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Performance testing of asphalt pavements with recycled asphalt shingles from multiple field trials

Andrew A. Cascione [a,*], R. Christopher Williams [b], Jianhua Yu [a]

[a] Department of Civil, Construction, and Environmental Engineering, Iowa State University, 174 Town Engineering Building, Ames, IA 50011, United States
[b] Department of Civil, Construction, and Environmental Engineering, Iowa State University, 490 Town Engineering Building, Ames, IA 50011, United States

HIGHLIGHTS

- RAS was successfully used in field trials with warm mix asphalt, RAP, and ground tire rubber.
- RAS was successfully used as a replacement for fibers and asphalt in stone matrix asphalt.
- RAS mixes demonstrated acceptable fatigue and fracture properties in laboratory tests.
- The addition of RAP to a RAS mix design decreased its fracture energy at low temperatures.
- Larger RAS particle sizes increased the amount of pavement transverse cracking.

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ABSTRACT

Transportation agencies have become increasingly interested in modifying hot mix asphalt (HMA) pavements with recycled asphalt shingles (RAS), yet they share common questions about the effect of RAS on the performance of HMA. In this study, the field and laboratory performance of RAS mixes produced from seven different transportation agencies are investigated as part of Transportation Pooled Fund TPF-5 (213). Field demonstration projects were conducted that evaluated multiple aspects of RAS that include RAS grind size, RAS percentage, RAS source, 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 and tested for their permanent deformation, fatigue cracking, and low temperature cracking performance. Recovered asphalt binder from each mix was also evaluated. Pavement condition surveys were conducted for each project after completion.

1. Introduction

Waste asphalt shingles have historically been considered a solid waste and placed in landfills. In the United States (US), nine million metric tons (Mt) of asphalt shingle waste are generated each year from the renovation and construction of roofs, and another 1 Mt of waste are produced during the manufacturing process of new shingles [1]. In total, asphalt roofing shingle waste represents up to 3% of all construction and demolition debris in the US [2].

A new sustainable construction technology emerging in the US is the recycling of asphalt roofing shingles for use in asphalt pavements. By diverting waste shingles from landfills and incorporating them into asphalt pavements, what was previously considered a solid waste can now be upcycled into the transportation network for constructing driving surfaces. This innovative technology reduces the environmental impacts resulting from road construction by reducing the amount of virgin materials used in hot mix asphalt (HMA) [3]. Replacing virgin materials with recycled asphalt shingles (RAS) saves resources, reduces the energy burned from using raw materials, eases landfill pressures, and reduces the demand of extraction [4,5]. Using RAS in asphalt pavements can also reduce greenhouse gas emissions produced during road construction by 9–12% [6].

Fluctuations in crude petroleum prices have considerably raised the cost of asphalt binder in the past several years. This increase, coupled with the advancement of shingle processing technology, has created favorable market conditions for RAS to be used in asphalt pavements [7,8]. From 2009 to 2012, the estimated amount of RAS annually used in asphalt pavements in the United States more than doubled, from 0.7 million tons to 1.9 tons [9].

* Corresponding author.
E-mail addresses: aacascio@iastate.edu (A.A. Cascione), rwilliam@iastate.edu (R.C. Williams).

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The components of RAS allow it to be a good candidate as a secondary (recycled) material in asphalt mixtures. Recycled roofing shingles contain between 19% and 31% asphalt and include fine angular granules which can improve the resistance to permanent deformation. Shingles also contain fiberglass or cellulose backing, that when crushed during the recycling process, break down into fiber-like particles that may improve the cracking resistance of asphalt [10]. Various fiber modifiers, such as cellulose, polyester, and mineral fibers, have been widely used in asphalt mixtures [11]. Putman and Amirkhanian [12] demonstrated that recycled fibers obtained from waste streams can increase the tensile strength of asphalt mixtures.

With these benefits in mind, more state highway agencies are beginning to see the potential impact RAS could have in lowering the costs of pavements. However, little information about the long-term performance of pavements with RAS is known because it is a new material that agencies are beginning to use. The challenge agencies have when implanting the use of RAS materials, is developing a construction specification for RAS mixtures that ensures a product with similar qualities and performance to non-RAS mixtures. Several aspects about the sourcing and processing of RAS make it important for agencies to understand which factors about RAS affect the material properties essential for good pavement performance. This led to the creation of Transportation Pooled Fund TPF-5(213), 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 demonstration projects focused on evaluating different factors of RAS to determine how they influence the performance of pavements. RAS factors addressed in the different demonstration projects included the evaluation of RAS grind size, percentage of RAS in hot mix asphalt (HMA), RAS source (post-consumer versus post-manufacturer), 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 recycled asphalt pavement (RAP) only or no recycled product to mix designs containing RAS.

2. Experimental plan

To evaluate how different factors of RAS materials effect pavement performance, an experimental plan was developed where each state highway agency in the pooled fund study proposed a unique field demonstration project that investigated a different aspect of RAS mixes. Field demonstration projects were sponsored by the Department of Transportation agencies in Missouri, Iowa, Minnesota, Indiana, Wisconsin, Colorado, and Illinois. The asphalt mixes evaluated from each field demonstration project (Table 1) show the experimental plan of each agency.

The Missouri demonstration project investigates how RAS grind size affects pavement performance and how replacing 5% RAP with RAS affects the properties of the asphalt pavement. The Iowa demonstration project investigates asphalt mixes with an increasing percentage of RAS. The Minnesota demonstration project investigates the difference between using post-consumer (PC) RAS and post-manufacturer (PM) RAS. The Indiana demonstration project investigates replacing RAS with RAP in asphalt mixes and the effect of producing RAS at reduced plant temperatures by using warm mix asphalt (WMA) foaming technology during production. The Wisconsin demonstration project investigates the effect of using Evotherm® 3G chemical WMA additive as a compaction aid at hot mix temperatures in mixes that contain RAS. The Colorado demonstration project investigates using 3% RAS as a replacement for 5% RAP. The Illinois demonstration project investigates using 5% RAS in stone asphalt matrix (SMA) in place of added fibers. While SMA mixes are always designed with fibers to prevent drain-down of the asphalt binder due to its gap-graded aggregate structure, the Illinois mixes did not contain any fibers since RAS has fibers in it. The Illinois project also contained different types of mixes to evaluate mixes produced with 0% RAP versus 11% RAP, mixes produced in the field versus mixes produced in the laboratory, and mixes produced with ground tire rubber (GTR) modified binder versus polymer modified binder.

During each field demonstration project, representative samples of each RAS source and asphalt mixture were collected for binder characterization and mixture laboratory performance testing. 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. Solvent was removed from the extract by following the rotovaper recovery process in ASTM D5404. 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 by following AASHTO T27. For the RAS samples, a dry gradation was conducted prior to extraction to evaluate the grind size distribution of 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.

2.1. Dynamic modulus

The dynamic modulus $|E^*|$ 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 that 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 pavement's strains and displacements.

The test was conducted by following AASHTO T342. Replicate test specimens of each asphalt mixture measured 100 mm in diameter and 150 mm in height at 7 ± 0.5% air voids. Specimens were tested by applying a continuous sinusoidal load at nine 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 microstrain in the sample. A servo-hydraulic testing machine capable of applying a load up to 25 kN was used to test the asphalt mixture specimens. The testing machine 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 dynamic modulus test data was used to construct master curves that plot dynamic modulus over a wide frequency range at a 21 °C reference temperature.

