99
3/8"	9.5	95	98
#4	4.75	84	95
#8	2.36	67	90
#16	1.18	44	72
#30	0.6	22	51
#50	0.3	10	40
#100	0.15	3	30
#200	0.075	0.6	21.3
	% Asphalt Content		21.7

B6. Asphalt Mix Design and Prodution Results

Two HMA mix designs were prepared by Tri-State for the demonstration project. The first mix design contained 0% RAS; the second mix design contained 5% RAS. Both mixes followed Iowa DOT specifications for a surface coarse mix designed for 1 million single equivalent axel loads (ESALS) with an aggregate frication category 4 and a ½ inch nominal maximum aggregate size (NMAS).

To produce the asphalt mixtures that contained the 4% and 6% RAS, Tri-State used the mix design containing 5% RAS and added the desired amount of RAS (either 4% or 6%) during production while adjusting the amount of virgin asphalt. The 5% RAS mix contained the same aggregates as the 0% RAS mix but with different percentages so the blend gradation matched the 0% RAS mix blend gradation. Gradations obtained from laboratory testing of the sampled asphalt mixtures are presented in Figure B6.1. As shown in the figure, the asphalt mixtures had similar aggregate structures with gradations passing above the restricted zone.

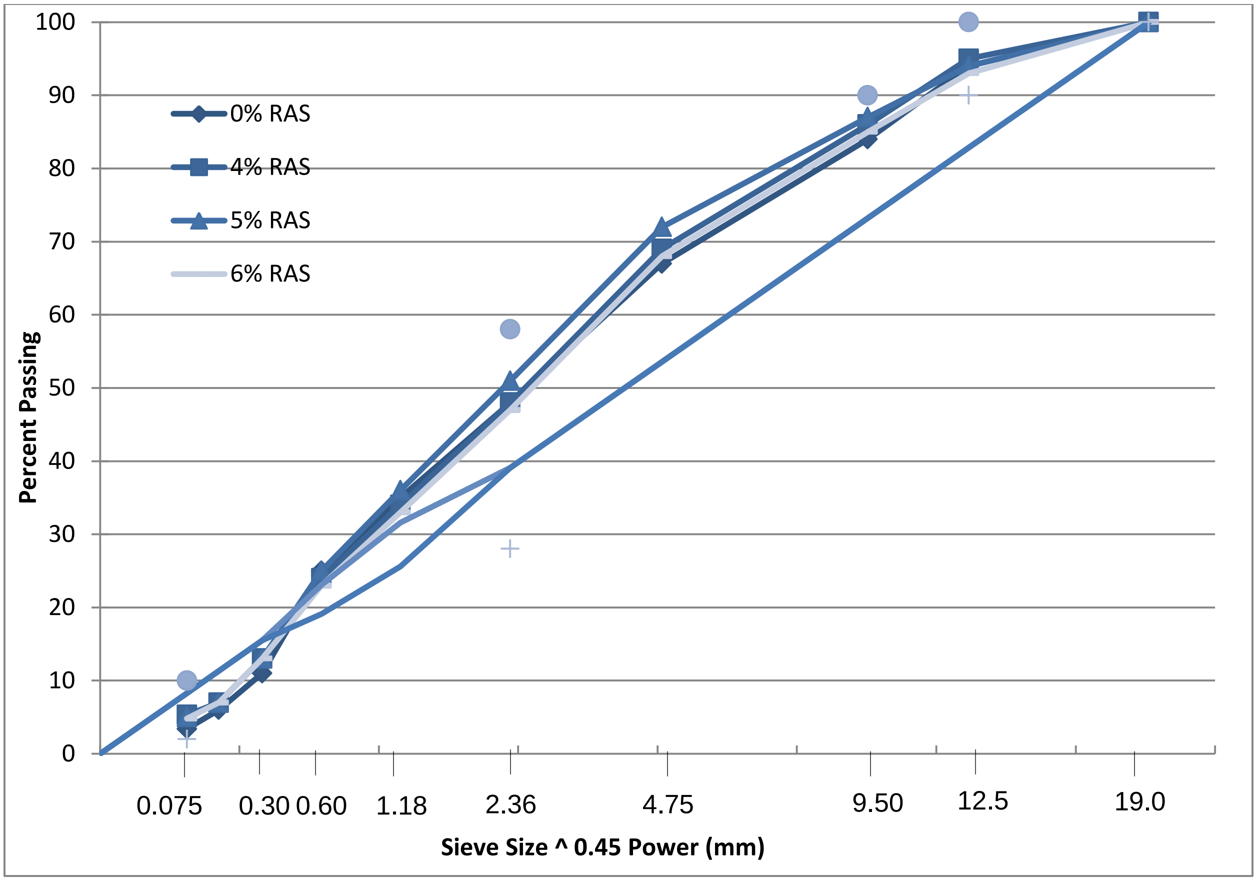


Figure B6.1. Asphalt gradations

The mix design properties are presented in Table B6.1. During the development of the mix design, the RAS contained 17.8% asphalt that contributed to the total asphalt in the mix. This resulted in a 15.6% binder replacement when 5% RAS was added to the mix design.

Table B6.1. Mixture asphalt demand properties

Mix Property	0% RAS	5% RAS
% RAS	0	5
% Total AC	5.90	5.73
% Virgin AC	5.90	4.84
% Binder Replacement	0	15.6
% Effective Asphalt	5.28	5.26
% Asphalt Absoprtion	0.62	0.47

Since the aggregates were adjusted in the 5% RAS mix design so the final gradation blend matched the 0% RAS mix design, the VMA in both asphalt designs remained relatively the same at 15.8/15.9% (Table B6.2). However, even with constant VMA, the optimum asphalt content (AC) decreased from 5.90% to 5.73% when 5% RAS was added to the mix design. That is a 0.17 difference, almost the same difference in asphalt absorption (0.15). Less asphalt absorption most likely reduced the total asphalt demand of the 5% RAS mix. Thus, the effective asphalt contents of the two mixes remained same (5.2%).

A comparison of the water absorption of the two mix designs reveals the 5% RAS mix contained more absorptive aggregates (0.98% absorption) than the 0% RAS mix (0.90% absorption). Therefore, the reduced asphalt absorption was not caused by the aggregates. It may be the result of the virgin binder blending with the RAS binder. The resulting blend would be a stiffer binder that would not be able to penetrate into the aggregates as deeply as the softer virgin binder would. If this is a correct hypothesis, then it would give evidence that the RAS binder and the virgin binder blended together well during laboratory mixing.

Table B6.2. Mixture design properties

Mix Property	0% RAS	5% RAS
Design Gyrations	76	76
NMAS (mm)	12.5	12.5
Virgin PG Grade	58-28	58-28
% Voids	4.0	4.0
% VMA	15.9	15.8
% VFA	75.0	74.7
-#200/Pbe	0.80	0.85

Measurements of the RAS, virgin binder, and HMA produced during production are presented in Table B6.3. Test strips of the 0% RAS and 5% RAS mix designs were paved before the start of the project to verify the field produced mixes. The laboratory voids were low in the mixes so the target asphalt content was reduced to 5.5% for the 0% RAS mix and 5.6% for the 5% RAS mix. Tri-State assumed the RAS contained 24% asphalt that would contribute to the total asphalt content of the mix. Thus, for a target of 5.6% asphalt content, Tri-state added 4.4% virgin binder when 5% RAS was used during production. The quality control results in Table B6.4 show the asphalt content of the 5% RAS mix was 5.4%. This equates to an average 19.4% binder replacement with 20.5% asphalt in the RAS contributing to the HMA (Table B6.3). The 20.5% asphalt in the RAS is a little shy of the 24% assumption by Tri-State and the 21.7% measurement by MNDOT but close enough to produce a mix meeting Iowa DOT specifications.

For the 4% and 6% RAS mixtures, Tri-State adjusted the virgin asphalt to 4.6% and 4.2% respectively to account for the change in RAS percentage. This resulted in an average of 16.3% binder replacement for the 4% RAS mix and a 22.8% binder replacement for the 6% RAS mix. Table B6.3 shows that for any RAS content between 4% and 6%, the RAS always contained approximately 20.5% asphalt that contributed to the mixture asphalt content.

The percent RAS added to the mix was not always exact but contained some variability. The 6% RAS mix produced on 7/7/10 contained a lower amount of RAS than the target percentage because the target asphalt content was lowered to 5.4%. To account for this change, Tri-State added the same amount of virgin AC while reducing the RAS percentage. The 6% RAS sample was obtained on 7/1/10 so the RAS fluctuation on 7/7/10 did not affect the laboratory portion of the study.

Table B6.3. Asphalt availability in RAS during production

Mix	Date	Tons of Mix Produced	Tons of RAS	% RAS	% Virgin AC added	% Binder Replacement	% AC in RAS
4% RAS	6/25/10	3,667.1	161.9	4.4	4.58	16.6	20.7
	6/30/10	4,000.4	171.3	4.3	4.60	16.0	20.4
5% RAS	6/22/10	4,021.4	195.8	4.9	4.42	18.9	20.4
	6/24/10	4,128.0	210.0	5.1	4.41	19.8	20.5
6% RAS	7/1/10	3,358.3	204.2	6.1	4.20	22.8	20.4
	7/7/10	3,123.1	154.1	4.9	4.20	19.4	20.6

Tri-State successfully produced the RAS mixtures within Iowa DOT specifications. Table B6.4 shows the QC results for the asphalt content and laboratory voids were close to the target values. The RAS mixtures were also compacted as well as the 0% RAS mixture, indicated by field voids measured from core samples which met the maximum 8.0% requirement.

Table B6.4. Mixture and construction quality control results

Mix	Date	Asphalt Content		Lab Voids		Field Voids	
		Target	QC Results	Target	QC Results(1)	Maximum	QC Results(2)
0% RAS	7/9/2010	5.5	5.47	4.0	3.2	8.0	5.5
	7/12/2010	5.5	5.34	4.0	4.1	8.0	7.5
	7/13/2010	5.5	5.51	4.0	4.1	8.0	7.1
	7/14/2010	5.5	5.48	4.0	4.5	8.0	7.2
	7/15/2010	5.5	5.71	4.0	3.7	8.0	6.1
4% RAS	6/25/2010	5.6	5.49	4.0	4.0	8.0	6.5
	6/30/2010	5.6	5.48	4.0	4.0	8.0	7.5
5% RAS	6/22/2010	5.6	5.42	4.0	3.7	8.0	6.6
	6/24/2010	5.6	5.45	4.0	4.0	8.0	7.6
6% RAS	7/1/2010	5.6	5.44	4.0	3.8	8.0	6.7
	7/7/2010	5.4	5.21	4.0	3.8	8.0	7.7

(1) Average of four lab density results per day
(2) Average of eight core density results per day

B7. Laboratory Test Results

Binder Testing

Performance Grade (PG) testing of the extracted binders was conducted to obtain their high, low, and intermediate PG temperatures as shown in Table B7.1. The high temperature performance grade of the RAS binder at 124.1°C is higher than a traditional paving grade binder. This is expected since the binder in roofing shingles is produced with an air-blowing process which oxidizes the asphalt. Additionally, the RAS used in the mix designs is from post-consumer shingles, so the binder in the RAS has experienced at least several years of aging.

Because the RAS mixtures are heated to high temperatures and placed in a centrifuge at high speeds during the recovery process, the RAS and virgin asphalt should be fully blended. A 58-28 virgin binder was used for the project. Heating of the HMA during production and reheating of the samples during laboratory extraction had an aging effect on the binder since the asphalt extracted from the 0% RAS mixture had a continuous PG of 73.0-19.7. Adding 4% RAS raised the PG to 75.8-19.1. At 5% RAS the continuous PG was 81.3-16.8, and at 6% RAS the PG increased again to 86.1-14.7.

Table B7.1. Performance grade of extracted binders

Material Identification	High PG Temp, °C	Intermediate PG Temp, °C	Low PG Temp, °C	Performance Grade
Virgin PG 58-28	61.1	17.9	-28.2	58-28
RAS	124.1	-	-	-
0% RAS	73.0	23.7	-19.7	72-16
4% RAS	75.8	21.3	-19.1	72-16
5% RAS	81.3	22.1	-16.8	76-16
6% RAS	86.1	24.4	-14.7	86-10

Dynamic Modulus

The dynamic modulus (|E*|) is a key material property that determines the stress-strain relationship of an asphalt mixture under continuous sinusoidal loading. A higher dynamic modulus indicates lower strains will result in a pavement structure when the asphalt mixture is stressed from repeated traffic loading. The mechanistic-empirical pavement design guide (MEPDG) uses |E*| as the stiffness parameter to calculate an asphalt pavements strains and displacements.

The test was conducted following AASHTO TP62 using five replicate samples of 150 mm in height and 100 mm in diameter. Each sample was compacted to 7 ± 0.5% air voids. Samples were tested by applying a continuous sinusoidal load at 9 different frequencies (0.1, 0.3, 0.5, 1, 3, 5, 10, 20, and 25 Hz) and three different temperatures (4, 21, and 37°C). Sample loading was adjusted to produce strains between 50 and 150 μstrain in the sample.

Master curves were constructed at a reference temperature of 21°C and plotted on a log-log scale for a general comparison as presented in Figure B7.1. At high temperatures, the addition of the RAS binder increases the overall stiffness of the mixture since the RAS mixtures have a higher dynamic modulus than the control mixture (0% RAS). Fibers in the RAS could also be providing a reinforcing effect that augments the mixture's modulus at high temperatures. Higher dynamic modulus values in the RAS mixtures indicate adding RAS to the mixture will improve its field rutting performance.

The plot in Figure B7.2 presents the mean dynamic modulus at 21°C and 37°C at specific frequencies for a more direct comparison**.** Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. At 21°C and 5 Hz, adding 4% RAS initially increased the mixture stiffness. However, increasing the RAS content above 4% decreased the mixture stiffness. At 6% RAS, the mixture is less stiff than the 0% RAS mixture. Low modulus values at this temperature are considered desirable in thin asphalt pavements (less than 4”) for fatigue cracking resistance. Mixtures with lower stiffness can deform more easily without building up large stresses. Lower modulus values in the RAS mixtures at this temperature, however, are counter intuitive because of the inclusion of a stiffer binder. Higher levels of RAS fibers may be affecting the overall material response during dynamic loading by reducing the modulus at intermediate temperatures.

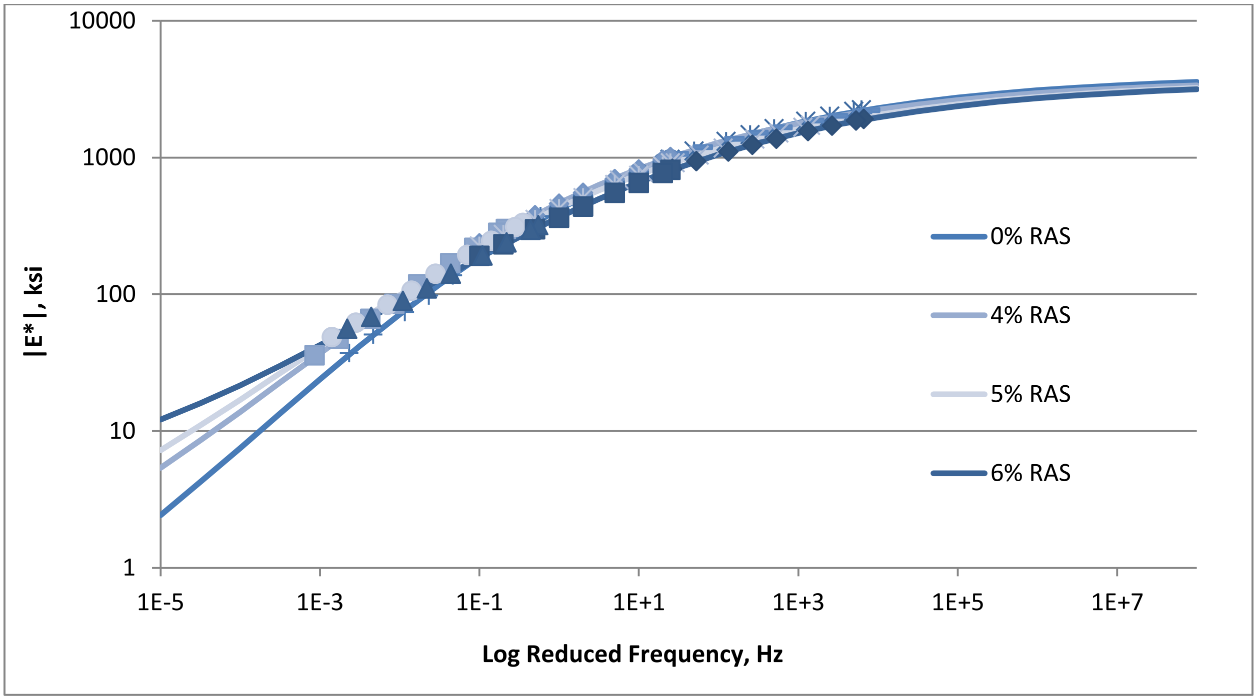


Figure B7.1. Comparison of master curves for Iowa DOT mixes

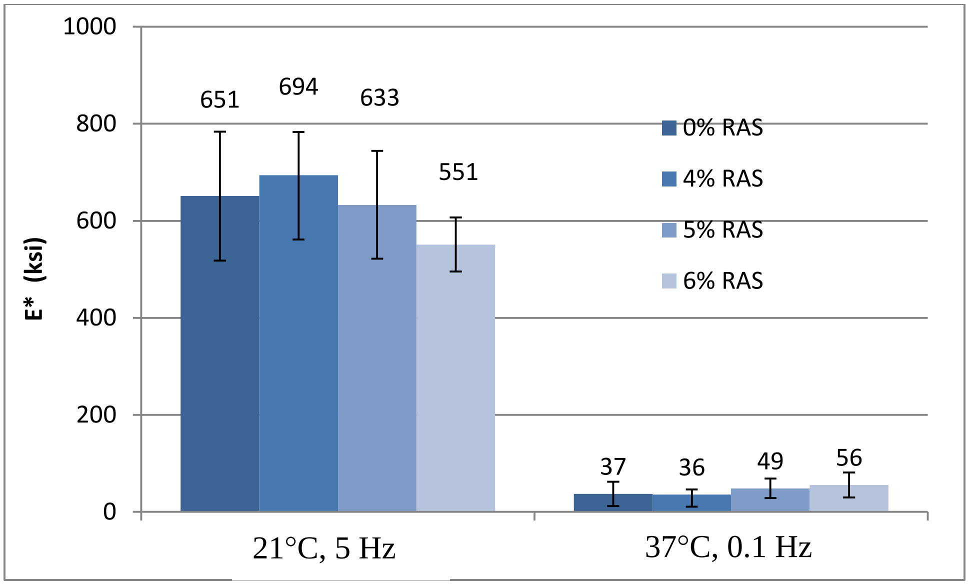


Figure B7.2. Dynamic modulus comparison at 21°C, 5 Hz and 37°C, 0.1 Hz

Flow Number

The flow number test measures the permanent deformation resistance of asphalt mixtures by applying a repeated dynamic load to a sample for up to several thousand load cycles. The flow number is defined as the number of load cycles an asphalt mixture can tolerate until it flows. Cumulative permanent deformation in the sample is plotted versus load cycles. The flow number is reached at the onset of tertiary flow.

Tests were conducted following procedures used in NCHRP Report 465. Samples used in dynamic modulus test were used for the flow number test since the dynamic modulus test is nondestructive. The samples were placed in a hydraulically loaded universal testing machine, unconfined, with a testing temperature of 37°C to simulate the climactic conditions that cause pavement to be susceptible to rutting. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 sec followed by 0.9 sec of dwell time. The loading cycle was repeated for a total of 10,000 load cycles. Three LVDT's were attached to each sample during the test to measure the cumulative strains.

Test results are presented in Figure B7.3. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. If the error bars of two mixtures do not overlap, then the difference of the two mixtures can be considered statistically significant at the 5% level.

The control mixture (0% RAS) is more susceptible to permanent deformation since it has a lower flow number than the RAS mixtures. The flow number of the mixture increases as the percentage of RAS increases in the mixture. The greatest increase in flow number occurs between the 4% RAS and 5% RAS mixtures. At 6% RAS, the flow number remains the same, but with less variation in the data. With flow numbers close to 6000, the 5 % and 6% RAS mixtures should be very resistant to permanent deformation.

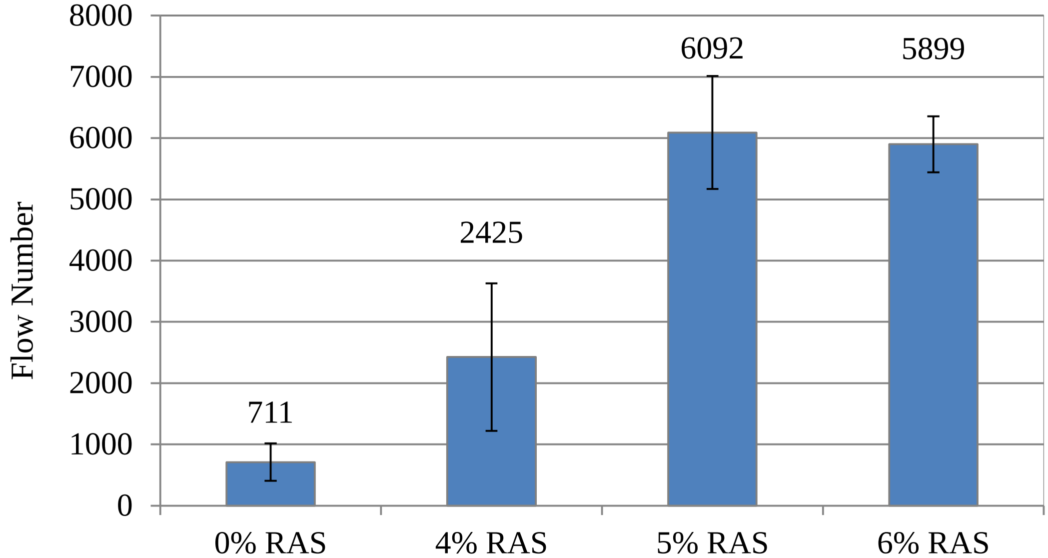


Figure B7.3. Flow number test results

Beam Fatigue

Fatigue cracking is the major cracking distress in asphalt pavements caused by repeated heavy traffic loads. Pavements that have a higher resistance to tensile strains that develop at the bottom of an asphalt layer due to repeated traffic will have a greater resistance to fatigue cracking. The four-point beam fatigue test following AASHTO T321 was conducted to evaluate the load associated cracking resistance of the mixtures.

The mixes were compacted in a linear kneading compactor to create slabs with 7% air voids. The slabs were saw-cut into beams with dimensions 15 inches in length, 2.5 inches in width, and 2 inches in height. The beams were tested in a strain controlled mode of loading at 20°C with haversine wave pulses applied to the beam at 10 Hz. Six beams were tested, each at a different strain level (375, 450, 525, 650, 800, and 1000 µstrain), until the flexural stiffness of the beam was reduced to 50% of the initial stiffness. A log-log regression was performed between strain and the number of cycles to failure (N_f). The relationship between strain and N_f can be modeled using the power law relationship as presented in Equation 1.

$$N_f = K1\left(\frac{1}{\varepsilon_o}\right)^{K2} \qquad (1)$$

where: N_f = cycles to failure; ε_o = flexural strain; and K1 and K2 = regression constants.

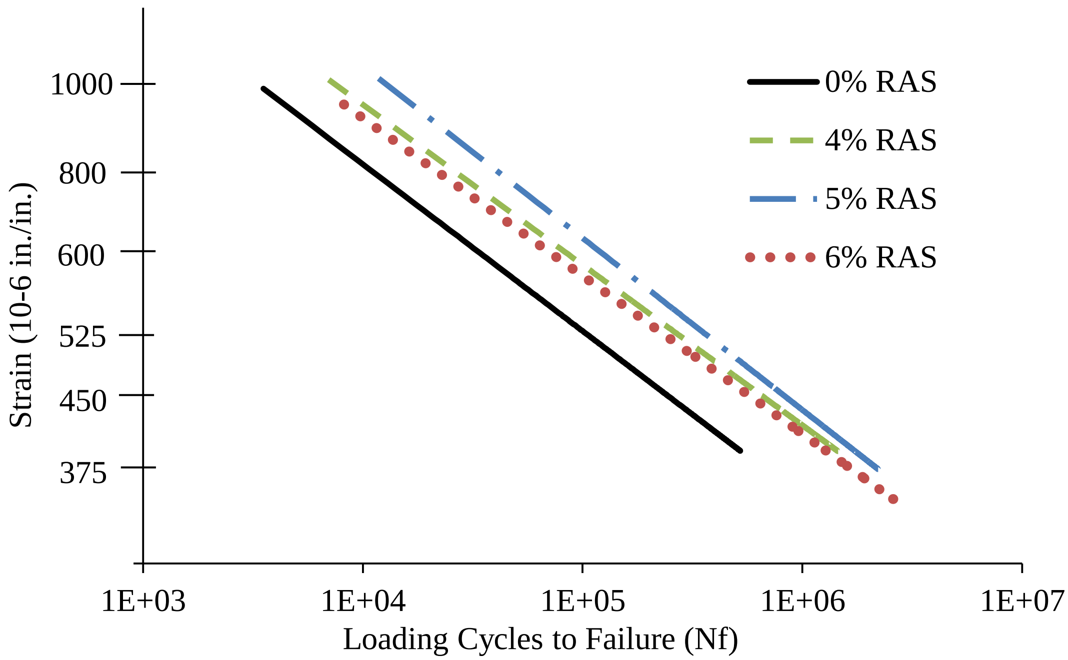


Figure B7.4. ε-N fatigue curves

The beam fatigue test results, as shown by strain versus "loading cycles to failure" curves, are presented in Figure B7.4. The fatigue curve model coefficients, average initial stiffness, and R^2 values are presented in Table B7.4. Because the fatigue life increases with the addition of RAS in a controlled strain mode of loading, the results indicate that RAS will improve the fatigue life of a thin lift pavement.

The four mixtures contain vary similar gradations and volumetric properties. They all have approximately the same asphalt content as shown in Table B6.4. The only difference between the mixtures is percentage of RAS. Because RAS contains stiffer binder than virgin binder, it is expected that an increase in RAS percentage would increase the stiffness of the mixture. Yet, the average initial beam stiffness of the 0% RAS mixture was 3497 MPa while the average initial beam stiffness of the 4%, 5%, and 6% RAS mixtures was 3090 MPa, 3106 MPa, and 3156 MPa respectively. Past beam fatigue studies in controlled strain mode of loading showed that when stiffness decreases from a change in binder type or grade, beam fatigue life is typically increased (SHRP-A-404). These results appear to follow the same trend as well, since the mixes with lower initial stiffness demonstrated longer fatigue lives. However, as the percentage of RAS increases from 0 to 4 to 5 percent in the mixture, which stiffens the binder grade, the fatigue life uncharacteristically increases. A possible explanation of this phenomenon, could be from the complex RAS-aggregate-binder interactions and the contribution of fibers from the RAS.

As the percent RAS content increases from 5% to 6%, the fatigue life no longer increases but decreases. While still significantly higher than the fatigue life of the 0% RAS mixtures, the decrease could result from the effect of the stiffer binder (now at 22.8 percent replacement) having a more influential effect on the fatigue properties.

The fatigue endurance limit (FEL) of each mixture was also predicted. If tensile strains are low enough in a pavement structure, the pavement has the ability to heal and therefore no damage cumulates over an indefinite number of load cycles. The level of this strain is referred to as the FEL. The FEL of each mixture was estimated using the lower 95% prediction limit at 50 million load cycles as proposed in NCHRP Report 646 and shown in Equation 2.

$$\text{Lower Prediction Limit} = \hat{y}_o - t_\alpha s \sqrt{1 + \frac{1}{n} + \frac{(x_o - \bar{x})^2}{S_{xx}}} \tag{2}$$

where:
y_o = the one-sided lower 95% prediction interval at the micro-strain level corresponding to 50,000,000 cycles;
t_α = value of t distribution for n-2 degrees of freedom for a significance level of 0.05;
s = standard error of the regression analysis;
n = number of samples;
S_{xx} = sum of squares of the x values;
x_o = log 50,000,000; and
$\bar{x}$ = average of the fatigue life results.

The FEL estimates are also presented in Table B7.4. The RAS mixtures exhibit higher and thus more desirable endurance limits, indicating that RAS may improve the FEL in the mixtures.

Table B7.4. Beam fatigue results

Mix ID	% Binder Replacement	Average Initial Stiffness (Mpa)	K1	K2	R^2	Endurance Limit (Micro-strain)
0% RAS	0	3497	1.43E-13	-5.45	0.987	144
4% RAS	16.3	3090	6.75E-14	-5.68	0.987	182
5% RAS	19.4	3106	1.97E-12	-5.27	0.982	175
6% RAS	22.8	3156	7.07E-14	-5.65	0.967	162

Semi-Circular Bending

The low temperature fracture properties of the mixtures were obtained from SCB tests by following the procedure in "Investigation of Low Temperature Cracking in Asphalt**"** **(**Marasteanu et al., 2007). Testing was conducted at four different low temperatures: -12°C, -18°C, -24°C, and -28°C.

All tests were performed inside an environmental chamber, and liquid nitrogen was used to obtain the required low temperature. The temperature was controlled by the environmental chamber temperature controller and verified using an independent platinum resistive-thermal-device (RTD) thermometer. The load line displacement (LLD) was measured on both faces of the test specimens using a vertically mounted Epsilon extensometer with 38 mm gage length and

±1 mm range. One end was mounted on a button that was permanently fixed on a specially made frame, and the other end was attached to a metal button glued to the sample. The average LLD measurement was used for each specimen. The crack mouth opening displacement (CMOD) was recorded by an Epsilon clip gage with 10 mm gage length and a +2.5 and -1.0 mm range. The clip gage was attached at the bottom of the specimen. A constant CMOD rate of 0.0005mm/s was used and the load and load line displacement (P-u), as well as the load versus LLD curves were plotted. A contact load with a maximum load of 0.3 kN was applied before the actual loading to ensure uniform contact between the loading plate and the specimen. The testing was stopped when the load dropped to 0.5 kN in the post peak region. The load and load line displacement data were used to calculate the fracture toughness and fracture energy (G_f).

The fracture energy (G_f) parameter is presented in Figure B7.5. The laboratory test results were analyzed to evaluate the effect of the various RAS treatments on the fracture energy. MacAnova statistical software package was utilized to perform a statistical analysis. The analysis of variance (ANOVA) was used to examine the differences among the mean response values of the different treatment groups. The significance of the differences was tested at 0.05 level of error. The analysis of variance was conducted with the assumption that the errors in the data are independently normal with constant variance.

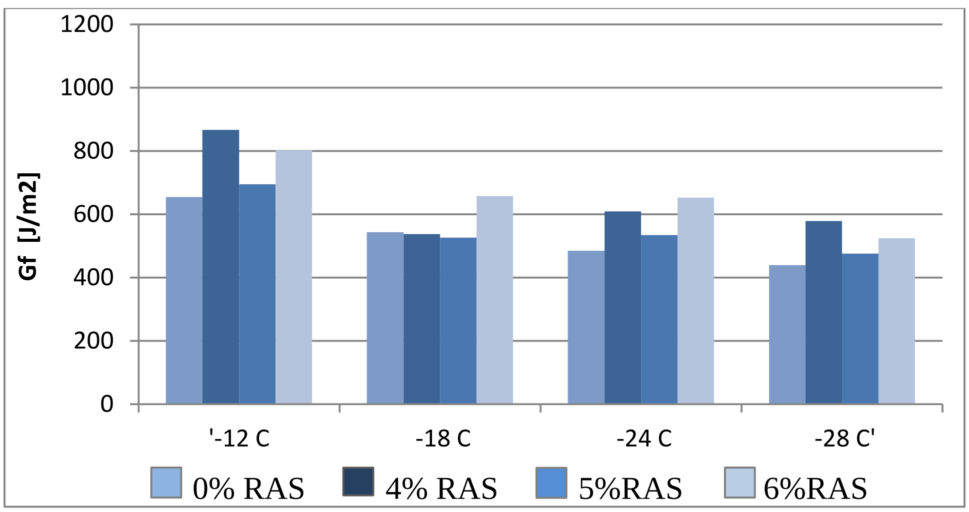


Figure B7.5. Iowa mixture fracture energy (G_f)

The Gf group means of each RAS treatment level was compared using a pair-wise comparison to rank the RAS treatment levels with regard to fracture energy. The outcome is reported in Table B7.5, in which statistically similar RAS treatments are grouped together. Letter A indicates the best performing group of mixtures; letter B the second best, and so on. Groups with the same letter are not statistically different, whereas mixtures with different letters are statistically different.

The mixture with 4% RAS has the highest fracture energy and the mixture with 0% RAS has the lowest fracture energy. The differences between the 4% RAS and 0% RAS are statistically significant. The ranking of the mixtures by fracture energy is almost identical to the ranking of the mixtures by fatigue life, where the RAS also had an effect on reducing the cracking propensity of the mix. These results indicate that small percentages of RAS will either decrease or have no detrimental effect on the cracking performance of asphalt pavements.

Table B7.5. Ranking of mixes by G_f mean value for -12, -18, -24, and -28°C temperatures

Rank	Treatment	Group mean G_f [J/m^2]
A	4% RAS	674
A/B	6% RAS	659
B/C	5% RAS	558
C	0% RAS	531

B8. Field Evaluations

The project team completed three distress surveys for the Iowa demonstration project test sections in December 2010, May 2011, and March 2012. Three 500-foot sections were randomly selected in each of the test sections: 0% RAS, 4% RAS, 5% RAS, and 6% RAS. For each of the sections, two of the surveys were completed in the EB lane and one in the WB lane. The surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* published by the Federal Highway Administration.

No distresses were found in any of the sections in the December 2010 survey. Over the next two years, the distress surveys found a progression of transverse cracking. These cracks are suspected to be caused by reflective cracking from differential movement of the concrete pavement below the overlay. Since no pre-condition survey was available, the project team was unable to ensure that the different survey sections contained similar levels of distress before the overlay.

After one year, the 4% RAS test sections contained the least amount of linear length of transverse cracking per 500 feet as shown in Figure B8.1. After two years, the 0% RAS sections contained the greatest amount of transverse cracking, followed by the 5% RAS and 6% RAS test sections respectively.

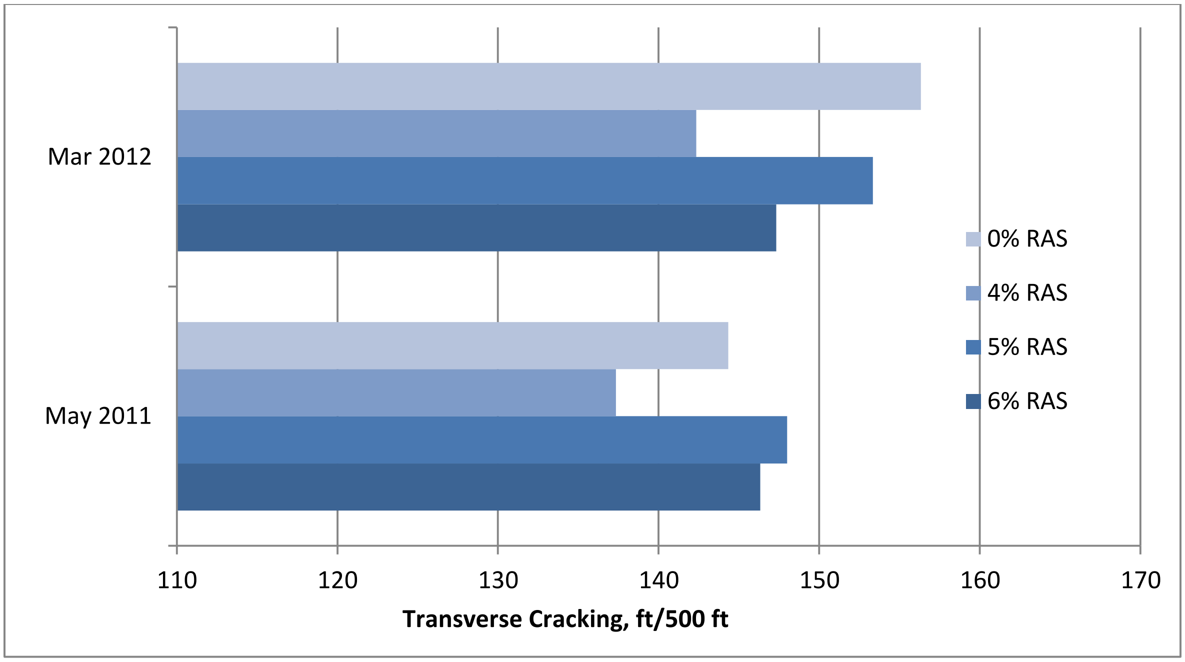


Figure B8.1. Iowa pavement evaluation

The amount of transverse cracking in each test section correlates well with the SCB fracture energy measured for each mixture, as shown in Figure B8.2. The 4% RAS showed the least amount of cracking in the field and had the highest fracture energy, whereas the 0% RAS showed the greatest amount of cracking in the field and had the lowest fracture energy. Both the laboratory fracture energy data and the field surveys indicate that adding RAS to the Iowa DOT mix design increases its ability to resist cracking.

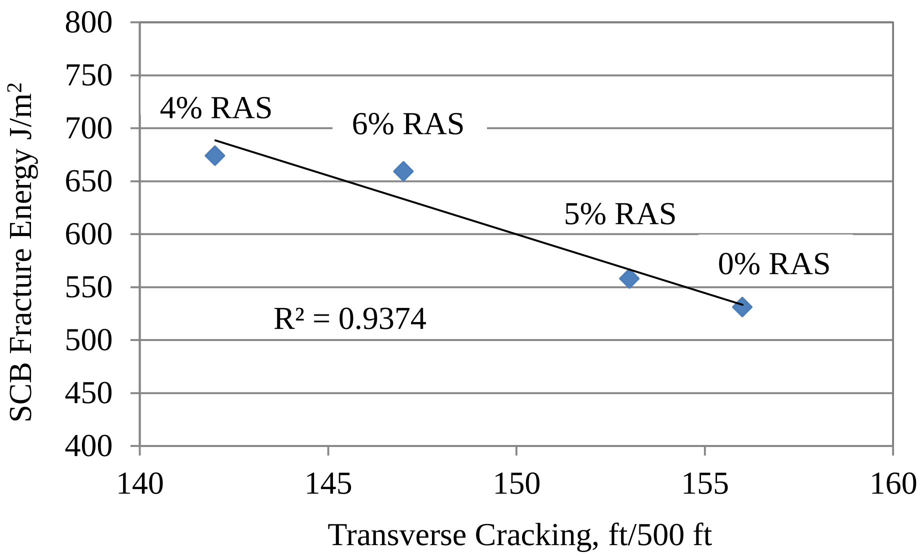


Figure B8.2. Transverse cracking versus SCB fracture energy

While measuring the length of transverse cracking in the pavements, the severity level of the cracks was also measured. Following the guidelines of the *Distress Identification Manual*, transverse cracks were categorized into three levels: low severity (crack widths ≤ 0.25 in), moderate severity (crack widths 0.25 in ≥ 0.75 in), and high severity (crack widths > 0.75 in). The severity levels of the transverse cracks measured in March 2012 are presented in Figure B8.3. Examples of the transverse cracks (TC) measured in the pavement test sections are presented in Figures B8.4 and B8.5.

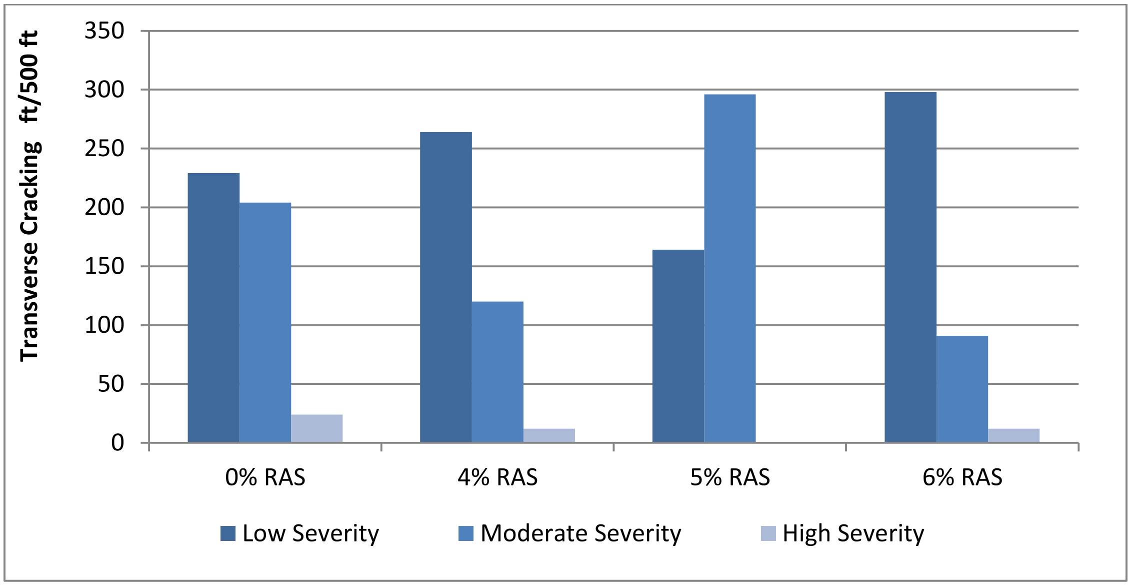


Figure B8.3. Severity level of transverse cracking (March 2012)

Figure B8.4. Low severity transverse crack (HMA 4% RAS)

Figure B8.5. High severity transverse crack (HMA 0% RAS)

In the 0% RAS section, longitudinal reflective cracking was observed near the white lane striping, as shown in Figure B8.5. This is the location of the edge of the concrete pavement slabs under the HMA layer. In the March 2012 survey, 165 feet per 500 feet of this type of cracking was identified in the 0% RAS section. No longitudinal cracking was observed in the RAS sections.

