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Performance Evaluation of Asphalt Mixtures with Reclaimed Asphalt Pavement and Recycled Asphalt Shingles in Missouri

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Abstract

This study investigates the performance of eighteen different dense-graded asphalt mixtures paved in Missouri. The sections contain a wide range of reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS), and different types of additives. The large number of sections investigated and the associated breadth of asphalt mixtures tested provided a robust data set to evaluate the range, repeatability, and relative values provided by modern mixture performance tests. As cracking is one of the most prevalent distresses in Missouri, performance tests such as the disk-shaped compact tension test (DC[T]) and Illinois flexibility index test (I-FIT) were used to evaluate the cracking potential of the sampled field cores. In addition, the Hamburg wheel tracking test (HWTT) was employed to assess rutting and stripping potential. Asphalt binder replacement (ABR) and binder grade bumping at low temperature were found to be critical factors in low-temperature cracking resistance as assessed by the DC(T) fracture energy test. Six sections were found to perform well in the DC(T) test, likely as a result of binder grade bumping (softer grade selection) or because of low recycling content. However, all of the sections were characterized as having brittle behavior by the I-FIT flexibility index. Service life and ABR were key factors in the I-FIT test. Finally, a performance-space diagram including DC(T) fracture energy and HWTT rut depth was used to identify mixtures with higher usable temperature interval (UTI_{mix}), some of which contained significant amounts of recycled material.

Asphalt concrete is the most recycled material in the world. Yet, after several decades of increased usage, the procedures for incorporating reclaimed asphalt pavement (RAP) into asphalt mix designs are not completely performance-based. Over 10 million tons of tear-off roofing shingles are currently stockpiled, creating the potential for large-scale recycling of recycled asphalt shingles (RAS). Recycling these materials in asphalt pavements is a potentially sustainable solution and can often yield performance benefits if used correctly (*1–3*). However, a lack of scientific test results and effective tests, especially to evaluate new products and manufacturing processes, and a lack of clear quantification of costs versus benefits impedes implementation by state transportation agencies and industry. The increased stiffness associated with RAP (*4*) can accelerate cracking rates if not designed properly, which can deter producers and state agencies from increasing RAP allowances (*1*, *5*). Similar issues arise with the use of RAS, and even more careful consideration is needed because of the high binder content and very high stiffness associated with the binder contained in RAS (*6–9*). Mixture performance testing can provide confidence when designing with these materials, especially in the case of high binder replacement levels.

Recently, Arshadi et al. conducted dynamic modulus, flow number, and SCB (Louisiana method) tests on Long-Term Pavement Performance, Specific Pavement Study (LTPP-SPS10) of five plant-produced mixtures in Oklahoma containing 13% and 14% asphalt binder replacement (ABR) by RAP and RAS, respectively (*10*). It appeared that virgin performance grade high-temperature (PGHT) drop or rejuvenator agent (RA) implementation could improve the cracking resistance. Researchers used four different dosages of a rejuvenator (0%, 3%, 6%, and 9% of binder weight) and tested

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plant-produced samples containing 14% RAP with two levels of aging, namely, unaged and short-term aged (*6*). Results from a semicircular bending (SCB) cracking test and Hamburg wheel track tests (HWTT) showed that with higher rejuvenator dosage, higher rutting and lower cracking was predicted. As expected, short-term aging of the mixtures resulted in a reduction of the flexibility index, as measured in the SCB, and rut depth. In another study, a suite of performance tests including rutting, bending, and tensile strength ratio (TSR) was performed on lab-produced warm-mix asphalt (WMA) mixtures containing two different warm-mix additives, along with 20%, 30%, and 40% RAP content with or without a rejuvenating agent (*7*). Using an ANOVA statistical test, it was concluded that RAP content was the only significant parameter in high- and low-temperature performance, whereas rejuvenator agent (or lack of) was the only significant parameter in moisture resistance (*7*). Hung et al. (*8*) investigated 10% RAP binder replacement into a gap-graded, rubberized hot-mix asphalt mixture (RHMA-G), which is typically used by the California Department of Transportation. The mix testing included dynamic modulus, beam flexural frequency sweep, flow number, and beam fatigue. Although the rutting performance of the tested mixture was improved, the authors did not recommend RAP addition because the fatigue life was reduced by up to 97% in some cases. However, this severe performance difference appears to be inconsistent with other RAP laboratory studies and with field experience.