2.2. Flow number

The flow number test was conducted to measure the permanent deformation resistance of asphalt mixtures. Specimens of 100 mm in diameter and 150 mm in height with 7 ± 0.5% air voids were placed in a servo-hydraulic testing machine, unconfined, with a testing temperature of 37 °C. An actuator applied a vertical haversine pulse load of 600 kPa for 0.1 s followed by 0.9 s of dwell time. The loading cycles were repeated for a total of 10,000 load cycles or until the specimen reached 5% cumulative strain. Three LVDT's were attached to each sample during the test to measure the cumulative strains. 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.

2.3. Four-point bending beam

Four-point bending beam 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 bending beam apparatus. The bending beam apparatus was housed in an environmental chamber maintained at the testing temperature of 20 ± 0.5 °C. 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.

Table 1
Mix design properties.

State agency	Mix ID	% RAS	% RAP	% Binder replaced	RAS source	Mix source	PG[e]	% GTR[f]	NMAS[g] (mm)	Design gyrations
Missouri	15 RAP	0	15	14.9	–	Plant	64–22	10	12.5	80
Missouri	5 FRAS[a]/10 RAP	5	10	30.2	PC[c]	Plant	64–22	10	12.5	80
Missouri	5 CRAS[b]/10 RAP	5	10	30.2	PC	Plant	64–22	10	12.5	80
Iowa	0 RAS	0	0	0	–	Plant	58–28	–	12.5	76
Iowa	4 RAS	4	0	16.3	PC	Plant	58–28	–	12.5	76
Iowa	5 RAS	5	0	19.4	PC	Plant	58–28	–	12.5	76
Iowa	6 RAS	6	0	22.8	PC	Plant	58–28	–	12.5	76
Minnesota	30 RAP	0	30	33.3	–	Plant	58–28	–	12.5	90
Minnesota	5 PC RAS	5	0	26.0	PC	Plant	58–28	–	12.5	90
Minnesota	5 PM RAS	5	0	18.8	PM[d]	Plant	58–28	–	12.5	90
Indiana	15 RAP HMA	0	15	19.3	–	Plant	70–22	–	9.5	100
Indiana	3 RAS HMA	3	0	12.9	PC	Plant	70–22	–	9.5	100
Indiana	3 RAS WMA	3	0	12.9	PC	Plant	70–22	–	9.5	100
Wisconsin	Evotherm® 3G	3	13	30.4	PC	Plant	58–28	–	12.5	75
Wisconsin	No Evotherm®	3	13	30.4	PC	Plant	58–28	–	12.5	75
Colorado	20 RAP	0	20	17.6	–	Plant	64–28	–	12.5	100
Colorado	3 RAS/15 RAP	3	15	23.1	PM	Plant	64–28	–	12.5	100
Illinois	0 RAS/5 RAP Field	5	0	21.0	PC	Plant	70–28	–	12.5	80
Illinois	0 RAS/5 RAP Lab	5	0	21.0	PC	Lab	70–28	–	12.5	80
Illinois	0 RAS/5 RAP GTR	5	0	21.0	PC	Lab	58–28	12	12.5	80
Illinois	11 RAS/5 RAP Field	5	11	35.0	PC	Plant	70–28	–	12.5	80
Illinois	11 RAS/5 RAP Lab	5	11	35.0	PC	Lab	70–28	–	12.5	80
Illinois	11 RAS/5 RAP GTR	5	11	35.0	PC	Lab	58–28	12	12.5	80

[a] FRAS – finely ground RAS.
[b] CRAS – coarsely ground RAS.
[c] C – post consumer.
[d] PM – post manufactured.
[e] PG – performance grade of asphalt binder.
[f] GTR – terminally blended ground tire rubber modifier.
[g] NMAS – nominal maximum aggregate size.

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% reduction of the initial beam flexural stiffness.

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 [13]. 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), (Fig. 1). The relationship between strain and N_f can be modeled using the power law relationship as presented in Eq. (1)

$$N_f = K1(1/\varepsilon_o)^{-K2} \quad (1)$$

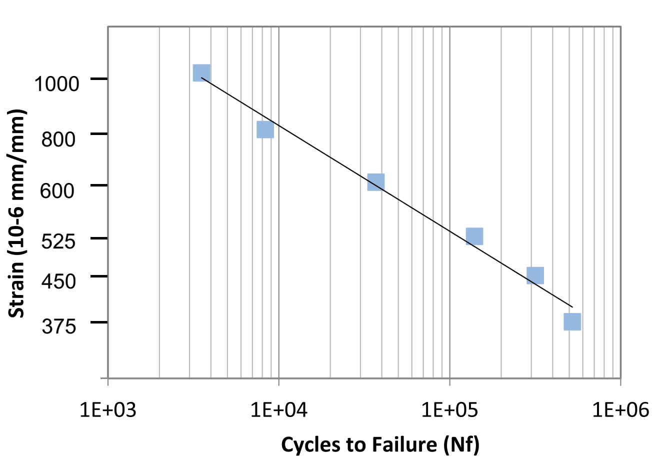

Fig. 1. Sample fatigue curve.

where N_f = cycles to failure, ε_o = 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.

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 [14] 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 in.) while the constant stress mode of loading best represents the performance of thick pavements (greater than 6 in.). 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. Prowell et al. [15] developed a practical equation for estimating the 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, the pavement could withstand an estimated 500 million loading cycles which represents 40 years of traffic. To calculate the FEL using this method, a linear regression is conducted on log transformed fatigue data (strain level and cycles to failure) to yield the t-distribution of strain level corresponding to 50 million load cycles. The FEL is determined as the strain level at the lower 95 percentile of the distribution. This technique uses Eq. (2) to estimate the FEL

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

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.

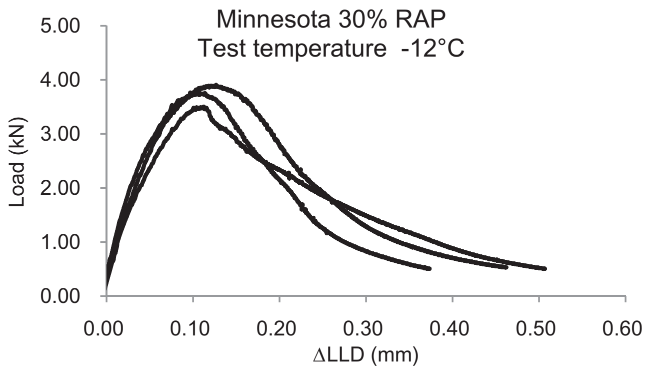

Fig. 2. Typical LLD versus load plot of three test replicates.

2.4. Semi-circular bending

To evaluate the low temperature fracture properties of the mixes, 150 mm diameter specimens containing 7 ± 0.5% air voids were compacted in Iowa State University's laboratory and delivered to the University of Minnesota for semi-circular bend (SCB) testing. SCB tests were conducted by following the procedure in "Investigation of Low Temperature Cracking in Asphalt" [16]. 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. Replicate 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 0.0005 mm/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). A typical load line displacement versus load plot is shown in Fig. 2.

2.5. Pavement condition surveys

Pavement condition surveys were conducted following the construction of each demonstration project and after each winter season for 1–4 years, depending on the project, to assess the field performance of the pavement concerning cracking, rutting, and raveling. The surveys were conducted in accordance with the *Distress Identification Manual for Long-Term Pavement Performance Program* by Federal Highway Administration. For each demonstration project, three 500-foot sections were randomly selected for each mix type paved. The surveys were conducted in these locations.

3. Results and discussion

3.1. RAS characterization

The results of the RAS gradation analysis are presented in Table 2. All the state agencies for the demonstration projects specified at least a 12.5 mm minus RAS grind size. Some RAS suppliers successfully produced a −9.5 mm RAS grind size. In the case of the

Table 2
RAS grind sizes.