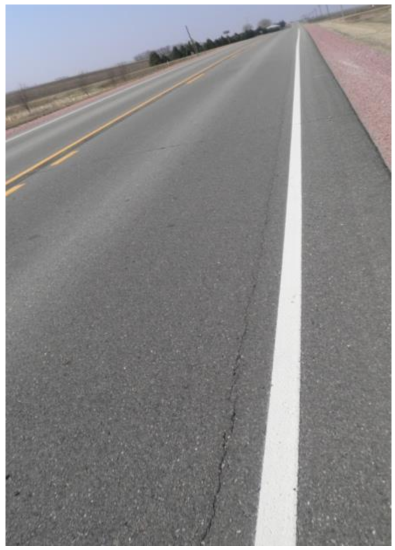


Figure B8.6. Longitudinal reflective cracking (HMA 0% RAS)

Small amounts of low severity raveling were also documented in the RAS test sections. Power-Point Presentations of the distress surveys by 500-foot sections are available for viewing on the TPF-5(213) website.

B9. Conclusions

An Iowa DOT demonstration project was conducted as part of Transportation Pooled Fund 5-213 to evaluate the effects of adding different percentages of post-consumer RAS in HMA. Four asphalt mixes were evaluated, a 0% RAS mix, a 4% RAS mix, a 5% RAS mix, and a 6% RAS mix. All four mixes were produced with the similar aggregate blend gradations, air voids, VMA, and total asphalt content. Field mixes of each pavement were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and recovery with subsequent binder characterization. The results of the study are summarized below:

- Observations from the demonstration project show the contractor successfully produced and constructed the RAS pavements while meeting Iowa DOT's quality assurance requirements.
- Nearly all the binder in the RAS was effective in reducing the laboratory air voids for all three RAS mixtures. Laboratory extraction of a RAS sample measured 21.7 percent asphalt in the RAS, and during production the RAS contributed approximately 20.5 percent asphalt to the HMA. The optimal asphalt content of the 0% RAS mixture and the RAS mixtures was approximately the same at 5.5 percent at 4 percent air voids.

- The performance grade of the total binder in the asphalt mixtures increased with the addition of RAS. The asphalt extracted from the 0% RAS mixture had a continuous PG of 73.0-19.7. Adding 4% RAS raised the PG to 75.8-19.1. At 5% RAS the continuous PG was 81.3-16.8, and at 6% RAS the PG increased again to 86.1-14.7.
- Adding RAS to the mix designs increased the dynamic modulus at high temperatures for improved rutting resistance. This was likely due to the stiffer RAS binder contained in the RAS. At intermediate temperatures, the dynamic modulus initially increased with 4% RAS, but then decreased with 5% RAS and 6% RAS.
- In the flow number test, the 0% RAS mixture had a relatively low flow number of 711 making it more susceptible to permanent deformation. By increasing the RAS content of the mixtures, the permanent deformation resistance also increased as measured by a larger flow number. The 4% RAS mixture had a flow number of 2425, the 5% RAS mixture had a flow number of 6092, and the 6% RAS mixture had a flow number of 5899.
- The four-point bending beam results showed that fatigue life of the asphalt mixture increased with the addition of RAS in a controlled strain mode of loading, the condition of thin lift pavements. This could be from fibers in the RAS providing additional ductility to the mixtures.
- The SCB test was conducted to measure the low temperature cracking susceptibility of the mixtures by measuring their fracture energy at -12°C, -18°C, -24°C, and -28°C. The mixture with 4% RAS had the highest fracture energy and the mixture with 0% RAS had the lowest fracture energy. These results indicate that small percentages of RAS will either decrease or have no detrimental effect on the cracking performance of asphalt pavements.
- Field condition surveys conducted one and two years after the demonstration project revealed the pavement section without RAS was the most susceptible to reflective cracking. The 4% RAS pavement sections displayed the least amount of reflective cracking, followed by the 6% RAS and 5% RAS pavement sections respectively. The amount of transverse cracking in each test section correlates well with the fracture energy measured for each mixture, since the ranking of the mixtures by the amount of measured transverse cracking is the same as the ranking of mixtures by their fracture energy. Both the laboratory fracture energy data and the field surveys indicate that adding RAS to the Iowa DOT mix design increases its ability to resist cracking.

B10. Iowa DOT Demonstration Project Acknowledgments

The researchers gratefully acknowledge the support of Scott Schram at the Iowa DOT. The research work was sponsored by Federal Highway Administration (FHWA) TPF-5(213) and the Transportation Pooled Fund (TPF) partners: Missouri (lead agency), California, Colorado, Illinois, Indiana, Iowa, Minnesota, and Wisconsin DOTs.

APPENDIX C. REPORT FOR THE MINNESOTA DEPARTMENT OF TRANSPORTATION SPONSORED DEMONSTRATION PROJECT

C1. Introduction

This report presents a summary of the results obtained from the field demonstration project sponsored by the Minnesota Department of Transportation (MnDOT) as part of Transportation Pooled Fund (TPF) 5-213 *Performance of Recycled Asphalt Shingles in Hot Mix Asphalt*. TPF 5-213 is a partnership of several state agencies with the goal of researching the effects of recycled asphalt shingles (RAS) on the performance of asphalt applications. As part of the Pooled Fund research program, multiple field demonstration projects were conducted to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements that include RAS.

Each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated different aspects of RAS mixes. For MnDOT's demonstration project, MnDOT selected in-service pavement sections at their MnROAD Cold Weather Road Research Facility pavement test track. The pavement sections were constructed in 2008 and included shoulder mixes and transition traffic lanes that used post-manufactured and post-consumer RAS. The pavement sections were selected to compare the performance of hot mix asphalt (HMA) containing post-manufactured RAS to HMA containing post-consumer RAS and to evaluate their performance to an asphalt mixture using recycled asphalt pavement (RAP) with no RAS.

C2. Experimental Plan

To evaluate the performance of HMA with post-manufactured RAS versus post-consumer RAS, MnDOT designed an experimental plan to address the following questions:

- Is there a difference in pavement performance when utilizing post-manufactured versus post-consumer RAS?
- When utilizing 5% RAS in HMA is there a difference in pavement performance when utilizing 30% RAP, specifically low temperature and reflective cracking?
- What are the differences in the asphalt contents of post-consumer and. post-manufactured RAS?

The experimental plan is presented in Table C2.1. In-service pavement sections selected from the MnRoads test track contained the following type of asphalt mixes: a mix with 30% RAP, a mix with 5% post-consumer RAS, and a mix with 5% post-manufactured RAS.

Table C2.1. Experimental plan

Mix ID	% RAS	% RAP	RAS Source
RAP	0	30	-
RAS	5	0	Post-Consumer
RAS	5	0	Post-Manufactured

During production of the asphalt mixtures in 2008, MnDOT collected samples of each asphalt mixture. These samples were sent to Iowa State University for laboratory testing in the fall of 2010. A portion of the samples were sent to the University of Minnesota and the Minnesota Department of Transportation (MnDOT) for Semi-Circular Bend (SCB) testing and binder extraction and recovery, respectively. The laboratory testing plan is presented in Table C2.2.

Table C2.2. Laboratory testing plan

	Laboratory Test	Iowa State University	University of Minnesota	Minnesota DOT
Processed Shingles	Binder Extraction			X
	High Temperature PG			X
	Gradation (Before Extraction)	X		
	Gradation (After Extraction)	X		
Mixture	Binder Extraction			X
	Binder PG Characterization	X		
	Gradation	X		
	Dynamic Modulus	X		
	Flow Number	X		
	Beam Fatigue	X		
	Semi-Circular Bending (SCB)		X	

The asphalt was recovered from the RAS and asphalt mixtures following AASHTO T164 Method A (Centrifuge Method) by using a blend of toluene and ethanol as the extraction solvent. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. At Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO R29 “Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder.”

Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution, and a washed gradation was conducted after extraction to evaluate the size distribution of the fine aggregates in the RAS product.

Performance testing was completed on the field produced asphalt mixtures at low, intermediate, and high temperatures. Dynamic Modulus and Flow Number were conducted at high temperatures to evaluate the modulus and rutting resistance of asphalt mixtures. The durability of the mixtures at intermediate temperatures was evaluated using the dynamic modulus and four-point beam fatigue test. The SCB test conducted at the University of Minnesota evaluated the fracture properties of the mixtures at low temperatures.

Starting in 2010, field evaluations were conducted on each test section to assess the field performance of the pavement concerning cracking, rutting, and raveling at two, three and four years after paving.

C3. Project Location

The MnROAD research center is located in Albertville, Minnesota, approximately 40 miles northwest of the Twin Cities in Wright County. The test sections selected for the Pooled Fund Study are located on the Interstate 94 mainline portion of the MnROAD facility which is a 3.5-mile, 2-lane road segment of the interstate that carries "live" traffic.

The test sections for the project include the westbound driving shoulders of MnRoads cell numbers 5, 6, 13-23, passing shoulder of cell number 20 and the east and west transitions which carry traffic from the interstate mainline to the test track mainline. The project limits are identified below in Figure C3.1.

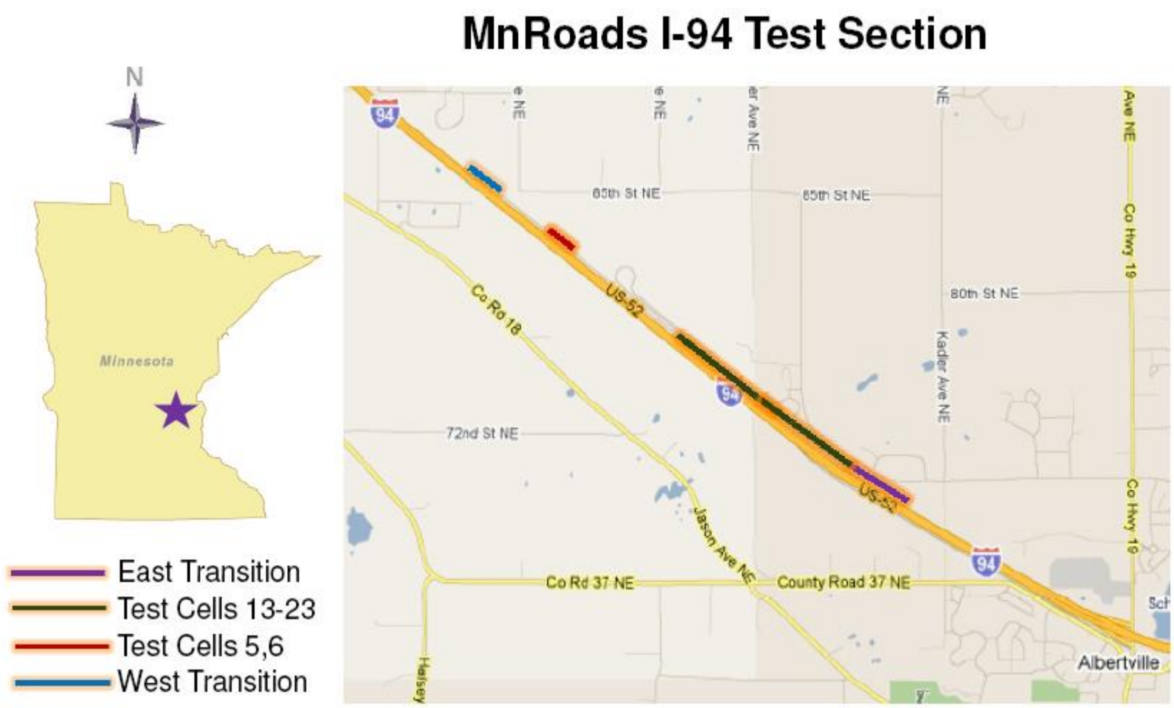


Figure C3.1. Project location

C4. Project Description

The pavement test sections at the MnROAD facility containing the RAS and RAP mixes were constructed by Hardrives, Inc. (Hardrives) in September and October of 2008. The driving shoulders of Cells 15-23 and the East transition area were paved with the 5% post-consumer RAS mix on September 19 and October 1-2, 2008 respectively. The driving shoulder of Cells 5, 6, 13, and 14 and the West transition area were paved with the 5% post-manufactured RAS mix on September 30, 2008. The passing shoulder of Cell 20 was paved with the 30% RAP mix on September 10, 2008. MnROAD cells are approximately 500 feet long. Weather conditions during all three paving days were fair with ambient temperatures in the mid 60's (°F).

Cell 5 westbound shoulder was a 3-inch bituminous overlay above a granular interlayer placed above existing bituminous shoulders. Several sensor instrumentation conduits were cut through the existing shoulder prior to paving, and wick drains were run from the mainline through the granular interlayer. Cell 15 westbound shoulder was a 3-inch bituminous overlay above an existing bituminous pavement. Cells 6 and 15-23 westbound shoulders (as well as the 30% RAP mix on the eastbound shoulder of Cell 20) were newly constructed shoulders with a 3-inch HMA layer over granular material. A summary and plan view of the MnROADS test sections is shown in Table C4.1 and Figure C4.1 respectively.

Table C4.1. Summary of MnROAD I-94 test sections

Mix Type	Location	Pavement Structure
5% Post-manufactured RAS	West tranisition	Newly constucted HMA
	Cell 5 driving shoulder	3" HMA over granular with wick drains over existing HMA
	Cell 6, 13, 14 driving shoulders	3" HMA over granular
5% Post-consumer RAS	East tranisition	Newly constucted HMA
	Cell 15 driving shoulder	3" HMA over existing cracked HMA
	Cell 16-23 driving shoulders	3" HMA over granular material
30% RAP	Cell 20 passing shoulder	Wearing course over HMA base course over granular material

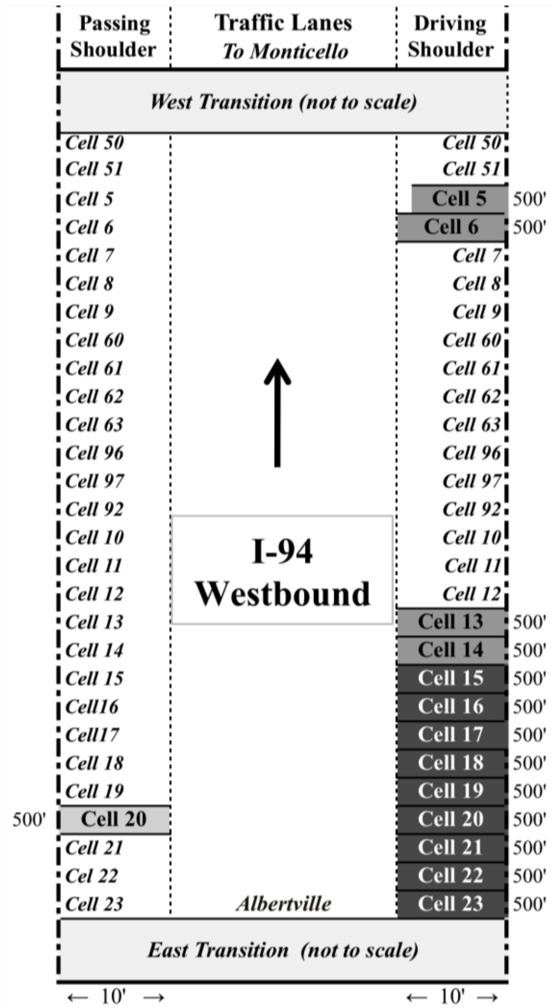


Figure C4.1. Plan view of MnROAD I-94 test sections

Hardrives used a single drum portable plant to produce the HMA. The RAS and RAP passed over a gator recycling breaker prior to being added in the recycled product column on the drum (Figures C4.2 and C4.3).

Figure C4.2. Portable single drum plant

Figure C4.3. RAP gator recycling breaker

A total of approximately 2,089 tons of HMA was placed for the surface course shoulder test sections. The test sections included a total of approximately 67 tons of post-consumer RAS, 26 tons of post-manufactured RAS and 36 tons of RAP. Tonnages for the RAS, RAP, and total HMA for each shoulder test section are summarized in Table C4.2 below.

Table C4.2. Project tonnages for driving and passing test cell shoulders

Material	Cell 5 (Ton)	Cell 6 (Ton)	Cell 13 (Ton)	Cell 14 (Ton)	Cell 15 (Ton)	Cell 16 (Ton)	Cell 17 (Ton)	Cell 18 (Ton)	Cell 19 (Ton)	Cell 20 (Ton)	Cell 21 (Ton)	Cell 22 (Ton)	Cell 23 (Ton)
Post-Cons. RAS	---	---	---	---	7.4	7.4	7.2	7.3	7.2	7.9	7.5	7.45	7.3
Post-Manuf. RAS	6.0	6.9	---	7.1	---	---	---	---	---	---	---	---	---
RAP	---	---	---	---	---	---	---	---	---	35.7	---	---	---
Total HMA	119	138	120	142	147	147	144	146	144	297	150	149	146

For the east transition, approximately 3,097 tons of HMA was placed for the surface course with approximately 155 tons of post-consumer RAS. For the West transition, approximately 981 tons of HMA was placed for the surface course with approximately 49 tons of post-manufactured RAS. Tonnages for the RAS and total HMA are summarized in Table C4.3 below.

Table C4.3. Project tonnages for the East and West transitions (driving lanes and shoulders)

Material	**East Transition (Tons)**	**West Transition (Tons)**
PC RAS	155	---
PM RAS	---	49
Total HMA	3,097	981

According to the 2008 MnROAD Phase II Construction Report, driving shoulder Cells 15-23 and the East transition were paved with no obstacles to the paver. The paving of the driving shoulders of Cells 5, 6, 13, and 14 were challenging due to the placement of LVDT boxes, maturity meter sensors and other instrumentation present. Handwork and protective measures were required to complete the paving. A single paving pass was used to place the passing shoulder of RAP Cell 20. The cells are designed to be in-place for five years. The mainline section was opened to traffic in early February 2009.

C5. Shingle Processing

The RAS was delivered and stockpiled at the MnROAD facility site prior to paving. The RAS was ground using an industrial grinder. RAS stockpiles were uncovered and open to all weather conditions. Pictures of the RAS stockpiles are presented in Figures C5.1 and C5.2. The RAP used in the project came from the mainline and shoulder millings removed in May 2008.

Figure C5.1. Post-consumer RAS stockpile

Figure C5.2. Post-manufactured RAS stockpile

The gradation and asphalt contents of the RAS and RAP products before extraction are presented in Table C5.1.

Table C5.1. RAS and RAP asphalt contents and gradations (% passing)

Sieve Size (US)	Sieve Size (mm)	Post-Manufactured RAS Before Extraction	Post-Manufactured RAS After Extraction	Post-Consumer RAS Before Extraction	Post-Consumer RAS After Extraction	RAP After Extraction
3/4"	19	100	100	100	100	100
1/2"	12.5	100	100	100	100	96
3/8"	9.5	95	100	99	100	89
#4	4.75	70	99	85	100	66
#8	2.36	56	97	73	99	49
#16	1.18	32	80	49	85	38
#30	0.6	12	58	24	65	29
#50	0.3	4	40	10	49	22
#100	0.15	1	28	3	35	16
#200	0.075	0.4	22.0	0.5	24.1	11.9
Asphalt Content (%)*		17.1		23.0		5.9

The post-consumer RAS contained 23.0% asphalt while the post-manufactured RAS contained 17.1% asphalt. The larger percentage of asphalt in the post-consumer RAS is likely due to presence of older shingles containing a cellulosic backing rather than a fiberglass backing which most newly manufactured shingles have today. A cellulosic backing will absorb more asphalt than the fiberglass backing, thus requiring more asphalt.

C6. Asphalt Mix Design and Prodution Results

Three HMA mix designs were prepared for the demonstration project. The two RAS mix designs followed MnDOT's classification for a SPWEB440(R) design, a 12.5mm wearing course for 3 – 10 million equivalent single axel loads (ESAL's) over a 20-year design period. The RAP mix design followed MnDOT's classification for a SPWEB440(B), also a 12.5mm wearing course for 3 – 10 million EASLs. The mix design gradations obtained from laboratory testing of the sampled asphalt mixtures are presented in Figure C6.1. All three mixes have a different gradation.

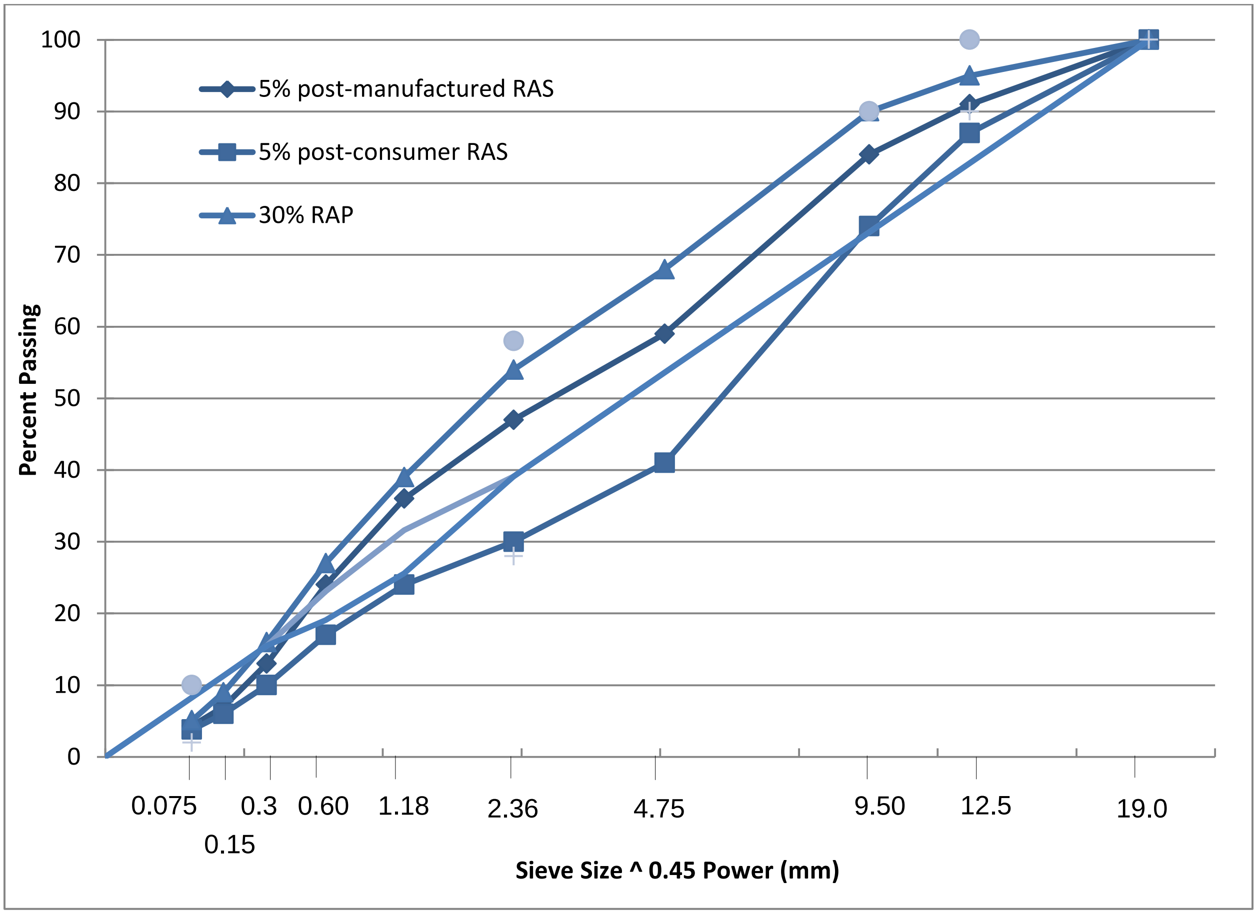


Figure C6.1. Asphalt gradations

The asphalt mix design properties are presented in Table C6.2. The target voids for the mixes was 4%. A PG 58-28 asphalt binder was used for the mix designs. When replacing 30% RAP with 5% RAS, the percent binder replacement of the mixtures decreased from 33.3% to 18.8% for the post-manufactured RAS mix and 26.0% for the post-consumer RAS mix.

Table C6.2. Asphalt mix design properties

Mix Property	Post-manufactured RAS	Post-consumer RAS	RAP
% RAS	5	5	0
% RAP	0	0	30
% Total AC	4.8	5.0	5.3
% Virgin AC	3.9	3.7	3.5
% Binder Replacement	18.8	26.0	33.3
Design Gyrations	90	90	90
NMAS (mm)	12.5	12.5	12.5
Virgin PG Grade	58-28	58-28	58-28
% Voids	4.0	4.0	4.0
% VMA	14.0	14.0	14.5

Production control and core density results are presented in Table C6.3. Laboratory test results are based on the average test results obtained by Hardrives during production. The density results are based on the average core measurements by MnDOT. The laboratory results show the RAS mixes were close to the mix design target values. There does not appear to be a large difference between the post-manufactured and post-consumer RAS mixes in their mix constructability. However, greater pavement densities were achieved with the post-consumer RAS mixture than the post-manufactured RAS mixture.

Table C6.3. Mix and construction quality control results(1)

Mix Type	Location	Paving Date	AC (%)	Voids (%)	VMA (%)	Mainline Density (2)	Long. joint Density(2)
Post-manufactured RAS	West tranisition	Sept. 30 2008	4.9	3.7	13.9	91.2%	90.9%
Post-manufactured RAS	Cell 5, 6, 13, 14 shoulders	Oct. 30 2008	-	-	-	91.4%	90.3%
Post-consumer RAS	East tranisition	Oct. 1-2 2008	5.2	4.1	15.1	92.3%	93.0%
Post-consumer RAS	Cell 15-23 shoulders	Sept. 19 2008	4.8	4.7	14.3	-	-
30% RAP wear	Cell 20 shoulder	Sept. 10 2008	5.0	4.7	14.7	92.5%	91.6%

(1) Average quality control test results obtained from Hardrives during production
(2) Average of core density results

C7. Laboratory Test Results

Binder Testing

Performance grade (PG) testing of the extracted binders was conducted to obtain their high, low, and intermediate PG temperatures as shown in Table C7.1. The high temperature performance grade of the RAS binders is higher than traditional paving grade binders. This is expected since the binder in roofing shingles is produced with an air-blowing process which oxidizes the asphalt. Additionally, the post-consumer RAS binder at 122.5°C is noticeably stiffer than the post-manufactured RAS binder at 109.1°C. It is stiffer because post-consumer RAS has been processed from in-service roofing shingles that have experienced at least several years of aging, while the post-manufactured RAS comes from waste produced during shingle manufacturing.

Because the RAS mixtures are heated to high temperatures and placed in a centrifuge at high speeds during the recovery process, the RAS and virgin asphalt are assumed to be fully blended. A 58-28 virgin binder was used for the project. When 5% post-consumer RAS was used in the mix design, the continuous performance grade of the blended asphalt was tested as a 71.1-21.2. When 5% post-manufactured RAS was used in the mix design, the continuous performance grade of the blended asphalt essentially remained the same as the 5% post-consumer RAS design at 71.3-21.7. While it is expected that the stiffer RAS binder would produce a final binder blend with a high PG, it appears the 13.4°C high PG difference between the post-consumer and post-manufactured RAS binder does not make a large difference on the final blend's PG when approximately 20 percent of the virgin binder is replaced with the RAS binder.

The binder extracted from the RAP contained a continuous PG of 73.5-10.8. The asphalt content of the RAP was 5.9%. When 30% RAP was utilized in the mix design, 33.3% of the base binder was replaced with the RAP binder. The continuous PG of the blended binder was 68.8-22.7, which is very similar to the binders in the RAS mixes. For this demonstration project, utilizing 5% RAS in the mix design produced comparable mix performance grades when utilizing 30% RAP with a 5.9% asphalt content in the mix design.

Table C7.1. Performance grade of extracted binders

Material Identification	High PG Temp, °C	Intermediate PG Temp, °C	Low PG Temp, °C	Performance Grade
Post-Manufactured RAS	109.1	-	-	-
Post-Consumer RAS	122.5	-	-	-
RAP	73.5	31.7	-10.8	70-10
5% Post-Manufactured RAS	71.3	18.5	-21.7	70-16
5% Post-Consumer RAS	71.1	19.7	-21.2	70-16
30% RAP Mix	68.8	20.6	-22.7	64-22

Dynamic Modulus

The dynamic modulus (|E*|) is a key material property that determines the stress-strain relationship of an asphalt mixture under continuous sinusoidal loading. A higher dynamic modulus indicates lower strains will result in a pavement structure when the asphalt mixture is stressed from repeated traffic loading. The mechanistic-empirical pavement design guide (MEPDG) uses |E*| as the stiffness parameter to calculate an asphalt pavements strains and displacements.

The test was conducted following AASHTO TP62 using five replicate samples at 7 ± 0.5% air voids with 150 mm in height and 100 mm in diameter. Samples were tested by applying a continuous sinusoidal load at 9 different frequencies (0.1, 0.3, 0.5, 1, 3, 5, 10, 20, and 25 Hz) and three different temperatures (4, 21, and 37°C). Sample loading was adjusted to produce strains between 50 and 150 μstrain in the sample.

Master curves were constructed at a reference temperature of 21°C and plotted on a log-log scale for a general comparison as presented in Figure C7.1. The 30% RAP mix has lower dynamic modulus values at low and intermediate frequency ranges than both of the RAS mixes.

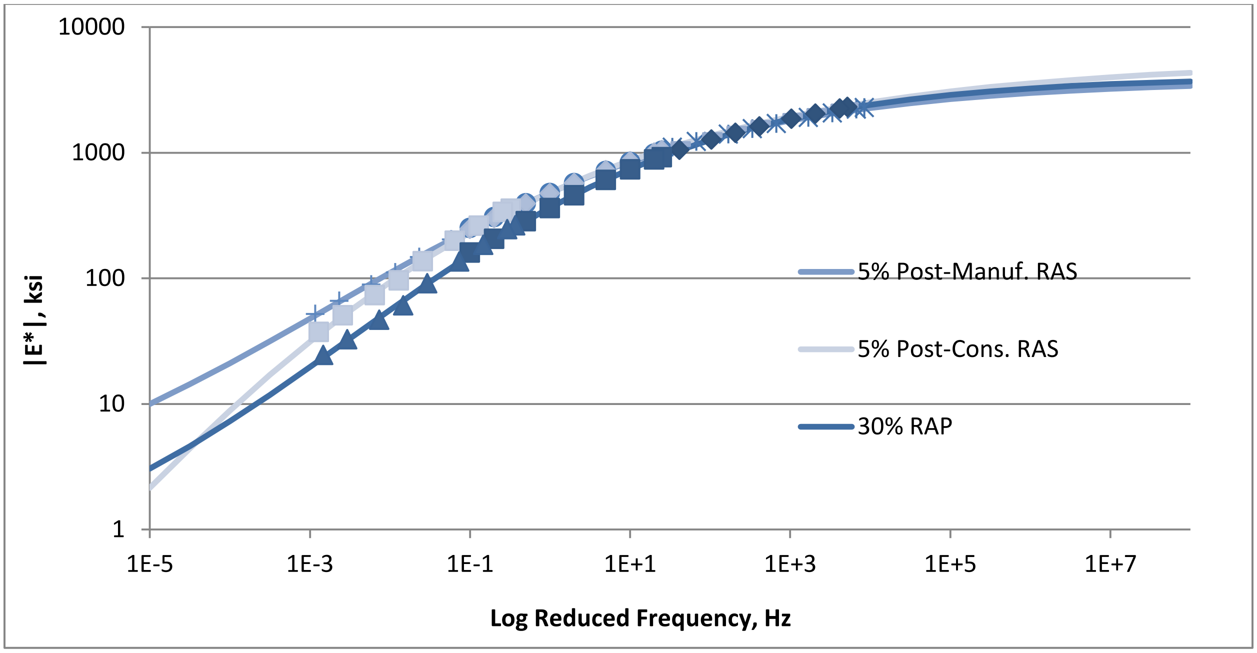


Figure C7.1. Comparison of master curves for MnDOT mixes

The plot in Figure C7.2 presents the mean dynamic modulus at 21°C and 37°C at specific frequencies for a more direct comparison. The dynamic modulus at 0.1 Hz at 37°C is closely related to the permanent deformation resistance of the mix, with a higher value indicating less strain is accumulated in the mix than a mix with a lower value. The dynamic modulus at 5 Hz at 21°C is related to the fatigue cracking resistance of the mix, with a higher value indicating the mix is stiffer and will therefore resist stresses in thick pavements better than a mix with a lower value. However, low modulus values at this temperature are considered desirable in thin asphalt pavements (less than 4”) for fatigue cracking resistance. Mixtures with lower stiffness can deform more easily without building up large stresses.

Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. The average values show that utilizing 5% RAS created a stiffer mix at both frequency/temperature levels than utilizing 30% RAP. However, there were no statistical differences at a 95% confidence level among the three mixtures.

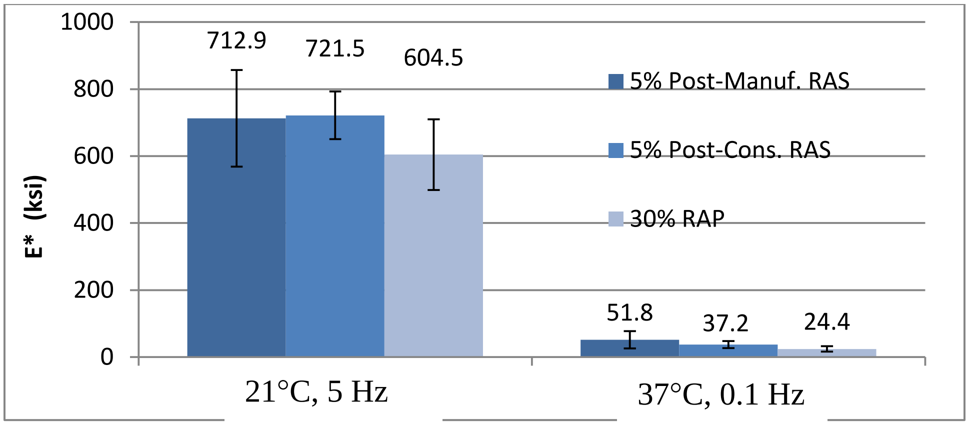


Figure C7.2. Dynamic modulus comparison at 21°C, 5 Hz and 37°C, 0.1 Hz

Flow Number

The flow number test measures the permanent deformation resistance of asphalt mixtures by applying a repeated dynamic load to a sample for up to several thousand load cycles. The flow number is defined as the number of load cycles an asphalt mixture can tolerate until it flows. Cumulative permanent deformation in the sample is plotted versus load cycles. The flow number is reached at the onset of tertiary flow.

Tests were conducted following procedures used in NCHRP Report 465. Samples used in the dynamic modulus test were used for the flow number test since the dynamic modulus test is nondestructive. The samples were placed in a hydraulically loaded universal testing machine, unconfined, with a testing temperature of 37°C to simulate the climactic conditions that cause pavement to be susceptible to rutting. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 sec followed by 0.9 sec of dwell time. The loading cycle was repeated for a total of 10,000 load cycles. Three LVDTs were attached to each sample during the test to measure the cumulative strains.

Test results are presented in Figure C7.3. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. If the error bars of two mixtures do not overlap, then the difference of the two mixtures can be considered statistically significant at the 5% level.

The 30% RAP mix is more susceptible to permanent deformation since its flow number of 767 is lower than the RAS mixes. The flow number of the post-consumer RAS mix at 2497 was also larger than the flow number of the post-manufactured RAS mix at 1705, indicating greater resistance to permanent deformation. While the two RAS mixes shared similar blended asphalt performance grades, the performance difference could be a result of different gradations in the mix designs. The post-consumer RAS mix contained a coarse gradation, and the post-manufactured RAS mix contained a fine gradation.

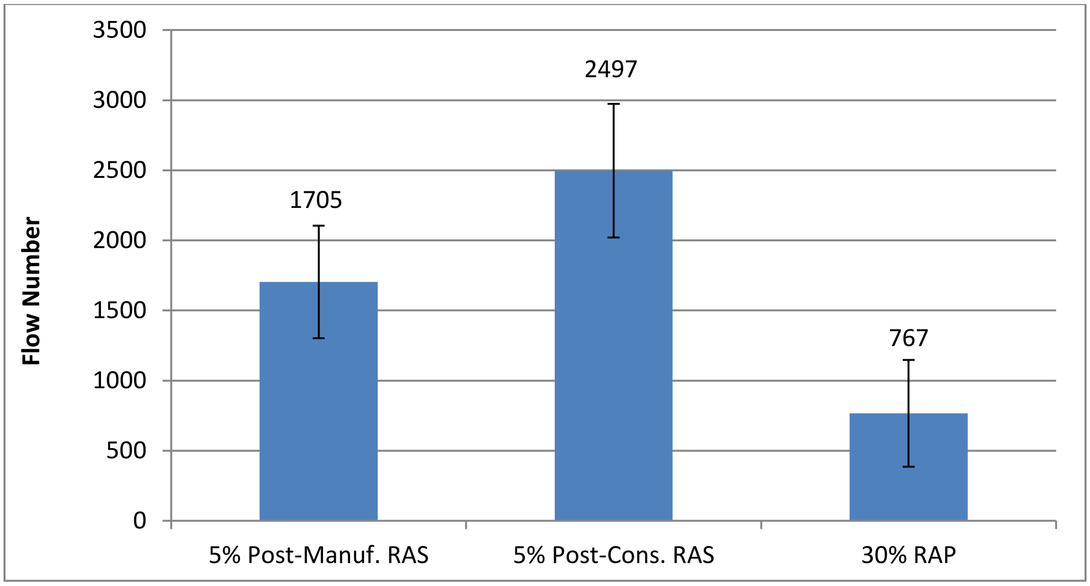


Figure C7.3. Flow number test results

Beam Fatigue

Fatigue cracking is the major cracking distress in asphalt pavements caused by repeated heavy traffic loads. Pavements that have a higher resistance to tensile strains that develop at the bottom of an asphalt layer due to repeated traffic will have a greater resistance to fatigue cracking. The four-point beam fatigue test following AASHTO T321 was conducted to evaluate the load associated cracking resistance of the mixtures.

The mixes were compacted in a linear kneading compactor to create slabs with 7% air voids. The slabs were saw-cut into beams with dimensions 15 inches in length, 2.5 inches in width, and 2 inches in height. The beams were tested in a strain controlled mode of loading at 20°C with haversine wave pulses applied to the beam at 10 Hz. Six beams were tested, each at a different strain level (375, 450, 525, 650, 800, and 1000 µstrain), until the flexural stiffness of the beam was reduced to 50% of the initial stiffness. A log-log regression was performed between strain and the number of cycles to failure (N_f). The relationship between strain and N_f can be modeled using the power law relationship as presented in Equation 1.

$$N_f = K1\left(\frac{1}{\varepsilon_o}\right)^{K2} \tag{1}$$

where: N_f = cycles to failure; ε_o = flexural strain; and K1 and K2 = regression constants.

The fatigue curves from beam fatigue test results are presented in Figure C7.4 with the fatigue model coefficients in Table C7.4.