Sieve size (mm)	Percent passing sieve size								
	MO PC-CRAS[a]	MO PC-FRAS[b]	IA PC[c]	MN PM[d]	MN PC	IN PC	WI PC	CO PM	IL PC
19	100	100	100	100	100	100	100	100	100
12.5	98	100	97	100	100	100	100	99	100
9.5	94	99	95	95	99	97	99	95	100
4.75	75	82	84	70	85	74	83	70	91
2.36	62	67	67	56	73	62	70	55	74
1.18	42	43	44	32	49	38	47	31	48
0.6	22	21	22	12	24	18	24	13	24
0.3	12	12	10	4	10	9	11	6	11
0.15	5	5	3	1	3	4	3	2	3
0.075	1.2	0.9	0.6	0.4	0.5	0.7	0.6	0.3	0.5

[a] CRAS – coarsely ground RAS.
[b] FRAS – finely ground RAS.
[c] PC– post consumer.
[d] PM – post manufactured.

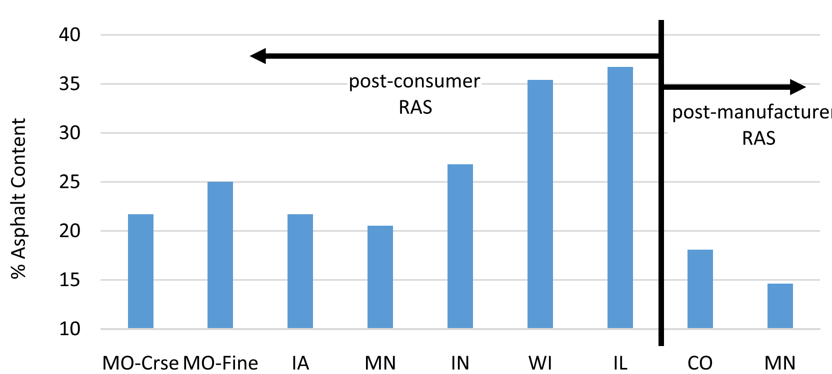

Fig. 3. RAS percent asphalt content.

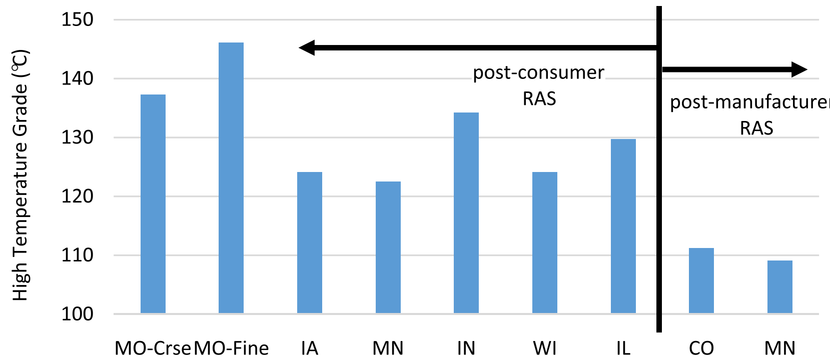

Fig. 4. RAS high temperature performance grade.

Table 3
Asphalt binder performance grades.

State agency	Mix ID	PG of base binder sampled from HMA plant		PG of extracted binder from field HMA sample	
		High temp (°C)	Low temp (°C)	High temp (°C)	Low temp (°C)
Missouri	15 RAP	70.3	−22.8	75.0	−16.8
Missouri	5 FRAS/10 RAP	70.3	−22.8	90.1	−8.7
Missouri	5 CRAS/10 RAP	70.3	−22.8	88.3	−4.9
Iowa	0 RAS	61.1	−17.9	73.0	−19.7
Iowa	4 RAS	61.1	−17.9	75.8	−19.1
Iowa	5 RAS	61.1	−17.9	81.3	−16.8
Iowa	6 RAS	61.1	−17.9	86.1	−14.7
Minnesota	30 RAP	58[a]	−28[a]	68.8	−22.7
Minnesota	5 PC RAS	58[a]	−28[a]	71.1	−21.2
Minnesota	5 PM RAS	58[a]	−28[a]	71.3	−21.7
Indiana	15 RAP HMA	72.2	−24.2	75.6	−20.1
Indiana	3 RAS HMA	72.2	−24.2	77.6	−14.2
Indiana	3 RAS WMA	72.2	−24.2	78.8	−15.1
Wisconsin	Evotherm® 3G	60.7	−29.1	68.5	−24.0
Wisconsin	No Evotherm®	60.7	−29.1	69.5	−22.5
Colorado	20 RAP	66.4	−34.8	67.6	−27.5
Colorado	3 RAS/15 RAP	66.4	−34.8	71.9	−21.1
Illinois	5 RAS/0 RAP Field	73.2	−29.9	72.8	−24.3
Illinois	5 RAS/0 RAP Lab	73.2	−29.9	72.7	−23.7
Illinois	5 RAS/0 RAP Lab-GTR	73.2	−29.9	77.2	−21.3
Illinois	5 RAS/11 RAP Field	73.2	−29.9	82.8	−18.1
Illinois	5 RAS/11 RAP Lab	73.2	−29.9	84.4	−14.5
Illinois	5 RAS/11 RAP Lab-GTR	73.2	−29.9	81.8	−17.7

[a] Asphalt binder sample was not available. PG58-28 was the specified base binder.

Missouri demonstration project, a −9.5 mm grind was compared to a −12.5 mm grind.

The asphalt contents of the post-manufacturer RAS sources (Minnesota and Colorado) range from 14.6% to 18.1% asphalt (Fig. 3). 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% 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.

All the RAS sources were tested for their high temperature PG using the dynamic shear rheometer (DSR) (Fig. 4). 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-manufacturer 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-manufacturer RAS comes from waste produced during shingle manufacturing.

3.2. Recovered asphalt binder

The performance grades of the binder extracted from the field samples and the asphalt binder used during production are presented in Table 3. When RAS and/or RAP is added to the mix designs of each state demonstration project, the asphalt binder high temperature and low temperature performance grades increase as expected. While the increase in the high temperature PG will stiffen the asphalt mixture to help reduce permanent deformation, the increase in the low temperature PG 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

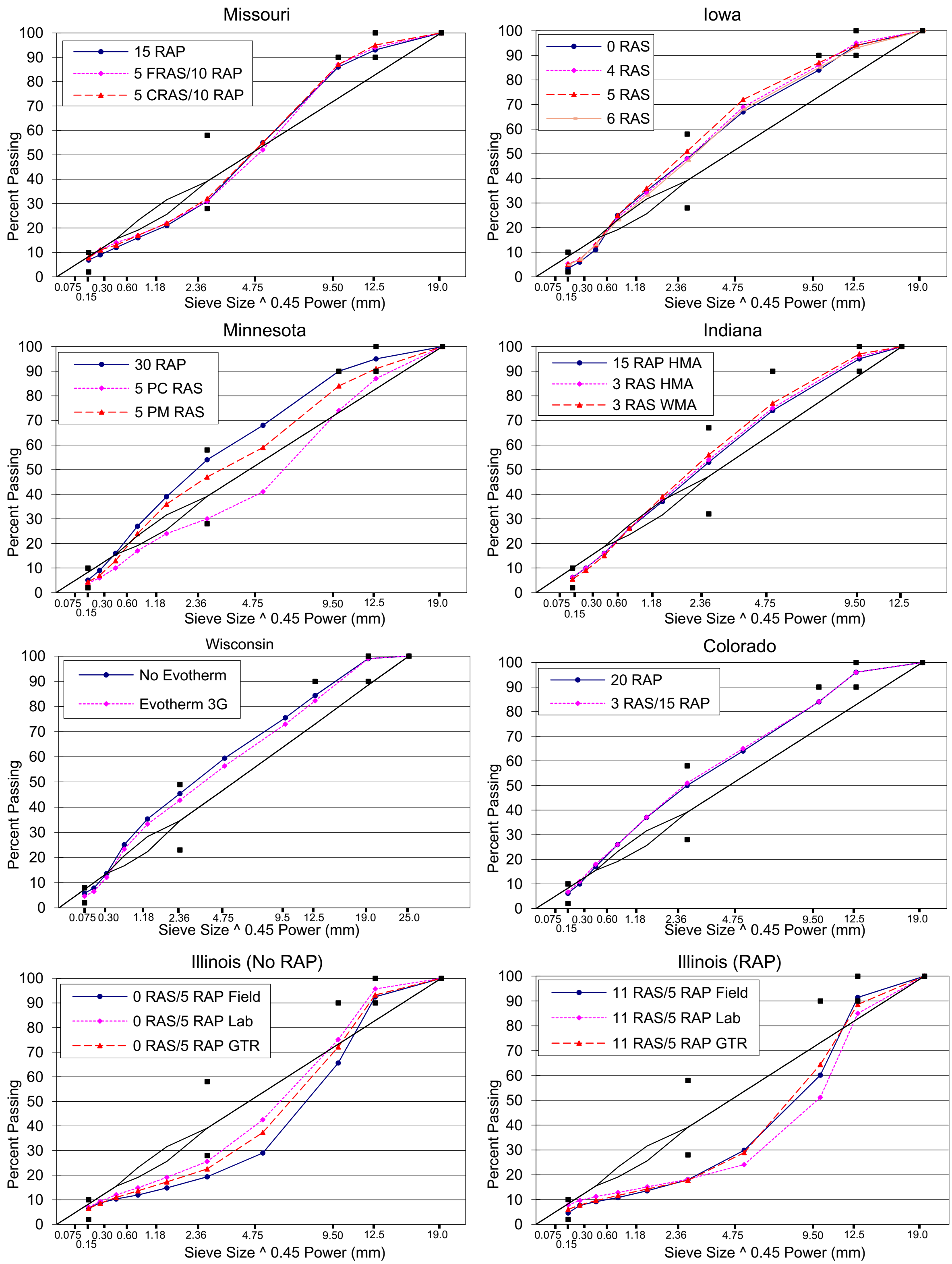

Fig. 5. Aggregate gradations.

to use a softer virgin binder with a lower PG. Since RAS and RAP have different performance grades and asphalt contents, it is important to know the specific properties of the recycled product to achieve the correct PG formulation.

The average results of all the mixes show that for every 1% increase in RAS, the low temperature grade will increase 1.9 °C; and for every 1% increase in RAP, the low temperature grade will increase 0.3 °C. Therefore, based on these mixes, 3% RAS or 20% 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% to 36.7%) 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. The average

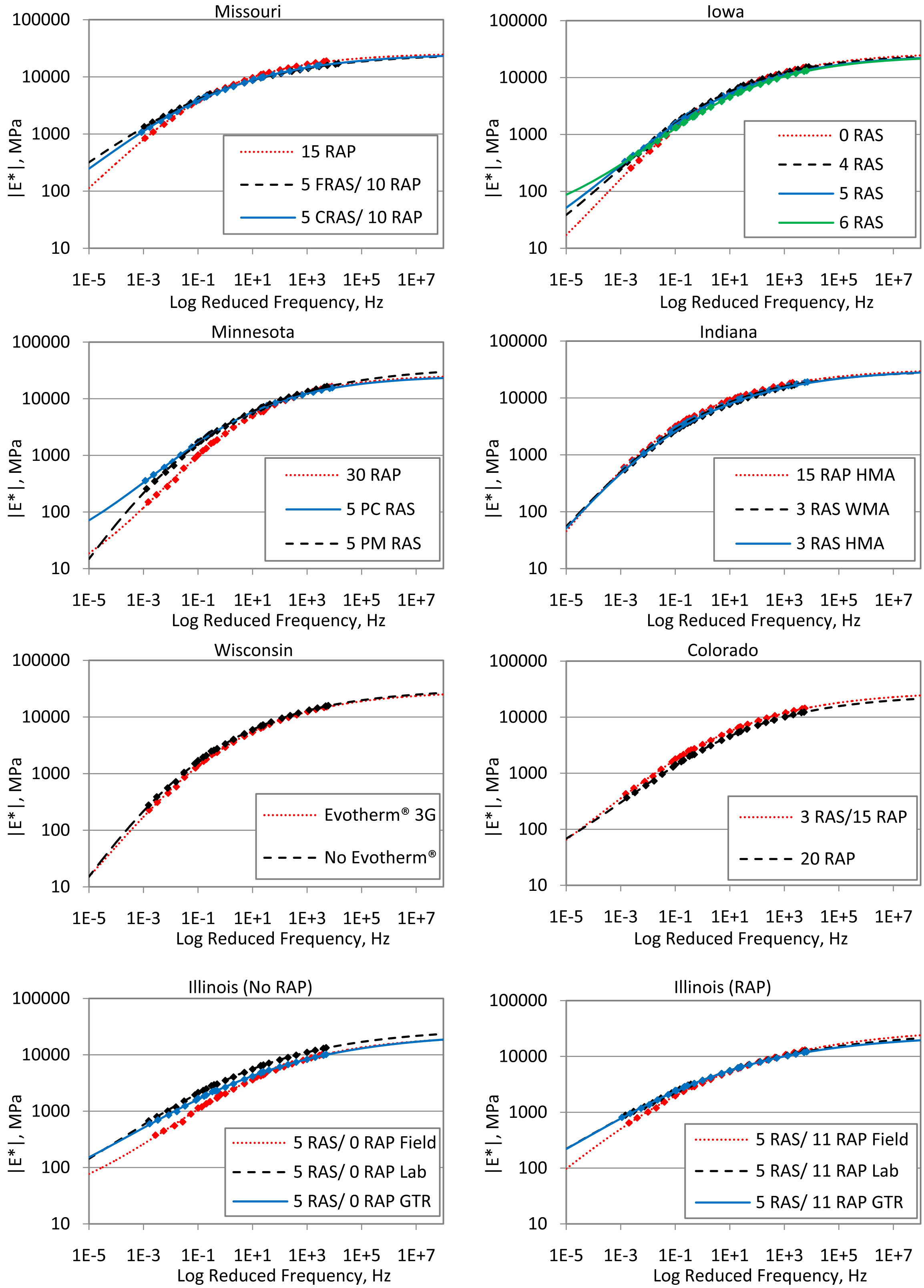

Fig. 6. Dynamic modulus master curves.

RAS asphalt content was 24.5% and the average optimum asphalt content of the mixtures was 5.5%. Using these values and the binder grading results, for every 1% increase in binder replacement with RAS, the low temperature grade will increase 0.43%. For every 1% increase in binder replacement with RAP, the low temperature grade will increase 0.3%. Therefore, to cap the increase in the low temperature performance grade by one grade bump (6 °C), either a maximum of 14% binder replacement with RAS binder could be used or a maximum of 20% binder replacement with RAP binder could be used.