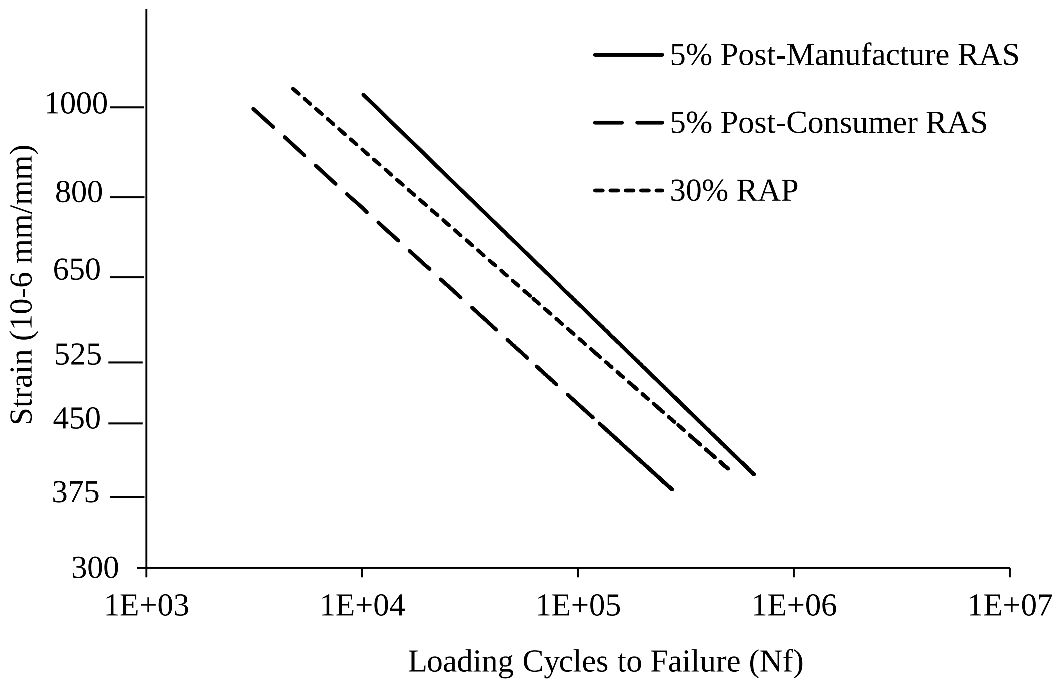


Figure C7.4. ε-N fatigue curves

The two different mixtures with RAS are compared to the mixture containing 30 percent RAP. In the controlled-strain mode of loading, both RAS mixes exhibit longer fatigue lives than the RAP mix. These results are counterintuitive when considering the RAS binder is substantially stiffer than the RAP binder. However, the RAS contains fibers, as a result of the shingle grinding process, which may be improving the fatigue performance of the RAS mixes by enhancing their ductile properties. The post-consumer RAS mixture has a longer fatigue life at higher strain levels than the post-manufacturer RAS mixture, but both mixes have similar fatigue lives at lower strain levels as indicated by the fatigue endurance limit.

The fatigue endurance limit (FEL) of each mixture was also predicted. If tensile strains are low enough in a pavement structure, the pavement has the ability to heal and therefore no damage cumulates over an indefinite number of load cycles. The level of this strain is referred to as the

FEL. The FEL of each mixture was estimated using the lower 95% prediction limit at 50 million load cycles as proposed in NCHRP Report 646 and shown in Equation 2.

$$\text{Lower Prediction Limit} = \hat{y}_o - t_\alpha s \sqrt{1 + \frac{1}{n} + \frac{(x_o - \bar{x})^2}{S_{xx}}} \tag{2}$$

where:
y_o = the one-sided lower 95% prediction interval at the micro-strain level corresponding to 50,000,000 cycles;
t_α = value of t distribution for n-2 degrees of freedom for a significance level of 0.05;
s = standard error of the regression analysis;
n = number of samples;
S_{xx} = sum of squares of the x values;
x_o = log 50,000,000; and
$\bar{x}$ = average of the fatigue life results.

The FEL estimates are presented in Table C7.2. The RAS mixes exhibit higher, and thus more desirable, endurance limits than the RAP mix indicating that more damage will accumulate in the RAP mix than the RAS mixes at low strain levels.

Table C7.2. Beam fatigue results

Mix ID	% Binder Replacement	K1	K2	R^2	Endurance Limit (Micro-strain)
5% Post-Manufactured RAS	18.8	9.19E-12	-4.90	0.994	131
5% Post-Consumer RAS	26.0	2.22E-09	-4.19	0.996	123
30% RAP	33.3	6.66E-11	-4.51	0.982	89

Semi-Circular Bending

The low temperature fracture properties of the mixtures were obtained from SCB tests by following the procedure in “Investigation of Low Temperature Cracking in Asphalt” (Marasteanu et al., 2007). Testing was conducted at four different low temperatures: -12°C, -18°C, -24°C, and -28°C.

All tests were performed inside an environmental chamber, and liquid nitrogen was used to obtain the required low temperature. The temperature was controlled by the environmental chamber temperature controller and verified using an independent platinum resistive-thermal-device (RTD) thermometer. The load line displacement (LLD) was measured on both faces of the test specimens using a vertically mounted Epsilon extensometer with 38 mm gage length and ±1 mm range. One end was mounted on a button that was permanently fixed on a specially made frame, and the other end was attached to a metal button glued to the sample. The average LLD measurement was used for each specimen. The crack mouth opening displacement (CMOD) was

recorded by an Epsilon clip gage with 10 mm gage length and a +2.5 and -1.0 mm range. The clip gage was attached at the bottom of the specimen. A constant CMOD rate of 0.0005mm/s was used and the load and load line displacement (P-u), as well as the load versus LLD curves were plotted. A contact load with a maximum load of 0.3 kN was applied before the actual loading to ensure uniform contact between the loading plate and the specimen. The testing was stopped when the load dropped to 0.5 kN in the post peak region. The load and load line displacement data were used to calculate the fracture toughness and fracture energy (G_f).

The fracture energy (G_f) parameter is presented in Figure C7.5. The laboratory test results were analyzed to evaluate the effect of the various RAS treatments on the fracture energy. MacAnova statistical software package was utilized to perform a statistical analysis. The analysis of variance (ANOVA) was used to examine the differences among the mean response values of the different treatment groups. The significance of the differences was tested at 0.05 level of error. The analysis of variance was conducted with the assumption that the errors in the data are independently normal with constant variance.

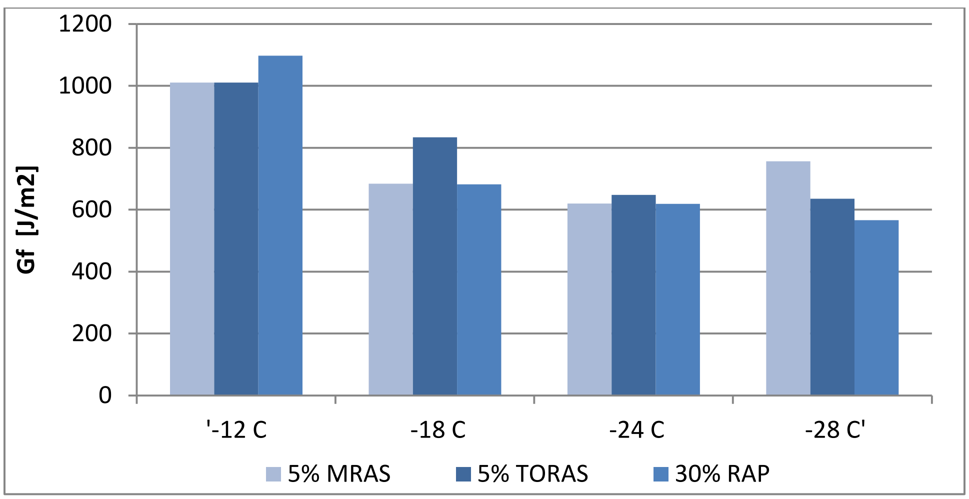


Figure C7.5. Fracture energy (G_f) of MnDOT mixes

The Gf group means of each RAS treatment levels was compared using a pair-wise comparison to rank the RAS treatment levels with regard to fracture energy. The outcome is reported in Table C7.3, in which statistically similar RAS treatments are grouped together. Letter A indicates the best performing group of mixtures; letter B the second best, and so on. Groups with the same letter are not statistically different, whereas mixtures with different letters are statistically different.

The results of the SCB test indicate similar low temperature cracking resistance of the RAS and RAP mixes. The 30% RAP mix has an average fracture energy of 741 J/m^2. When 5% RAS was used in the mix design in place of 30% RAP, the fracture energy increased to 768 J/m^2 for the post-manufactured RAS mix and **777** J/m^2 for the post-consumer RAS mix. Since all the mixes are statistically ranked with the letter A, no statistical differences existed between the results of the three mixes.

Table C7.3. Ranking of mixes by G_f mean value for -12, -18, -24, and -28°C temperatures

Rank	Treatment	Group mean G_f [J/m^2]
A	5% Post-Consumer RAS	777
A	5% Post-Manufacturer RAS	768
A	30% RAP	741

C8. Field Evaluations

Pavement distress surveys for the Minnesota DOT demonstration project were completed in July 2010, July 2011 and March 2012 on each test Cell shoulder included in the study. MnROAD test Cells are 500 feet long. Surveys for the East and West transitions were completed in July 2011 and March 2012. They were conducted on the first 500 feet of the transition and included the traffic lanes and both shoulders. Surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* by FHWA.

Although only three mix designs were evaluated for this project, the pavement surveys were categorized into 7 different sections. At the MnROAD facility, the same mix was placed in multiple Cells on the shoulder. Additionally, some of the shoulder Cells have different pavement structures, and are also adjacent to different types of pavement structures in the mainline. Therefore, to effectively report the condition of the three mix types in the field, the pavement sections were divided into 7 categories as shown in Table C8.1.

Table C8.1. Summary of MnROAD I-94 test sections

Mix Type	Location	Pavement Structure
5% Post-manufactured RAS	West transition	Newly constucted HMA
5% Post-manufactured RAS	Cell 5 driving shoulder	3" HMA over granular with wick drains over existing HMA adjacent to jointed concrete pavement in the mainline
5% Post-manufactured RAS	Cell 6, 13, 14 driving shoulders	3" HMA over granular constructed adjacent to jointed concrete pavement in the mainline
5% Post-consumer RAS	East transition	Newly constucted HMA
5% Post-consumer RAS	Cell 15 driving shoulder	3" HMA over existing cracked HMA adjacent to HMA in the mainline
5% Post-consumer RAS	Cell 16-23 driving shoulders	3" HMA over granular material Adjacent to HMA in the mainline
30% RAP	Cell 20 passing shoulder	Newly constructed HMA

The primary distress recorded in the test sections was transverse cracking. The amount of transverse cracking for each shoulder section is presented in Figure C8.1. For the post-consumer RAS sections, 4 feet of cracking no cracking occurred in Cells 16-23 during the three year pavement survey period. These shoulder sections were newly constructed and contained 3" of HMA over granular material. Cell 15 also contained post-consumer RAS in the HMA, however, this section contained 141 linear feet of transverse cracking. Cell 15 consisted of 3” of HMA over existing cracked HMA. The amount of distress was likely caused by cracks in the existing HMA layer reflecting into the 3” surface course.

With respect to the post-manufactured RAS test sections, shoulder Cells 5, 6, 13, and 14 each contained HMA with post-manufactured RAS. These Cells were adjacent to a jointed concrete pavement in the mainline, which likely contributed to the transverse cracking in the Cells. Figure C8.4 through C8.7 shows deterioration of the shoulder/mainline joint and transverse cracking reflecting from the concrete joints into the HMA shoulder. Cell 5 contained more transverse cracking than Cells 6, 13, and 14 combined. Cell 5 contained wick drains in the pavement shoulder aggregate base which may be a factor in its distress.

With respect to the 30% RAP section in the passing shoulder of Cell 20, no cracking occurred during the three year pavement survey period. The fact that this was a newly constructed HMA shoulder adjacent to a new constructed HMA mainline, likely contributed to its superior performance.

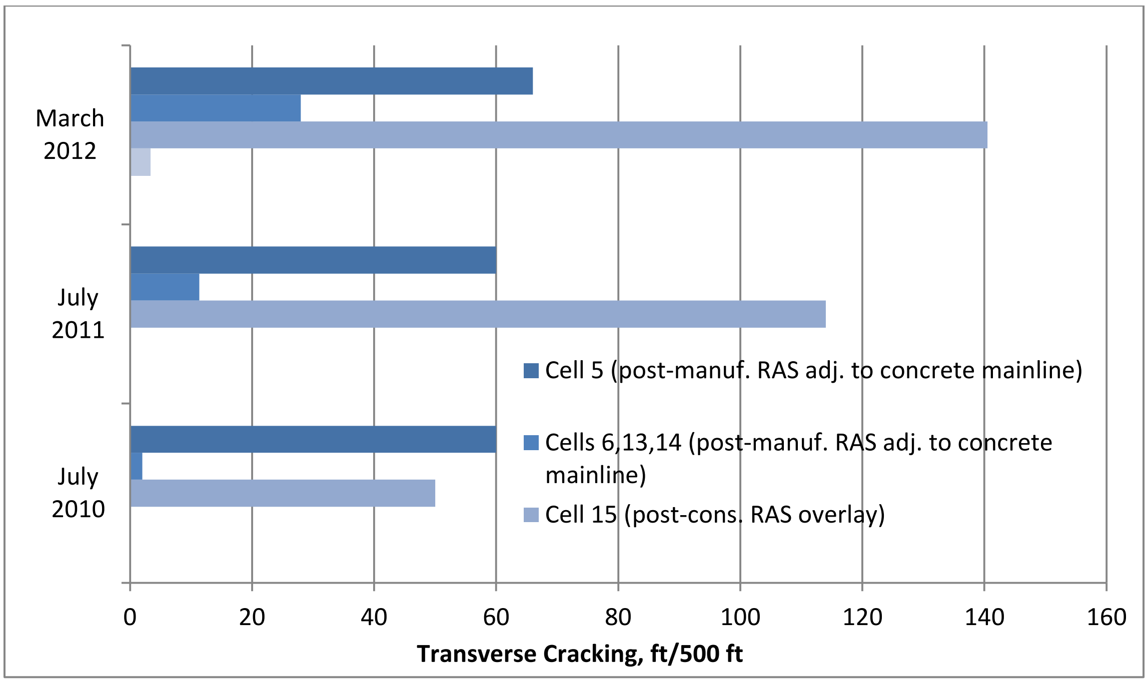


Figure C8.1. Shoulder transverse cracking

The east and west transitions contained a greater amount of transverse cracking than the shoulder section as presented in Figure C8.2. After three years in-service, the west transition, which contained post-manufactured RAS, was found to have a greater amount of transverse cracking (199 linear feet) than the east transition (173 linear feet), which contained post-consumer RAS.

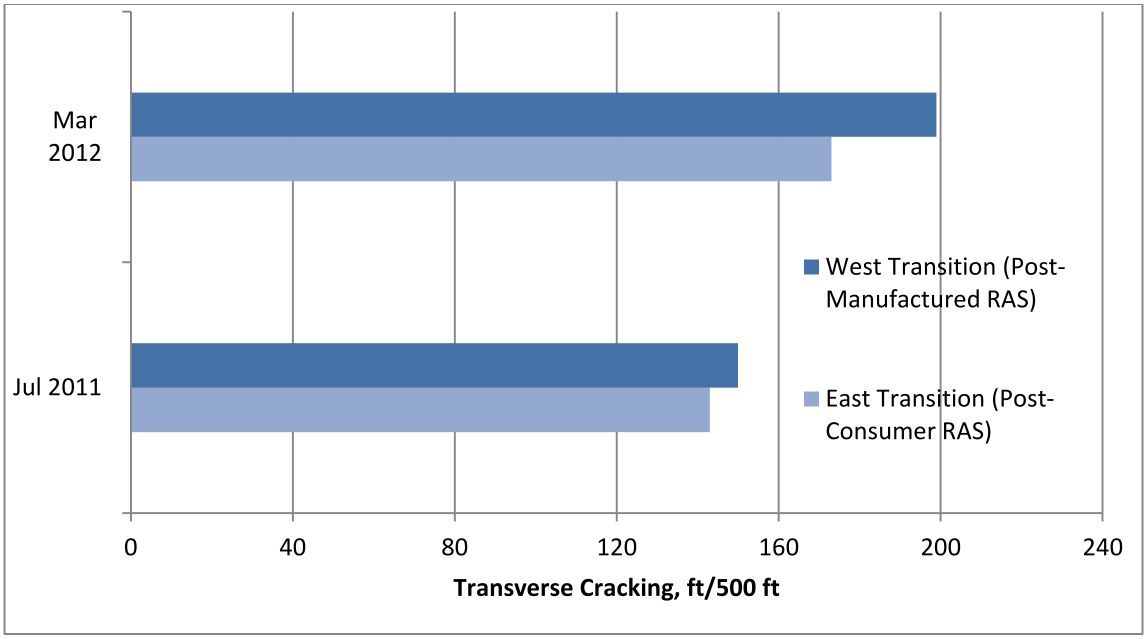


Figure C8.2. East and west transition transverse cracking

While measuring the length of transverse cracking in the pavements, the severity level of the cracks was also measured. Following the guidelines of the Distress Identification Manual, transverse cracks were categorized into three levels: low severity (crack widths ≤ 0.25 in), moderate severity (crack widths 0.25 in ≥ 0.75 in), and high severity (crack widths > 0.75 in).

The severity level of transverse cracks documented in the March 2012 pavement survey is presented in Figure C8.3. Out of all the shoulder sections, Cell 15 (placed over in-place HMA) had the greatest percentage of moderate or high severity transverse cracks. With respect to the transitions, the west transition contained more transverse cracking than the east transition, however, the east transition contained a greater percentage of cracks with a moderate to high severity.

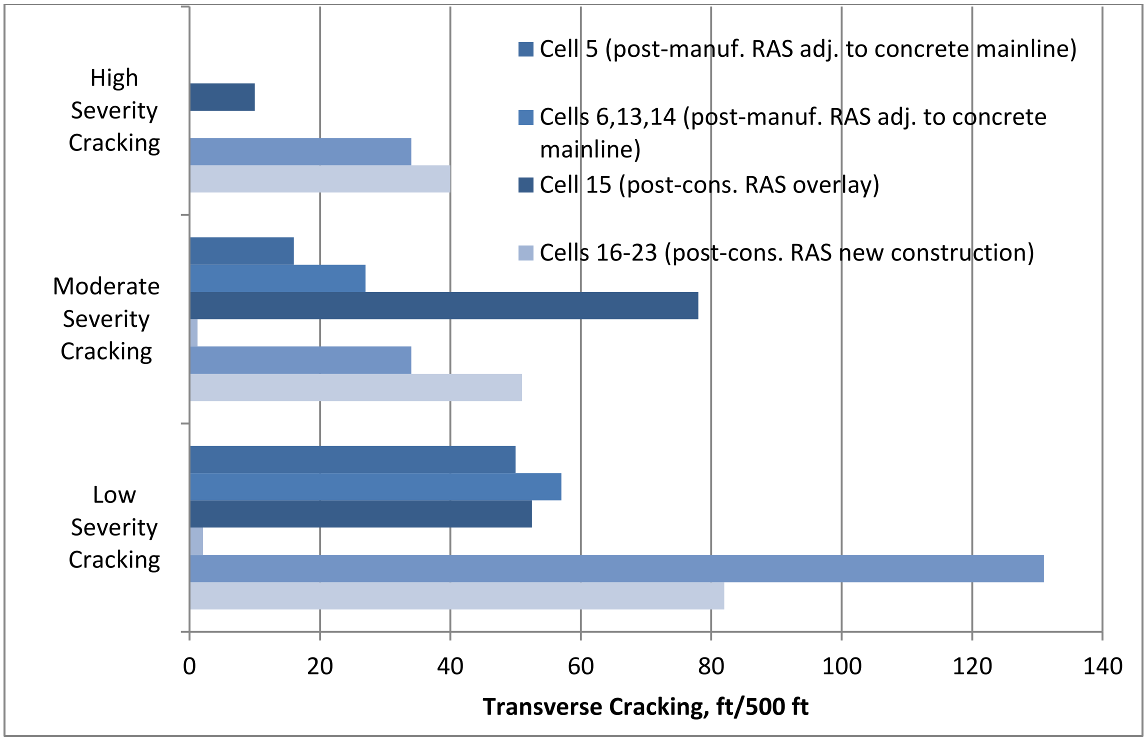


Figure C8.3. Severity level of transverse cracking (March 2012)

Pictures of Cells 13 and 14 from the March 2012 survey are presented in Figures C8.4 through C8.7. They show the majority of the cracking was found at the concrete joints where low to high severity block cracking occurred.

Figure C8.4. Transverse crack Cell 14

Figure C8.5. Block cracking Cell 13

Figure C8.6. Alligator cracking Cell 13

Figure C8.7. Alligator cracking Cell 14

Low to high severity raveling was also documented in Cells 16-23 and the West Transition as shown in Figure C8.8. Low severity raveling was documented in Cells 13-15 and the East Transition. Power-Point Presentations of the distress surveys by 500-foot sections are available for viewing on the TPF-5(213) website.

Figure C8.8. Medium raveling west transition

C9. Conclusions

A Minnesota DOT demonstration project was conducted as part of Transportation Pooled Fund 5-213 to evaluate the effects of post-consumer RAS and post-manufactured RAS in HMA. Three different mixes were constructed and evaluated along the MnROAD facility I-94 shoulder and transitions. One mix contained 5% post-consumer RAS, one mix contained 5% post-manufactured RAS, and another mix contained 0% RAS and 30% RAS. Each mix contained a virgin PG 58-28 binder. Field mixes of each pavement were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and recovery with subsequent binder characterization. The results of the study are summarized below:

- Observations from the demonstration project show the contractor successfully produced and constructed the RAS pavements while meeting MNDOT's quality assurance requirements.
- The performance grade of the blended binder extracted from asphalt mixtures showed that utilizing 5% RAS in the mix design will produce comparable mix performance grades when utilizing 30% RAP in the mix design. When 5% post-consumer RAS was used in the mix design, the continuous PG was 71.1-21.2; when 5% post-manufactured RAS was used in the mix design, the continuous PG was 71.3-21.7; and when 30% RAP was used in the mix design the continuous PG was 68.8-22.7.
- The average dynamic modulus values at high and intermediate temperature ranges showed that utilizing 5% RAS created a slightly stiffer mix than the one that utilized 30% RAP, however, the results were not statistically different among the three mixtures.
- The flow number test showed the 30% RAP mix is more susceptible to permanent deformation since its flow number of 767 is lower than the RAS mixes. The flow number of the post-consumer RAS mix at 2497 was also larger than the flow number of the post-manufactured RAS mix at 1705, indicating greater resistance to permanent deformation. While the two RAS mixes shared similar blended asphalt performance grades, the performance difference could be a result of different gradations in the mix designs. The post-consumer RAS mix contained a coarse gradation, and the post-manufactured RAS mix contained a fine gradation.
- The four-point bending beam results showed that both RAS mixes exhibited longer fatigue lives than the RAP mix. These results are counterintuitive when considering the RAS binder is substantially stiffer than the RAP binder. However, the RAS contains fibers, as a result of the shingle grinding process, which may be improving the fatigue performance of the RAS mixes by enhancing their ductile properties. The post-manufacturer RAS mixture has a longer fatigue life at higher strain levels than the post-consumer RAS mixture, but both mixes have similar fatigue lives at lower strain levels.
- The results of the SCB test indicate similar low temperature cracking resistance of the RAS and RAP mixes. The 30% RAP mix has an average fracture energy of 741 J/m^2. When 5% RAS was used in the mix design in place of 30% RAP, the fracture energy increased to 768 J/m^2 for the post-manufactured RAS mix and **777** J/m^2 for the post-consumer RAS mix. Since all the mixes are statistically ranked with the letter A, no statistical differences existed between the results of the three mixes.
- Field condition surveys conducted two, three, and four years after construction revealed similar performance in the shoulders for the post-consumer RAS pavement section and the RAP pavement section. The post-manufactured RAS sections performed substantially lower, however, the shoulders containing the post-manufactured RAS mix design were adjacent to a jointed concrete pavement in the mainline which seemed to accelerate the cracking in the HMA shoulder. When comparing the mainline transitions, the post-consumer RAS transition contained slightly less transverse cracking (173 linear feet) than the post-manufactured RAS transition (199 linear feet).

C10. MnDOT Demonstration Project Acknowledgments

The researchers gratefully acknowledge the support of the Minnesota DOT. The research work was sponsored by Federal Highway Administration (FHWA) TPF-5(213) and the Transportation Pooled Fund (TPF) partners: Missouri (lead agency), California, Colorado, Illinois, Indiana, Iowa, Minnesota, and Wisconsin DOTs.

APPENDIX D. REPORT FOR THE INDIANA DEPARTMENT OF TRANSPORTATION SPONSORED DEMONSTRATION PROJECT

D1. Introduction

This report presents a summary of the results obtained from the field demonstration project sponsored by the Indiana Department of Transportation (INDOT) as part of Transportation Pooled Fund (TPF) 5-213 *Performance of Recycled Asphalt Shingles in Hot Mix Asphalt*. TPF 5-213 is a partnership of several state agencies with the goal of researching the effects of recycled asphalt shingles (RAS) on the performance of asphalt applications. As part of the Pooled Fund research program, multiple field demonstration projects were conducted to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements that include RAS.

Each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated different aspects of asphalt mixes containing RAS. The field demonstration project sponsored by INDOT investigated using RAS in combination with foaming Warm Mix Asphalt (WMA) technology. The objective of this demonstration project was twofold: first, to evaluate the performance of WMA containing RAS, and second, to compare a typical INDOT mix design that contains recycled asphalt pavement (RAP) to a mix design that contains RAS.

D2. Experimental Plan

To evaluate the compatibility of RAS with WMA, INDOT designed an experimental plan to address the following questions:

- Does replacing 15 percent RAP with three percent RAS affect pavement performance in hot mix asphalt (HMA)?
- Can RAS be used in a mix design that uses foaming WMA technology?

The experimental plan is presented in Table D2.1. The plan was implemented during the demonstration project by producing three asphalt mixtures: HMA-RAP, HMA-RAS, and WMA-RAS.

Table D2.1. Experimental plan

Mix ID	% RAS	% RAP	RAS Source	WMA Technology
HMA-RAP	0	15	-	None
HMA-RAS	3	0	Post-Consumer	None
WMA-RAS	3	0	Post-Consumer	Foaming

During production of the asphalt mixtures, INDOT collected samples of each mixture and delivered them to Iowa State University for laboratory testing. A portion of the samples were sent to the University of Minnesota and the Minnesota Department of Transportation (MNDOT) for Semi-Circular Bend (SCB) testing and binder extraction and recovery, respectively. The laboratory testing plan is presented in Table D2.2.

Table D2.2. Laboratory testing plan

	Laboratory Test	Iowa State University	University of Minnesota	Minnesota DOT
Processed Shingles	Binder Extraction			X
	High Temperature PG			X
	Gradation (Before Extraction)	X		
	Gradation (After Extraction)	X		
Mixture	Binder Extraction			X
	Binder PG Characterization	X		
	Gradation	X		
	Dynamic Modulus	X		
	Flow Number	X		
	Beam Fatigue	X		
	Semi-Circular Bending (SCB)		X	

The asphalt was recovered from the RAS and asphalt mixtures following AASHTO T164 Method A (Centrifuge Method) by using a blend of toluene and ethanol as the extraction solvent. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. At Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO R29 “Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder.”

Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution, and a washed gradation was conducted after extraction to evaluate the size distribution of the fine aggregates in the RAS product.

Performance testing was completed on the field produced asphalt mixtures at low, intermediate, and high temperatures. Dynamic Modulus and Flow Number were conducted at high temperatures to evaluate the modulus and rutting resistance of asphalt mixtures. The durability of the mixtures at intermediate temperatures was evaluated using the dynamic modulus and four-point beam fatigue test. The SCB test conducted at the University of Minnesota evaluated the fracture properties of the mixtures at low temperatures.

After construction of the pavement for the demonstration project, field evaluations were conducted on each pavement test section one and two years after paving to assess the field performance of the pavement concerning cracking, rutting, and raveling.

D3. Project Location

The field demonstration project was completed on US Route 6 (a two-lane highway) east of Nappanee, Indiana located in Elkhart County. The test sections were placed in the eastbound and westbound lanes starting 1.25 miles east of State Road (SR) 19 and ending just west of SR 15, for a total length of approximately 6.8 miles. The project limits are identified below in Figure D3.1.

Figure D3.1. Project location

D4. Project Description

The demonstration project was conducted by Phend & Brown, Inc. in July and August of 2009. The existing pavement structure consisted of an HMA overlay placed over a concrete pavement. For the demonstration project, 1.5 inches of the HMA was to be milled and replaced with 1.5 inches of one of the three test experimental mix designs: an HMA Control section with 15% RAP only (0% RAS), an HMA test section with 3% RAS only (0% RAP), and a WMA test section with 3% RAS only (0% RAP). Mainline pavement cores identified the existing HMA thickness to range from 8.0 to 12.5 inches in the eastbound EB lane and from 11.5 to 15.5 inches in the WB lane. The existing concrete thickness ranged from 7.5 to 8.5 inches in the both the EB and WB lane. Cross-sections of the EB and WB lanes are shown in Figure D4.1.

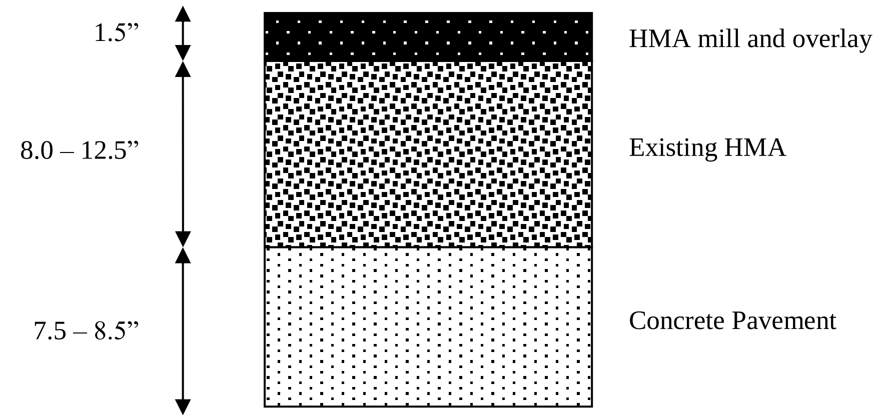


Figure D4.1a. Eastbound pavement cross-section

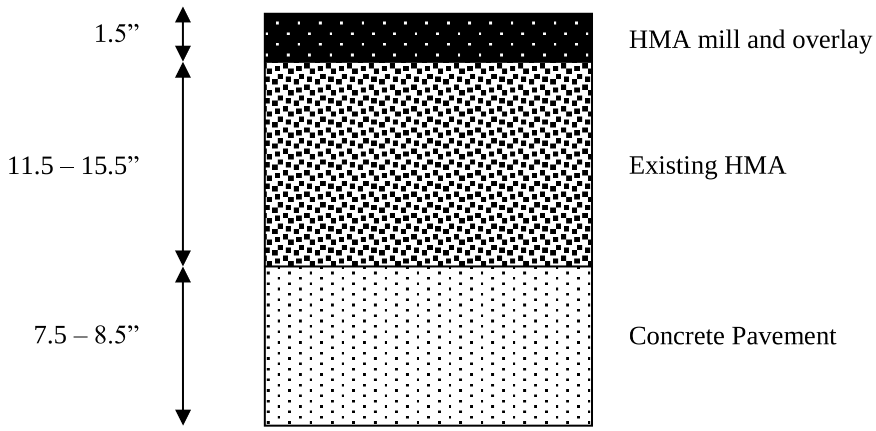


Figure D4.1b. Westbound pavement cross-section

Phend & Brown paved the surface course test sections in July 2009. The HMA-RAP section was paved in the entire EB lane (6.8 miles). In the WB lane, the HMA-RAP section was paved from Station 426.5 to 178.4, approximately 4.7 miles; the HMA-RAS test section was paved from Station 178.4 to 102.32, approximately 1.1 miles; and the WMA-RAS section was paved from Station 120.32 to 67.5 (the end of the project) approximately 1.0 miles. A plan view of the test section on US Route 6 is shown in Figure D4.2.

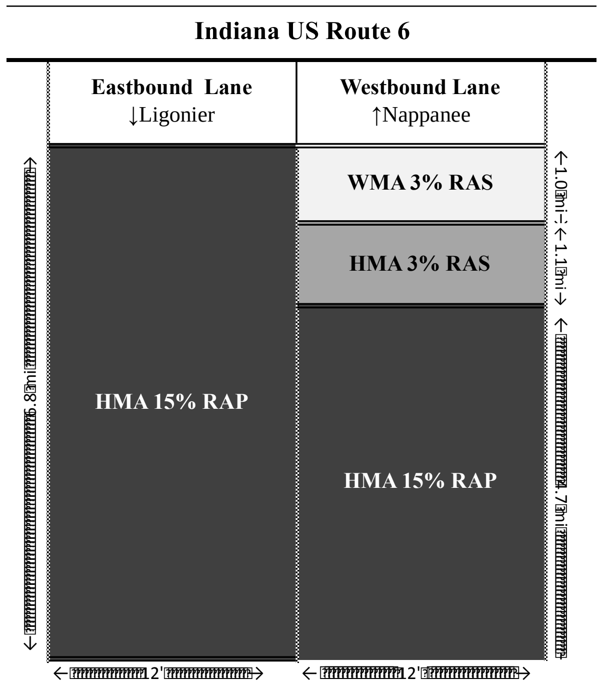


Figure D4.2. Plan view of US Route 6 project test sections

The Weather conditions during the paving of the test sections were noted as sunny with an ambient temperature of 85 degrees Fahrenheit. The asphalt plant for the project was located in Leesburg, Indiana. The haul distance to the project sections, or furthest point from the plant was 15 miles. Production rates averaged 300 ton per hour. A total of approximately 7,760 tons of HMA was placed in the demonstration project with a total of approximately 36 tons of RAS and 990 tons of RAP. Tonnages of RAS, RAP and total HMA for each test section are summarized below in Table D4.1 below.

Table D4.1. Project tonnages

Material	HMA-RAP (Tons)	HMA-RAS (Tons)	WMA-RAS (Tons)
RAS	---	17.5	17.3
RAP	990	---	---
Total HMA	6603	580	574

D5. Shingle Processing

The post-consumer RAS came from Touby Pike Recycling Center, LLC (Touby Pike Recycling) located in Kokomo, Indiana. As required by the Indiana Department of Environmental Management, the RAS used in the project came from shingles on single family homes built after 1980 with one shingle roofing layer. Shingles from other residential or commercial facilities were permitted to be used proved that were sampled, tested, and found to contain less than 1% asbestos containing materials (ACM). No rolled roofing was accepted for recycling at their facility. Both the pre-processed and post-processed asphalt shingles were stored either in a building or in a covered containment unit. Random samples were collected from the stockpiles of pre-processed asphalt shingles and tested by a certified laboratory for ACM using the polarized light microscopy method. All samples were found to be negative.

Sample buckets of the final ground RAS were delivered to Iowa State University. A picture of the INDOT RAS is shown below in Figure D5.1. The test result of the RAS gradation before extraction was completed by Iowa State University; it is presented in Table D5.1. Four RAS gradations after burnoff in the ignition oven were also completed by INDOT. The average of the test results are also presented in Table D5.1. The RAS contained an asphalt content of 26.8%.

Figure D5.1. INDOT post-consumer RAS

Table D5.1. RAS gradations

Sieve Size (US)	Sieve Size (mm)	RAS (Before Extraction)	RAS (After Extraction)
3/4"	19	100	100
1/2"	12.5	100	100
3/8"	9.5	97	99
#4	4.75	74	90
#8	2.36	62	87
#16	1.18	38	69
#30	0.6	18	47
#50	0.3	9	40
#100	0.15	4	34
#200	0.075	0.7	26.5
	% Asphalt Content		26.8

D6. Asphalt Mix Design and Prodution Results

Two HMA mix designs were prepared by for the demonstration project. The first mix design contained 15% RAP and 0% RAS. The second HMA mix design contained 3% RAS and 0% RAP. To produce the WMA with 3% RAS, Phend & Brown used the job mix formula for the HMA with 3% RAS with a foamed liquid asphalt. The mix design gradations obtained from laboratory testing of the sampled asphalt mixtures are presented in Figure D6.1. As shown in the figure, the asphalt mixtures had similar aggregate structures with gradations passing above the maximum density line.

Asphalt demand properties of the mixtures are presented in Table D6.1. When replacing 15% RAP with 3% RAS, the percent binder replacement of the mixtures decreased from 19.3% to 12.9%. The effective asphalt content of all the designs was approximately the same at 5.1/5.2%, with the RAS mix design having a higher total asphalt content of 0.5%.

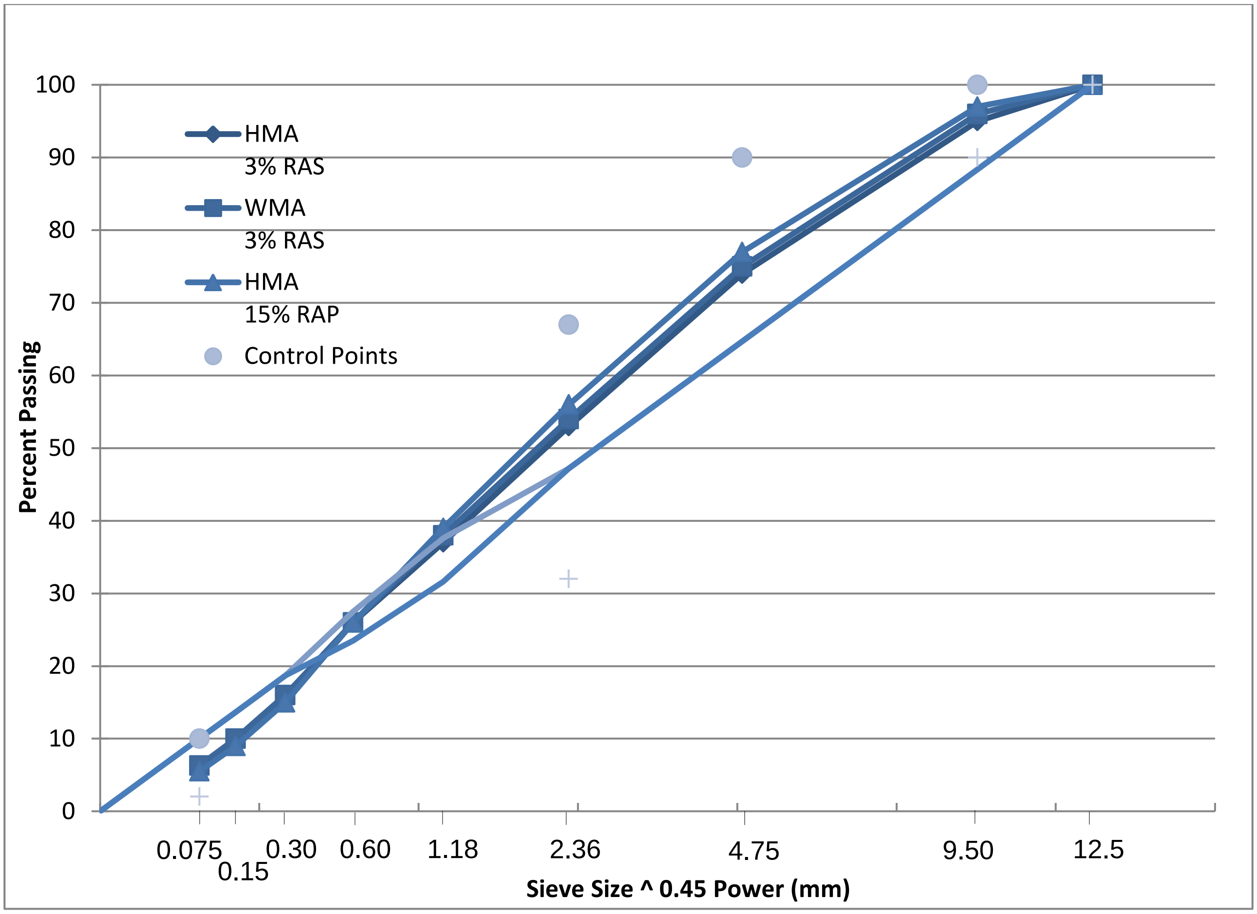


Figure D6.1. Asphalt gradations

Table D6.1. Mixture asphalt demand properties

Mix Property	HMA-RAP	HMA-RAS	WMA-RAS
% RAS	0	3	3
% RAP	15	0	0
% Total AC	5.7	6.2	6.2
% Virgin AC	4.6	5.4	5.4
% Binder Replacement	19.3	12.9	12.9
% Effective Asphalt	5.1	5.2	5.2
% Asphalt Absorption	0.8	1.0	1.0

The asphalt mix design volumetric properties are presented in Table D6.2. The designs were dense-graded Superpave bituminous mixtures, following INDOT's 401 specification for the project. The mix designs met INDOT's Category 4 design traffic level, which corresponds to 10 million <30 million equivalent single axel loads (ESAL's) over a 20-year design period. The target voids for all mixes were 4%. A PG 70-22 virgin asphalt binder was used for the mix designs.

Table D6.2. Mixture design volumetric properties

Mix Property	HMA-RAP	HMA-RAS	WMA-RAS
Design Gyrations	100	100	100
NMAS (mm)	9.5	9.5	9.5
Virgin PG Grade	70-22	70-22	70-22
% Voids	4.0	4.0	4.0
% VMA	15.3	15.1	15.1
% VBE	10.2	9.9	9.9
% VFA	73.9	73.5	73.5
-#200/Pbe	1.2	1.1	1.1

Production control results by Phend & Brown are presented in Table D6.3. The results are based on the first quality control tests conducted during the production of each of the three mixes. All three mixes were produced with asphalt contents and volumetric properties close to the job mix formula (JMF). Pay factors for mix properties ranged from 0.98 to 1.05. Density results obtained from field cores show the contractor was able to successfully compact all three mixes. Although the WMA-RAS mix had lower production temperatures (274°F) than the HMA-RAS mix (297°F), the WMA had a higher density after compaction. Since WMA technology increases the workability of an asphalt mixture at lower temperatures, it may have helped the contractor achieve a great density of the RAS mixture by opening up the compaction window of the mix.