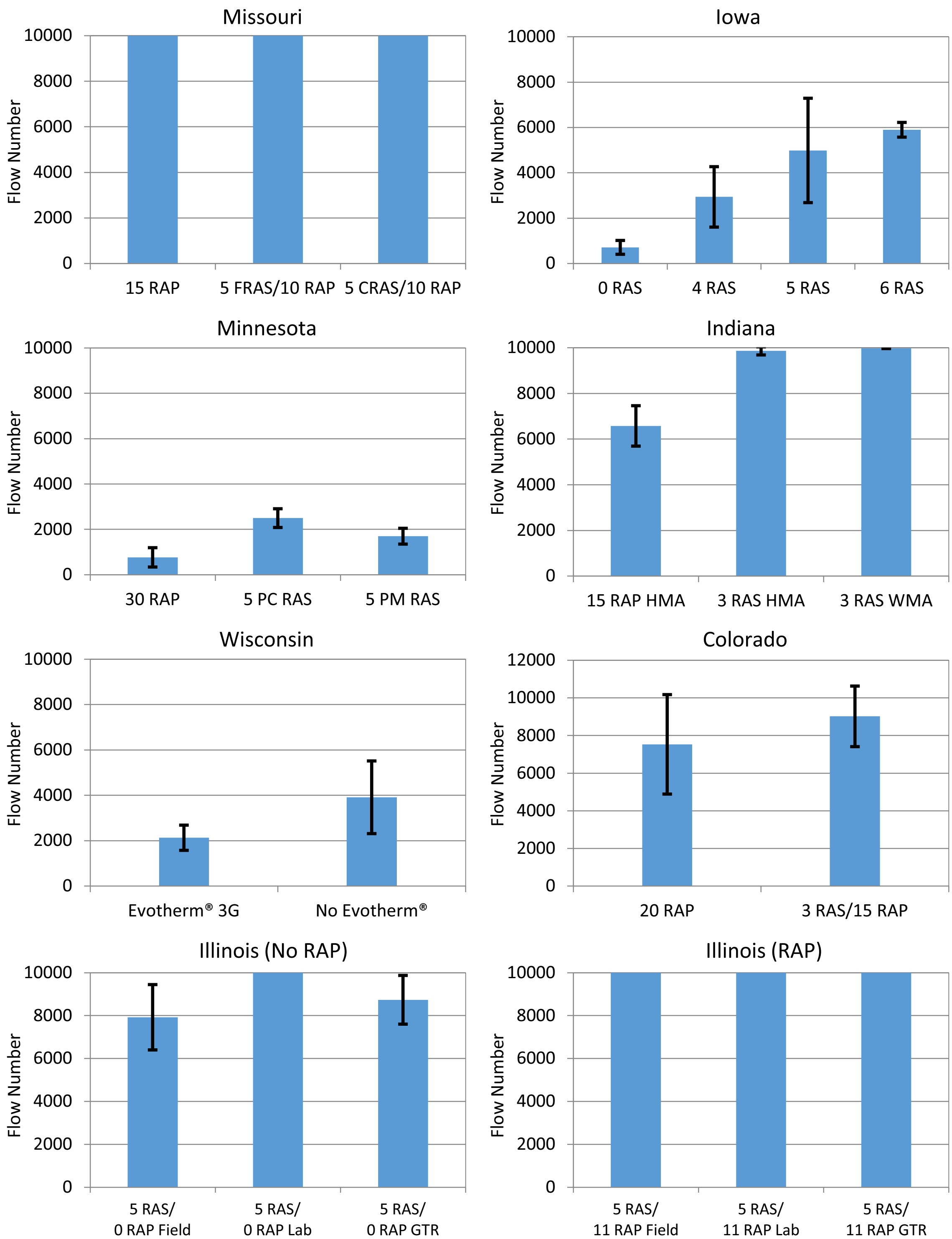

Fig. 7. Flow number.

This 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 for state highway agencies to consider multiple factors when developing a RAS construction specification for asphalt pavements.

3.3. Aggregate gradations

The aggregate gradations presented in Fig. 5 demonstrate the wide range of aggregate structures for asphalt mixes utilized in different states. Iowa, Indiana, Minnesota, Wisconsin, and Colorado utilize mixes with fine aggregate gradations that extend above the restricted zone on a 0.45 power chart, while Missouri mixes

Table 4
Flow number statistical grouping.

State agency	Mix ID	ANOVA *p*-value	Tukey statistical rank	Group mean flow number
Missouri	15 RAP	*	*	>10,000
Missouri	5 FRAS/10 RAP		*	>10,000
Missouri	5 CRAS/10 RAP		*	>10,000
Iowa	6 RAS	0.0007	A	5899
Iowa	5 RAS		A/B	4988
Iowa	4 RAS		B/C	2938
Iowa	0 RAS		C	711
Minnesota	5 PC RAS	<0.0001	A	2497
Minnesota	5 PM RAS		B	1700
Minnesota	30 RAP		C	767
Indiana	3 RAS WMA	<0.0001	A	9986
Indiana	3 RAS HMA		A	9865
Indiana	15 RAP		B	6578
Wisconsin	Evotherm® 3G	0.1425	A	2128
Wisconsin	No Evotherm®		A	3912
Colorado	20 RAP	0.4521	A	7533
Colorado	3 RAS/15 RAP		A	9020
Illinois	5 RAS/0 RAP Field	*	*	7923
Illinois	5 RAS/0 RAP Lab		*	>10,000
Illinois	5 RAS/0 RAP Lab-GTR		*	8737
Illinois	5 RAS/11 RAP Field		*	>10,000
Illinois	5 RAS/11 RAP Lab		*	>10,000
Illinois	5 RAS/11 RAP Lab-GTR		*	>10,000

* ANOVA analysis wasn't conducted since flow number reached maximum value of 10,000

Table 5
Fatigue model coefficients and predicted endurance limit.

State agency	Mix ID	$K1$	$K2$	R^2	Endurance limit (microstrain)
Missouri	15 RAP	5.15E−17	6.40	0.968	139
Missouri	5 FRAS/10 RAP	7.25E−19	6.91	0.992	145
Missouri	5 CRAS/10 RAP	2.07E−20	7.37	0.968	159
Iowa	0 RAS	1.43E−13	5.45	0.987	144
Iowa	4 RAS	6.75E−14	5.68	0.987	182
Iowa	5 RAS	1.97E−12	5.27	0.982	175
Iowa	6 RAS	7.07E−14	5.65	0.967	162
Minnesota	30 RAP	6.66E−11	4.51	0.982	89
Minnesota	5 PC RAS	2.22E−09	4.19	0.996	123
Minnesota	5 PM RAS	9.19E−12	4.90	0.994	131
Indiana	15 RAP HMA	7.04E−12	4.87	0.993	114
Indiana	3 RAS HMA	1.41E−11	4.77	0.970	118
Indiana	3 RAS WMA	1.17E−11	4.81	0.985	110
Wisconsin	Evotherm® 3G	1.70E−11	4.74	0.976	74
Wisconsin	No Evotherm®	3.75E−10	4.32	0.984	53
Colorado	20 RAP	2.34E−13	5.69	0.907	195
Colorado	3 RAS/15 RAP	9.22E−14	5.89	0.907	244
Illinois	5 RAS/0 RAP Field	5.97E−16	6.51	0.946	195
Illinois	5 RAS/0 RAP Lab	2.92E−11	5.07	0.907	138
Illinois	5 RAS/0 RAP GTR	2.15E−11	4.86	0.593	152
Illinois	5 RAS/11 RAP Field	2.61E−13	5.64	0.985	208
Illinois	5 RAS/11 RAP Lab	5.26E−27	9.95	0.996	359
Illinois	5 RAS/11 RAP GTR	8.29E−20	7.56	0.735	204

and one mix from Minnesota utilize course gradations that plot below the restricted zone. The SMA mixes from Illinois plot well below the maximum density line to achieve the gap graded aggregate structure for SMA mixes.

3.4. Dynamic modulus

The dynamic modulus master curves presented in Fig. 6 show the moduli of the asphalt mixes decrease with temperature and increase with frequency. The Missouri mixes have the highest dynamic modulus values which correlate to their high asphalt binder performance grade and coarse aggregate structure. Their high dynamic modulus values seem reasonable since they were designed for hotter climates and heavier traffic compared to mixes with lower modulus values such as Wisconsin or Iowa. For the Missouri, Iowa, Minnesota, and Colorado mixes, using RAS in the mix design increased the dynamic modulus values of the mixes indicating a greater resistance to permanent deformation. For the Minnesota mixes, the HMA with post-consumer RAS exhibited higher dynamic modulus values than the HMA with post-manufactured RAS. For the Indiana mixes, using WMA foaming

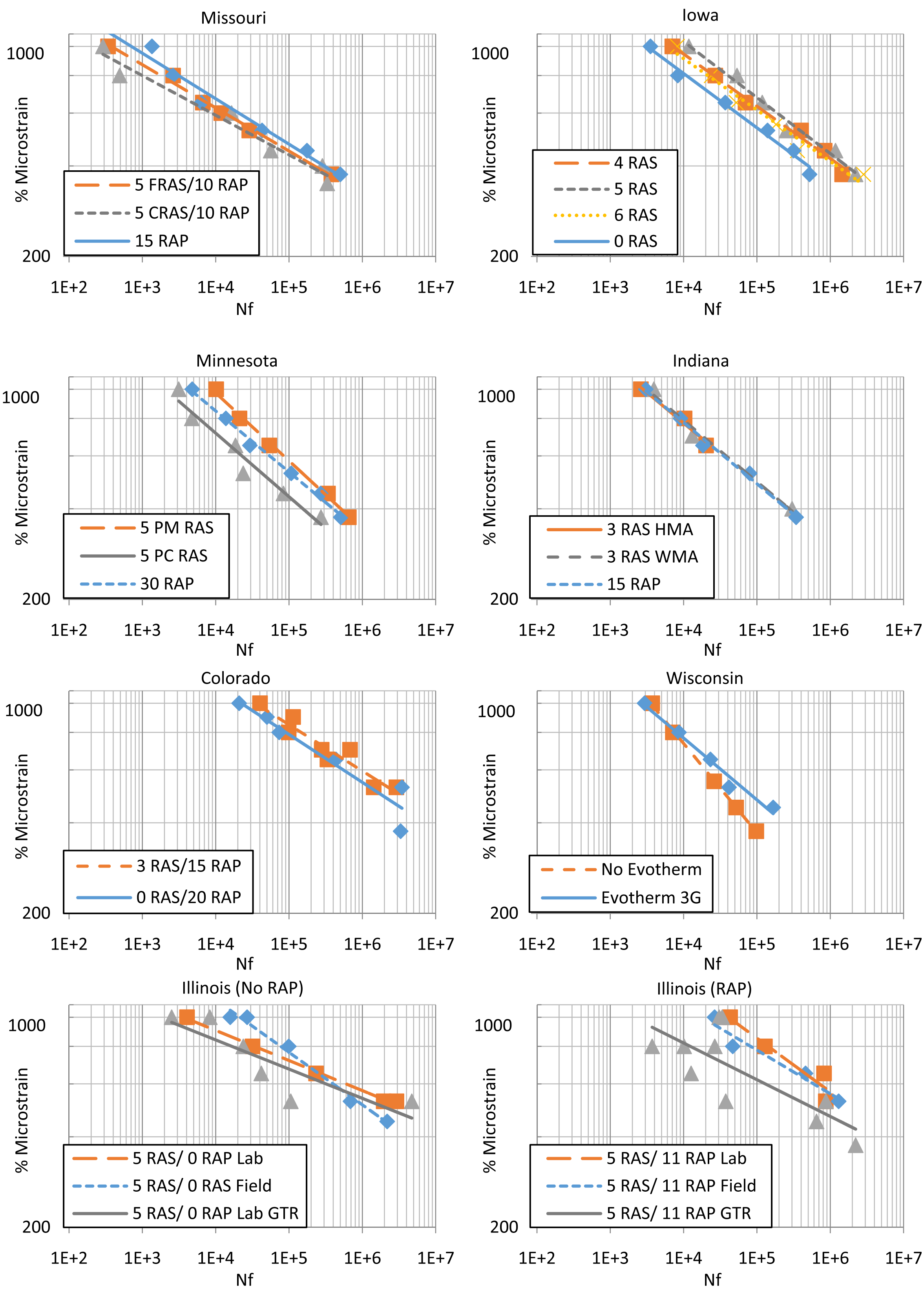

Fig. 8. Four-point bending beam fatigue curves.

technology did not impact the dynamic modulus values of the asphalt mixture. However, for the Wisconsin mixes, using Evotherm® 3G slightly decreased the dynamic modulus of the HMA at high temperatures. For the Illinois mixes, when 11% RAP is added to the mixes, there is an increase in dynamic modulus. There is also a difference between lab and field dynamic modulus values in the Illinois asphalt samples with no RAP. Since the PG of extracted binders was essentially the same, the difference may be the result of the slightly different aggregate grading between the lab and field sample.