Table D6.3. Mixture and construction quality assurance results

Mix Property	HMA-RAP 7/28/2009		HMA-RAS 7/30/2009		WMA-RAS 7/30/2009	
	JMF	QC Results	JMF	QC Results	JMF	QC Results
% Total AC(1)	5.7	6.1	6.2	6.3	6.2	6.0
% Voids(1)	4.0	3.0	4.0	2.8	4.0	3.6
% VMA(1)	15.3	15.1	15.1	14.8	15.1	14.8
% Density(2)	-	92.1	-	93.3	-	94.4
Ave Load Temp (°F)	-	-	-	297	-	274

(1) First quality control test result during production
(2) Average of core density results (% of Gmm)

D7. Laboratory Test Results

Binder Testing

Performance Grade (PG) testing of the extracted binders was conducted to obtain their high, low, and intermediate PG temperatures as shown in Table D7.1. The high temperature PG of the RAS binder at 134.2°C is higher than traditional paving grade binders. This is expected since the binder in roofing shingles is produced with an air-blowing process which oxidizes the asphalt. Additionally, the RAS used in the mix designs is from post-consumer shingles, so the binder in the RAS has experienced at least several years of aging.

Because the RAS mixtures are heated to high temperatures and placed in a centrifuge at high speeds during the recovery process, the RAS and virgin asphalt should be fully blended. The addition of 15% RAP binder raised the low temperature PG one grade higher to -16°C while keeping the high temperature PG the same at 70°C. The continuous PG for the HMA-RAP mixture was 75.6-20.1, while the HMA-RAS mixture was 77.6-14.2 and the WMA-RAS mixture was 78.8-15.1. Both RAS mixtures contained similar performance grades indicating that foaming WMA technology doesn't change the properties of the blended binder.

Table D7.1. Performance grade of extracted binders

Material Identification	High PG Temp, °C	Intermediate PG Temp, °C	Low PG Temp, °C	Performance Grade
PG 70-22	72.2	25.3	-24.2	70-22
RAS	134.2	-	-	-
HMA-RAP	75.6	26.2	-20.1	70-16
HMA-RAS	77.6	26.2	-14.2	76-10
WMA-RAS	78.8	26.3	-15.1	76-10

Dynamic Modulus

The dynamic modulus (|E*|) is a key material property that determines the stress-strain relationship of an asphalt mixture under continuous sinusoidal loading. A higher dynamic modulus indicates lower strains will result in a pavement structure when the asphalt mixture is stressed from repeated traffic loading. The mechanistic-empirical pavement design guide (MEPDG) uses |E*| as the stiffness parameter to calculate an asphalt pavements strains and displacements.

The test was conducted following AASHTO TP62 using five replicate samples at 7 ± 0.5% air voids with 150 mm in height and 100 mm in diameter. Samples were tested by applying a continuous sinusoidal load at 9 different frequencies (0.1, 0.3, 0.5, 1, 3, 5, 10, 20, and 25 Hz) and three different temperatures (4, 21, and 37°C). Sample loading was adjusted to produce strains between 50 and 150 μstrain in the sample.

Master curves were constructed at a reference temperature of 21°C and plotted on a log-log scale for a general comparison as presented in Figure D7.1. The RAP and RAS mixtures appear to have similar dynamic modulus values across a wide frequency range. While the RAS binder has a higher stiffness modulus than the RAP binder, more virgin binder is replaced in the RAP mixture than the RAS mixtures. This created mixtures similar binder performance grades (Table D7.1), resulting in mixtures with similar modulus values. The results also show that the WMA technology did not change the dynamic modulus of the mixture.

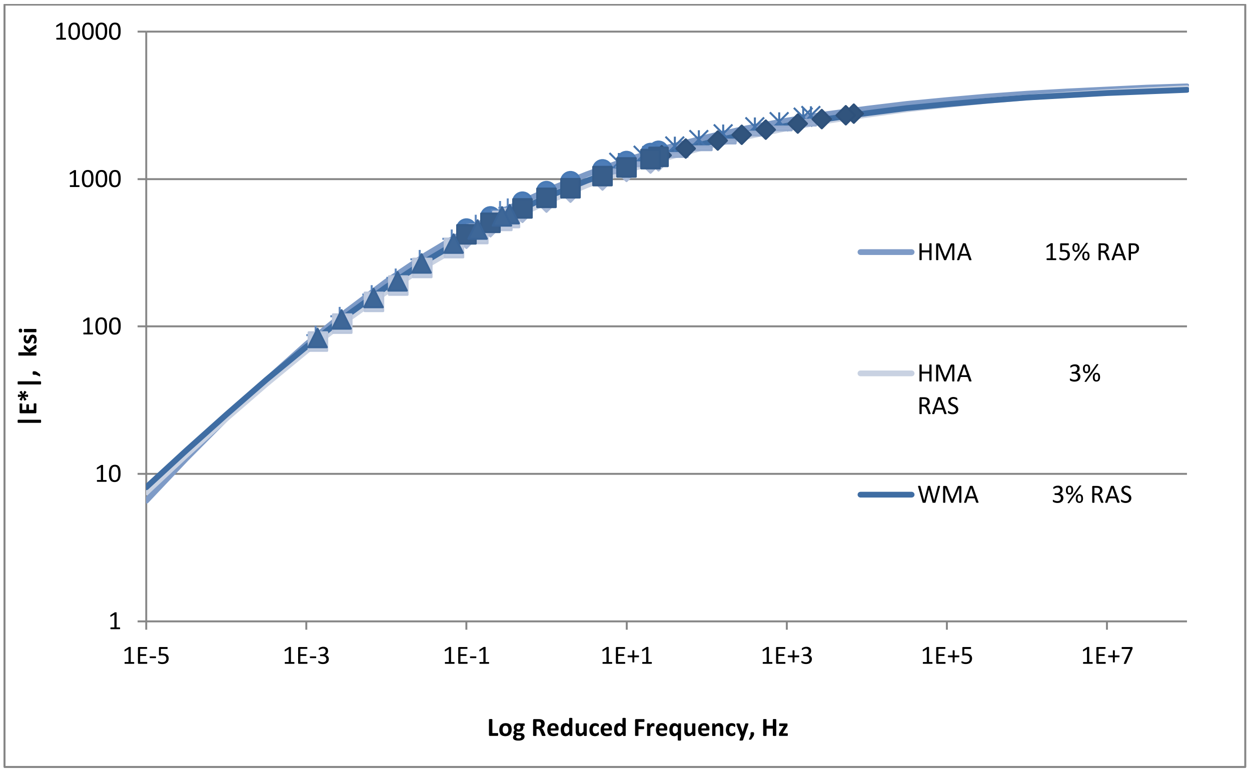


Figure D7.1. Comparison of master curves for MoDOT mixes

The plot in Figure D7.2 presents the mean dynamic modulus at 21°C and 37°C at specific frequencies for a more direct comparison. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. There were no statistical differences at a 95% confidence level among the three mixtures except the difference between the HMA-RAP and HMA-RAS mixture at the intermediate 21°C temperature. Low modulus values at this temperature are considered desirable in thin asphalt pavements (less than 4”) for fatigue cracking resistance. Mixtures with lower stiffness and can deform more easily without building up large stresses. The RAS fibers may be affecting the overall material response during dynamic loading by reducing the modulus at intermediate temperatures.

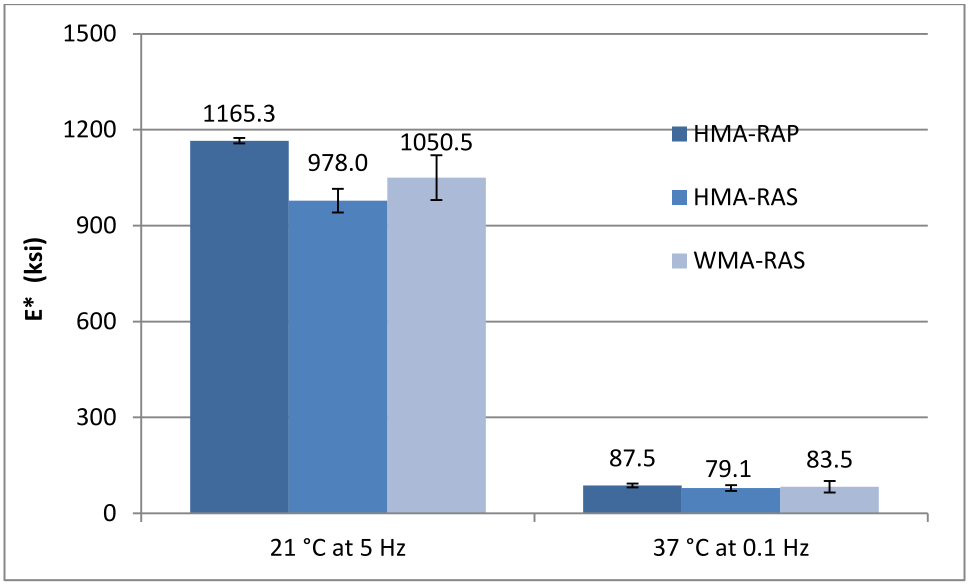


Figure D7.2. Dynamic modulus comparison at 21°C, 5 Hz and 37°C, 0.1 Hz

Flow Number

The flow number test measures the permanent deformation resistance of asphalt mixtures by applying a repeated dynamic load to a sample for up to several thousand load cycles. The flow number is defined as the number of load cycles an asphalt mixture can tolerate until it flows. Cumulative permanent deformation in the sample is plotted versus load cycles. The flow number is reached at the onset of tertiary flow.

Tests were conducted following procedures used in NCHRP Report 465. Samples used in the dynamic modulus test were used for the flow number test since the dynamic modulus test is nondestructive. The samples were placed in a hydraulically loaded universal testing machine, unconfined, with a testing temperature of 37°C to simulate the climactic conditions that cause pavement to be susceptible to rutting. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 sec followed by 0.9 sec of dwell time. The loading cycle was repeated for a total of 10,000 load cycles. Three LVDT’s were attached to each sample during the test to measure the cumulative strains.

All the mixtures performed very well in the test with high flow numbers; therefore, all three mixtures should be very resistant to permanent deformation. However, higher flow numbers in the RAS mixtures indicate that mixtures with 3% RAS will be more rut resistant than mixtures with 15% RAP. The test results also show that adding foaming WMA technology did not change the mixture's resistance to permanent deformation.

Test results are presented in Figure D7.3. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. Since the error bars of the RAS mixtures do not overlap with the error bars of the HMA-RAP mixture, the RAS mixtures performed statistically better than the HMA-RAP mixture at a 5 percent Type I error level.

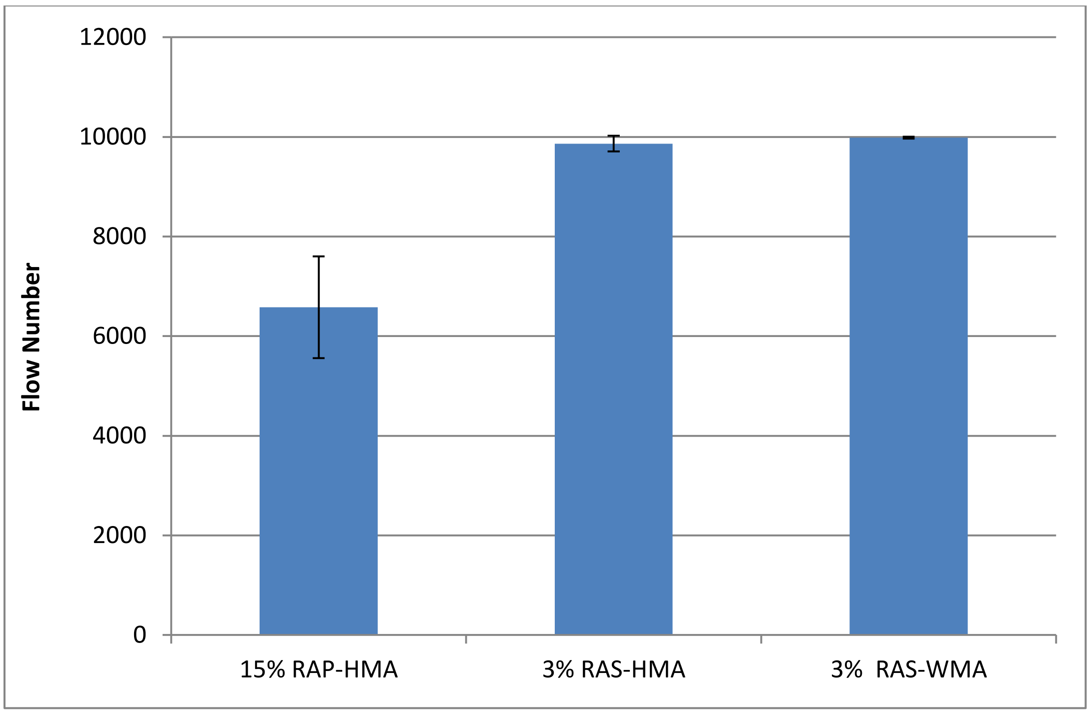


Figure D7.3. Flow number test results

Beam Fatigue

Fatigue cracking is the major cracking distress in asphalt pavements caused by repeated heavy traffic loads. Pavements that have a higher resistance to tensile strains that develop at the bottom of an asphalt layer due to repeated traffic will have a greater resistance to fatigue cracking. The four-point beam fatigue test following AASHTO T321 was conducted to evaluate the load associated cracking resistance of the mixtures.

The mixes were compacted in a linear kneading compactor to create slabs with 7% air voids. The slabs were saw-cut into beams with dimensions 15 inches in length, 2.5 inches in width, and 2 inches in height. The beams were tested in a strain controlled mode of loading at 20°C with haversine wave pulses applied to the beam at 10 Hz. Six beams were tested, each at a different strain level (375, 450, 525, 650, 800, and 1000 μstrain), until the flexural stiffness of the beam was reduced to 50% of the initial stiffness. A log-log regression was performed between strain and the number of cycles to failure (N_f). The relationship between strain and N_f can be modeled using the power law relationship as presented in Equation 1.

$$N_f = K1\left(\frac{1}{\varepsilon_o}\right)^{K2} \tag{1}$$

where: N_f = cycles to failure; ε_o = flexural strain; and K1 and K2 = regression constants.

The fatigue curves from beam fatigue test results are presented in Figure D7.4 with the fatigue model coefficients in Table D7.4. Here, hot mix asphalt with RAS is compared to warm mix asphalt with RAS. Both mixtures are also compared to the conventional 15% RAP mixture in Indiana. The fatigue curves of all three mixtures are essentially the same. The HMA with RAS has similar fatigue properties as the HMA with no RAS. Additionally, the WMA mixture with RAS performed the same as the other mixes.

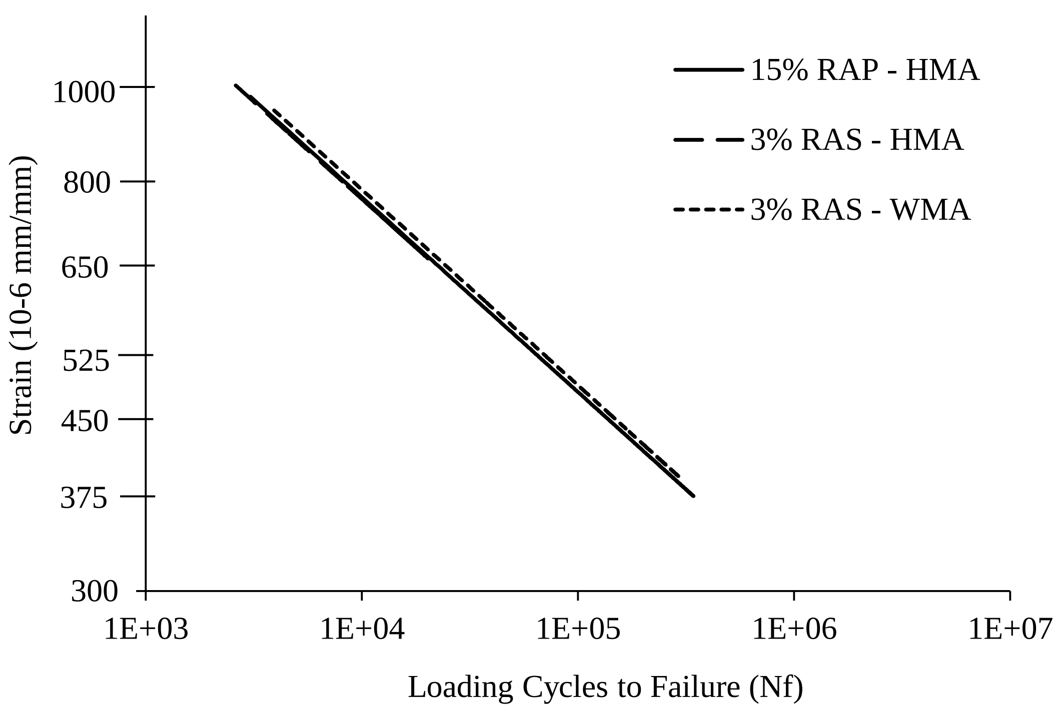


Figure D7.4. ε-N fatigue curves

The fatigue endurance limit (FEL) of each mixture was also predicted. If tensile strains are low enough in a pavement structure, the pavement has the ability to heal and therefore no damage cumulates over an indefinite number of load cycles. The level of this strain is referred to as the

FEL. The FEL of each mixture was estimated using the lower 95% prediction limit at 50 million load cycles as proposed in NCHRP Report 646 and shown in Equation 2.

$$\text{Lower Prediction Limit} = \hat{y}_o - t_\alpha s\sqrt{1+\frac{1}{n}+\frac{(x_o-\bar{x})^2}{S_{xx}}} \qquad (2)$$

where:
y_o = the one-sided lower 95% prediction interval at the micro-strain level corresponding to 50,000,000 cycles;
t_α = value of t distribution for n-2 degrees of freedom for a significance level of 0.05;
s = standard error of the regression analysis;
n = number of samples;
S_{xx} = sum of squares of the x values;
x_o = log 50,000,000; and
$\bar{x}$ = average of the fatigue life results.

The FEL estimates are presented in Table D7.4. All three mixes exhibit similar long-term endurance limits indicating that damage will accumulate in the 3% RAS mixture (HMA or WMA) at the same level as the 15% RAS mixture.

Table D7.4. Beam fatigue results

Mix ID	% Binder Replacement	K1	K2	R^2	Endurance Limit (Micro-strain)
15% RAP - HMA	19.3	7.04E-12	-4.87	0.993	114
3% RAS - HMA	12.9	1.41E-11	-4.77	0.970	118
3% RAS - WMA	12.9	1.17E-11	-4.81	0.985	110

Semi-Circular Bending

The low temperature fracture properties of the mixtures were obtained from SCB tests by following the procedure in “Investigation of Low Temperature Cracking in Asphalt” (Marasteanu et al., 2007). Testing was conducted at four different low temperatures: -6°C, -12°C, -18°C, and -22°C.

All tests were performed inside an environmental chamber, and liquid nitrogen was used to obtain the required low temperature. The temperature was controlled by the environmental chamber temperature controller and verified using an independent platinum resistive-thermal-device (RTD) thermometer. The load line displacement (LLD) was measured on both faces of the test specimens using a vertically mounted Epsilon extensometer with 38 mm gage length and ±1 mm range. One end was mounted on a button that was permanently fixed on a specially made frame, and the other end was attached to a metal button glued to the sample. The average LLD measurement was used for each specimen. The crack mouth opening displacement (CMOD) was

recorded by an Epsilon clip gage with 10 mm gage length and a +2.5 and -1.0 mm range. The clip gage was attached at the bottom of the specimen. A constant CMOD rate of 0.0005mm/s was used and the load and load line displacement (P-u), as well as the load versus LLD curves were plotted. A contact load with a maximum load of 0.3 kN was applied before the actual loading to ensure uniform contact between the loading plate and the specimen. The testing was stopped when the load dropped to 0.5 kN in the post peak region. The load and load line displacement data were used to calculate the fracture toughness and fracture energy (G_f).

The fracture energy (G_f) parameter is presented in Figure D7.5. The laboratory test results were analyzed to evaluate the effect of the various RAS treatments on the fracture energy. MacAnova statistical software package was utilized to perform a statistical analysis. The analysis of variance (ANOVA) was used to examine the differences among the mean response values of the different treatment groups. The significance of the differences was tested at 0.05 level of error. The analysis of variance was conducted with the assumption that the errors in the data are independently normal with constant variance.

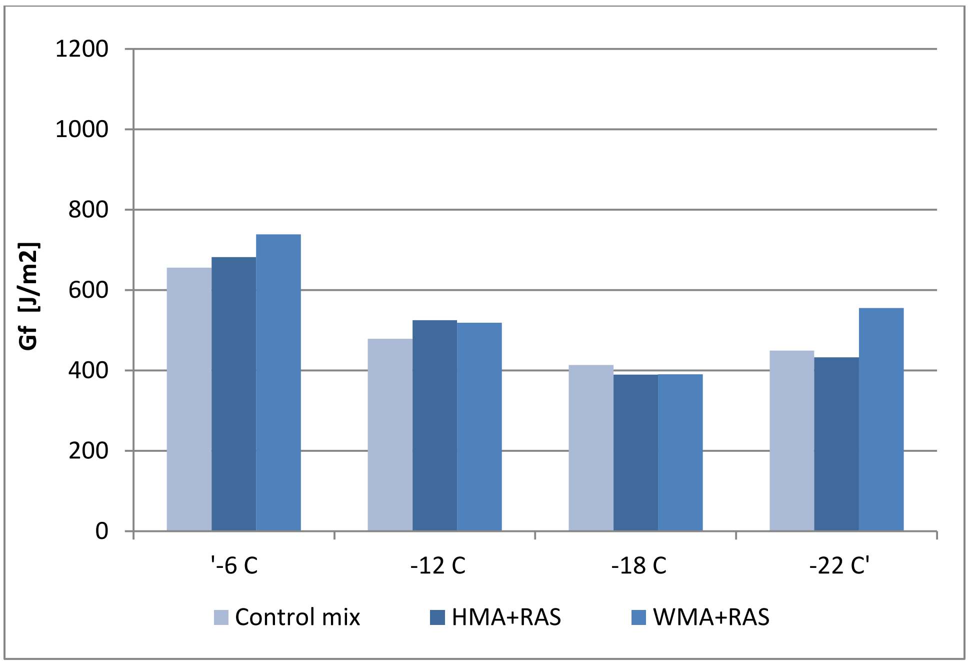


Figure D7.5. Indiana mixture fracture energy (G_f)

The Gf group means of each RAS treatment levels was compared using a pair-wise comparison to rank the RAS treatment levels with regard to fracture energy. The outcome is reported in Table D7.5, in which statistically similar RAS treatments are grouped together. Letter A indicates the best performing group of mixtures; letter B the second best, and so on. Groups with the same letter are not statistically different, whereas mixtures with different letters are statistically different.

When 15% RAP was replaced with 3% RAS, the fracture energy decreased from 551 to 502 **J/m^2** although the difference was not statistically significant. Overall, the results of the SCB test did not detect any difference in low temperature cracking performance when either RAS or WMA technology was used in the mixtures.

Table D7.5. Ranking of mixes by G_f mean value for -6, -12, -18, and -22°C temperatures

Rank	Treatment	Group mean G_f [J/m^2]
A	HMA-RAP (Control)	551
A	HMA-RAS	502
A	WMA-RAS	500

D8. Field Evaluations

The project team completed three distress surveys for the Indiana demonstration project test sections in November 2010, May 2011, and March 2012. During the surveys, the traffic level of trucks and heavy farm equipment on U.S. Route 6 was documented as can be seen below in Figure D8.1.

Figure D8.1. Heavy farm equipment and trucks traveling on US 6 (May 2011 survey)

Three 500-foot sections were randomly selected in each of the test sections: HMA-RAP, HMA-RAS, and WMA-RAS. For the HMA-RAP sections, two of the surveys were completed in the EB lane and one in the WB lane. Since the HMA-RAS and WMA-RAS sections were only in the WB lane, all the surveys for those sections were conducted in the WB lane. The surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* by FHWA.

The distress surveys found a progression of transverse cracking over the three years within the three sections. These cracks are suspected to be caused by reflective cracking from differential movement of the concrete and HMA pavement below the overlay. Since no pre-condition survey was available, the project team was unable to ensure that the different survey sections contained similar levels of distress before the overlay. Many of the cracks were sealed at some point between the November 2010 and May 2011 surveys.

The HMA-RAP and HMA-RAS test sections were found to have comparable linear length of transverse cracking per 500-feet, whereas the WMA RAS test section was found to have a higher linear length of transverse cracking per 500-feet, as shown in Figure D8.1 below.

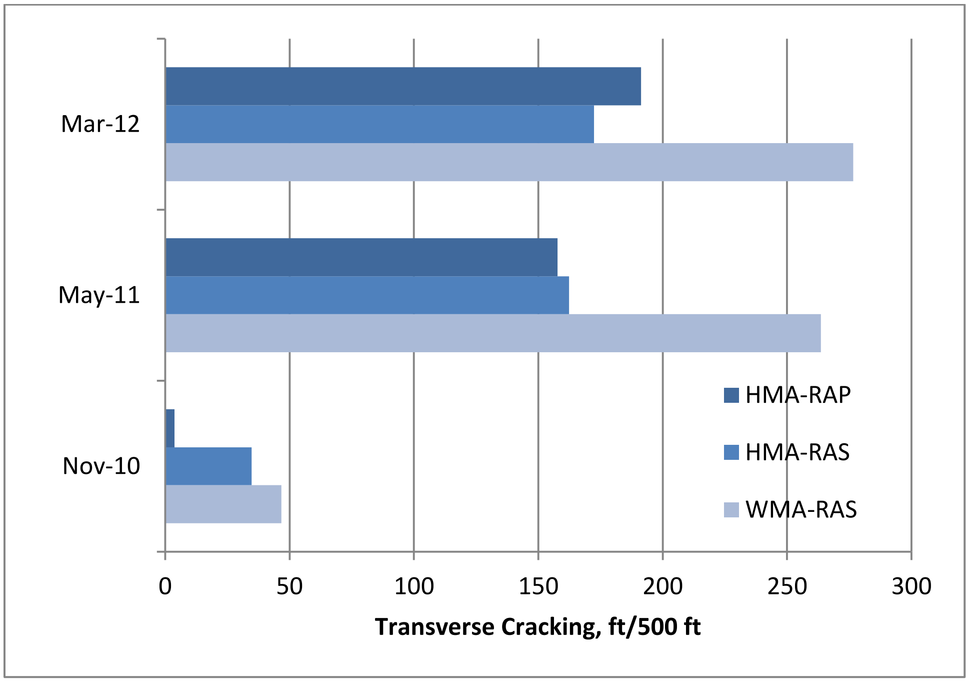


Figure D8.2. Indiana pavement evaluation

While measuring the length of transverse cracking in the pavements, the severity level of the cracks was also measured. Following the guidelines of the Distress Identification Manual, transverse cracks were categorized into three levels: low severity (crack widths ≤ 0.25 in), moderate severity (crack widths 0.25 in ≥ 0.75 in), and high severity (crack widths > 0.75 in).

Although the WMA-RAS pavement sections contained more transverse cracking than the other sections after two years, most of the cracks in the WMA-RAS sections were of low severity while the HMA-RAP and HMA-RAS sections had a greater percentage of cracks with a moderate to high severity. As shown in Figure D8.3, 65% of the transverse cracks measured in HMA-RAP sections have a moderate or greater severity level, 60.5% of the transverse cracks measured in HMA-RAS sections have a moderate or greater severity level, and 28% of the transverse cracks measured in WMA-RAS sections have a moderate or greater severity level. Whether the low severity cracks in the RAS sections will expand into moderate of high severity cracks remains to be seen. Meanwhile, the current pavement survey data suggests that replacing RAS may help reduce low severity cracks from expanding into a higher level of severity. The addition of fibers from the RAS could help prevent existing cracks from expanding.

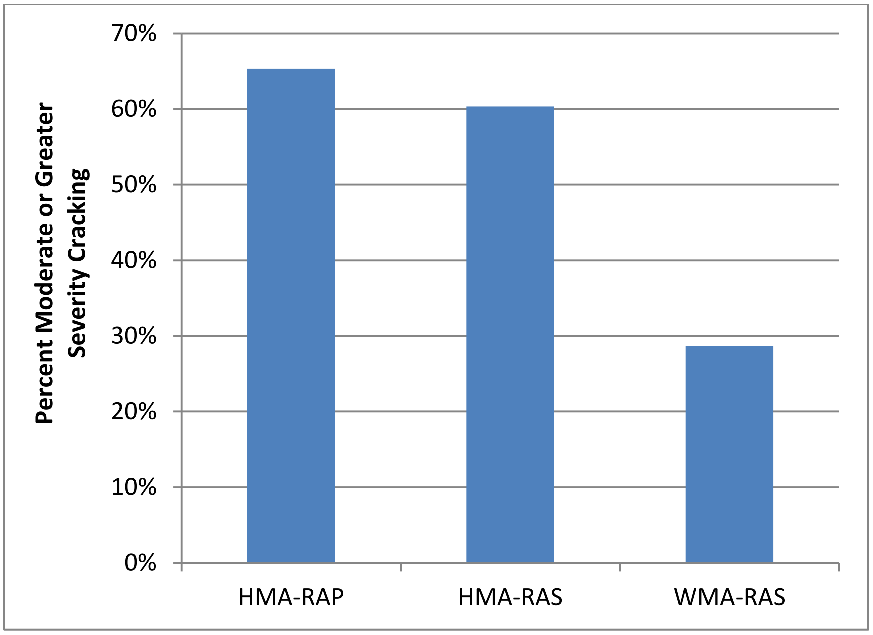


Figure D8.3. Percent of transverse cracks with moderate severity or greater (March 2012)

Examples of the transverse cracks (TC) measured in the pavement test sections are presented in Figure D8.4 and D8.5.

Figure D8.4. Low severity TC (WMA-RAS)

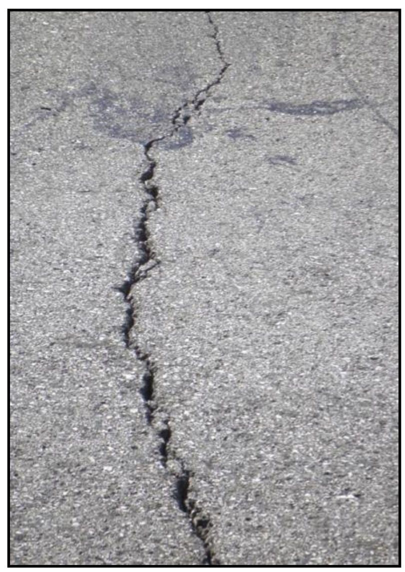


Figure D8.5. High severity TC (HMA-RAS)

Low severity fatigue cracking, identified by random longitudinal cracking in the wheel path, was also documented in all three sections shown in Figures D8.5 through D8.8. The greatest amount of longitudinal cracking was documented in the HMA-RAS and WMA-RAS sections (Figure D8.9).

Figure D8.6. Fatigue cracking (HMA-RAP)

Figure D8.7. Fatigue cracking (HMA-RAS)

Figure D8.8 Fatigue cracking (WMA-RAS)

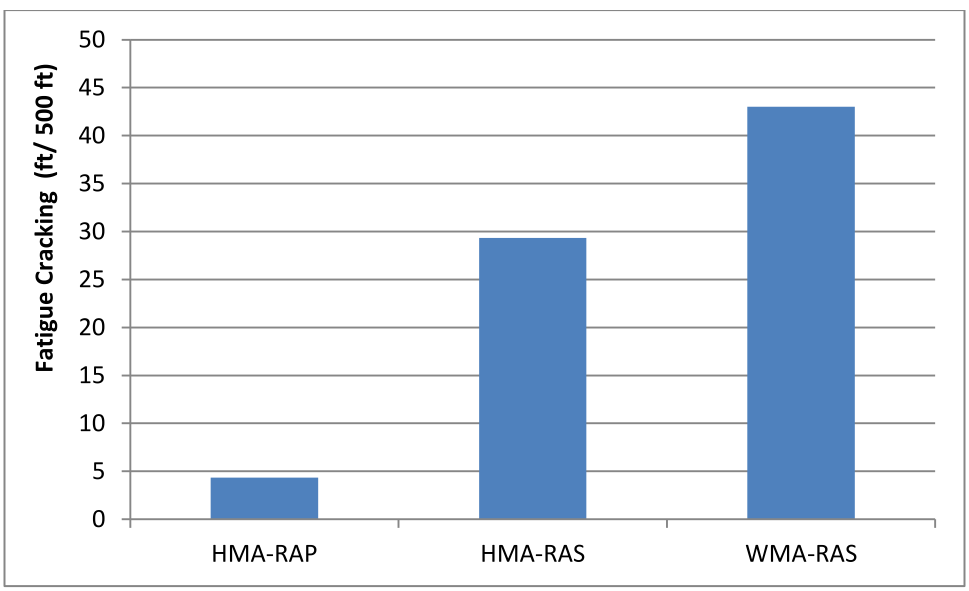


Figure D8.9. Fatigue cracking (ft/500 ft) (March 2012)

Longitudinal and block cracking emanating from the white striping was also documented in all three-test sections as shown in Figure D8.10. The HMA-RAS and WMA-RAS sections had the greatest amount of this type of cracking (Figure D8.11).

Figure D8.10. Longitudinal/block cracking near adjacent striping

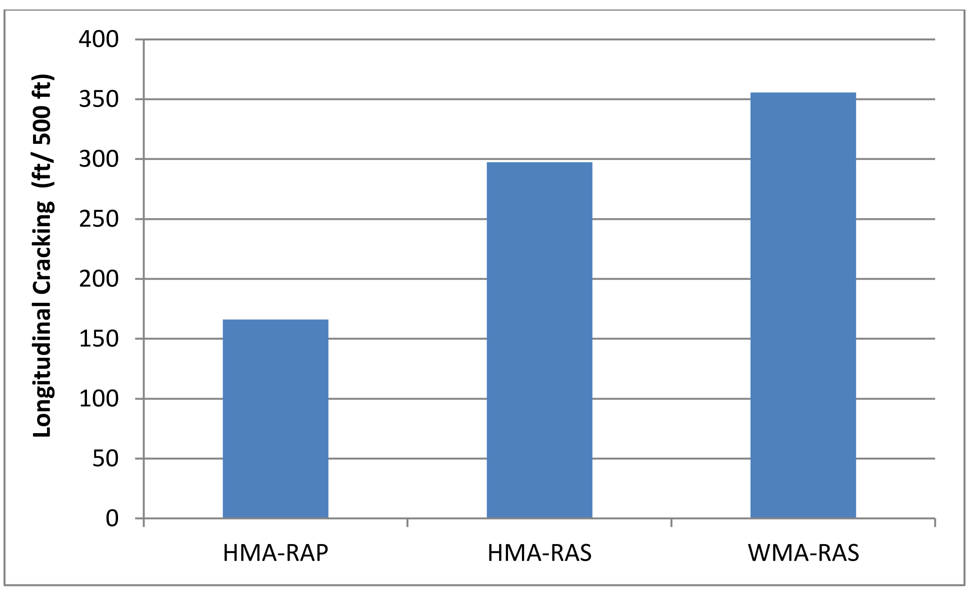


Figure D8.11. Longitudinal/block cracking adjacent to white striping (ft/500 ft) (March 2012)

Low severity raveling was documented in all test sections. Power-Point Presentations of the distress surveys by 500-foot sections are available for viewing on the TPF-5(213) website.

D9. Conclusions

An Indiana DOT demonstration project was conducted as part of Transportation Pooled Fund 5-213 to evaluate the performance of asphalt pavements with 15% RAP compared to pavements with 3% RAS - with or without foaming warm mix technology. Three asphalt mix designs were evaluated, a control mixture containing 15% RAP and no RAS, a mixture containing 3% RAS and no RAP, and a mixture containing 3% RAS and no RAP produced with foaming warm mix technology. Field mixes of each pavement were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and characterization. The results of the study are summarized below:

- Observations from the demonstration project show that the contractor successfully produced and constructed the HMA-RAS and WMA-RAS pavements while meeting INDOT’s quality assurance requirements.
- The performance grade of the blended binder extracted from asphalt mixtures slightly increased when 3% RAS replaced 15% RAP. The continuous PG for the HMA-RAP mixture was 75.6-20.1, while the HMA-RAS mixture was 77.6-14.2 and the WMA-RAS mixture was 78.8-15.1.
- The RAP and RAS mixtures have similar dynamic modulus values across a wide frequency range, likely due to the similar stiffness properties of the blended binders for the different mixtures. The dynamic modulus results show that the WMA technology did not change the dynamic modulus of the mixture.
- In the flow number test, all three mixtures had high flow numbers, and therefore should be very resistant to permanent deformation. Higher flow numbers in the RAS mixtures though, indicate that mixtures with 3% RAS will be more rut resistant than mixtures with 15% RAP. The test results also show that adding foaming WMA technology did not change the mixture’s resistance to permanent deformation.
- The four-point bending beam results showed that the HMA with RAS has similar fatigue properties as the HMA with RAP. Additionally, the WMA mixture with RAS performed the same as the other mixes.
- The SCB test was performed to measure the low temperature cracking susceptibility of the mixtures by measuring their fracture energy at -6°C, -12°C, -18°C, and -22°C. When 15% RAP was replaced with 3% RAS, the fracture energy decreased from 551 to 502 J/m^2 although the difference was not statistically significant. Overall, the results of the SCB test did not detect any difference in low temperature cracking performance when either RAS or WMA technology was used in the mixtures.
- Field condition surveys conducted one, two, and three years after the demonstration project revealed that all three pavement sections are susceptible to transverse cracking, longitudinal cracking, and fatigue cracking. The transverse cracking is most likely caused by differential movement of the concrete and HMA pavement below the asphalt overlay. The RAS pavement sections displayed a greater amount of distress than the RAP pavement sections. However, the Control pavement sections exhibited the greatest percentage of transverse

cracking with a moderate or greater severity level (65%). In contrast, the transverse cracking exhibited in the HMA-RAS and WMA-RAS pavement sections contained 60.5% and 28%, respectively, cracks with a moderate or greater severity level.

D10. INDOT Demonstration Project Acknowledgments

The researchers gratefully acknowledge the support of Mike Prather at the Indiana DOT. The research work was sponsored by Federal Highway Administration (FHWA) TPF-5(213) and the Transportation Pooled Fund (TPF) partners: Missouri (lead agency), California, Colorado, Illinois, Indiana, Iowa, Minnesota, and Wisconsin DOTs.

APPENDIX E. REPORT FOR THE WISCONSIN DEPARTMENT OF TRANSPORTATION SPONSORED DEMONSTRATION PROJECT

E1. Introduction

This report presents a summary of the results obtained from the field demonstration project sponsored by the Wisconsin Department of Transportation (WisDOT) as part of Transportation Pooled Fund (TPF) 5-213 *Performance of Recycled Asphalt Shingles in Hot Mix Asphalt*. TPF 5-213 is a partnership of several state agencies with the goal of researching the effects of recycled asphalt shingles (RAS) on the performance of asphalt applications. As part of the pooled fund research program, multiple field demonstration projects were conducted to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements that include RAS.

Each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated different aspects of asphalt mixes containing RAS. The field demonstration project sponsored by WisDOT investigated the effect of using Evotherm® warm mix asphalt technology as a compaction aid in hot mix asphalt (HMA) containing post-consumer RAS. The objective of this demonstration project was to evaluate the performance of a typical WisDOT mix design containing RAS, with and without Evotherm®, at hot mix production and compaction temperatures during late season construction (November).

E2. Experimental Plan

To evaluate the performance of HMA with RAS and Evotherm®, WisDOT designed an experimental plan to address the following questions:

- How is the performance of HMA containing RAS affected when Evotherm® is used as a compaction aid?
- Will using Evotherm® affect the laboratory performance of the mixture?

The experimental plan is presented in Table E2.1. The plan was implemented during the demonstration project by producing two asphalt mixtures: a mixture with Evotherm® and a mixture with no Evotherm®. Both mixtures contained 3% post-consumer RAS and 13% fractionated recycled asphalt pavement (FRAP).

Table E2.1. Experimental plan

Mix ID	% RAS	% FRAP	RAS Source	WMA Technology
Evo	3	13	Post-Consumer	Evotherm®
No Evo	3	13	Post-Consumer	None

During production of the asphalt mixtures, Iowa State University obtained samples of each mixture for laboratory testing. A portion of the samples were sent to the University of Minnesota and the Minnesota Department of Transportation (MnDOT) for Semi-Circular Bend (SCB) testing and binder extraction and recovery, respectively. The laboratory testing plan is presented in Table E2.2.

The asphalt was recovered from the RAS and asphalt mixtures following AASHTO T164 Method A (Centrifuge Method) by using a blend of toluene and ethanol as the extraction solvent. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. At Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO R29 "Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder."