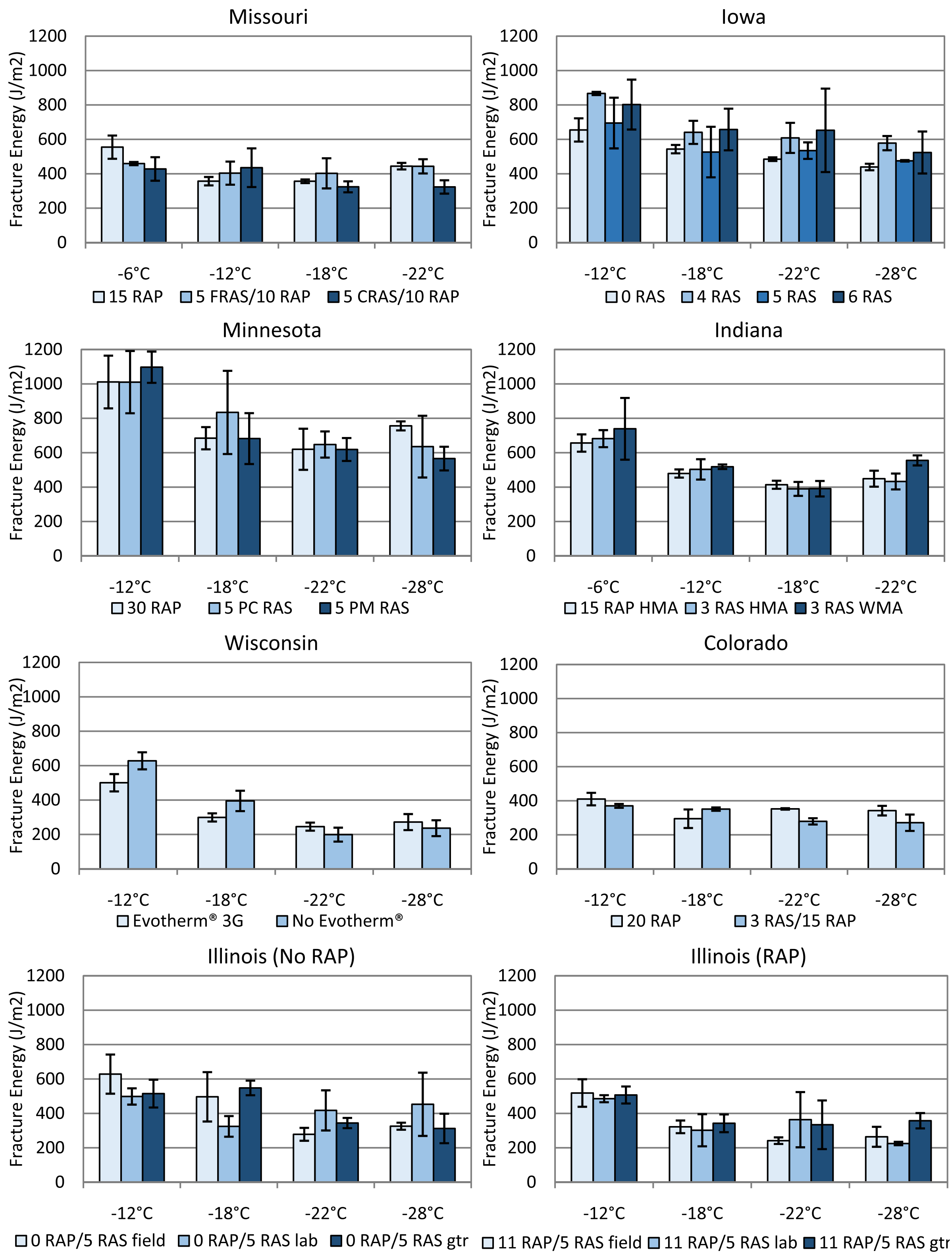

Fig. 9. Fracture energy (G_f) of mixes from each state.

3.5. Flow number

The mean flow numbers are presented in Fig. 7 with standard deviation error bars. A statistical analysis was conducted using a one-way analysis of variance to determine statistical differences among the mean flow number values at a 95% significance level. A pair-wise comparison was then performed to compare and rank the mix treatment levels within each state with regard to flow number. The outcome is reported in Table 4, in which statistically similar treatments are grouped together. Letter A indicates the best

Table 6
Ranking of mixes by G_f mean for each demonstration project.

State agency	Mix ID	Tukey statistical rank	Group mean G_f (J/m^2)	Sample size at each temp.	Test temperatures (°C)
Missouri	15 RAP	A	428	3	−6, −12, −18, −22
Missouri	5 FRAS/10 RAP	A	427	3	
Missouri	5 CRAS/10 RAP	A	378	3	
Iowa	4 RAS	A	674	3	−12, −18, −24, −28
Iowa	6 RAS	A/B	659	3	
Iowa	5 RAS	A/B	558	3	
Iowa	0 RAS	B	531	3	
Minnesota	30 RAP	A	741	3	−12, −18, −24, −28
Minnesota	5 PC RAS	A	777	3	
Minnesota	5 PM RAS	A	768	3	
Indiana	15 RAP HMA	A	551	3	−6, −12, −18, −22
Indiana	3 RAS HMA	A	502	3	
Indiana	3 RAS WMA	A	500	3	
Wisconsin	Evotherm® 3G	A	329	2	−12, −18, −24, −28
Wisconsin	No Evotherm®	A	364	2	
Colorado	20 RAP	A	350	2	−12, −18, −24, −28
Colorado	3 RAS/15 RAP	A	318	2	
Illinois	5 RAS/0 RAP Field	A	482	2	−12, −18, −24, −28
Illinois	5 RAS/0 RAP Lab	A	432	2	
Illinois	5 RAS/0 RAP Lab-GTR	A	430	2	
Illinois	5 RAS/11 RAP Field	B	337	2	
Illinois	5 RAS/11 RAP Lab	B	369	2	
Illinois	5 RAS/11 RAP Lab-GTR	B	385	2	

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 demonstrate that higher amounts of RAS and/or RAP will increase the flow number, and thus the permanent deformation resistance, of the asphalt mixture. For example, as RAS is increased in the mix design for the Iowa project, the flow number increases. There is a statistical increase in flow number at the 95% confidence level when comparing the means of the 0% RAS mixes to the 6% RAS mixes. Likewise, when 30% RAP was replaced with 5% RAS in the Minnesota mixes, the flow number also increased. The Minnesota mixes also demonstrate that using post-consumer RAS can improve the flow number of an asphalt mix to a greater extent than using post-manufacturer RAS. For the Indiana mixes, replacing RAP with RAS also improved the flow number of the asphalt mixes. In the case of the Missouri and Illinois mixes, the flow numbers of the mixes reached the end of the test at 10,000 load cycles without reaching tertiary flow. The strains accumulated in the mixes after 10,000 load cycles was very small at less than 1% strain. Hence, the Missouri and Illinois mixes exhibited the highest flow numbers.

3.6. Four-point bending beam

The four-point bending beam results, as presented by the strain versus "loading cycles to failure" curves, are reported in Fig. 8. The $K1$ and $K2$ coefficients, R^2 value, and predicted endurance limit for all the mixes are presented in Table 5. With exception of the Illinois SMA mixes with GTR, all fatigue curves have an R^2 value above 0.9. The laboratory compacted SMA samples with GTR contained greater amounts of variability in air voids resulting in higher R^2 values. This was most likely due to the GTR's ability to absorb asphalt and swell when heated resulting in non-homogeneity in the sample. The $K1$ coefficient of the fatigue model characterizes the flexural modulus, and the $K2$ coefficient indicates the rate of damage accumulation in a sample. When using this relationship as failure criterion for a pavement design, a lower $K2$ value is more conservative as it assumes faster accumulation of fatigue damage. Carpenter [17] recommended the Illinois Department of Transportation use a $K2$ value in the range of 3.5–4.5. All the mixes, with or without RAS, had $K2$ coefficients above 4.

With respect to the predicted fatigue endurance limit of the mixes, the SMA mixes from Illinois which used 5% RAS exhibit good fatigue properties. In the case of the Iowa, Missouri, Minnesota, and Colorado projects, the RAS mixes exhibit better fatigue lives and higher predicted endurance limits than the non-RAS mixes. These results demonstrate that mixes with RAS can possess similar or better fatigue properties to mixes without RAS. This is both consistent and contradictory with other researchers' results. Cooper et al. [18] concluded from SCB testing that asphalt mixtures containing 5% recycled shingles showed no adverse effects on intermediate-temperature properties (fracture resistance) when compared with a control mixture containing no RAS. Based on monotonic fatigue tests, Wu et al. [19] found no statistically significant differences between mixtures with RAS and without RAS. However, Admad et al. [20] concluded from complex shear modulus fatigue tests on fine asphalt mixtures that fatigue life decreased as the percentage RAS content increased.

For the Iowa mixes in this study, fatigue life increases with the addition of RAS. Since the fatigue tests were conducted in a controlled-strain mode of loading, the results indicate that RAS will improve the fatigue life of a thin lift pavement. The four Iowa mixes contain very similar gradations and volumetric properties. They all have approximately the same asphalt content. The only difference between the mixes 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 mixes. Yet, the average initial beam stiffness of the 0% RAS mix was 3497 MPa, while the average initial beam stiffness of the 4%, 5%, and 6% RAS mixes was 3090 MPa, 3106 MPa, and 3156 MPa respectively. In the mixes containing RAS, beam stiffness increases as the percentage of RAS increases. However, these values were lower than the 0% RAS mix. These results are unexpected since the stiffer RAS binder should increase the flexural stiffness of the 0% RAS mix. A possible explanation for the decrease in flexural stiffness in the mixes with RAS could be from the RAS-aggregate–binder interactions and the contribution of fibers from the RAS. With respect to the fatigue life of the mixes, past beam fatigue studies in controlled strain mode of loading showed that a decrease in flexural stiffness due to a softer binder will increase fatigue resistance (SHRP-A-404). These results follow this trend since the mixes with lower initial stiffness demonstrated longer fatigue lives.

It is also of interest that the 11% RAP mixes for Illinois have higher endurance limits than the 0% RAP mixes. These results are counter intuitive since a higher percentage of recycled binder can increase the stiffness of an asphalt mix and reduce its fatigue life in a strain-controlled mode of loading. The RAP mixes may possess higher endurance limits because they have a higher total binder content than the non-RAP mixes, 6.3% versus 5.6%.

The SMA mixes exhibited high $K2$ values (e.g. 4.86–9.95) indicating superior fatigue performance. The $K2$ values of 7.56 and 9.95 for two of the SMA mixes with RAS and RAP are comparable to values obtained by Varvrick et al. [21]. Varvrick obtained a $K2$ value of 8.89 when testing a laboratory produced SMA mixes with 5% RAS and 15% RAP in the four-point bending beam. However, $K2$ values in this range are unusually high compared to other $K2$ values of SMA mixes with RAP and not RAS tested by Varvrick et al. [22] which had a range of 5.00–5.45.

3.7. Semi-circular bending

The fracture energy results from the semi-circular bend (SCB) tests for each state's mixes are shown in Fig. 9. The SCB samples from each state were used to conduct a completely randomized two-way factorial statistical experiment with mix type and temperature as the treatment groups. The temperature of the SCB test had a significant impact of the facture energy of the mixes from all the states. This indicates that the mixes have a reduced fracture energy, and thus reduced cracking resistance, as their temperature decreases. Using a pair-wise comparison of the mix type group mean from each state, the mixes are ranked according to their fracture energy in Table 6. Mixes from Minnesota have the highest fracture energy whereas mixes from Missouri and neighboring states have much lower fracture energies. Interestingly, this trend is associated with the geographic location where the mixes were designed and placed. For the northern climate states, softer asphalt binders were used in the mixes. As demonstrated by the results, the use of a softer asphalt binder resulted in mixes with a greater resistance to cracking.

With respect to the Missouri mixes, when 5% RAP is replaced with RAS, the fracture energy does not change. While the mixture with a coarse grind RAS decreases the fracture energy from 427 to 378 J/m^2, the difference is not statistically significant.

For the Iowa mixes, the 4% RAS mix has the highest fracture energy and the 0% RAS mix has the lowest fracture energy. The differences are statistically significant. The ranking of the mixtures by fracture energy is almost identical to the ranking of the mixtures by fatigue endurance limit, where RAS also has an effect on reducing the cracking susceptibility of the mix. These results indicate that small percentages of RAS will either decrease or have no effect on the low temperature cracking resistance of the mixes prior to long-term aging.

The results of the Minnesota mixes indicate similar low temperature cracking resistance between the RAS and RAP mixes. The 30% RAP mix has an average fracture energy of 741 J/m^2. When 5% RAS is used in the mix design in place of 30% RAP, the fracture energy increased to 768 J/m^2 for the post-manufacturer 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 exist between the results of the three mixes.

For the Indiana mixes, when 15% RAP is replaced with 3% RAS, the fracture energy decreases from 551 to 502 J/m^2, although the differences are not statistically significant. The SCB test does not detect any difference in low temperature cracking performance when either RAS or WMA technology are used in the mixes.

For the Wisconsin mixes, when Evotherm® 3G is added to the HMA as a compaction aid, the fracture energy does not change. While the Evotherm® 3G mix does have a lower fracture energy

Table 7
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	–	–
Missouri	5 FRAS/10 RAP	0	52	97	–	–
Missouri	5 CRAS/10 RAP	0	41	139	–	–
Iowa	0 RAS	0	144	156	–	–
Iowa	4 RAS	0	137	142	–	–
Iowa	5 RAS	0	148	153	–	–
Iowa	6 RAS	0	146	147	–	–
Minnesota	30 RAP	–	–	–	0	0
Minnesota	5 PC RAS	–	–	–	143	173
Minnesota	5 PM RAS	–	–	–	150	199
Indiana	15 RAP HMA	–	4	158	191	–
Indiana	3 RAS HMA	–	35	162	172	–
Indiana	3 RAS WMA	–	47	264	277	–
Wisconsin	Evotherm® 3G	0	0	–	–	–
Wisconsin	No Evotherm®	0	0	–	–	–
Colorado	20 RAP	0	0	–	–	–
Colorado	3 RAS/15 RAP	0	25	–	–	–
Illinois	5 RAS/0 RAP Field	0	0	–	–	–
Illinois	5 RAS/0 RAP Lab	0	0	–	–	–
Illinois	5 RAS/0 RAP GTR	0	0	–	–	–
Illinois	5 RAS/11 RAP Field	0	0	–	–	–
Illinois	5 RAS/11 RAP Lab	0	0	–	–	–
Illinois	5 RAS/11 RAP GTR	0	0	–	–	–

(329 J/m^2) than the non-Evotherm® 3G mix (364 J/m^2), the difference is not statistically significant.

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

For the Illinois mixes, when 11% RAP is added to the mixes, the fracture energy significantly decreases, resulting in a mix with a greater susceptibility to cracking. This is most likely due to an increase in asphalt binder replacement from the addition of RAP. The main effects of binder modification type (GTR versus polymer) and sample type (laboratory versus field) were not significant.

3.8. Pavement field performance

The results of the pavement condition surveys for each demonstration project are reported in Table 7. The number of pavement surveys conducted for each project was dependent on the timing, location, and scope of the project. For example, the Wisconsin project was an intermediate course that was paved over with a surface course the following year, and the Colorado project was milled after 2 years as part of a complete pavement reconstruction.

During each survey, there was no measureable amount of permanent deformation. 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.

For the Missouri and Minnesota projects, the RAS pavements exhibited more cracking than the non-RAS pavements. However, for the Iowa and Indiana projects RAS pavements exhibited the same amount of cracking or less than the non-RAS pavements. The Indiana WMA pavement with RAS exhibited more cracking than the HMA pavement with RAS. In the case of the Wisconsin project, using Evotherm® with RAS did not decrease the pavement performance after one winter season. In the Minnesota project, slightly more cracking was observed in the mix using post-manufacturer RAS compared to the mix using post-consumer RAS. 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 about RAS pavements solely based on the surveys should be reserved.

4. Conclusions

The Transportation Pooled Fund (TPF)-5(213) demonstration projects show that pavements with RAS can be successfully produced and meet state agency quality assurance requirements for asphalt content, gradation, and volumetrics. This includes the SMA mixes produced in Illinois which used 5% RAS in place 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, and GTR.

When RAS is used in HMA, the shingle binder increases the high and low temperature performance grade (PG) of the base binder. For every 1% increase in RAS, the low temperature grade of the base binder will increase 1.9 °C; and for every 1% increase in RAP, the low temperature grade of the base binder will increase 0.3 °C. Therefore, on average, 3% RAS or 20% 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% binder replacement when using RAS and a 20% binder replacement when using RAP, when considering the average asphalt content values for all the mix designs. When estimating how RAS will affect an HMA binder, agencies should consider the RAS source (post-manufacturer 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 that were evaluated for multiple years.

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 and no added fibers exhibited good 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.

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 differences in fracture energy for the projects in Missouri, Minnesota, Indiana, Wisconsin, and Colorado. However, there were differences in fracture energy for the projects in Iowa and Illinois. For the Iowa mixes, the 4% RAS mix had a statistically higher fracture energy than the 0% RAS mix which suggests that RAS can improve the fracture resistance of HMA prior to long-term aging. For the Illinois mixes, adding 11% RAP to the mixes with 5% RAS decreased the fracture energy. The increase in recycled binder content from the RAP likely caused the fracture energy to drop since the asphalt binder replacement increased from 21% to 35% due to RAP. Based on these results, it is possible for recycled mixes with RAS to have acceptable resistance to fracture, but a combination of RAS and RAP and a high asphalt binder content replacement can result in a lower 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-manufacturer 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 about RAS pavements solely based on the surveys should be reserved.

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