Table E2.2. Laboratory testing plan

	Laboratory Test	Iowa State University	University of Minnesota	Minnesota DOT
Processed Shingles	Binder Extraction			X
	High Temperature PG			X
	Gradation (Before Extraction)	X		
	Gradation (After Extraction)	X		
Mixture	Binder Extraction			X
	Binder PG Characterization	X		
	Gradation	X		
	Dynamic Modulus	X		
	Flow Number	X		
	Beam Fatigue	X		
	Semi-Circular Bending (SCB)		X	

Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution, and a washed gradation was conducted after extraction to evaluate the size distribution of the fine aggregates in the RAS product.

Performance testing was completed on the field produced asphalt mixtures at low, intermediate, and high temperatures. Dynamic Modulus and Flow Number were conducted at high temperatures to evaluate the modulus and rutting resistance of asphalt mixtures. The durability of the mixtures at intermediate temperatures was evaluated using the dynamic modulus and four-point beam fatigue test. The SCB test conducted at the University of Minnesota evaluated the fracture properties of the mixtures at low temperatures.

After construction of the pavement for the demonstration project, field evaluations were conducted on each pavement test section after one winter season after paving to assess the field performance of the pavement concerning cracking, rutting, and raveling.

E3. Project Location

The field demonstration project was completed on State Trunk Highway (STH) 144 northeast of West Bend, Wisconsin in Washington County located in the southeast corner of the state. The test sections were placed on the northbound (NB) and southbound (SB) lanes of STH 144, a two-lane highway. The project starts in the City of Barton at Station 885+49 and moves north approximately 8 miles, ending at the Washington County line at Station 1328+14.1. The project limits are identified below in Figure E3.1.

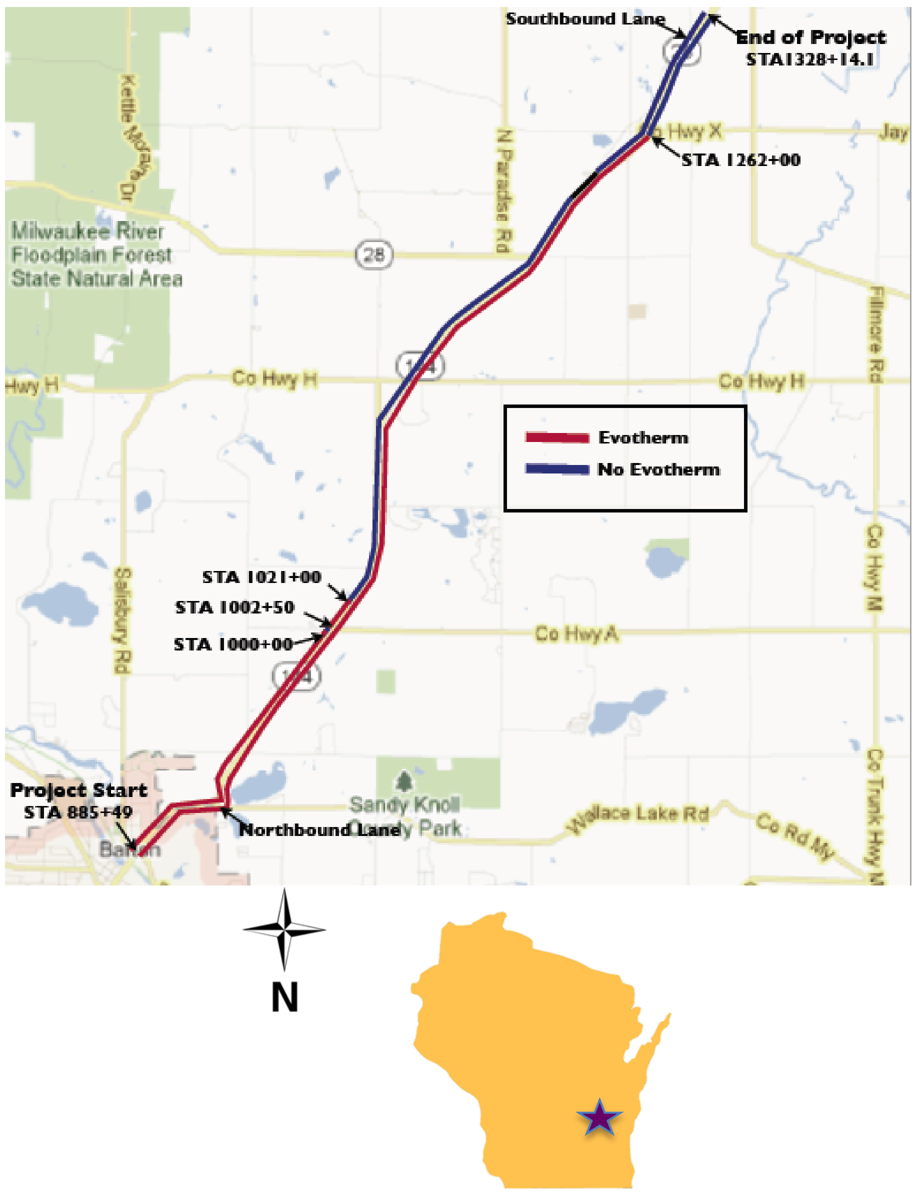


Figure E3.1. Project location (STH 141)

E4. Project Description

The demonstration project was conducted by Payne & Dolan, Inc. (Payne & Dolan) in October and November of 2011. The existing pavement structure on STH 144 consisted of two different profiles: six inches of HMA over 12 inches of dense base aggregate and 4.5 inches of HMA over eight inches of concrete. For the demonstration project, 1.5 inches of existing HMA was milled and resurfaced with 2.75 – 3.25 inches of an experimental HMA leveling course (lower layer) containing RAS and FRAP. A two inch surface course (upper layer) was scheduled to be placed over the leveling course during the 2012 construction season. Only the leveling course mix is included in the scope of the demonstration project. Cross-sections of the pavement designs are shown in Figures E4.1a – E4.1c.

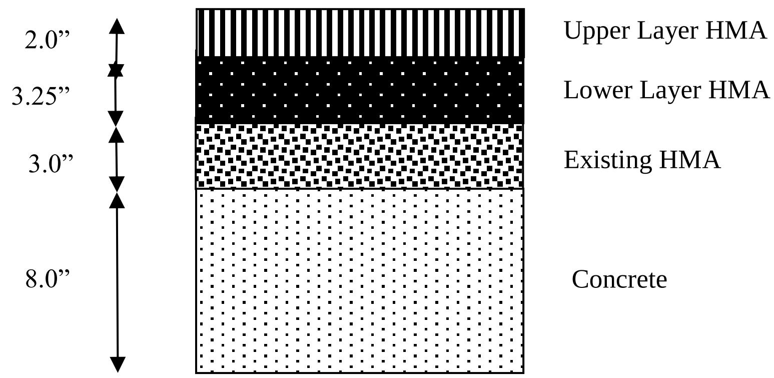


Figure E4.1a. Pavement resurfacing cross-section West Bend to CTH “A” (STA. 885+49 to STA. 1006+04)

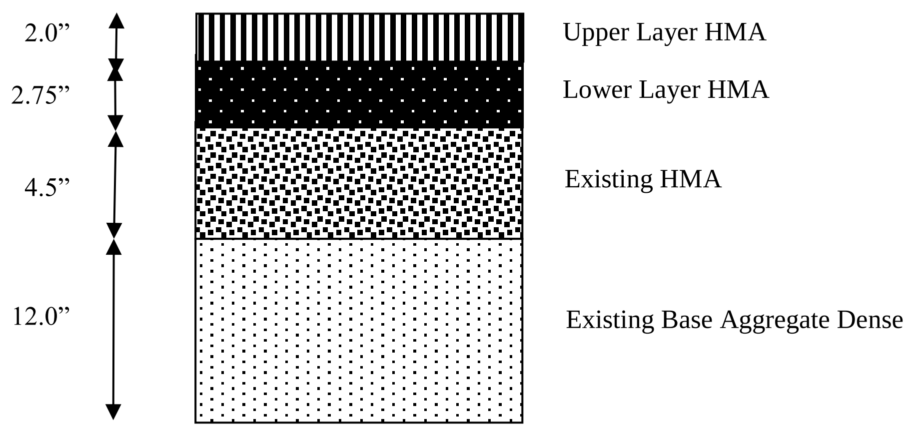


Figure E4.1b. Pavement resurfacing cross-section CTH “A” to North County (STA. 1006+04 to STA. 1328+14)

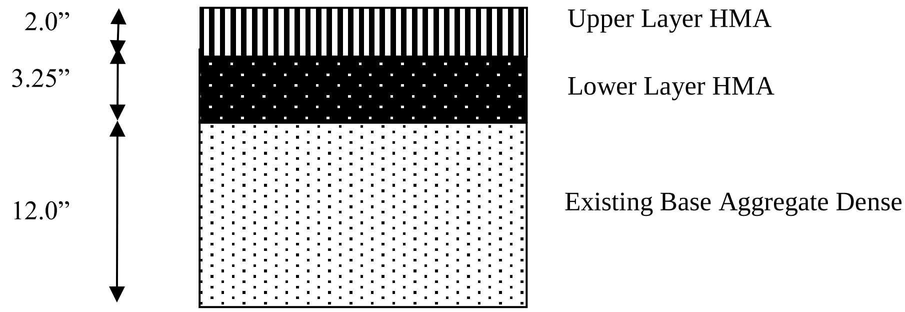


Figure E4.1c. Pavement reconstruction cross-section (STA. 907+25 to STA.921+00 and STA. 977+55 to STA. 999+00)

A plan view of the test sections on STH 144 is shown in Figure E4.2. Test sections with no Evotherm® were paved in the northbound lane from Station (STA) 1262+00 to the end of the project (STA 1328+14.1) and in the southbound lane from STA 1000+00 to 1002+50 and 1021+00 to the end of the project. Test sections with Evotherm® were placed in the northbound lane from the start of the project (STA 885+49) to STA 1262+00 and in the southbound lane from the start of the project to STA 1000+00 and from STA 1002+50 to STA 1021+00. Approximately 9.5 lane miles were paved with Evotherm® and 7.5 miles were paved without Evotherm®, for a total of 17.0 lane miles.

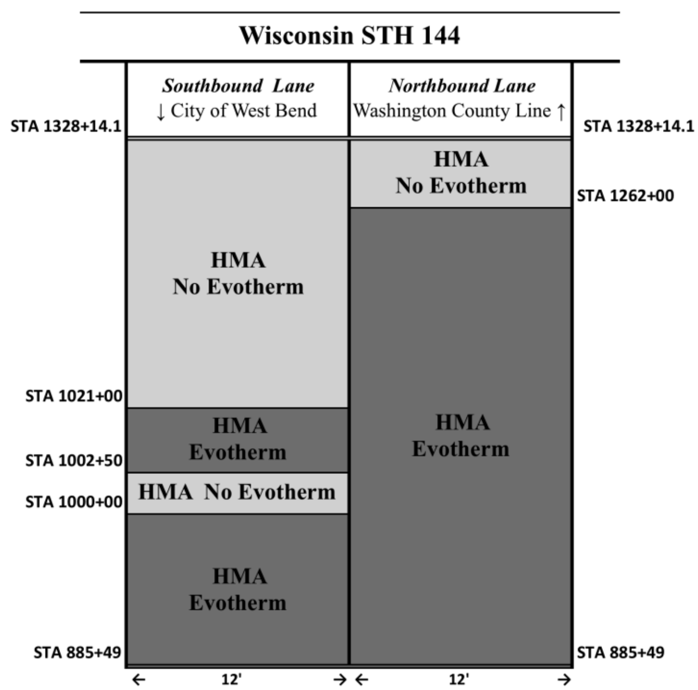


Figure E4.2. Plan view of Wisconsin STH 144 project test sections

E5. HMA Production and Shingle Processing

Payne and Dolan used a portable drum asphalt plant to produce the HMA (Figures E5.1 - E5.2). The plant was located northeast of Campbellsport, Wisconsin off of State Highway 45. The haul distance to the project test sections was approximately 15 miles. Production temperatures averaged 315 degrees Fahrenheit with a plant capacity to produce 300 tons HMA per hour.

Figure E5.1. Payne and Dolan portable plant

Figure E5.2. Evotherm® meter attachment to asphalt tank

During the project, Payne and Dolan produced 24,950 tons of HMA without Evotherm® and 14,389 tons of HMA with Evotherm®, for a total of 39,339 tons. Approximately 20,000 tons were placed on the mainline and 19,300 tons on the shoulders, side roads and intersections. 180 tons of RAS and 3,923 tons of FRAP were utilized for the project. Tonnages of RAS, FRAP and total HMA for each test section are summarized below in Table E5.1.

Table E5.1 Project tonnages

Material	No Evotherm (Tons)	Evotherm (Tons)
RAS	432	749
FRAP	1,619	2,304
Total HMA	14,389	24,950

Samples of both types of HMA mixes were obtained on November 21, 2011 when Payne and Dolan paved 3,633 tons of mix using Evotherm® and 116 ton of mix without Evotherm®. The samples were shipped to Iowa State University for laboratory testing.

Weather conditions during the paving of the test sections ranged in temperatures from 19 to 62 degrees Fahrenheit. During paving of the HMA with Evotherm®, the mean temperatures ranged 28 to 43 degrees Fahrenheit; during paving of the HMA without Evotherm®, the mean temperatures ranged from 32 to 51.

The HMA mix design used for the project contained a combination of RAS and FRAP, both of which Payne and Dolan crushed and processed. Payne and Dolan collected post-consumer shingles and processed them in a grinder to produce a final product with essentially a minus 3/8" material (99% was passing the 3/8" sieve). The RAP was fractionated to pass the 3/4" screen. A picture of the RAS stockpile used for the project is shown in Figure E5.3. The asphalt content and gradation test results of the RAS before and after extraction, and of the FRAP after extraction, are presented in Table E5.2. Asphalt extracted from the RAS by MnDOT was measured to be 35.4%.

Figure E5.3. Recycled asphalt shingles (RAS) stockpile

Table E5.2. RAS and RAP gradations (percent passing)

Sieve Size (US)	Sieve Size (mm)	RAS (Before Extraction)	RAS (After Extraction)	RAP (After Extraction)
3/4"	19	100	100	100
1/2"	12.5	100	100	97
3/8"	9.5	99	99	91
#4	4.75	83	99	73
#8	2.36	70	89	53
#16	1.18	47	71	38
#30	0.6	24	47	29
#50	0.3	11	39	23
#100	0.15	3	31	18
#200	0.075	0.6	23.0	13.3
	% Asphalt Content		35.4	4.1

E6. Asphalt Mix Design and Prodution Results

The project mix design followed WisDOT specifications for a lower layer E-3 HMA designed for one to three million single equivalent axel loads (ESALs) with a nominal maximum aggregate size (NMAS) of 19.0 mm (Table E6.1). Gradations obtained from laboratory testing of the asphalt mixture with and without Evotherm® are presented in Figure E6.1 on a 0.45 power chart. As shown in the figure, the asphalt mixtures share the same aggregate structures with gradations passing above the restricted zone.

Table E6.1. Mixture design properties

Mix Property	Value
Design Gyrations	75
NMAS (mm)	19.0
Virgin PG Grade	58-28
% Voids	4.0
% VMA	13.5
% VFA	70.4
-#200/Pbe	1.1

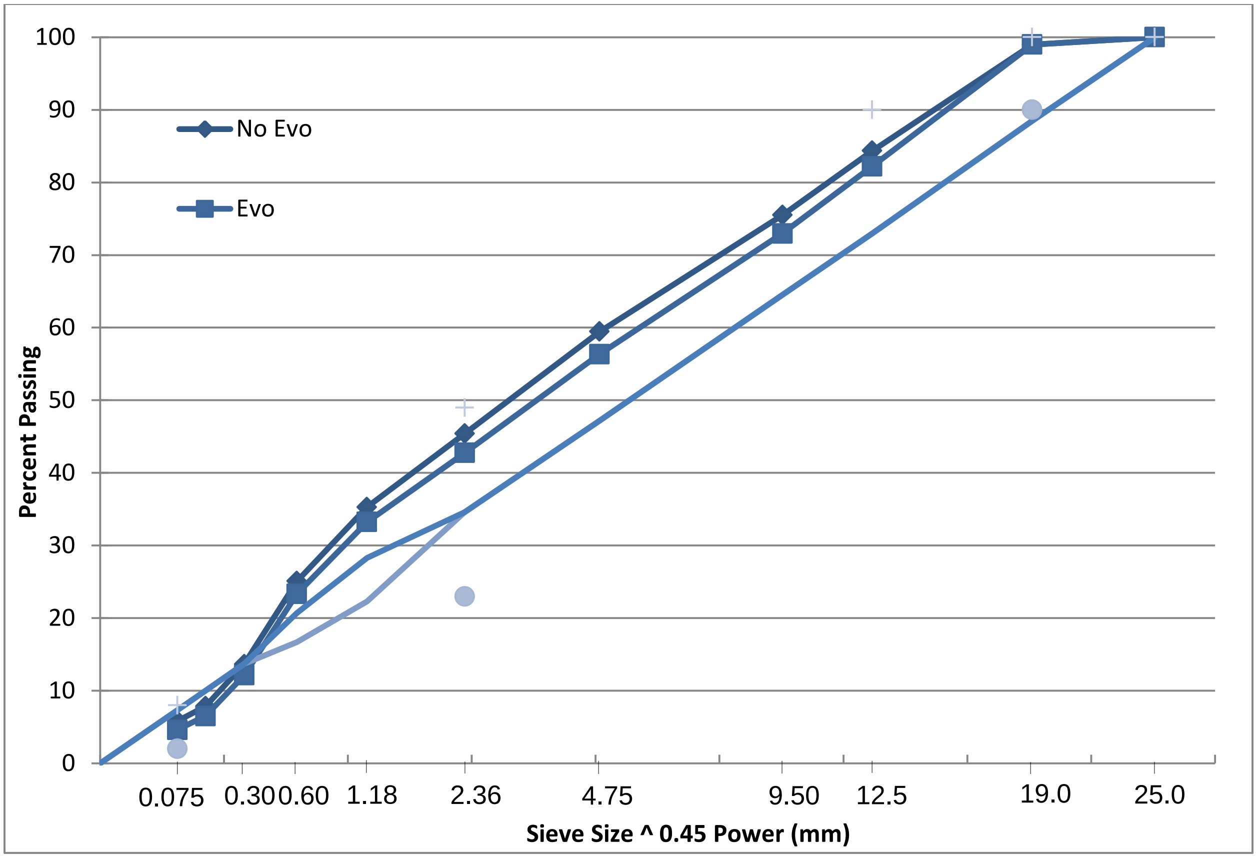


Figure E6.1. Asphalt mix design gradations

The amount of recycled products in the mix design is presented in Table E6.2. WisDOT specifications allow a mix design for a lower layer to have up to 35 percent binder replacement when RAS and RAP are used in combination (with a maximum of five percent RAS). During the development of the mix design, the RAS contained 29.1 percent asphalt and the RAP contained 4.1 percent asphalt that contributed to the total asphalt in the mix. This resulted in a 30.4% binder replacement when 3% RAS and 13% RAP were added to the mix design. The optimum asphalt content is 4.60 percent.

Table E6.2. Amount of recycled materials in the mix design

Mix Property	Value
% RAS	3
% RAP	13
% Total binder	4.60
% Virgin binder	3.20
% Binder Replacement	30.4

Payne and Dolan successfully produced the HMA, with and without Evotherm®, within WisDOT specifications. Table E6.3 shows the average of the quality control results for the HMA mix produced with Evotherm®. The asphalt content, laboratory voids, and voids in the mineral aggregate (VMA) were close to target design values. Asphalt contents were slightly higher than the target value of 4.6 percent. The MnDOT extraction test results on the mixes sampled on 11/21/2011 confirm this, since the measured asphalt content for the Evotherm® and the non-Evotherm® mix was 4.7 and 4.8 percent, respectively.

Table E6.3. Quality control results of HMA with Evotherm®

Warm Mix Additive	Date Sampled	Asphalt Content		Lab Voids		VMA	
		Target	QA Results(1)	Target	QA Results(1)	JMF Value	QA Results(1)
Evotherm®	10/27/2011	4.6	4.7	4.0	3.5	13.5	14.8
	10/28/2011		4.8		4.1		15.2
	10/29/2011		4.8		3.9		15.0
	11/04/2011		4.9		4.1		14.9
	11/21/2011		4.8		3.3		13.9
	11/22/2011		4.8		3.6		14.3
	11/23/2011		4.8		4.2		14.5

(1) Average of contractor control and quality verification tests for each day's production

Pavement density results obtained after compaction with a nuclear density gauge are presented in Table E6.4. In the table, the density results of the HMA mix with no Evotherm® is delinated from the density results of the HMA containing Evotherm®. Payne and Dolan satisfied the WisDOT density specifications for each mixture.

The WisDOT specification state that asphalt paving is not allowed after October 15th or if the outside temperature is below 36°F without approval from the Engineer. Daily mean temperatures during paving included in Table E6.4 show the temperatures, particularly in late November, were in the mid to low 30's (°F). During these days, Payne and Dolan used Evotherm® and they were able to achieve the minimum density requirement.

Table E6.4. Pavement density summary

Warm Mix Additive	Paving Date	Mainline: Minimum % Density Req'd	Mainline: % Density(1)	Edge of Pavement: Minimum % Density Req'd	Edge of Pavement: % Density(1)	Daily Mean Temp (°F)
No Evotherm®	10/26/2011	91.5	92.3	89.5%	91.3	42
	10/31/2011		93.8		92.5	39
	11/1/2011		94.0		91.9	46
	11/3/2011		94.0		92.1	39
Evotherm®	10/27/2011		93.3		92.0	40
	10/28/2011		93.9		93.0	40
	10/29/2011		93.5		92.2	41
	11/4/2011		93.8		92.2	38
	11/21/2011		93.9		92.3	34
	11/22/2011		92.9		91.1	34
	11/23/2011		93.7		92.2	37
	11/28/2011		92.2(2)		-	29
	11/29/2011		92.1(2)		-	30
	12/2/2011		93.0(2)		-	29

(1) Lot average of nuclear gauge density tests
(2) Side roads and shoulders

E7. Laboratory Test Results

Binder Testing

Performance Grade (PG) testing of the extracted binders was conducted to obtain their high, low, and intermediate PG temperatures as shown in Table E7.1. The high temperature performance grade of the RAS binder at 124.1°C is higher than a traditional paving grade binder. This is expected since the binder in roofing shingles is produced with an air-blowing process which oxidizes the asphalt. Additionally, the RAS used in the mix designs is from post-consumer shingles, so the binder in the RAS has experienced at least several years of aging.

A 58-28 virgin binder was used for the project and combined with recycled binder from the RAS and FRAP materials. Because the HMA samples are heated to high temperatures and placed in a centrifuge at high speeds during the recovery process, the RAS, FRAP, and virgin asphalt should be fully blended. Adding 3% RAS and 13% FRAP for a combination of 30.4 percent replaced binder raised the PG of the HMA with no Evotherm® from a continuous PG of 60.7-29.1 to a continuous PG of 68.5-27.4 resulting in one grade bump on the high and low side (64-22). The PG of the HMA with Evotherm® was very similar with a continuous PG of 69.5-25.9 also resulting in one grade bump on the high and low side (64-22). These results indicate that adding Evotherm® to HMA will not have a large impact on the binder grade.

By bumping the grade from a 58-28 to a 64-22, the addition of the RAS and FRAP binders to the HMA did not have a large impact on the total HMA mixture's performance grade. The increase from a 58 to a 64 on the high side will help increase the rutting resistance of the HMA. While the increase from a -28 to -22 on the low side decreases the low temperature cracking resistance of the mix, having only a single grade bump on the low side is excellent for a 30.4 percent binder replacement in the mix. Since this mix is a lower layer HMA in the pavement structure, the effects of the grade bump will be less significant on pavement performance. In-situ temperatures will be less than the ambient air temperature at the surface of the pavement.

Table E7.1. Performance grade of extracted binders

Material Identification	High PG Temp, °C	Intermediate PG Temp, °C	Low PG Temp, °C	Performance Grade
Virgin Binder	60.7	18.0	-29.1	58-28
RAS	124.1	-	-	-
Mix Sample (No Evo)	68.5	18.7	-24.0	64-22
Mix Sample (Evo)	69.5	20.3	-22.5	64-22

Dynamic Modulus

The dynamic modulus (|E*|) is a key material property that determines the stress-strain relationship of an asphalt mixture under continuous sinusoidal loading. A higher dynamic modulus indicates lower strains will result in a pavement structure when the asphalt mixture is stressed from repeated traffic loading. The mechanistic-empirical pavement design guide (MEPDG) uses |E*| as the stiffness parameter to calculate an asphalt pavements strains and displacements.

The test was conducted following AASHTO TP62 using three replicate samples of 150 mm in height and 100 mm in diameter. Each sample was compacted to 7 ± 0.5% air voids. Samples were tested by applying a continuous sinusoidal load at 9 different frequencies (0.1, 0.3, 0.5, 1, 3, 5, 10, 20, and 25 Hz) and three different temperatures (4, 21, and 37°C). Sample loading was adjusted to produce strains between 50 and 150 μstrain in the sample.

Master curves were constructed at a reference temperature of 21°C and plotted on a log-log scale for a general comparison as presented in Figure E7.1. Both mixes plot very close to each other over a wide frequency range indicating similar performance between the two mixtures.

The plot in Figure E7.2 presents the mean dynamic modulus at 21°C and 37°C at specific frequencies for a more direct comparison. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. At 21°C and 5 Hz, the mixture with Evotherm® has a statistically lower dynamic modulus than the mixture without Evotherm®. Low modulus values at this temperature are considered desirable in thin asphalt pavements (less than 4") for fatigue cracking resistance. Mixtures with lower stiffness can deform more easily without building up large stresses.

As shown in Figure E7.2, the dynamic modulus at 37°C and 0.1 Hz was analyzed since the modulus of asphalt mixtures at high temperatures and low frequencies is an indicator of rutting resistance. Since there are no statistical differences between the dynamic modulus of the mixtures at this frequency and temperature, the addition of Evotherm® to the HMA did not impact the rutting resistance of the asphalt mixture, based on the dynamic modulus test.

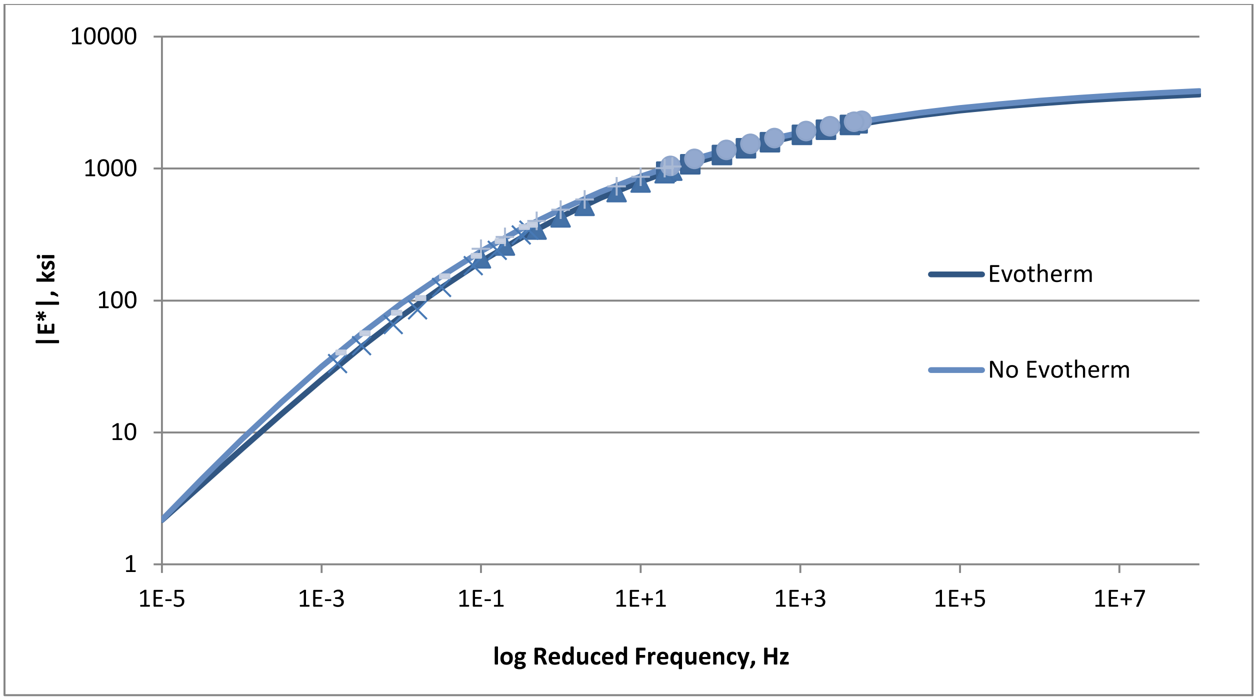


Figure E7.1. Comparison of dynamic modulus master curves

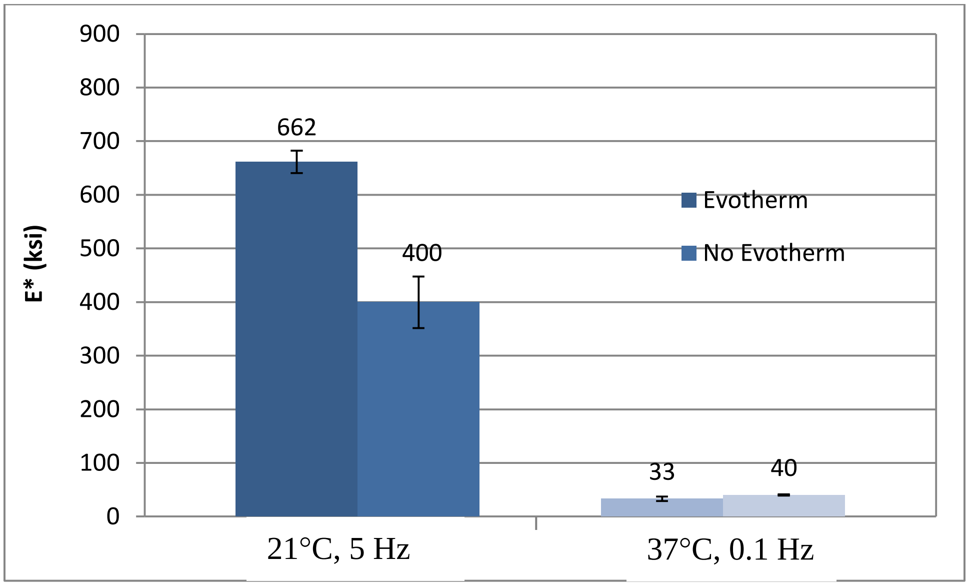


Figure E7.2. Dynamic modulus comparison at 21°C, 5 Hz and 37°C, 0.1 Hz

Flow Number

The flow number test measures the permanent deformation resistance of asphalt mixtures by applying a repeated dynamic load to a sample for up to several thousand load cycles. The flow number is defined as the number of load cycles an asphalt mixture can tolerate until it flows. Cumulative permanent deformation in the sample is plotted versus load cycles. The flow number is reached at the onset of tertiary flow.

Tests were conducted following procedures used in NCHRP Report 465. Samples used in dynamic modulus test were used for the flow number test since the dynamic modulus test is nondestructive. The samples were placed in a hydraulically loaded universal testing machine, unconfined, with a testing temperature of 37°C to simulate the climactic conditions that cause pavement to be susceptible to rutting. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 sec followed by 0.9 sec of dwell time. The loading cycle was repeated for a total of 10,000 load cycles. Three LVDT's were attached to each sample during the test to measure the cumulative strains.

Test results are presented in Figure E7.3. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. If the error bars of two mixtures do not overlap, then the difference of the two mixtures can be considered statistically significant at the 5% level.

The average flow number for the HMA with Evotherm® and without Evotherm was measured as 2462 and 3902, respectively. No statistical differences were measured between the two mixtures. Therefore, based on the flow number test, the addition of Evotherm® did not affect the permanent deformation performance of the mixtures.

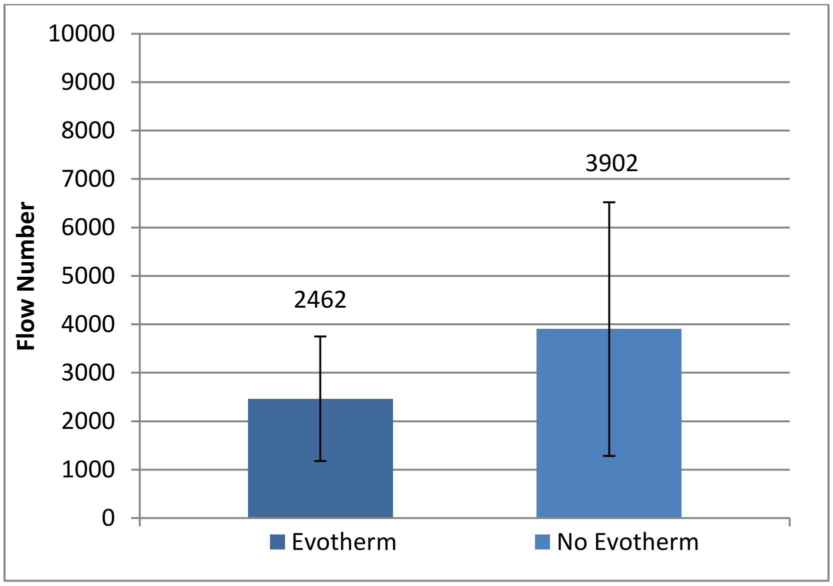


Figure E7.3. Flow number test results

Beam Fatigue

Fatigue cracking is the major cracking distress in asphalt pavements caused by repeated heavy traffic loads. Pavements that have a higher resistance to tensile strains that develop at the bottom of an asphalt layer due to repeated traffic will have a greater resistance to fatigue cracking. The four-point beam fatigue test following AASHTO T321 was conducted to evaluate the load associated cracking resistance of the mixtures.

The mixes were compacted in a linear kneading compactor to create slabs with 7% air voids. The slabs were saw-cut into beams with dimensions 15 inches in length, 2.5 inches in width, and 2 inches in height. The beams were tested in a strain controlled mode of loading at 20°C with haversine wave pulses applied to the beam at 10 Hz. Five beams were tested, each at a different strain level (375, 450, 525, 800, and 1000 μstrain), until the flexural stiffness of the beam was reduced to 50% of the initial stiffness. A log-log regression was performed between strain and the number of cycles to failure (N_f). The relationship between strain and N_f can be modeled using the power law relationship as presented in Equation 1.

$$N_f = K1\left(\frac{1}{\varepsilon_o}\right)^{K2} \quad (1)$$

where: N_f = cycles to failure; ε_o = flexural strain; and K1 and K2 = regression constants.

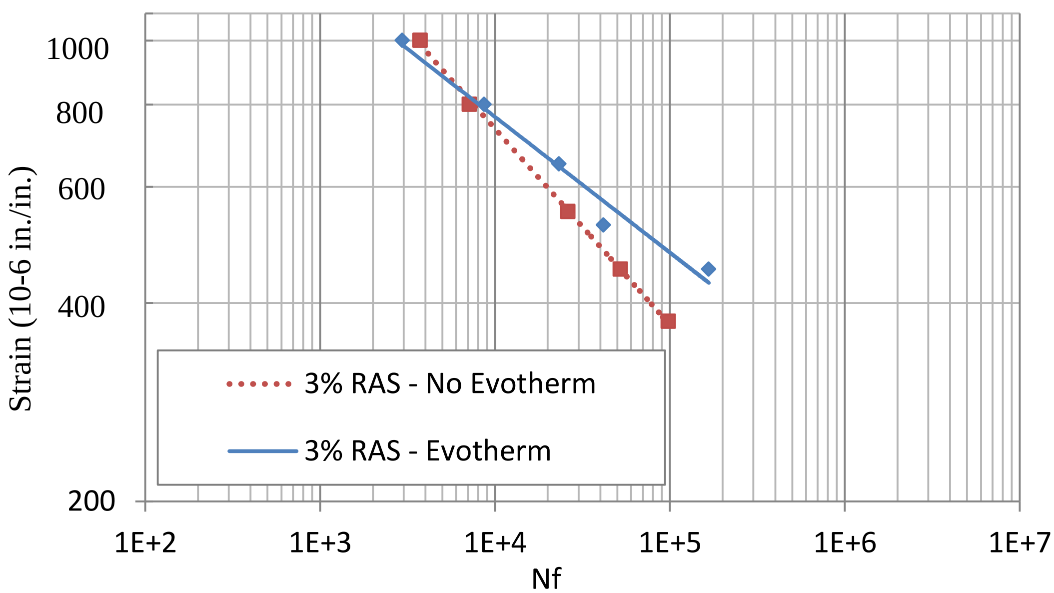


Figure E7.4. ε-N fatigue curves

The beam fatigue test results, as shown by strain versus "loading cycles to failure" curves (Nf), are presented in Figure E7.4. The fatigue curve model coefficients, average initial stiffness, and R^2 values are presented in Table E7.4. The HMA with Evotherm® exhibits a longer fatigue life than the HMA without Evotherm® at simialar strain levels in a strain-controlled mode of loading. This indicates that Evotherm increases the fatigue life of HMA for a thin lift pavement.

The fatigue endurance limit (FEL) of each mixture was also predicted. If tensile strains are low enough in a pavement structure, the pavement has the ability to heal and therefore no damage cumulates over an indefinite number of load cycles. The level of this strain is referred to as the FEL. The FEL of each mixture was estimated using the lower 95% prediction limit at 50 million load cycles as proposed in NCHRP Report 646 and shown in Equation 2.

$$\text{Lower Prediction Limit} = \hat{y}_o - t_\alpha s\sqrt{1+\frac{1}{n}+\frac{(x_o-\bar{x})^2}{S_{xx}}} \tag{2}$$

where:
y_o = the one-sided lower 95% prediction interval at the micro-strain level corresponding to 50,000,000 cycles;
t_α = value of t distribution for n-2 degrees of freedom for a significance level of 0.05;
s = standard error of the regression analysis;
n = number of samples;
S_{xx} = sum of squares of the x values;
x_o = log 50,000,000; and
$\bar{x}$ = average of the fatigue life results.

The FEL estimates are also presented in Table E7.2. The HMA with Evotherm® has a higher and thus more desirable endurance limit, indicating that Evotherm® may improve FEL in the mixture.

Table E7.2. Beam fatigue results

Mix ID	% Binder Replacement	Average Initial Stiffness (Mpa)	K1	K2	R^2	Endurance Limit (Micro-strain)
Evo	30.4	2950	1.70E-11	-4.74	0.976	74
No Evo	30.4	2992	3.75E-10	-4.32	0.984	53

Semi-Circular Bending

The low temperature fracture properties of the mixtures were obtained from SCB tests by following the procedure in “Investigation of Low Temperature Cracking in Asphalt**”** **(**Marasteanu et al., 2007). Testing was conducted at four different low temperatures: -12°C, -18°C, -24°C, and -28°C.

All tests were performed inside an environmental chamber, and liquid nitrogen was used to obtain the required low temperature. The temperature was controlled by the environmental chamber temperature controller and verified using an independent platinum resistive-thermal-device (RTD) thermometer. The load line displacement (LLD) was measured on both faces of the test specimens using a vertically mounted Epsilon extensometer with 38 mm gage length and ±1 mm range. One end was mounted on a button that was permanently fixed on a specially made frame, and the other end was attached to a metal button glued to the sample. The average LLD measurement was used for each specimen. The crack mouth opening displacement (CMOD) was recorded by an Epsilon clip gage with 10 mm gage length and a +2.5 and -1.0 mm range. The clip gage was attached at the bottom of the specimen. A constant CMOD rate of 0.0005mm/s was used and the load and load line displacement (P-u), as well as the load versus LLD curves were plotted. A contact load with maximum load of 0.3 kN was applied before the actual loading to ensure uniform contact between the loading plate and the specimen. The testing was stopped when the load dropped to 0.5 kN in the post peak region. The load and load line displacement data were used to calculate the fracture toughness and fracture energy (G_f).

The fracture energy (G_f) parameter is presented in Figure E7.5. The laboratory test results were analyzed to evaluate the effect of the various RAS treatments on the fracture energy. MacAnova statistical software package was utilized to perform a statistical analysis. The analysis of variance (ANOVA) was used to examine the differences among the mean response values of the different treatment groups. The significance of the differences was tested at 0.05 level of error. The analysis of variance was conducted with the assumption that the errors in the data are independently normal with constant variance.

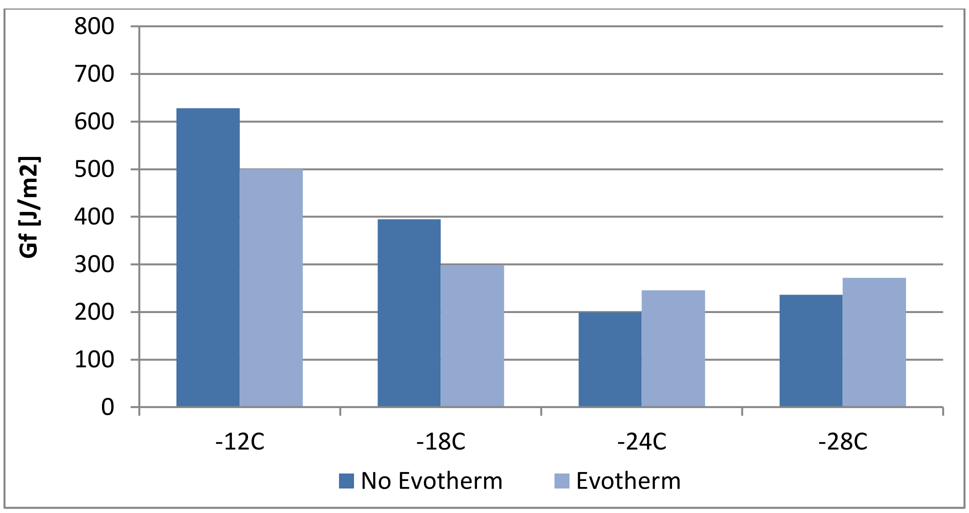


Figure E7.5. Iowa mixture fracture energy (G_f)

The Gf group means of each RAS treatment level was compared using a pair-wise comparison to rank the RAS treatment levels with regard to fracture energy. The outcome is reported in Table E7.3, in which statistically similar RAS treatments are grouped together. Letter A indicates the best performing group of mixtures; letter B the second best, and so on. Groups with the same letter are not statistically different, whereas mixtures with different letters are statistically different.

When Evotherm® was added to the HMA as a compaction aid, the fracture energy did not change. While the Evotherm® mixture did have a lower fracture energy (329 J/m^2) than the non-Evotherm® mixture (364 J/m^2), the difference was not statistically significant. Although not statistically significant at the 95% confidence level, these results do correlate well with the PG of the extracted binders. The low temperature performance grade of the extracted HMA binder containing Evotherm® was higher than the extracted HMA binder not containing Evotherm®, see Table E7.1., thus also indicating slightly lower resistance to cracking at low temperatures.

Table E7.3. Ranking of mixes by G_f group mean for -12, -18, -24, and -28°C temperatures

Rank	Treatment	Group mean G_f [J/m^2]
A	Evotherm	329
A	No Evotherm	364

E8. Field Evaluations

The project team completed one distress survey for the Wisconsin demonstration project test sections in March 2012, prior to the surface mix placement. Three 500-foot sections were randomly selected in each of the test sections: HMA with Evotherm®, HMA with no Evotherm®. Three surveys were completed in the northbound lanes and three surveys were completed in the southbound lanes. The surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* published by the Federal Highway Administration. No distresses were found in any of the sections (Figures E8.4 and E8.5). A PowerPoint presentation of the distress survey is available for viewing on the TPF-5(213) website.

Figure E8.4. Northbound lane HMA with Evotherm® STA 1205+00

Figure E8.5. Southboud lane HMA with Evotherm® STA 1282+00

E9. Conclusions

A WisDOT demonstration project was conducted as part of Transportation Pooled Fund 5-213 to evaluate the effects of adding Evotherm® warm mix asphalt technology as a compaction aid in HMA containing post-consumer RAS. Two asphalt mixtures were evaluated: a mixture with Evotherm® and a mixture with no Evotherm®. Both mixtures contained 3% post-consumer RAS and 13% fractionated recycled asphalt pavement (FRAP). Field mixes of each pavement were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and recovery with subsequent binder characterization. A pavement condition survey of the demonstration project test sections was also conducted after the first winter season after paving. The results of the study are summarized below:

- Observations from the demonstration project show the contractor successfully produced and constructed the HMA with a combination of post-consumer RAS and FRAP (30.5% binder replacement) during late season paving by using Evotherm® as a compaction aid. The contractor met WisDOT's quality verification requirements on mix properties and pavement density.
- The performance grade of the total binder in the asphalt mixtures increased from a PG 58-28 to a PG 64-22 with the addition of RAS and FRAP. The addition of Evotherm® did not significantly impact the HMA performance grade. Asphalt extracted from the mixture with Evotherm® had a continuous PG of 69.5-25.9, and asphalt extraction from the mixture without Evotherm® had a continuous PG 68.5-27.9.
- Adding Evotherm® to the HMA did not affect the mixture's rutting resistance based on the dynamic modulus and flow number test results. Both laboratory tests indicated similar stiffness and permanent deformation resistance at high temperatures
- Adding Evotherm® to the HMA improved its fatigue properties and slightly increased its fatigue life in a controlled strain mode of loading, based on the four-point bending beam test and the dynamic modulus at intermediate temperatures. The predicted fatigue endurance limit, from the four-point bending beam test, of the HMA with Evotherm® was lower than the HMA without Evotherm®. Likewise, the dynamic modulus of the Evotherm® mixture at intermediate temperatures was measured to be statistically lower than the dynamic modulus of the mixture without Evotherm®, which may also improve the fatigue properties of the mixture.
- The SCB test was conducted to measure the low temperature cracking susceptibility of the mixtures by measuring their fracture energy at -12°C, -18°C, -24°C, and -28°C. The two HMA mixtures exhibited similar low temperature cracking resistance as there was no statistical difference in fracture energy between the two mixtures. Evotherm® did not impact the fracture energy of the HMA.
- Field condition surveys conducted one winter season after the demonstration project revealed no pavement distresses in test section.

E10. WisDOT Demonstration Project Acknowledgments

The researchers gratefully acknowledge the support of Judith Ryan at the Wisconsin DOT. The research work was sponsored by Federal Highway Administration (FHWA) TPF-5(213) and the Transportation Pooled Fund (TPF) partners: Missouri (lead agency), California, Colorado, Illinois, Indiana, Iowa, Minnesota, and Wisconsin DOTs.

APPENDIX F. REPORT FOR THE COLORADO DEPARTMENT OF TRANSPORTATION SPONSORED DEMONSTRATION PROJECT

F1. Introduction

This report presents a summary of the results obtained from the field demonstration project sponsored by the Colorado Department of Transportation (CDOT) as part of Transportation Pooled Fund (TPF) 5-213 *Performance of Recycled Asphalt Shingles in Hot Mix Asphalt*. TPF 5-213 is a partnership of several state agencies with the goal of researching the effects of recycled asphalt shingles (RAS) on the performance of asphalt applications. As part of the pooled fund research program, multiple field demonstration projects were conducted to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements that include RAS.

Each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated different aspects of asphalt mixes containing RAS. The field demonstration project sponsored by CDOT investigated the economic and performance benefits when replacing recycled asphalt pavement (RAP) with RAS in hot mix asphalt (HMA). The objective of this demonstration project was to compare a typical CDOT mix design that contains 20 percent RAP to a mix design that contains 15 percent RAP and 3 percent post-manufactured RAS.

F2. Experimental Plan

To evaluate the performance of HMA with post-manufactured RAS and RAP, CDOT designed an experimental plan to address the following questions:

- Can a quality HMA product containing post-manufactured RAS be produced and placed in Colorado and meet CDOT construction specifications?
- How will replacing five percent RAP with three percent RAS affect the performance of the HMA?

The experimental plan is presented in Table F2.1. The plan was implemented during the demonstration project by producing two asphalt mixtures: a mixture with only RAP and a mixture with RAS and RAP.

Table F2.1. Experimental plan

Mix ID	% RAP	% RAS	RAS Source
RAP Only	20	0	---
RAS/RAP	15	3	post-manufactured

During production of the asphalt mixtures, Iowa State University collected samples of each mixture for laboratory testing. A portion of the samples were sent to the University of Minnesota and the Minnesota Department of Transportation (MNDOT) for Semi-Circular Bend (SCB) testing and binder extraction and recovery, respectively. The laboratory testing plan is presented in Table F2.2.

The asphalt was recovered from the RAS and asphalt mixtures following AASHTO T164 Method A (Centrifuge Method) by using a blend of toluene and ethanol as the extraction solvent. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. At Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO R29 "Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder."

Table F2.2. Laboratory testing plan

	Laboratory Test	Iowa State University	University of Minnesota	Minnesota DOT
Processed Shingles	Binder Extraction			X
	High Temperature PG			X
	Gradation (Before Extraction)	X		
	Gradation (After Extraction)	X		
Mixture	Binder Extraction			X
	Binder PG Characterization	X		
	Gradation	X		
	Dynamic Modulus	X		
	Flow Number	X		
	Beam Fatigue	X		
	Semi-Circular Bending (SCB)		X	

Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution, and a washed gradation was conducted after extraction to evaluate the size distribution of the fine aggregates in the RAS product. Performance testing was completed on the field produced asphalt mixtures at low, intermediate, and high temperatures. Dynamic Modulus and Flow Number were conducted at high temperatures to evaluate the modulus and rutting resistance of asphalt mixtures. The durability of the mixtures at intermediate temperatures was evaluated using the dynamic modulus and four-point beam fatigue test. The SCB test conducted at the University of Minnesota evaluated the fracture properties of the mixtures at low temperatures.

After construction of the pavement for the demonstration project, field evaluations were conducted on each pavement test section prior to the first winter season and the following spring after paving to assess the field performance of the pavement concerning cracking, rutting, and raveling.

F3. Project Location

The field demonstration project was completed on US Route 36 (Denver-Boulder Turnpike) south of Boulder, CO, located in the north central region of the state. The test sections were placed on the eastbound (EB) and westbound (WB) lanes of US Route 36. The project started at the intersection of US Route 36 and State Highway 121 in Broomfield, CO, and continued west approximately three miles to the intersection of US Route 36 and South 88th Street. The project limits are identified below in Figure F3.1.

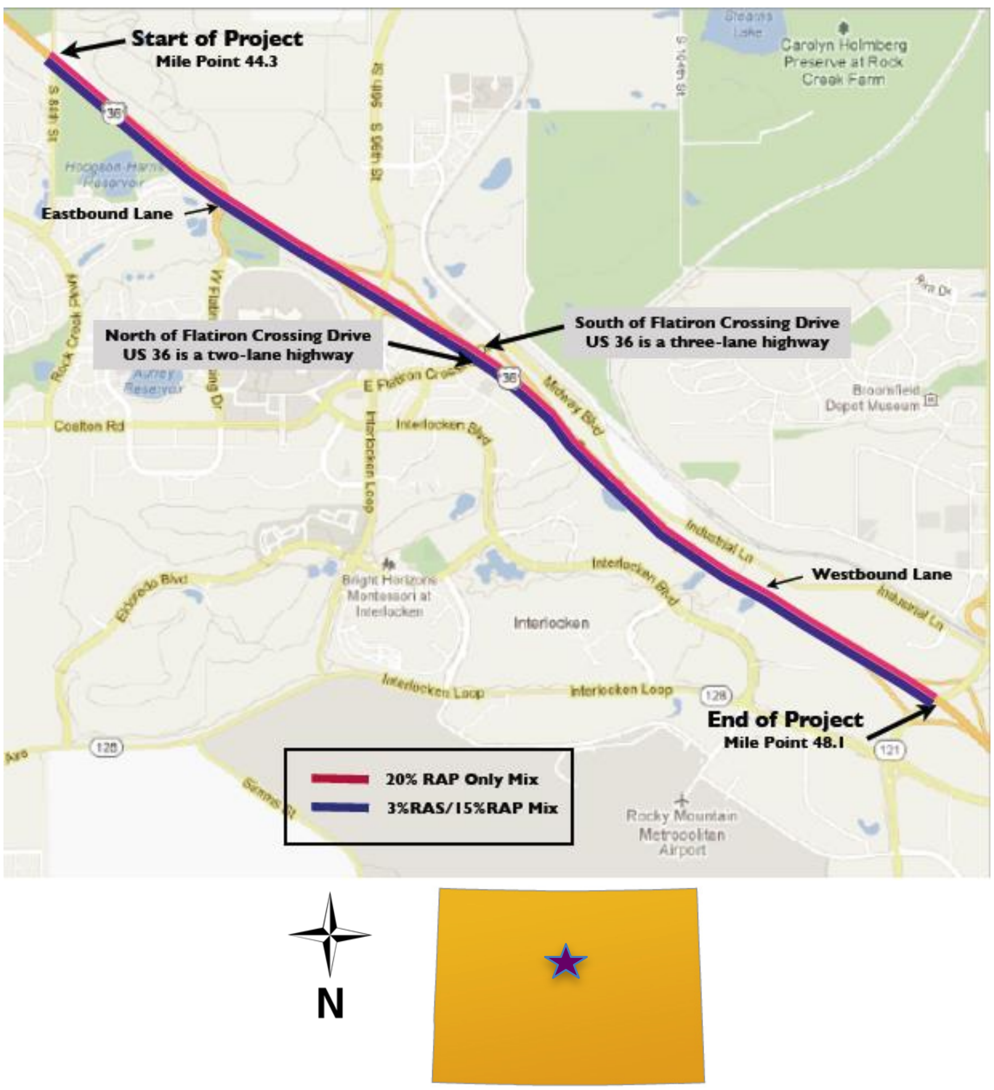


Figure F3.1. Project location (US 36)

F4. Project Description

The demonstration project, CDOT number STA0361-095, was completed by Asphalt Specialties Company, Inc. (Asphalt Specialties). The demonstration project was a temporary placement of HMA prior to a reconstruction project scheduled for 2013-2014 to widen the highway from a four-lane to a six-lane highway. The existing pavement structure consisted of six inches of HMA over concrete pavement. For the demonstration project, two-inches of the HMA was milled and replaced with two-inches of one of the two mix designs. A cross-section is shown in Figure F4.1.

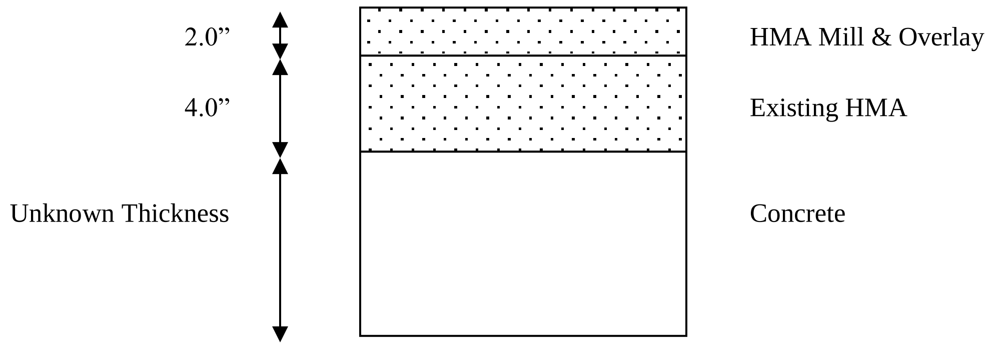


Figure F4.1. Pavement cross-section

Asphalt Specialties milled and placed the surface course test sections in June-August 2011. The RAP only mix was placed on the eastbound lane and the RAP/RAS mix was placed on the westbound lane. US Route 36 is currently a six-lane highway east of Flatiron Crossing Drive and a four-lane highway west of Flatiron Crossing Drive. A plan view of the test sections on US Route 36 is shown in Figure F4.2.

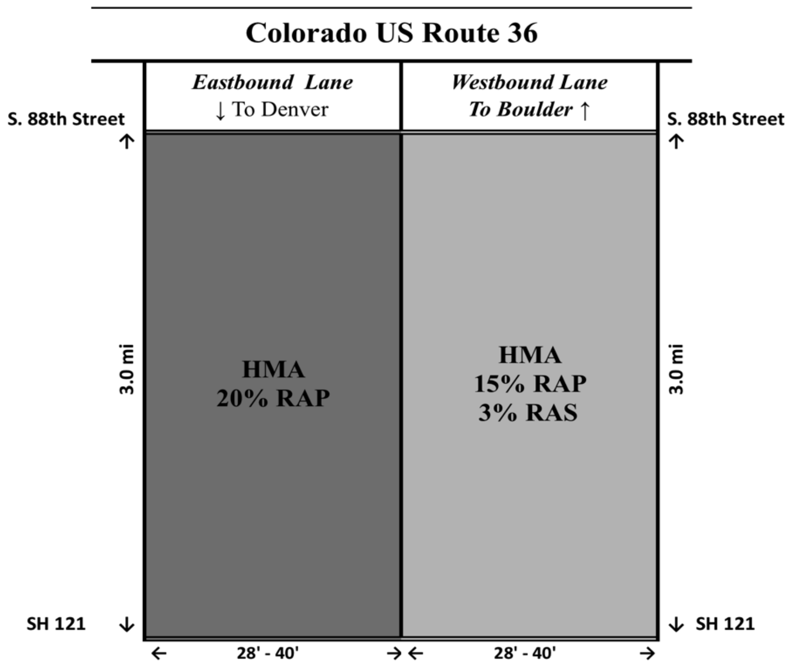


Figure F4.2. Plan view of Highway 10 project test sections

F5. HMA Production and Shingle Processing

The asphalt plant for the project was located in Henderson, CO, next to US 76. The haul distance from the plant to the furthest project point was 18 miles. The plant is a parallel flow drum plant with a capacity to produce up to 300 tons of HMA per hour (Figure F5.1).

Figure F5.1. Henderson plant

For production of the RAS/RAP mix, the RAS was augured onto a conveyor belt which carried the RAS to the RAP conveyor and then over a vibrating screen (grizzly) to remove any clumps that may have occurred in the stockpiles during the holding time from delivery to plant usage (Figure F5.2).

Figure F5.2. RAS screening

Approximately 23,760 tons of HMA was placed for the demonstration project. This included 357 tons of RAS and 4,158 tons of RAP. Tonnages of the RAS, RAP and HMA for each test section are summarized below in Table F5.1.

Table F5.1. Project tonnages

Material	RAP Only (Tons)	RAP-RAS (Tons)
RAP	2,376	1,782
RAS	---	357
Total HMA	11,880	11,880

Wet summer weather conditions created delays and extended the project into August 2011. Weather conditions during the paving were ambient temperatures ranging from 69-95 degrees Fahrenheit with sunny to cloudy skies and moderate to high humidity. Paving was completed over-night to reduce traffic delays. Due to lane closures for paving, traffic was limited to the shoulders and controlled by flaggers.

Temperatures for the RAP mix were approximately 300°F in the trucks and ranged from 285°F to 290°F behind the paver. Temperatures for the RAS/RAP mix were slightly higher at 335°F in the trucks and ranged from 315°F to 320°F behind the paver.

The HMA mix design used for the project contained a combination of RAS and RAP, both of which Asphalt Specialties crushed and processed. Asphalt Specialties collected manufactured shingles and processed them in an industrial grinder to produce a final product with essentially a minus 1/2" material (99% was passing the 1/2" sieve). The RAP was fractionated to pass the 1/2" screen. A picture of the RAS stockpile used for the project is shown in Figure F5.3. The asphalt content and gradation test results of the RAS before and after extraction, and of the RAP after extraction, are presented in Table F5.2. From MnDOT's extractions, the asphalt content of the RAS was measured to be 18.1 percent and the asphalt content of the RAP was measured to be 4.5 percent. These asphalt contents slightly varied from the asphalt contents of the materials used during the mix design, which were 18.5 percent for the RAS and 5.8 percent for the RAP.

Figure F5.3. Recycled asphalt shingles (RAS) stockpile

Table F5.2. RAS and RAP gradations (percent passing)

Sieve Size (US)	Sieve Size (mm)	RAS (Before Extraction)	RAS (After Extraction)	RAP (After Extraction)
3/4"	19	100	100	100
1/2"	12.5	99	100	100
3/8"	9.5	95	100	99
#4	4.75	70	95	88
#8	2.36	55	93	74
#16	1.18	31	74	60
#30	0.6	13	54	45
#50	0.3	6	46	30
#100	0.15	2	35	19
#200	0.075	0.3	26.4	16.5
% Asphalt Contents measured by MnDOT			18.1	4.5
% Asphalt Contents recorded in the mix designs			18.5	5.8

During night paving of the RAS mix, individual tabs of RAS were visible on the pavement surface (Figure F5.4) indicating the 1/2 inch minus grind size of the RAS was too large to adequately blend with the virgin materials.

Figure F5.4. RAS tabs visible during paving

F6. Asphalt Mix Design and Prodution Results

Two HMA mix designs were prepared by Earth Engineering Consultants, Inc. (EEC) for the demonstration project. The mix designs were completed in general accordance with Section 401 of CDOT Standard Specifications and the Asphalt Institute Superpave Mix Design No. 2 mix design procedures. The first mix design contained 20 percent RAP; the second mix design contained 15 percent RAP and three percent RAS. Both mixes were designed with a 1/2 inch nominal maximum aggregate size (NMAS). Gradations obtained from laboratory testing of the sampled asphalt mixtures are presented in Figure F6.1. As shown in the figure, the asphalt mixtures had similar aggregate structures with gradations passing above the restricted zone.

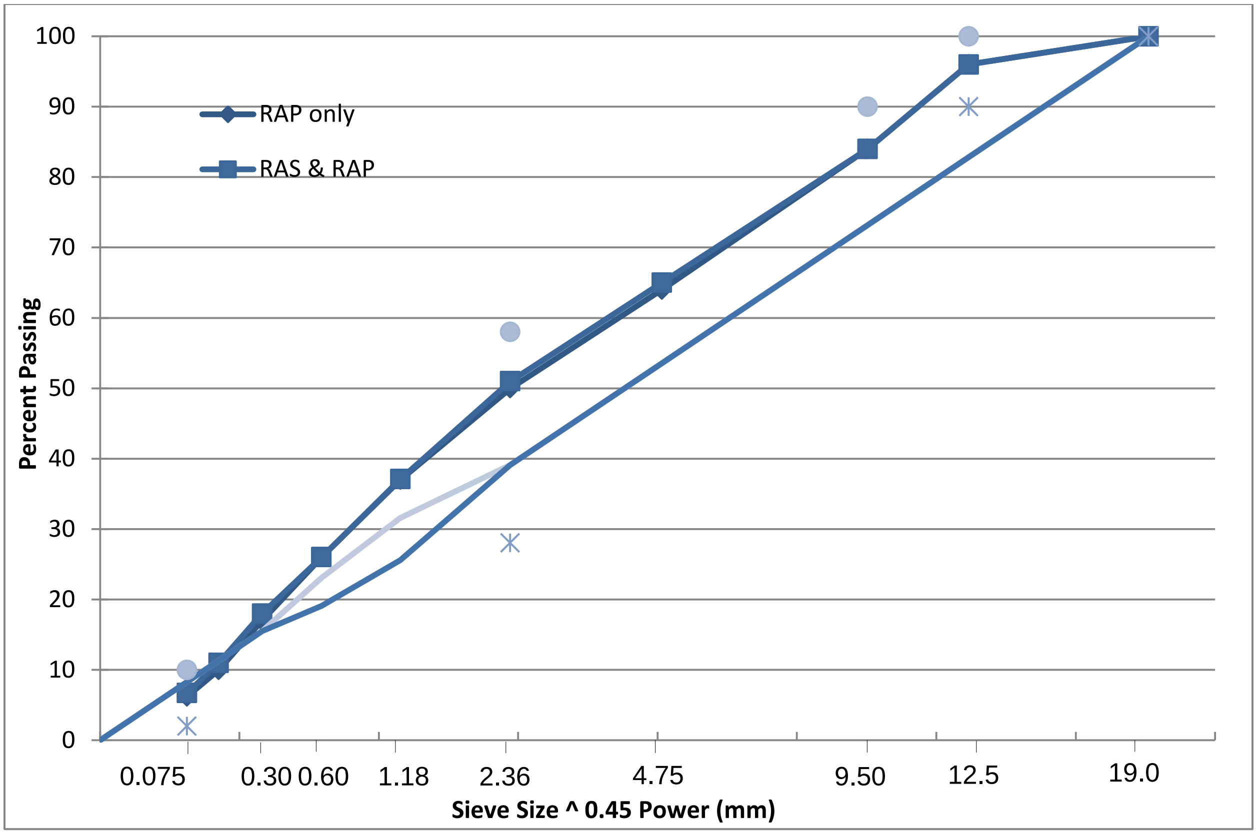


Figure F6.1. Asphalt mix design gradations

The asphalt demand properties of the mixes are presented in Table F6.1. The optimum asphalt contents for the RAP and RAP/RAS mixes were 5.1 and 5.2 percent, respectively. The contributions of the recycled binder from the RAS and RAP products resulted in a 23.5 percent binder replacement when 20 percent RAP was added to the mix and 26.9 percent binder replacement when 15 percent RAP and 3 percent RAS were added to the mix.

Table F6.1. Mixture asphalt demand properties

Mix Property	RAP only	RAS/RAP
% RAS	0	3
% RAP	20	15
% Total AC	5.1	5.2
% Virgin Binder(1)	3.9	3.8
% Binder Replacement(1)	23.5	26.9

(1) Calculated from asphalt contents reported in EEC's mix designs (assumes |100% of the RAS binder was effective in the mix)

The one tenth larger optimum asphalt content in the RAS mix is likely due to the 0.3% larger VMA (Table F6.2). Adding RAS to an asphalt mix design can increase the VMA due to the crushed aggregate particles in the RAS.

Table F6.2. Mixture design properties

Mix Property	RAP only	RAS/RAP
Design Gyrations	100	100
NMAS (mm)	12.5	12.5
Virgin Binder PG	64-28	64-28
% Voids	4.0	4.0
% VMA	14.7	15.0
% VFA	74	74
-#200/Pbe	1.1	1.2

Adjustments to the mix designs were made during production. For the RAP/RAS mix, the asphalt content target was increased to 5.4 percent and the voids were targeted at 3.4 percent. The production targets and quality control test results for the RAP/RAS mix produced on 6/21/11 and 6/27/11 are presented in Table F6.3. On 6/21/11, the asphalt content was slightly high resulting in low air voids (1.0 to 1.5 percent). Since shingle tabs containing asphalt were visibly protruding from the mat during paving of the RAP/RAS mix on 6/21/11 (see Figure F5.3), the mix would most likely be deficient in effective asphalt and possess a high laboratory air voids. The quality control results with lower air void contents, however, indicate sufficient effective asphalt present in the field mix.

Table F6.3. Quality control test results for the RAP/RAS mix(1)

Date	%AC		%Voids		VMA		VFA	
	Result	Target	Result	Target	Result	Target	Result	Target
6/21/11	5.66	5.4±0.3	1.4	3.4±1.2	14.2	14.5-16.9	90.1	65-75
6/21/11	5.72		1.5		14.3		89.6	
6/21/11	5.41		1.3		13.9		90.6	
6/21/11	5.66		1.0		13.9		92.5	
6/21/11	5.74		1.3		14.2		90.7	
6/27/11	5.32		3.1		15.1		79.2	

(1) Each result represents the average of three test results obtained from one sample

The quality control test results for the 20 percent RAP mix are shown in Table F6.4. The asphalt content for this mix was also adjusted. The target asphalt content during production was set at 5.2 percent.

Table F6.4. Quality control test results for the 20 percent RAP mix[1]

Date	%AC		%Voids		VMA		VFA	
	Result	**Target**	**Result**	**Target**	**Result**	**Target**	**Result**	**Target**
6/3/11	4.83	5.2 ± 0.3	3.9	3.6 ± 1.2	14.3	14.2-16.6	72.5	65-75
6/3/11	5.29		3.6		15.2		76.3	
6/3/11	5.18		3.2		14.8		78.0	
6/3/11	5.41		3.4		15.1		77.5	
6/15/11	5.28		3.2		14.8		78.2	
6/17/11	5.19		4.0		15.5		74.1	
6/21/11	5.31		1.9		14.4		86.9	
6/21/11	5.43		2.2		14.7		84.7	
6/21/11	5.06		4.1		14.7		72.0	
6/23/11	5.17		2.9		14.9		80.6	
6/23/11	5.22		3.0		14.9		79.8	
6/24/11	5.18		3.5		15.6		77.5	

Each result represents the average of three test results obtained from one sample

F7. Laboratory Test Results

Binder Testing

Performance Grade (PG) testing of the extracted binders was conducted to obtain their high, low, and intermediate PG temperatures as shown in Table F7.1. The high temperature performance grade of the RAS binder, measured at 111.2°C, is higher than a traditional paving grade binder. This is expected since the binder in roofing shingles is produced with an air-blowing process which oxidizes the asphalt.

A 64-28 virgin binder was used for both asphalt mixes. A sample of the virgin binder used during production was delivered to Iowa State's asphalt laboratory and tested as a continuous performance grade of 66.4-34.8.

HMA extraction samples were heated and placed in a centrifuge at high speeds during the recovery process, so the RAS, RAP, and virgin asphalt in the binder samples should be fully blended. The binder extracted from the RAP contained a continuous performance grade of 73.5-10.8. When 20 percent RAP was used in the mix design, the continuous performance grade of the blended asphalt was determined to be 67.6-27.5. When 15 percent RAP and 3 percent post-manufactured RAS was used in the mix design, the continuous performance grade of the blended asphalt changed to 71.9-21.1.

Table F7.1. Performance grade of extracted binders

Material Identification	High PG Temp, °C	Intermediate PG Temp, °C	Low PG Temp, °C	Performance Grade
PG 64-28	66.4	12.4	-34.8	64-34
RAS	111.2	-	-	-
RAP	77.7	26.5	-18.8	76-16
RAP HMA	67.6	18.7	-27.5	64-22
RAS/RAP HMA	71.9	19.7	-21.1	64-16

Dynamic Modulus

The dynamic modulus (|E*|) is a key material property that determines the stress-strain relationship of an asphalt mixture under continuous sinusoidal loading. A higher dynamic modulus indicates lower strains will result in a pavement structure when the asphalt mixture is stressed from repeated traffic loading. The mechanistic-empirical pavement design guide (MEPDG) uses |E*| as the stiffness parameter to calculate an asphalt pavements strains and displacements.

The test was conducted following AASHTO TP62 using five replicate samples of 150 mm in height and 100 mm in diameter. Each sample was compacted to 7 ± 0.5% air voids. Samples were tested by applying a continuous sinusoidal load at 9 different frequencies (0.1, 0.3, 0.5, 1, 3, 5, 10, 20, and 25 Hz) and three different temperatures (4, 21, and 37°C). Sample loading was adjusted to produce strains between 50 and 150 µstrain in the sample.

Master curves were constructed at a reference temperature of 21°C and plotted on a log-log scale for a general comparison as presented in Figure F7.1. The 20 percent RAP mix has higher dynamic modulus values than the RAS/RAP mix at low and intermediate frequency ranges.

The plot in Figure F7.2 presents the mean dynamic modulus at 21°C and 37°C at specific frequencies for a more direct comparison. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. At 21°C and 5 Hz, the mixture with 15 percent RAP and 3 percent RAS has a statistically lower dynamic modulus than the mixture with 20 percent RAP. Low modulus values at this temperature are considered desirable in thin asphalt pavements (less than 4”) for fatigue cracking resistance. Mixtures with lower stiffness can deform more easily without building up large stresses.

The dynamic modulus of the two mixes at 37°C and 0.1 Hz was analyzed since the modulus of asphalt mixtures at high temperatures and low frequencies is an indicator of rutting resistance (Figure F7.2). Since there are no statistical differences between the dynamic modulus of the mixes at this frequency and temperature, replacing 5 percent RAP with 3 percent RAS in the HMA did not impact the rutting resistance of the asphalt mixture, based on the dynamic modulus test results.

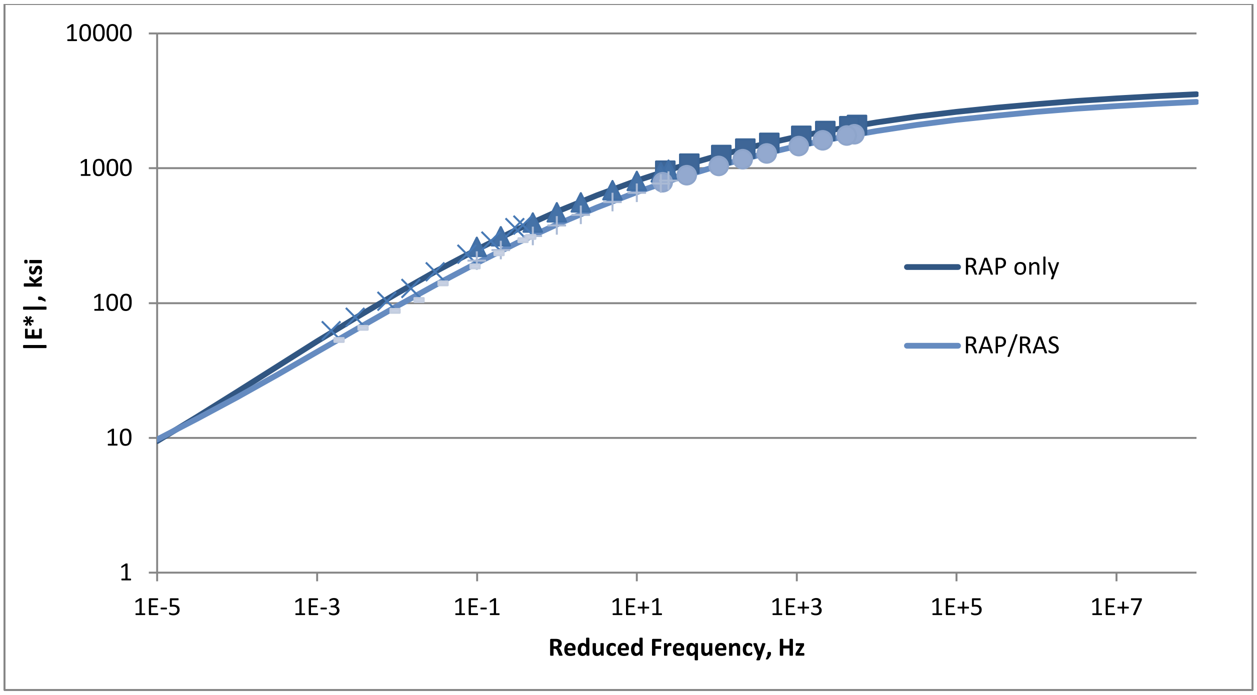


Figure F7.1. Comparison of dynamic modulus master curves

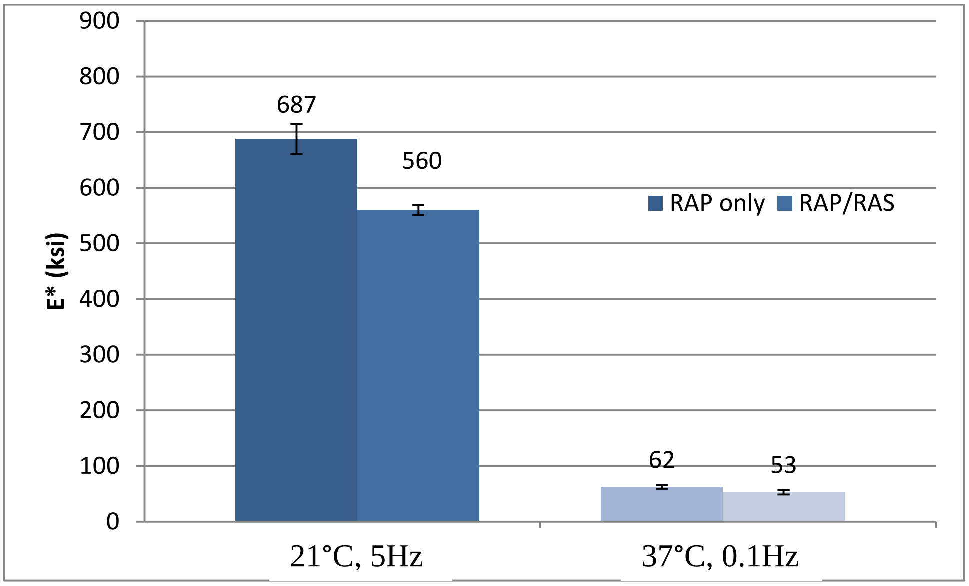


Figure F7.2. Dynamic modulus comparison at 21°C, 5 Hz and 37°C, 0.1 Hz

Flow Number

The flow number test measures the permanent deformation resistance of asphalt mixtures by applying a repeated dynamic load to a sample for up to several thousand load cycles. The flow number is defined as the number of load cycles an asphalt mixture can tolerate until it flows.

Cumulative permanent deformation in the sample is plotted versus load cycles. The flow number is reached at the onset of tertiary flow.

Tests were conducted following procedures used in NCHRP Report 465. Samples used in dynamic modulus test were used for the flow number test since the dynamic modulus test is nondestructive. The samples were placed in a hydraulically loaded universal testing machine, unconfined, with a testing temperature of 37°C to simulate the climactic conditions that cause pavement to be susceptible to rutting. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 sec followed by 0.9 sec of dwell time. The loading cycle was repeated for a total of 10,000 load cycles. Three LVDTs were attached to each sample during the test to measure the cumulative strains.

Test results are presented in Figure F7.3. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. If the error bars of two mixtures do not overlap, then the difference of the two mixtures can be considered statistically significant at the 5% level.

The flow numbers for both mixes were very high indicating good resistance to permanent deformation. The average flow number for the HMA with 20 percent RAP was measured as 8033, while the average flow number for the HMA with 15 percent RAP and 3 percent RAS was measured as 7687. There were no statistical differences between the two mixtures. Therefore, based on the flow number test results, replacing 5 percent RAP with 3 percent RAS did not have an effect on the permanent deformation performance of the mixtures. It is important to point out that there is more variability in the flow number results for the mixture with the 3% RAS.

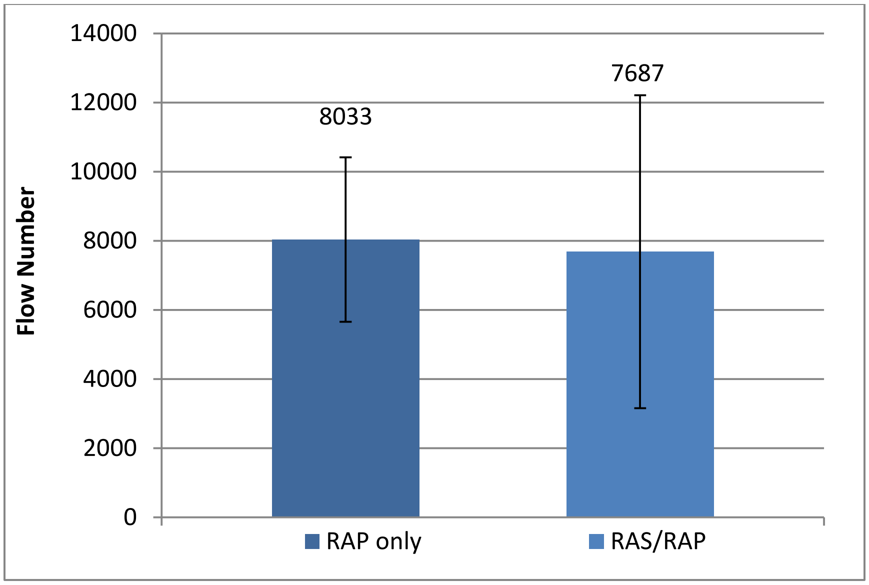


Figure F7.3. Flow number test results

Beam Fatigue

Fatigue cracking is the major cracking distress in asphalt pavements caused by repeated heavy traffic loads. Pavements that have a higher resistance to tensile strains that develop at the bottom of an asphalt layer due to repeated traffic will have a greater resistance to fatigue cracking. The four-point beam fatigue test following AASHTO T321 was conducted to evaluate the load associated cracking resistance of the mixtures.

The mixes were compacted in a linear kneading compactor to create slabs with 7% air voids. The slabs were saw-cut into beams with dimensions 15 inches in length, 2.5 inches in width, and 2 inches in height. The beams were tested in a strain controlled mode of loading at 20°C with haversine wave pulses applied to the beam at 10 Hz. Six beams were tested, each at a different strain level (375, 450, 525, 650, 800, and 1000 µstrain), until the flexural stiffness of the beam was reduced to 50% of the initial stiffness. A log-log regression was performed between strain and the number of cycles to failure (N_f). The relationship between strain and N_f can be modeled using the power law relationship as presented in Equation 1.

$$N_f = K1\left(\frac{1}{\varepsilon_o}\right)^{K2} \tag{1}$$

where: N_f = cycles to failure; ε_o = flexural strain; and K1 and K2 = regression constants.

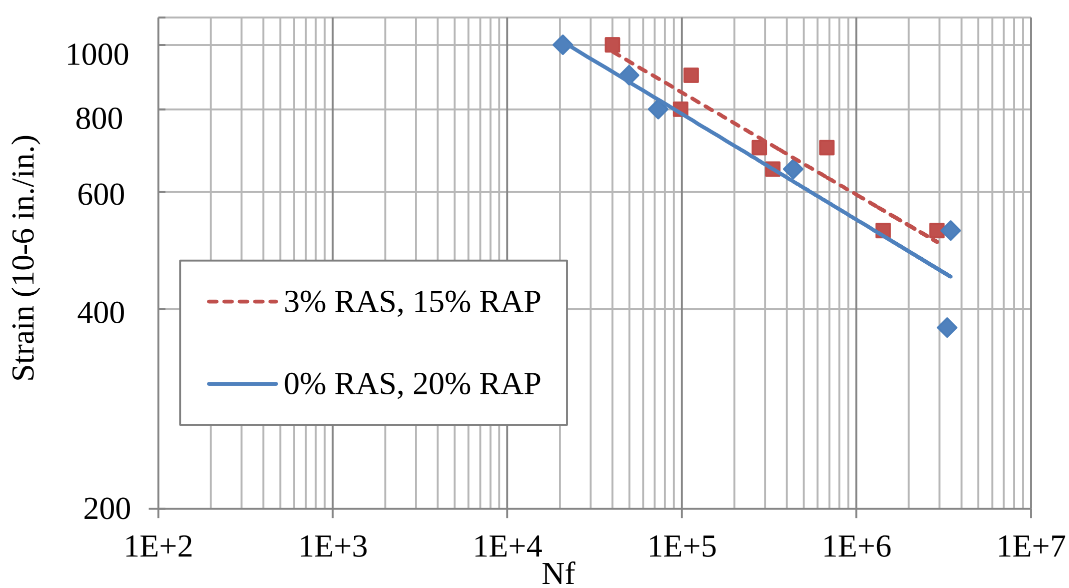


Figure F7.4. ε-N fatigue curves

The beam fatigue test results, as shown by strain versus "loading cycles to failure" curves, are presented in Figure F7.4. The fatigue curve model coefficients, average initial stiffness, and R^2

values are presented in Table F7.4. The HMA with RAS exhibits a longer fatigue life than the HMA with only RAP at similar strain levels in a strain-controlled mode of loading. This indicates that adding post-manufactured RAS increases the fatigue life of HMA for a thin lift pavement.

The fatigue endurance limit (FEL) of each mixture was also predicted. If tensile strains are low enough in a pavement structure, the pavement has the ability to heal and therefore no damage cumulates over an indefinite number of load cycles. The level of this strain is referred to as the FEL. The FEL of each mixture was estimated using the lower 95% prediction limit at 50 million load cycles as proposed in NCHRP Report 646 and shown in Equation 2.

$$\text{Lower Prediction Limit} = \hat{y}_o - t_\alpha s \sqrt{1 + \frac{1}{n} + \frac{(x_o - \bar{x})^2}{S_{xx}}} \tag{2}$$

where:
y_o = the one-sided lower 95% prediction interval at the micro-strain level corresponding to 50,000,000 cycles;
t_α = value of t distribution for n-2 degrees of freedom for a significance level of 0.05;
s = standard error of the regression analysis;
n = number of samples;
S_{xx} = sum of squares of the x values;
x_o = log 50,000,000; and
$\bar{x}$ = average of the fatigue life results.

The FEL estimates are also presented in Table F7.2. The RAS mixture exhibits higher and thus more desirable endurance limits, indicating that RAS may improve the FEL in the mixture.

Table F7.2. Beam fatigue results

Mix ID	% Binder Replacement	Average Initial Stiffness (Mpa)	K1	K2	R^2	Endurance Limit (Micro-strain)
RAP only	17.6	2299	2.34E-13	5.69	0.907	195
RAS/RAP	23.1	2605	9.22E-14	5.89	0.907	244

Semi-Circular Bend Test

The low temperature fracture properties of the mixtures were obtained from SCB tests by following the procedure in “Investigation of Low Temperature Cracking in Asphalt**”** **(**Marasteanu et al., 2007). Testing was conducted at four different low temperatures: -12°C, -18°C, -24°C, and -28°C.

All tests were performed inside an environmental chamber, and liquid nitrogen was used to obtain the required low temperature. The temperature was controlled by the environmental chamber temperature controller and verified using an independent platinum resistive-thermal-device (RTD) thermometer. The load line displacement (LLD) was measured on both faces of the test specimens using a vertically mounted Epsilon extensometer with 38 mm gage length and ±1 mm range. One end was mounted on a button that was permanently fixed on a specially made frame, and the other end was attached to a metal button glued to the sample. The average LLD measurement was used for each specimen. The crack mouth opening displacement (CMOD) was recorded by an Epsilon clip gage with 10 mm gage length and a +2.5 and -1.0 mm range. The clip gage was attached at the bottom of the specimen. A constant CMOD rate of 0.0005mm/s was used and the load and load line displacement (P-u), as well as the load versus LLD curves were plotted. A contact load with a maximum load of 0.3 kN was applied before the actual loading to ensure uniform contact between the loading plate and the specimen. The testing was stopped when the load dropped to 0.5 kN in the post peak region. The load and load line displacement data were used to calculate the fracture toughness and fracture energy (G_f).

The fracture energy (G_f) parameter is presented in Figure F7.5. The laboratory test results were analyzed to evaluate the effect of the various RAS treatments on the fracture energy. MacAnova statistical software package was utilized to perform a statistical analysis. The analysis of variance (ANOVA) was used to examine the differences among the mean response values of the different treatment groups. The significance of the differences was tested at 0.05 level of error. The analysis of variance was conducted with the assumption that the errors in the data are independently normal with constant variance.

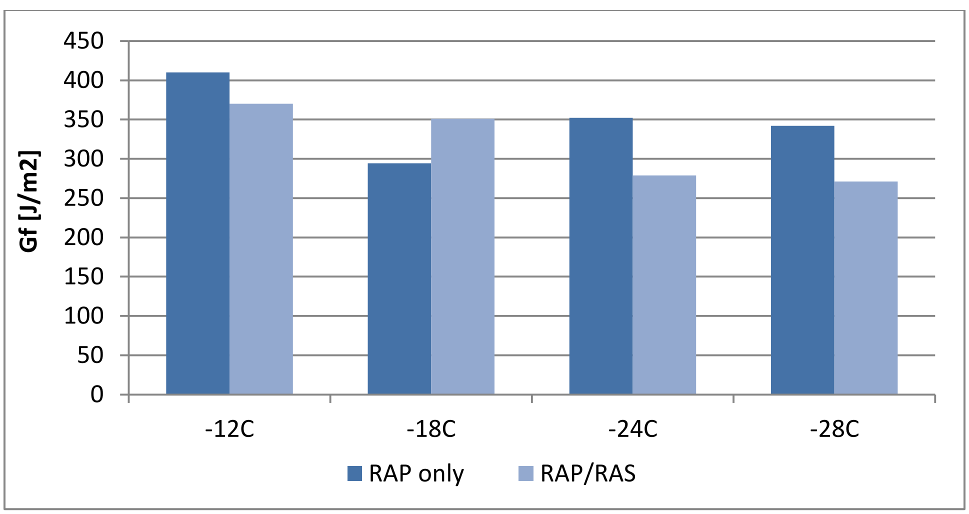


Figure F7.5. Colorado mixture fracture energy (G_f)

The Gf group means of each RAS treatment level was compared using a pair-wise comparison to rank the RAS treatment levels with regard to fracture energy. The outcome is reported in Table F7.3, in which statistically similar RAS treatments are grouped together. Letter A indicates the best performing group of mixtures; letter B the second best, and so on. Groups with the same letter are not statistically different, whereas mixtures with different letters are statistically different.

When 5 percent RAS with replaced with 3 percent RAS in the HMA, the fracture energy did not statistically change. While the RAS/RAP mixture did have a lower fracture energy (318 **J/m^2)** than the RAP only mixture (350 **J/m^2),** the difference was not statistically significant. Although not statistically significant at the 95% confidence level, these results do correlate well with the PG of the extracted binders. The low temperature performance grade of the extracted HMA binder containing RAP and RAS was higher than the extracted HMA binder containing RAP only (see Table F7.1.), thus also indicating slightly lower resistance to cracking at low temperatures.

Table F7.3. Ranking of mixes by G_f group mean for -12, -18, -24, and -28°C temperatures

Rank	Treatment	Group mean G_f [J/m^2]
A	RAP only	350
A	RAP/RAS	318

F8. Field Evaluations

Prior to the demonstration project, CDOT conducted precondition surveys of the pavement prior to the mill and overlay. The survey revealed a high amount of distress (cracking, rutting, and patching) present in the pavement (Figures F8.1 and F8.2). It was noted by onsite engineers that in some areas rutting was still present after the two-inch milling.

Figure F8.1. Precondition survey (2011) (westbound lane)

Figure F8.2. Precondition survey (2011) (eastbound lane)

The project team completed two distress surveys for the Colorado demonstration project test sections in October 2011 and March 2012. Three 500-foot sections were randomly selected in each of the test sections: 20 percent RAP and 15 percent RAP with 5 percent RAS. The surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* published by the Federal Highway Administration.

No distresses were found in any of the sections in the October 2011 survey. The second survey was completed in March 2012, the spring following the first winter season after paving. A minor amount of transverse cracking was documented (Figure F8.3) during the survey. Cracks identified in the driving lanes were shown to propagate from shoulder cracks. The shoulders were not repaved during the demonstration project.

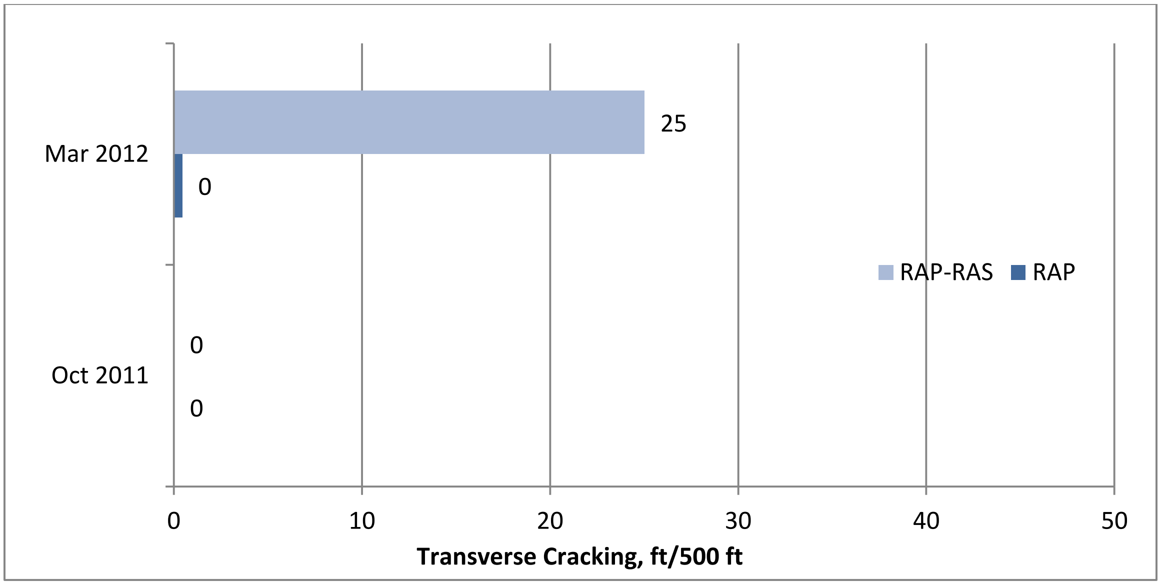


Figure F8.3. Colorado pavement evaluation

While measuring the length of transverse cracking in the pavements, the severity level of the cracks was also measured. Following the guidelines of the *Distress Identification Manual*, transverse cracks were categorized into three levels: low severity (crack widths ≤ 0.25 in), moderate severity (crack widths 0.25 in ≥ 0.75 in), and high severity (crack widths > 0.75 in). All transverse cracks measured in March 2012 were low severity levels of the transverse cracks. Examples of the transverse cracks measured in both pavement test sections are presented in Figures F8.4 and F8.5.

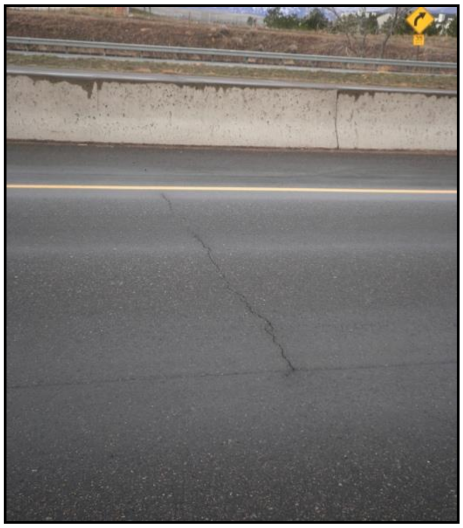


Figure F8.4. Low severity transverse crack (RAS/RAP)

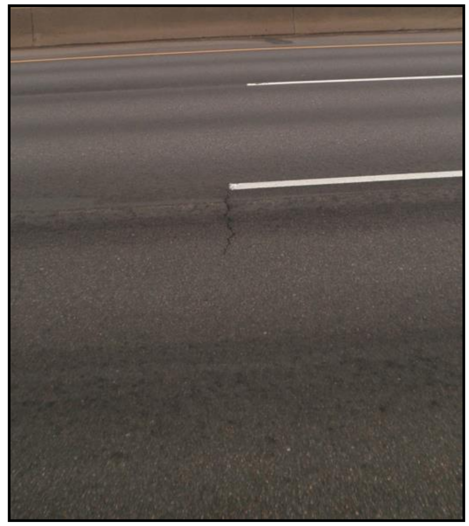


Figure F8.5. Low severity transverse crack (RAP only)

Figure F8.6. Low severity raveling (RAP)

Small amounts of low severity raveling were also documented in the RAP test sections (Figure F8.6). PowerPoint presentations of the distress surveys by 500-foot sections are available for viewing on the TPF-5(213) website.

F9. Conclusions

A CDOT demonstration project was conducted as part of Transportation Pooled Fund 5-213 to evaluate the effects of replacing of portion of RAP in a typical CDOT mix design with post-manufactured RAS. Two asphalt mixtures were evaluated: HMA containing 20 percent RAP and HMA containing 15 percent RAP and 3 percent post-manufactured RAS. Field mixes of each pavement were sampled for conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and recovery with subsequent binder characterization. Two pavement condition surveys of test sections were also conducted after paving. The results of the study are summarized below:

- Observations from the demonstration project show the contractor successfully produced and constructed both HMA mixes; however, there were several production days where not all volumetric requirements were met.
- During night paving of the RAS mix, individual tabs of RAS were visible on the pavement surface (Figure F5.4) indicating the 1/2 inch minus grind size of the RAS was too large to adequately blend with the virgin materials. A finer RAS grind of 3/8 inch minus or less will help reduce the presence of tabs during paving.
- The addition of RAP and RAS in the HMA increased the performance grade of the 64-28 binder in the asphalt mixtures. When 20 percent RAP was used in the mix design, the continuous performance grade of the blended asphalt was tested as a 67.6-27.5. When 15 percent RAP and 3 percent post-manufactured RAS was used in the mix design, the continuous performance grade of the blended asphalt increased to 71.9-21.1.
- There were no differences in rutting resistance between the two mixtures, based on the dynamic modulus and flow number test results. Both laboratory tests indicated similar stiffness and permanent deformation resistance at high temperatures
- Replacing 5 percent RAP with 3 percent RAS in the HMA improved its fatigue properties and increased its fatigue life in a controlled strain mode of loading, based on the four-point bending beam test and the dynamic modulus at intermediate temperatures. The predicted fatigue endurance limit, from the four-point bending beam test, of the HMA with RAS and RAP was higher than the HMA with RAP only. Likewise, the dynamic modulus of the RAS and RAP mixture at intermediate temperatures was measured to be statistically lower than the dynamic modulus of the mixture with RAP only, which may also improve the fatigue properties of the mixture.
- The SCB test was conducted to measure the low temperature cracking susceptibility of the mixtures by measuring their fracture energy at -12°C, -18°C, -24°C, and -28°C. The two HMA mixtures exhibited similar low temperature cracking resistance as there was no statistical difference in fracture energy between the two mixtures. Replacing 5 percent RAP with three percent RAS did not impact the fracture energy of the HMA.
- Field condition surveys conducted one winter season after the demonstration project revealed minor transverse cracking in the RAS and RAP mixture (25 linear feet of cracking per 500 feet), and very little transverse cracking in the RAP only mixture (1.5 linear feet of cracking per 500 feet).

F10. CDOT Demonstration Project Acknowledgments

The researchers gratefully acknowledge the support of Roberto DeDios at the Colorado DOT. The research work was sponsored by Federal Highway Administration (FHWA) TPF-5(213) and the Transportation Pooled Fund (TPF) partners: Missouri (lead agency), California, Colorado, Illinois, Indiana, Iowa, Minnesota, and Wisconsin DOTs.

APPENDIX G. REPORT FOR THE ILLINOIS DEPARTMENT OF TRANSPORTATION SPONSORED DEMONSTRATION PROJECT

G1. Introduction

This report presents a summary of the results obtained from the field demonstration project sponsored by the Illinois Department of Transportation (IDOT) as part of Transportation Pooled Fund (TPF) 5-213 *Performance of Recycled Asphalt Shingles in Hot Mix Asphalt*. TPF 5-213 is a partnership of several state agencies with the goal of researching the effects of recycled asphalt shingles (RAS) on the performance of asphalt applications. As part of the pooled fund research program, multiple field demonstration projects were conducted to provide adequate laboratory and field test results to comprehensively answer design, performance, and environmental questions about asphalt pavements that include RAS.

Each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated different aspects of asphalt mixes containing RAS. The field demonstration project sponsored by IDOT investigated the economic and performance benefits of replacing fibers and virgin asphalt with RAS in stone mastic asphalt (SMA). Several different plant and laboratory SMA mixes were produced using post-consumer RAS with different types of base binders. Some mixes also contained ground tire rubber (GTR) and recycled asphalt pavement (RAP). The objective of this demonstration project was to evaluate the performance of SMA mixtures using post-consumer RAS, GTR, and RAP with different base binders and to investigate the performance differences between laboratory produced SMA-RAS mixes to plant produced SMA-RAS mixes.

G2. Experimental Plan

To evaluate the performance of the SMA mixes, IDOT designed an experimental plan to address the following questions:

- What are the performance properties of SMA that uses 5 percent post-consumer RAS in place of fibers?
- What are the performance differences between an SMA-RAS mix using a polymer modified PG 70-28 and an SMA-RAS mix using a PG 58-28 with 12 percent GTR?
- What are the performance differences between a laboratory produced SMA-RAS mix versus a plant produced SMA-RAS mix?
- How will adding 11 percent fine RAP to an SMA-RAS mix affect its performance?

The experimental plan was implemented during the demonstration project with the production of two SMA mixes using a polymer modified PG 70-28. “D” Construction, Inc. (D Construction) produced an SMA mix with 5 percent RAS, and Curran Contracting Company, Inc. (Curran) produced an SMA mix with 5 percent RAS and 11 percent fine RAP. Each SMA mix was also produced in the laboratory by S.T.A.T.E. Testing, LLC. using two different asphalt binders: a polymer modified PG 70-28 and a PG 58-28 with 12 percent GTR. The experimental plan is presented in Table G2.1. All the mixes contained 5 percent post-consumer RAS to replace 100 percent of the fibers normally used in SMA.

Table G2.1. Experimental plan

Mix ID	Binder PG	% RAP	% RAS	RAS Source	Mix Type	Contractor
Dcon 70-28P	70-28	0	5	post-consumer	Plant	D Construction
Dcon 70-28L	70-28	0	5	post-consumer	Lab	D Construction
Dcon 58-28L	58-28 w/ 12% GTR	0	5	post-consumer	Lab	D Construction
Curran 70-28P	70-28	11	5	post-consumer	Plant	Curran
Curran 70-28L	70-28	11	5	post-consumer	Lab	Curran
Curran 58-28L	58-28 w/ 12% GTR	11	5	post-consumer	Lab	Curran

During production of the SMA, IDOT collected samples of each mixture and sent them to Iowa State University for laboratory testing. A portion of the samples were sent to the University of Minnesota and the Minnesota Department of Transportation (MNDOT) for Semi-Circular Bend (SCB) testing and binder extraction and recovery, respectively. The laboratory testing plan is presented in Table G2.2.

The asphalt was recovered from the RAS and asphalt mixtures following AASHTO T164 Method A (Centrifuge Method) by using a blend of toluene and ethanol as the extraction solvent. The fines were removed from the binder extract by using a centrifuge at high speeds. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. At Iowa State University, the Performance Grade (PG) of the extracted asphalt binders was determined by following AASHTO R29 “Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder.”

Table G2.2. Laboratory testing plan

	Laboratory Test	Iowa State University	University of Minnesota	Minnesota DOT
Processed Shingles	Binder Extraction			X
	High Temperature PG			X
	Gradation (Before Extraction)	X		
	Gradation (After Extraction)	X		
Mixture	Binder Extraction			X
	Binder PG Characterization	X		
	Gradation	X		
	Dynamic Modulus	X		
	Flow Number	X		
	Beam Fatigue	X		
	Semi-Circular Bending (SCB)		X	

Washed gradations of the aggregates after extractions were also conducted at Iowa State University by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution, and a washed gradation was conducted after extraction to evaluate the size distribution of the fine aggregates in the RAS product.

Performance testing was completed on the field produced asphalt mixtures at low, intermediate, and high temperatures. Dynamic Modulus and Flow Number were conducted at high temperatures to evaluate the modulus and rutting resistance of asphalt mixtures. The durability of the mixtures at intermediate temperatures was evaluated using the dynamic modulus and four-point beam fatigue test. The SCB test conducted at the University of Minnesota evaluated the fracture properties of the mixtures at low temperatures.

After pavement construction for the demonstration project, field evaluations were conducted on the pavement test section prior to the winter season and the following spring after paving to assess the field performance of the pavement concerning cracking, rutting, and raveling.

G3. Project Location

The demonstration project completed by D Construction took place on Interstate 80 (I-80) east of Joliet, IL, located in the northeast corner of the state. Test sections were placed on the eastbound (EB) and westbound (WB) lanes of I-80, starting at the intersection of I-80 and the Grundy County Line and continuing east approximately 14 miles to US Route 30 (Figure G3.1)

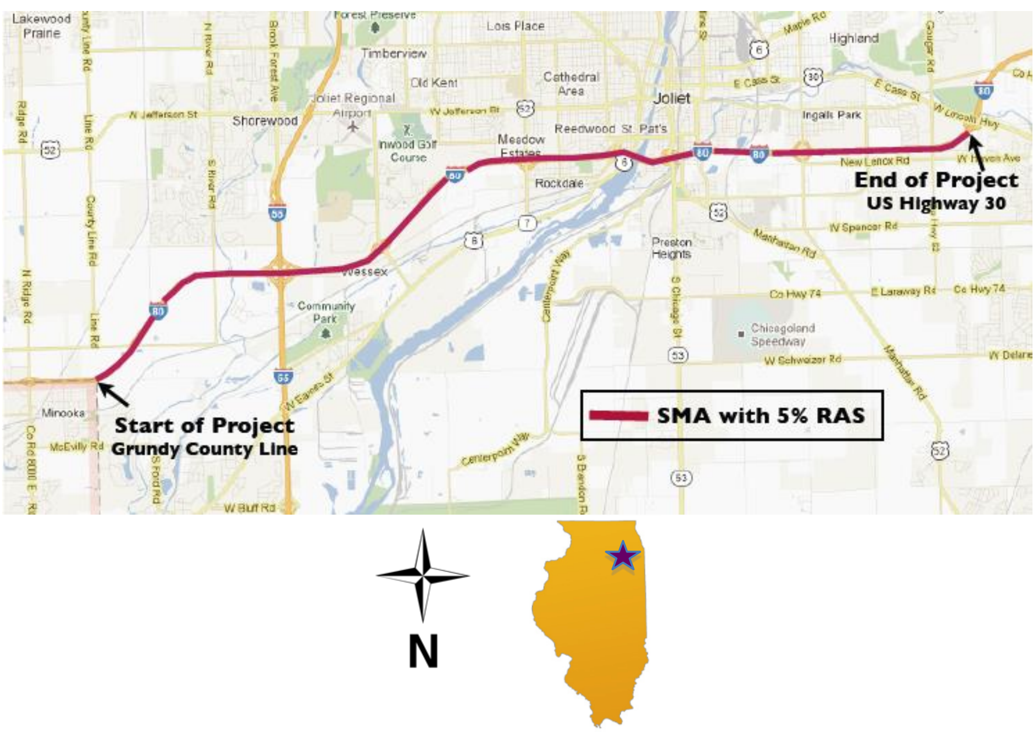


Figure G3.1. Project location on I-80 (SMA produced by D Construction)

Three additional mixes were evaluated from a concurrent SMA overlay project completed by Curran on the Jane Addams Memorial Tollway, a segment of Interstate 90 (I-90), in Hoffman Estates, IL, located in the northeast corner of the state. Although this was not the official IDOT sponsored demonstration project, the Curran SMA mixes were included in the pooled fund study since they were very similar to the D Construction SMA mixes. They also contained 5 percent post-consumer RAS in place of fibers. The difference between the Curran and D Construction SMA mixes was the addition of 11 percent fine RAP in the Curran SMA mixes.

The test section for the SMA mixes placed by Curran was located on the eastbound (EB) and westbound (WB) lanes of I-90, starting at the intersection of I-90 and Barrington Road and continuing west approximately 9.5 miles to US Route 31. The project limits are identified below in Figure G3.2.

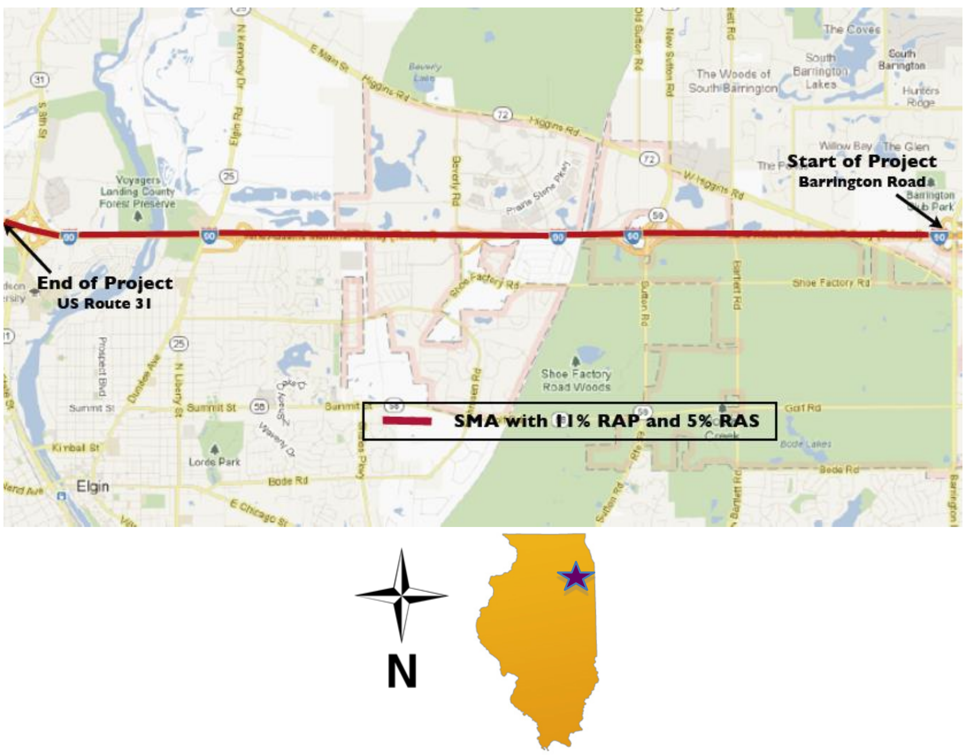


Figure G3.2. Project location on Jane Addams Memorial Tollway (I-90) (SMA produced by Curran)

G4. Project Description

For the demonstration project on I-80, IDOT number 18435R, the existing pavement structure consisted of 3.5 inches of SMA placed over 9.25 inches of continuous reinforced concrete pavement. D construction milled and replaced the 3.5 inches of asphalt with two two-inch lifts of SMA. Only the binder course was sampled and sent to Iowa State University for testing. The surface friction course was a similar SMA mix design but included slag aggregates. The project cross-section is shown in Figure G4.1.

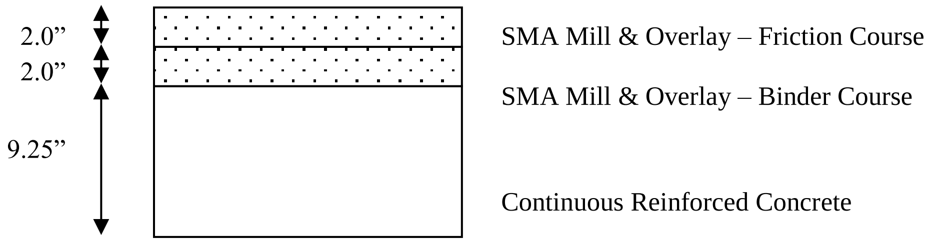


Figure G4.1. I-80 pavement cross-section

For the project on the Jane Addams Memorial Tollway (I-90), the existing pavement structure consisted of 2.25 - 5.50 inches of HMA placed over 10 inches of jointed reinforced concrete pavement. Curran milled and replaced the asphalt with two two-inch lifts of SMA. The binder course was sampled and sent to Iowa State University for testing. A cross-section is shown in Figure G4.2.

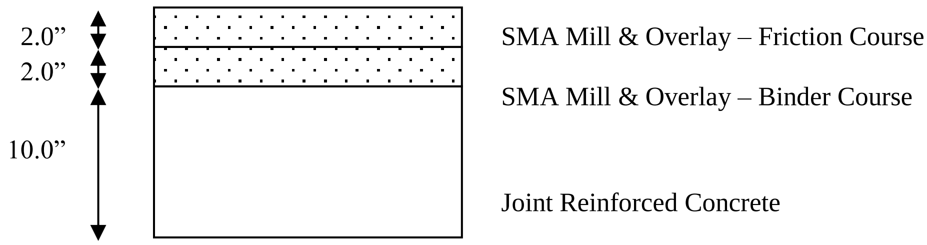


Figure G4.2. I-90 pavement cross-section

D Construction and Curran milled and placed the test sections in July through October 2011. All lanes were paved with the SMA-RAS mixture. In the areas where the test sections are located, I-80 contains four lanes (two in each direction) and I-90 contains six lanes (three in each direction). A plan view of the I-80 project is shown below in Figure G4.3.

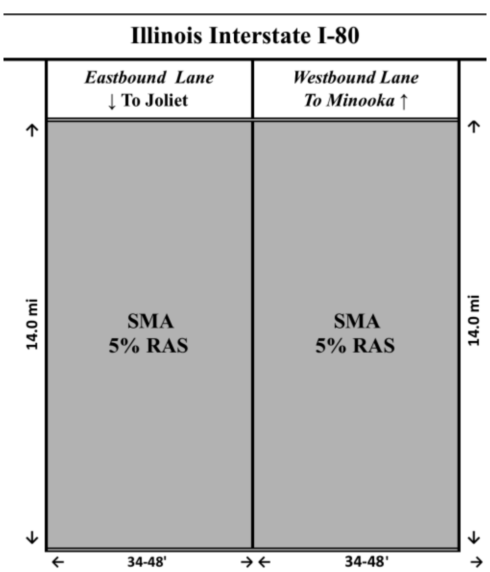


Figure G4.3. Plan view of I-80 project

G5. HMA Production and Shingle Processing

The asphalt plant for the I-80 demonstration project was located in Rockdale, IL adjacent to I-80. It is a counter flow drum plant with a capacity to produce up to 500 tons of HMA per hour (Figure G5.1). The farthest haul distance from the plant to the project was approximately nine-miles. Weather conditions during paving were ambient temperatures ranging from 40-96 degrees Fahrenheit with sunny to cloudy skies and moderate humidity. The plant production temperature was 360 degrees Fahrenheit.

Figure G5.1. Rockdale plant

The RAS was metered onto a conveyor belt and run over a vibrating screen (grizzly) to remove any clumps that may have occurred in the stockpiles during the holding time from delivery to plant usage. Approximately 160,000 tons of SMA and 8,000 tons of RAS were placed for the I-80 demonstration project as summarized in Table G4.1 (includes ramps and shoulders).

Table G5.1. Project tonnages

Material	RAS (Tons)
RAS	8,000
Total SMA	160,000

D Construction and Curran received their post-consumer RAS from Southwind RAS, LLC. The RAS was processed using an industrial grinder then screened to produce a final product with a minus 3/8" material. A picture of the RAS stockpiled at D Construction's plant is shown in Figure G5.2.

Figure G5.2. Post-consumer RAS stockpile

The asphalt content and gradation test results of the RAS before and after extraction, and the RAP after extraction, are presented in Table G5.2. From MnDOT's extractions, the asphalt content of the RAS was measured to be 36.7 percent and the asphalt content of the RAP was measured to be G7.1 percent. The asphalt content of the RAS measured by MnDOT varied from the asphalt content of the RAS used for the mix designs which was 26.0 percent. The asphalt content in the RAP used for the mix designs was G7.2 percent.

It is possible that higher asphalt contents were present in the RAS during the mix design, and not all the shingle asphalt binder was contributing to the final binder blend. Rather than 100 percent of the shingle asphalt binder releasing into the mix to coat virgin aggregates, some of the RAS particles may have been coated with asphalt. In this scenario, the RAS would have an effective asphalt content of 26.0 percent in the mix, but a total asphalt content of 36.7 percent.

Table G5.2. RAS and RAP gradations (percent passing)

Sieve Size (US)	Sieve Size (mm)	RAS (Before Extraction)	RAS (After Extraction)	RAP (After Extraction)
3/4"	19	100	100	100
1/2"	12.5	100	100	100
3/8"	9.5	100	100	100
#4	4.75	91	97	97
#8	2.36	74	91	70
#16	1.18	48	74	48
#30	0.6	24	52	34
#50	0.3	11	44	25
#100	0.15	3	36	18
#200	0.075	0.5	27.8	13.9
% Asphalt Contents measured by MnDOT			36.7	7.1
% Asphalt Contents measured by IDOT			26.0	7.2

G6. Asphalt Mix Design and Prodution Results

IDOT prepared the two SMA binder course mix designs, one for D Construction and one for Curran. Both mixes were designed with a 1/2 inch nominal maximum aggregate size (NMAS). Gradations obtained from laboratory testing of the samples delivered to Iowa State University are presented in Figures G6.1 and G6.2.

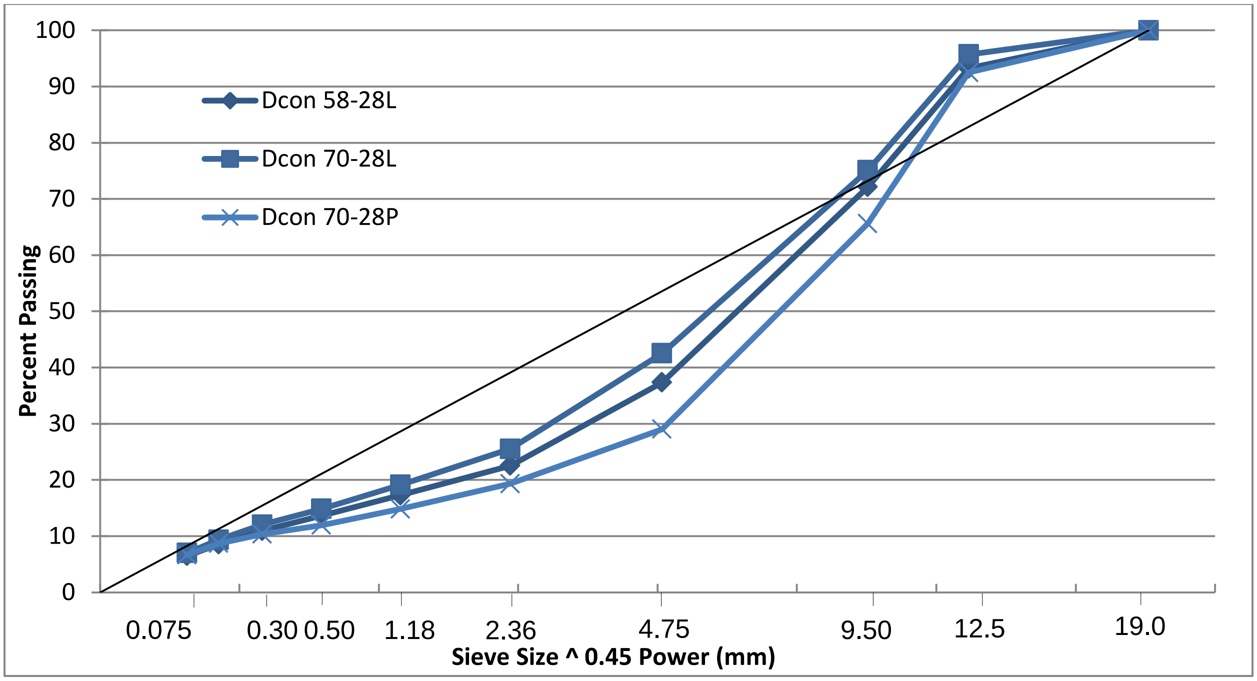


Figure G6.1. D Construction SMA gradations

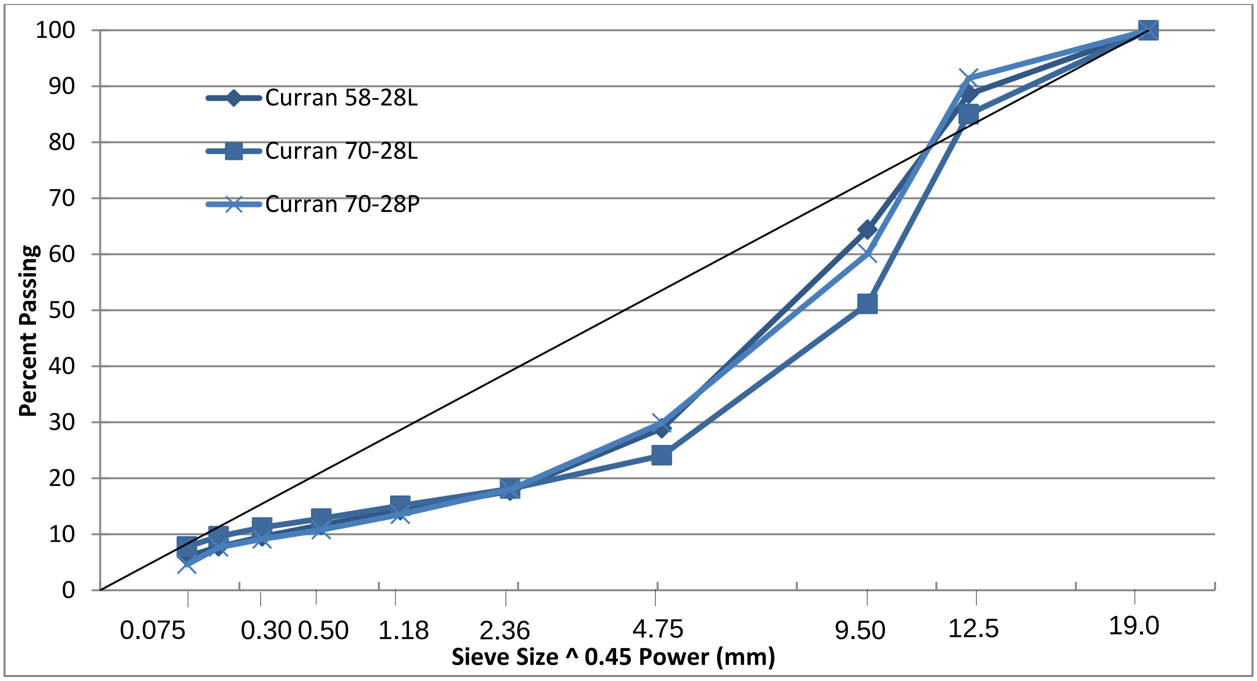


Figure G6.2. Curran SMA gradations

The asphalt demand properties of the SMA binder course mixes are presented in Table G6.1. The optimum asphalt contents for the D Construction and Curran mixes were 6.2 and 6.0 percent, respectively. The contributions of the recycled binder from the RAS and RAP products resulted in a 21.0 percent binder replacement when 5 percent RAS was added to the mix and a 35.0 percent binder replacement when 5 percent RAS and 11 percent RAP were added to the mix.

Table G6.1. SMA binder course asphalt demand properties

Mix Property	D Construction SMA	Curran SMA
% RAS	5	5
% RAP	0	11
% Total AC	6.2	6.0
% Virgin Binder(1)	4.9	3.9
% Binder Replacement(1)	21.0	35.0

(1) Values obtained from IDOT mix designs

The volumetric properties of the mixes presented in Table G6.2 possess typical values for IDOT SMA binder course mixes. IDOT requires that SMA mixes contain stabilizing additives such as cellulose or mineral fibers to prevent draindown greater than 0.3 percent. In these mixes, RAS was used in place of fibers, since RAS contains fibers as a result of grinding the cellulose shingle backing material. The draindown test results of 0.00 and 0.02 percent show that the RAS helped prevent draindown in the mixes (Table G6.2).

Table G6.2. SMA binder course mix design properties

Mix Property	D Construction SMA	Curran SMA
Design Gyrations	80	80
NMAS (mm)	12.5	12.5
% Voids	3.5	3.5
% VMA	15.6	15.8
% VFA	77.6	77.9
-#200/Pbe	1.21	1.31
% Draindown	0.00	0.02

G7. Laboratory Test Results

Binder Testing

Performance Grade (PG) testing of the extracted binders was conducted on the plant and laboratory produced SMA samples to obtain their high, low, and intermediate PG temperatures. The PG test results for the D Construction and Curran SMA mixes are presented in Table G7.1 and G7.2, respectively. The high temperature performance grade of the RAS binder, measured at 129.7°C, is higher than a traditional paving grade binder. This is expected since the binder in roofing shingles is produced with an air-blowing process which oxidizes the asphalt. Additionally, the RAS used in the mix designs is from post-consumer shingles, so the binder in the RAS has experienced several years of aging.

For the D Construction mixes, the PG of the modified 70-28 used during production was tested as a continuous 73.2-29.2, and the PG of the 58-28 binder with 12 percent GTR used for laboratory mixing was tested as a continuous 78.3-26.1. Blending 12 percent GTR to a PG 58-28 binder increased the PG to a 76-22.

There are several observations to note when 5 percent RAS was used in the D Construction SMA mix designs. First, there was no difference in the blended PG between the plant produced 70-28 SMA and the laboratory produced 70-28 SMA. Both had a blended PG of a 70-22. Therefore, the RAS only impacted the low PG side of the mix by one grade bump. Second, the same is true for the 58-28 SMA mix. Adding 5 percent RAS bumped the PG from a 76-22 to a 76-16. Again, only the low PG was impacted by one grade bump. Third, these results can be used to mathematically back-calculate the low temperature grade of the RAS. The average critical low temperature of the RAS binder for the D Construction mixes is -2.3°C.

Table G7.1. Performance grade of extracted binders for D Construction SMA mixes

Material Identification	High PG Temp, °C	Intermediate PG Temp, °C	Low PG Temp, °C	PG
PG 70-28(1)	73.2	15.5	-29.9	70-28
PG 58-28 with 12% GTR(2)	78.3	16.5	-26.1	76-22
Southwind Post-Consumer RAS	129.7	-	-	-
SMA mix for Dcon 70-28P	72.8	21.0	-24.3	70-22
SMA mix for Dcon 70-28L	72.7	19.1	-23.7	70-22
SMA mix for Dcon 58-28L	77.2	18.5	-21.3	76-16

(1) Binder sampled during D Construction's plant operations
(2) The same laboratory blended binder was used for both D Construction and Curran SMA mixes

For the Curran mixes, the PG of the modified 70-28 binder used during production was tested as a continuous 73.1-29.2. The same 58-28 (w/ 12% GTR) binder used to mix the D Construction 58-28 SMA in the laboratory was used to mix the Curran 58-28 SMA in the laboratory, so they both share the same continuous PG of 78.3-26.1.

The RAP in the Curran mixes was tested to have a continuous PG of 78.5-19.2. Adding the RAP and RAS to the 70-28 SMA during production increased the low and high temperature PG two grade bumps to a PG 82-16 (Table G7.2). For the laboratory produced 70-28 SMA, the blended PG was slightly stiffer than plant produced 70-28 SMA, but still within expected variability ranges for two different sample sources. The continuous PG of the plant produced SMA was 82.8-18.1 and the continuous PG of the laboratory produced SMA was 84.4-14.5.

The PG 58-28 SMA with 12 percent GTR contained a very similar blended binder grade as the plant produced polymer modified PG 70-28 SMA. This shows that a mix designer can use a softer base binder with GTR, RAP, and RAS to produce a mix with the same PG as mix that uses a polymer modified PG 70-28 binder.

Although the post-consumer RAS used for the D Construction and Curran mixes both came from Southwind RAS, LLC, there was some differences in the RAS low temperature properties for the two mixes. The average critical low temperature of the RAS binder for the Curran mixes is +8.7°C, approximately 11°C higher than the value calculated for the RAS used in the D Construction mixes. When considering the D Construction and Curran SMA mixes together, the average critical low temperature for the Southwind post-consumer RAS is +3.2°C. Therefore, for every 1 percent increase in RAS, the low temperature PG of the SMA mixes will increase 1.4°C; and for every 1 percent increase in RAP, the low temperature of the PG of the SMA mixes will increase about 0.1°C.

Table G7.2. Performance grade of extracted binders for Curran SMA mixes

Material Identification	High PG Temp, °C	Intermediate PG Temp, °C	Low PG Temp, °C	Performance Grade
PG 70-28(1)	73.2	15.5	-29.2	70-28
PG 58-28 with 12% GTR(2)	78.3	16.5	-26.1	76-22
Southwind Post-Consumer RAS	129.7	-	-	-
Type 1 Fine RAP	78.5	27.0	-19.2	76-16
SMA mix for Curran 70-28P	82.8	26.8	-18.1	82-16
SMA mix for Curran 70-28L	84.4	25.7	-14.5	82-10
SMA mix for Curran 58-28L	81.8	23.5	-17.7	76-16

(1) Binder sampled during Curran's plant operations
(2) The same laboratory blended binder was used for both D Construction and Curran SMA mixes

Dynamic Modulus

The dynamic modulus (|E*|) is a key material property that determines the stress-strain relationship of an asphalt mixture under continuous sinusoidal loading. A higher dynamic modulus indicates lower strains will result in a pavement structure when the asphalt mixture is stressed from repeated traffic loading. The mechanistic-empirical pavement design guide (MEPDG) uses |E*| as the stiffness parameter to calculate an asphalt pavements strains and displacements.

The test was conducted following AASHTO TP62 using three replicate samples of 150 mm in height and 100 mm in diameter. Each sample was compacted to 7 ± 0.5% air voids. Samples were tested by applying a continuous sinusoidal load at 9 different frequencies (0.1, 0.3, 0.5, 1, 3, 5, 10, 20, and 25 Hz) and three different temperatures (4, 21, and 37°C). Sample loading was adjusted to produce strains between 50 and 150 µstrain in the sample.

Master curves were constructed at a reference temperature of 21°C and plotted on a log-log scale for a general comparison as presented in Figure G7.1 and G7.2. The Curran SMA mixes in Figure G7.2 have a greater dynamic modulus than the D Construction mixes in Figure G7.1. The RAP in the Curran mixes has a stiffer binder which contributes to the increased modulus. A larger modulus at higher temperatures and/or low frequency helps the pavement resist against permanent deformation.

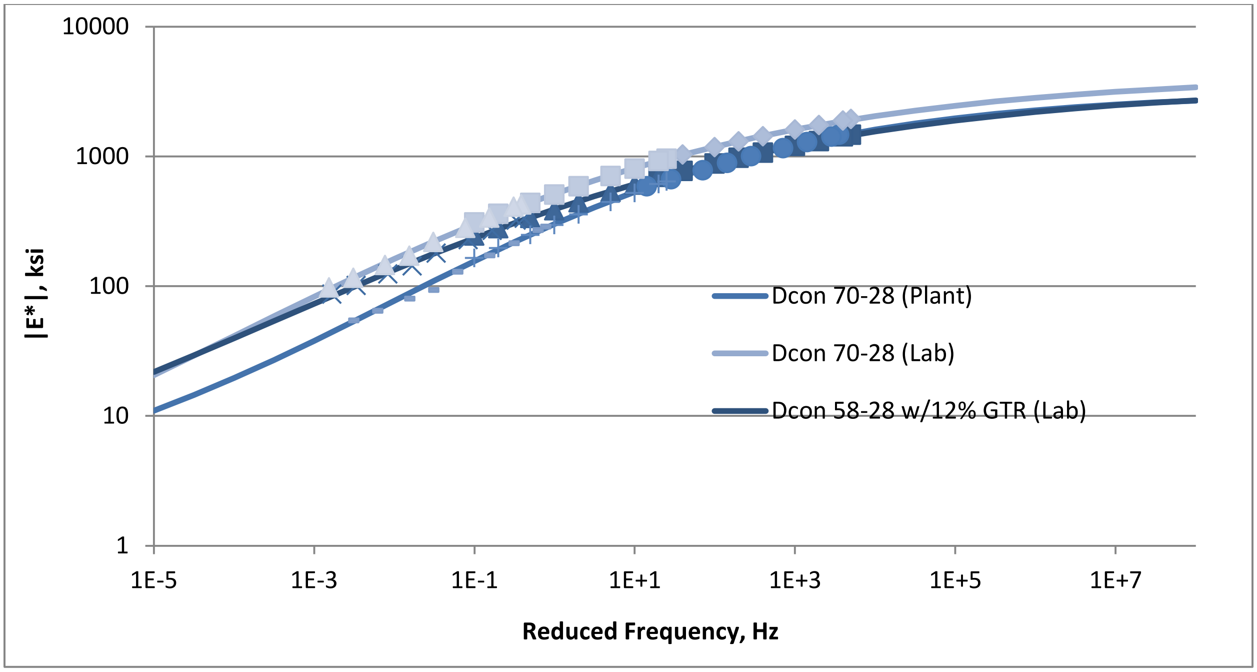


Figure G7.1. Comparison of dynamic modulus master curves (D Construction)

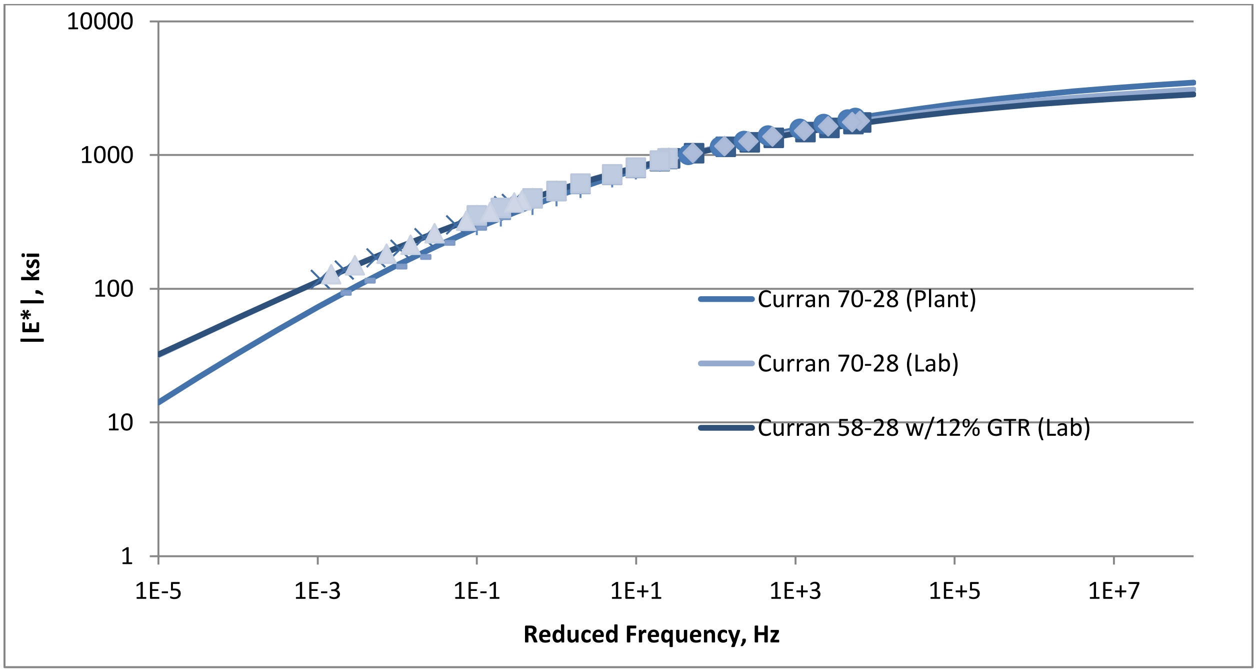


Figure G7.2. Comparison of dynamic modulus master curves (Curran)

A common trend in the D Construction and Curran mixes is that the plant produced PG 70-28 SMA possesses a lower dynamic modulus than the laboratory produced PG 70-28 SMA. This can be explained with one of two hypotheses. First, the higher laboratory modulus values may be due to the aging procedures used to cure the laboratory produced SMA. The samples could be over-cured and not match the actual short-term aging that took place during construction.

However, the laboratory loose mix was not cured following mixing to prevent over-curing. Rather, the mix was placed in 5-gallon metal buckets after mixing and delivered to Iowa State University for testing. Iowa State University reheated the laboratory and plant produced mixes at 300 to 310°F for 4 hours to compact the dynamic modulus samples.

For the second Hypothesis, the higher laboratory modulus values may be due to the RAS binder melting and blending with the virgin binder more effectively during laboratory mixing than during plant production. When more RAS binder releases from the shingle particles and blends with the virgin binder, the overall binder blend and SMA mixture will be stiffer and possess a higher modulus. RAS mix designs are conducted in a carefully controlled environment to optimize the blending of all materials. The same type of blending cannot always be exactly replicated during production. Several factors can affect the blending of the RAS and virgin binder during production: these include dwell time in the drum, RAS moisture content, RAS grind size, location of the burner, and plant temperature.

The plots in Figure G7.3 and G7.4 present the mean dynamic modulus at 21°C and 37°C at specific frequencies for a more direct comparison. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. Low modulus values at intermediate temperatures and frequencies (21°C and 5 Hz in Figure G7.3) are considered desirable in thin asphalt pavements (less than 4”) for fatigue cracking resistance. Mixtures with a lower stiffness can deform more easily without building up large stresses. The D Construction PG 70-28 plant SMA and PG 58-28 SMA mixes have lower dynamic modulus values than their Curran counterpart mixes, but the differences are not statistically significant. Adding 11 percent RAP to the SMA mixes did not significantly impact their dynamic modulus values at intermediate temperatures.

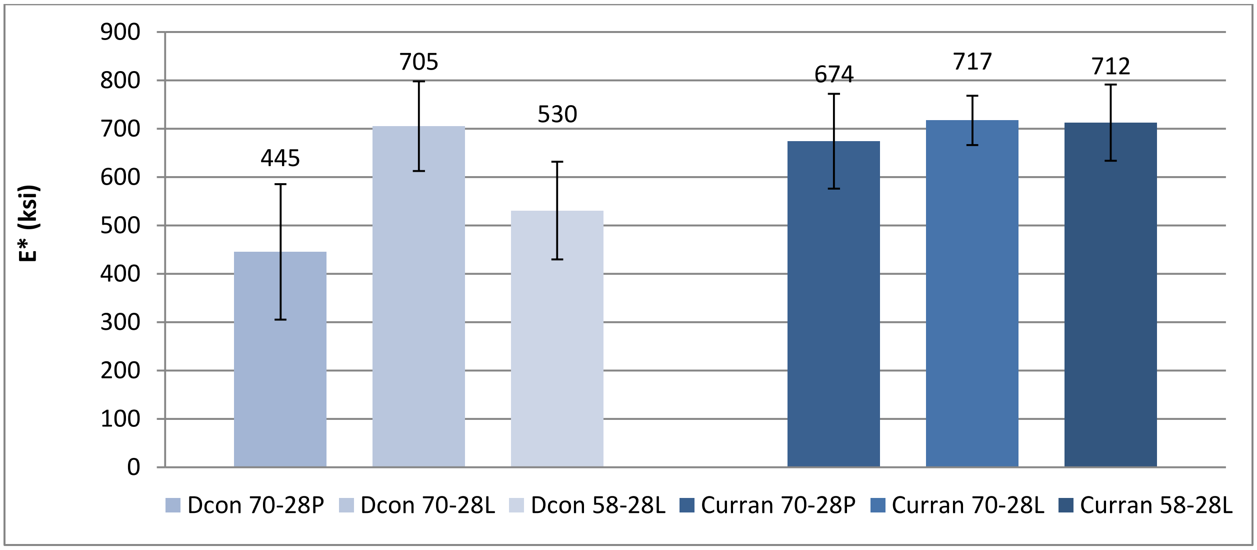


Figure G7.3. Dynamic modulus comparison at 21°C, 5 Hz

The dynamic modulus at 37°C and 0.1 Hz is evaluated in Figure G7.4 since the modulus of asphalt mixtures at high temperatures and low frequencies is an indicator of rutting resistance.

While the Curran mixes have larger dynamic modulus values than the D Construction mixes, there are no statistical differences. Using a PG 58-28 with 12% GTR in the SMA produced similar modulus values as using the polymer modified PG 70-28.

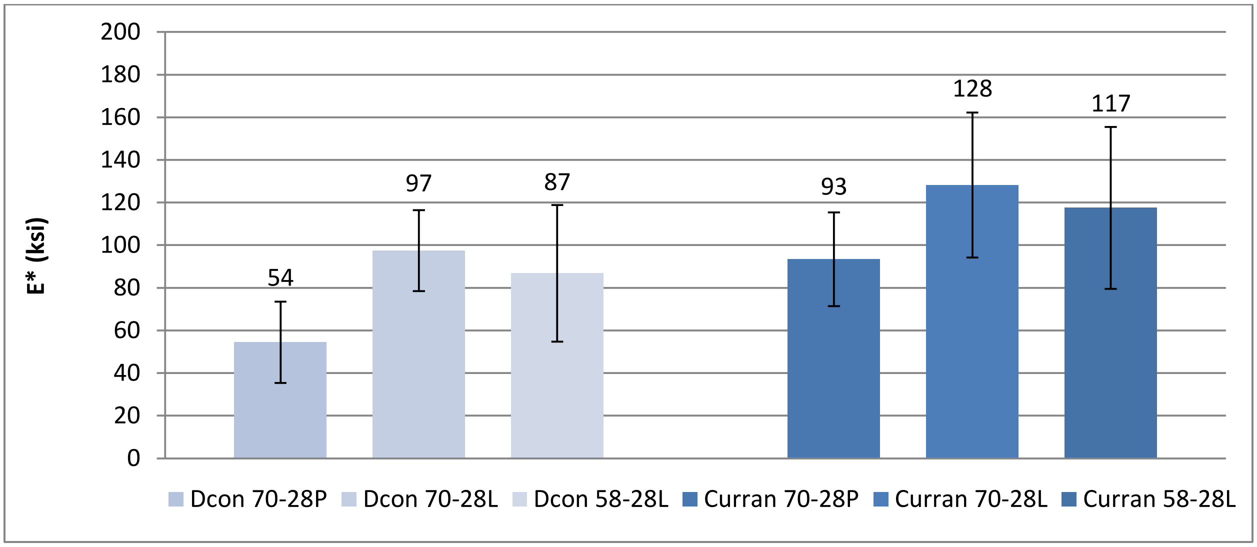


Figure G7.4. Dynamic modulus comparison at 37°C, 0.1 Hz

Flow Number

The flow number test measures the permanent deformation resistance of asphalt mixtures by applying a repeated dynamic load to a sample for up to several thousand load cycles. The flow number is defined as the number of load cycles an asphalt mixture can tolerate until it flows. Cumulative permanent deformation in the sample is plotted versus load cycles. The flow number is reached at the onset of tertiary flow.

Tests were conducted following procedures used in NCHRP Report 465. Samples used in dynamic modulus test were used for the flow number test since the dynamic modulus test is nondestructive. The samples were placed in a hydraulically loaded universal testing machine, unconfined, with a testing temperature of 37°C to simulate the climactic conditions that cause pavement to be susceptible to rutting. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 sec followed by 0.9 sec of dwell time. The loading cycle was repeated for a total of 10,000 load cycles. Three LVDT's were attached to each sample during the test to measure the cumulative strains.

Test results are presented in Figure G7.5. Error bars on the chart represent a distance of two standard errors from the mean for an estimate of the 95% confidence interval. The flow numbers for all six mixes were very high indicating good resistance to permanent deformation. Four of the mixes have flow numbers greater than 10,000 since they did not have a cumulative strain greater than 5 percent after 10,000 load cycles. Just as the D Construction plant produced PG 70-28 SMA and the PG 58-28 SMA had the lowest dynamic modulus values, they also had the lowest flow number values, albeit still very high.

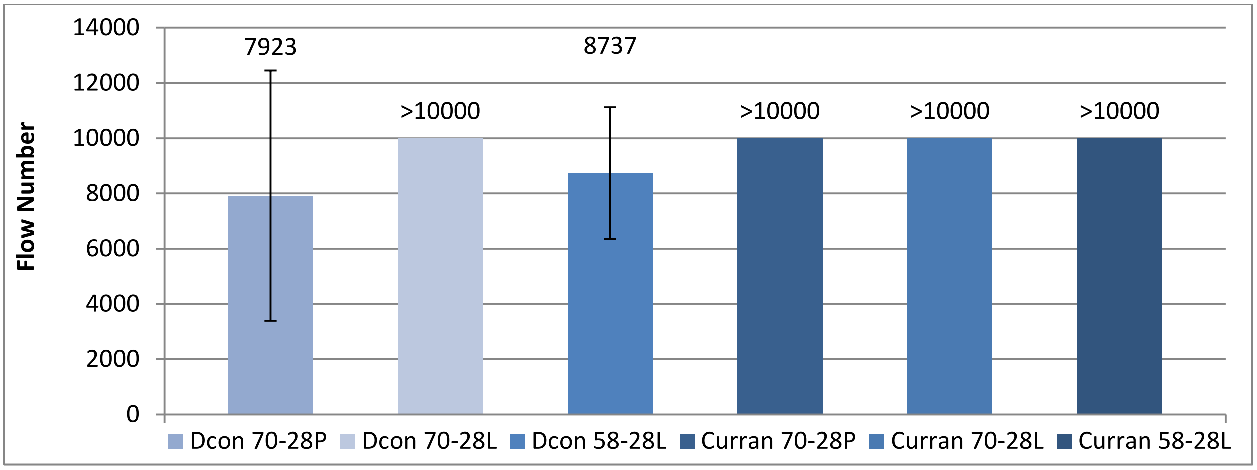


Figure G7.5. Flow number test results

Beam Fatigue

Fatigue cracking is the major cracking distress in asphalt pavements caused by repeated heavy traffic loads. Pavements that have a higher resistance to tensile strains that develop at the bottom of an asphalt layer due to repeated traffic will have a greater resistance to fatigue cracking. The four-point beam fatigue test following AASHTO T321 was conducted to evaluate the load associated cracking resistance of the mixtures.

The mixes were compacted in a linear kneading compactor to create slabs with 7% air voids. The slabs were saw-cut into beams with dimensions 15 inches in length, 2.5 inches in width, and 2 inches in height. The beams were tested in a strain controlled mode of loading at 20°C with haversine wave pulses applied to the beam at 10 Hz. Six beams were tested, each at a different strain level (375 to 1000 µstrain), until the flexural stiffness of the beam was reduced to 50% of the initial stiffness. A log-log regression was performed between strain and the number of cycles to failure (N_f). The relationship between strain and N_f can be modeled using the power law relationship as presented in Equation 1.

$$N_f = K1\left(\frac{1}{\varepsilon_o}\right)^{K2} \qquad (1)$$

where: N_f = cycles to failure; ε_o = flexural strain; and K1 and K2 = regression constants.

The beam fatigue test results, as shown by strain versus “loading cycles to failure” curves, are presented in Figure G7.6 and G7.7. The fatigue curve model coefficients, average initial stiffness, and R^2 values are presented in Table G7.4. All the RAS-SMA mixes possess excellent fatigue properties in a strain-controlled mode of loading. As binder course mixes in a four inch asphalt overlay, this will help reduce the build-up of large stresses in the pavement.

The PG 58-28 with 12 percent GTR mixes contained higher than usual variability, most likely due to the high amount of recycled products (GTR, RAP, RAS) present in the mixes. Even with the high amount of recycled products, the GTR mixes still exhibited fatigue performance similar to the polymer modified SMA mixes. The GTR mixes had the lowest average initial flexural stiffness of all the mixes (Table G7.2) indicating they are a more ductile and compliant mix at intermediate temperatures.

For the D Construction 70-28 SMA mixes, the plant produced SMA performed similar to the laboratory produced SMA. However, for the Curran 70-28 SMA mixes, the plant produced SMA mixes exhibited lower fatigue lives at lower strain levels than the laboratory produced SMA.

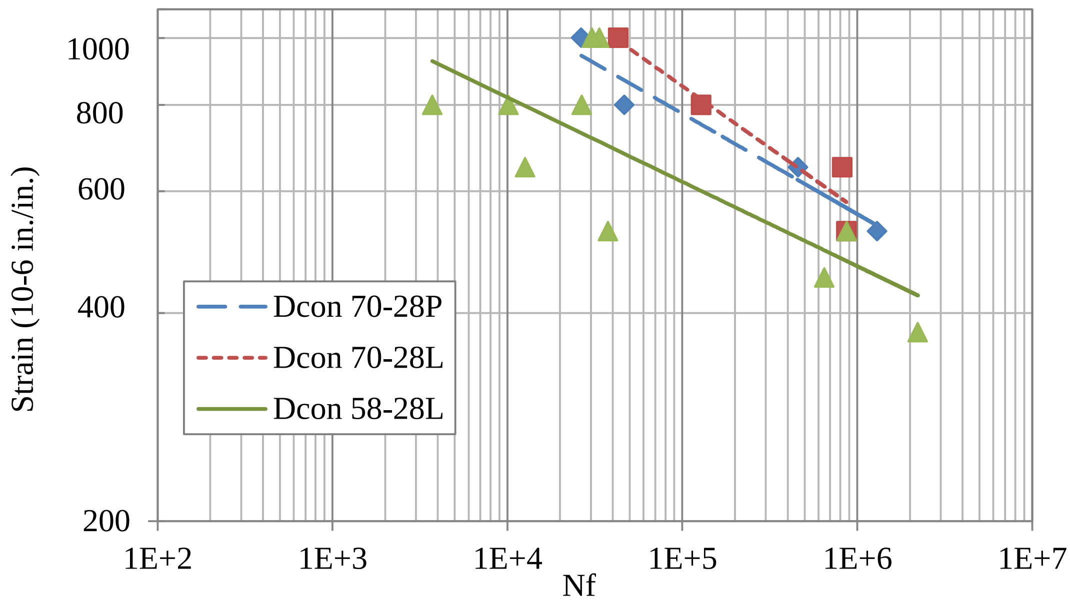


Figure G7.6. ε-N fatigue curves (D Construction)

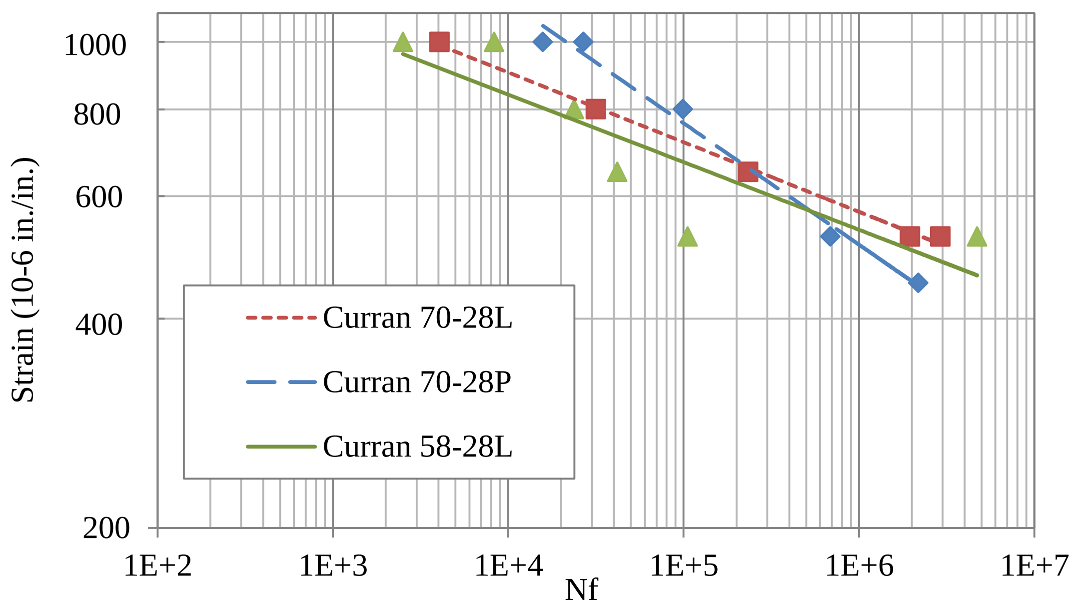


Figure G7.7. ε-N fatigue curves (Curran)

The fatigue endurance limit (FEL) of each mixture was also predicted. If tensile strains are low enough in a pavement structure, the pavement has the ability to heal and therefore no damage cumulates over an indefinite number of load cycles. The level of this strain is referred to as the FEL. The FEL of each mixture was estimated using the lower 95% prediction limit at 50 million load cycles as proposed in NCHRP Report 646 and shown in Equation 2.

$$\text{Lower Prediction Limit} = \hat{y}_o - t_\alpha s\sqrt{1+\frac{1}{n}+\frac{(x_o-\bar{x})^2}{S_{xx}}} \quad (2)$$

where:
y_o = the one-sided lower 95% prediction interval at the micro-strain level corresponding to 50,000,000 cycles;
t_α = value of t distribution for n-2 degrees of freedom for a significance level of 0.05;
s = standard error of the regression analysis;
n = number of samples;
S_{xx} = sum of squares of the x values;
x_o = log 50,000,000; and
$\bar{x}$ = average of the fatigue life results.

The FEL estimates are also presented in Table G7.3. All the Curran mixes exhibited a higher and thus more desirable endurance limits than the D Construction mixes, even with 11 percent added RAP and higher binder replacements. These results are counter intuitive since a higher percentage of recycled binder can increase the stiffness of an asphalt mixture and reduce its fatigue life in a strain-controlled mode of loading. The Curran mixes may possess higher endurance limits because they have a higher total binder content than the D Construction mixes (6.2 versus 6.0).

Table G7.3. Beam fatigue results

Mix ID	% Binder Replacement	Average Initial Stiffness (Mpa)	K1	K2	R^2	Endurance Limit (Micro-strain)
Dcon 70-28P	21.0	2222	5.97E-16	6.51	0.946	195
Dcon 70-28L	21.0	2024	2.92E-11	5.07	0.907	138
Dcon 58-28L	21.0	1666	2.15E-11	4.86	0.593	152
Curran 70-28P	35.0	2233	2.61E-13	5.64	0.985	208
Curran 70-28L	35.0	2085	5.26E-27	9.95	0.996	359
Curran 58-28L	35.0	1823	8.29E-20	7.56	0.735	204

Semi-Circular Bend Test

The low temperature fracture properties of the mixtures were obtained from SCB tests by following the procedure in "Investigation of Low Temperature Cracking in Asphalt**"** **(**Marasteanu et al., 2007). Testing was conducted at four different low temperatures: -12°C, -18°C, -24°C, and -28°C, with two replicate samples tested at each temperate.

All tests were performed inside an environmental chamber, and liquid nitrogen was used to obtain the required low temperature. The temperature was controlled by the environmental chamber temperature controller and verified using an independent platinum resistive-thermal-device (RTD) thermometer. The load line displacement (LLD) was measured on both faces of the test specimens using a vertically mounted Epsilon extensometer with 38 mm gage length and ±1 mm range. One end was mounted on a button that was permanently fixed on a specially made frame, and the other end was attached to a metal button glued to the sample. The average LLD measurement was used for each specimen. The crack mouth opening displacement (CMOD) was recorded by an Epsilon clip gage with 10 mm gage length and a +2.5 and -1.0 mm range. The clip gage was attached at the bottom of the specimen. A constant CMOD rate of 0.0005mm/s was used and the load and load line displacement (P-u), as well as the load versus LLD curves were plotted. A contact load with a maximum load of 0.3 kN was applied before the actual loading to ensure uniform contact between the loading plate and the specimen. The testing was stopped when the load dropped to 0.5 kN in the post peak region. The load and load line displacement data were used to calculate the fracture toughness and fracture energy (G_f).

The fracture energy (G_f) parameter is presented in Figures G7.8 and G7.9. The laboratory test results were analyzed to evaluate the effect of the various RAS treatments on the fracture energy. MacAnova statistical software package was utilized to perform a statistical analysis. The analysis of variance (ANOVA) was used to examine the differences among the mean response values of the different treatment groups. The significance of the differences was tested at 0.05 level of error. The analysis of variance was conducted with the assumption that the errors in the data are independently normal with constant variance.

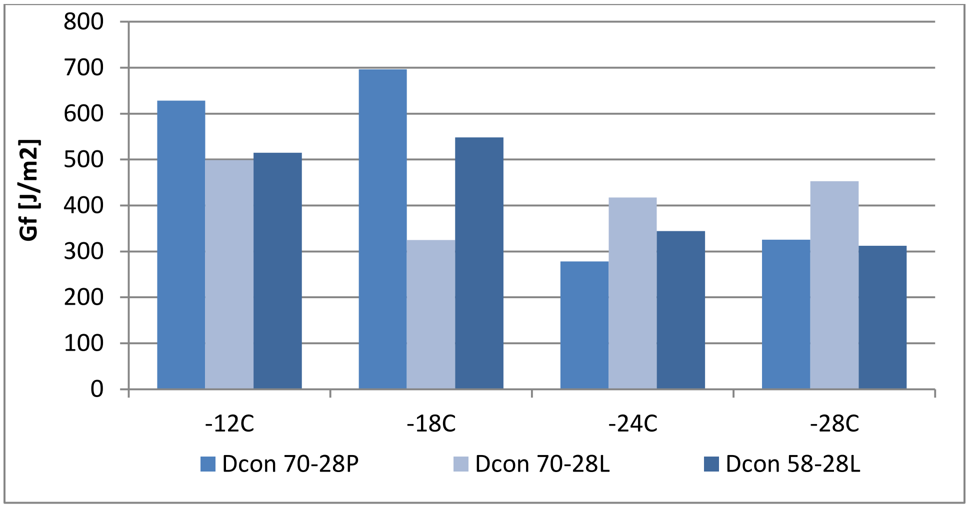


Figure G7.8. SMA fracture energy, G_f (D Construction)

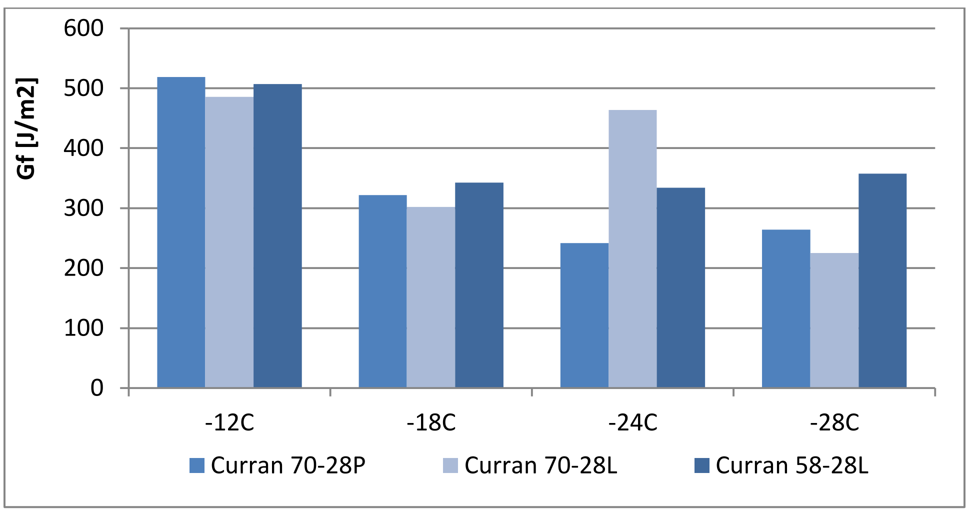


Figure G7.9. SMA fracture energy, G_f (Curran)

The Gf group means of each RAS treatment level was compared using a pair-wise comparison to rank the RAS treatment levels with regard to fracture energy. The outcome is reported in Table G7.4 for the D Construction mixes and Table 7.5 for the Curran mixes, in which statistically similar RAS treatments are grouped together. Letter A indicates the best performing group of mixtures; letter B the second best, and so on. Groups with the same letter are not statistically different, whereas mixtures with different letters are statistically different.

The fracture energy results for the D Construction mixes in Table G7.4 show there are no statistical differences between the three mix types. Likewise, fracture energy results for the Curran mixes in Table G7.5 also show there are no statistical differences between the three mix types. Using a PG 58-28 (w/ GTR) in place of a polymer modified PG 70-28 did not affect the fracture energy of the SMA. Additionally, the PG 70-28 SMA mixes produced in the field had a similar low temperature fracture energy as the PG 70-28 SMA mixes produced in the laboratory. Although the D Construction SMA mixes have higher fracture energies than the Curran SMA mixes, the difference between the group means between these two mix types was not statistically significant at the 95 percent confidence level. The p-value was 0.0674. Therefore, adding 11 percent RAP to the SMA mix design did not change its fracture energy.

Table G7.4. Ranking of D Con mixes by G_f mean value for -12, -18, -24, and -28°C temps

Rank	Treatment	Group mean, G_f [J/m^2]
A	Dcon 70-28P	482
A	Dcon 70-28L	432
A	Dcon 58-28L	430

Table G7.5 Ranking of Curran mixes by G_f mean value for -12, -18, -24, and -28°C temps

Rank	Treatment	Group mean, G_f [J/m^2]
A	Curran 70-28P	337
A	Curran 70-28L	369
A	Curran 58-28L	385

G8. Field Evaluations

The project team completed two pavement condition surveys for the IDOT I-80 demonstration project and the Jane Addams Memorial Tollway project. The first surveys were completed in October 2011 following construction and in March 2012, the spring following the first winter season after paving. Two 500-foot sections were randomly selected in the eastbound and westbound lanes on I-80 in the location where D Construction placed the PG 70-28 SMA with five percent RAS, and on the Jane Addams Memorial Tollway where Curran placed the PG 70-28 SMA with five percent RAS and 11 percent RAP.

The surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* published by the Federal Highway Administration. The field condition surveys conducted one winter season after the demonstration project revealed no pavement distresses in the I-80 and Jane Addams Memorial Tollway test sections. Pictures of the I-80 tests section are shown below in Figures G8.1 and G8.2. A Power-Point Presentation of the condition surveys by 500-foot sections are available for viewing on the TPF-5(213) website.

Figure G8.1. EB lane I-80 mile 135.5 **Figure G8.2. WB lane I-80 mile 124.0**

G9. Conclusions

An IDOT demonstration project was conducted as part of Transportation Pooled Fund 5-213 to evaluate the performance benefits of replacing fibers and virgin asphalt with post-consumer RAS in SMA. Several different plant and laboratory SMA mixes were produced using post-consumer RAS with two types of binders, a polymer modified PG 70-28 and a PG 58-28 with 12 percent GTR. The SMA mix design used by D Construction contained 5 percent RAS and the SMA mix design used by Curran contained 5 percent RAS and 11 percent RAP. Laboratory and plant produced mixes were evaluated by conducting the following tests: dynamic modulus, flow number, four-point beam fatigue, semi-circular bending, and binder extraction and characterization. Two pavement condition surveys of test sections were also conducted after paving. The results of the study are summarized below:

- Observations from the demonstration project show the SMA pavements with RAS were successfully produced and constructed while meeting IDOT's quality assurance requirements. The SMA's did not have any binder drain-down when 5 percent RAS was utilized as a stabilizer.
- The 58-28 SMA with 12 percent GTR exhibited similar rutting resistance and low temperature cracking properties as the polymer modified PG 70-28 SMA. The SMA with GTR also exhibited longer fatigue lives than the PG 70-28 SMA but contained more variability. This shows that a softer, base binder with GTR, RAP, and RAS can be used to produce an SMA with similar performance properties as an SMA that uses a more expensive polymer modified PG 70-28.
- The addition of 5 percent RAS in the D Construction SMA mixes increased total binder blend from a PG 70-28 to a PG 70-22. The addition of 5 percent RAS and 11 percent RAP in the Curran SMA mixes increased the total binder blend from a PG 70-28 to a PG 82-16. Blending 12 percent GTR to the PG 58-28 increased its binder grade to a PG 76-22, and adding 5 percent RAS and 11 percent RAP to the GTR-SMA mix increased its binder grade to a PG 76-16. For every 1 percent increase in RAS, the low temperature PG of the SMA mixes will increase 1.4°C; and for every 1 percent increase in RAP, the low temperature of the PG of the SMA mixes will increase about 0.1°C.

- The PG 58-28 with GTR and the PG 70-28 SMA mixes displayed excellent rutting resistance, based on the flow number test results.
- The Curran SMA mixes with 11 percent RAP had a greater dynamic modulus than the D Construction mixes with no RAP. The plant produced SMA mixes had a lower dynamic modulus than the laboratory produced SMA mixes. This may be due to the RAS binder melting and blending with the virgin binder more effectively during laboratory mixing than during plant production.
- All the RAS-SMA mixes exhibited excellent fatigue properties in a strain-controlled mode of loading. For the D Construction PG 70-28 SMA mixes, the plant produced SMA performed similar to the laboratory produced SMA. However, for the Curran PG 70-28 SMA mixes, the plant produced SMA mixes exhibited lower fatigue lives at lower strain levels than the laboratory produced SMA.
- The SCB test results for the SMA mixes show there were no statistical differences at the 95 percent confidence level in low temperature fracture energy between the PG 58-28 (w/ GTR) and PG 70-28 SMA mixes, between the plant produced PG 70-28 and laboratory produced PG 70-28 SMA mixes, nor between the Curran mixes with 11 percent RAP and the D Construction mixes with no RAP.
- Field condition surveys conducted one winter season after the demonstration project revealed no pavement distresses in the I-80 and Jane Addams Memorial Tollway test sections.

G10. IDOT Demonstration Project Acknowledgments

The researchers gratefully acknowledge the support of Abdul Dahhan at the Illinois DOT. The research work was sponsored by Federal Highway Administration (FHWA) TPF-5(213) and the Transportation Pooled Fund (TPF) partners: Missouri (lead agency), California, Colorado, Illinois, Indiana, Iowa, Minnesota, and Wisconsin DOTs.