Behnia et al. used the disk-shaped compact tension (DC[T]) test to evaluate the effect of addition of RAP in asphalt mixtures on thermal cracking (*11*). The authors found that the addition of RAP beyond 10% significantly decreased the fracture energy of the specimen. Yang et al. compared the DC(T) fracture energy of field cores and lab-produced samples and observed that both crumb rubber modified WMA and crumb rubber modified hot-mix asphalt (HMA) performed better than control HMA (*9*). Arnold et al. showed that addition of RAS to asphalt mixture specimens led to an increase in the peak load and a decrease in the overall fracture energies (*12*). Dave et al. used DC(T) for low-temperature fracture characterization of nine mixes with varying RAP content, aging, and air void content (*13*). The results showed that the DC(T) test was successfully able to capture the effects of temperature, varying content of RAP, binder modifiers, and aging. Buttlar et al. developed the performance-space diagram, a graphical tool designed to capture high- and low-temperature mixture performance test results in a single visual diagram (*14*). For instance, the diagram schematically demonstrates the asphalt mixture counterpart to the asphalt binder concept of usable temperature interval, or UTI. Mixtures with better performance at both high and low temperatures tend to plot in the upper-right portion of the diagram (*14*). This is similar to the once-popular concept of temperature susceptibility, according to which lower temperature susceptibility binders were expected to perform better in climates with hot summers and colder winters (for instance, mid-continental locations). The performance-space diagram was used in (*15*) to study the effects of re-refined engine oil bottoms on mixture performance after aging.

This limited sampling of recent literature, combined with the increasing diversity of recycled materials and additives available to the mixture designer, suggests that additional work is needed to evaluate the effects of recycled material and rejuvenator agents on asphalt mixture performance. This also suggests the need for accurate performance evaluation of asphalt concrete at different environmental and loading conditions. To this end, this study investigated the cracking and rutting potential of field cores obtained from the routes across Missouri using modern mixture performance tests. Asphalt mixtures from the selected sections contained a wide variety of recycled materials, additives, and virgin binder grades. This allowed for distinguishing good and bad performers and developing strategies to further boost their performance.

Materials and Sample Preparation

Field Section Details

In this study, eighteen sections across Missouri were investigated. The project sections were spread across the state, and included MO13, MO52, US63, US50, and US54. The selected sections cover a wide range of recycled materials and additives, namely, RAP, RAS, WMA additives, and rejuvenators. Table 1 provides details for each of the eighteen tested sections. Most of the sections (13 out of eighteen sections) were constructed in 2016 and cored soon thereafter (within two weeks after construction), which represents the short-term aging condition. The other five sections are highlighted in different colors to easily distinguish between the recently constructed and aged sections. The oldest section investigated was US54_7, which was constructed in 2003 and did not contain any RAP and RAS. ABR by RAP and RAS, total percentage of asphalt content by mixture mass (P_b), performance grade (PG) of the virgin binder, the type and dosage of additives, and nominal maximum aggregate size (NMAS) are summarized in Table 1. For convenience, the percentages of ABR, and RAP and RAS contents have been included in the section labels. As an example, MO13_1 (17-17-0) has an ABR of 17% (first number in parenthesis), resulting from 17% replacement by RAP (second number in parenthesis), and 0% by RAS (third number in parenthesis).

Table 1. Section Properties

Number	Construction year	Section	ABR %		Total P_b %[a]	Virgin binder	Additive	NMAS (mm)
			%RAP	%RAS				
1	2016	MO13_1 (17-17-0)	17	0	5.7	PG64-22 H[b]	Type 1:0.5%	9.5
2	2016	US63_1 (35-35-0)	35	0	5.1	PG58-28	Type 2:0.5% + Type 3:1.75%	12.5
3	2016	US54_6 (31-31-0)	31	0	5.1	PG58-28	Type 1:1%	12.5
4	2016	US54_1 (33-0-33)	0	33	5.2	PG58-28	Type 4:2.5% + Type 5:3.5% + Type 1:1.5%	12.5
5	2011	US50_1 (25-25-0)	25	0	4.5	PG64-22	Type 6:1.5% + Type 7:1%	12.5
6	2010	MO52_1 (34-0-34)	0	34	4.8	PG64-22	Type 6: 1.5% Type 7:0.8%	12.5
7	2008	US63_2 (30-20-10)	20	10	5.6	PG64-22	Type 6: 1.5% + Type 7: 0.5%	12.5
8	2016	US54_2 (33-33-0)	33	0	5.3	PG58-28	Type 1: 1%	12.5
9	2016	US54_3 (33-18-15)	18	15	5.2	PG58-28	Type 1: 1%	12.5
10	2016	US54_4 (35-35-0)	35	0	4.8	PG64-22 H	Type 5:3% + Type 1:1%	12.5
11	2016	US54_5 (0-0-0)	0	0	5.4	PG64-22 H	Type 1:1%	12.5
12	2003	US54_7 (0-0-0)	0	0	6.2	PG64-22	Type 8:0.25%	12.5
13	2006	US 54_8 (9-9-0)	9	0	5.6	PG70-22	Type 7:0.5%	12.5
14	2016	SPS10-1 (24-24-0)	24	0	5.2	PG64-22 H	Type 1:1%	12.5
15	2016	SPS10-2 (25-25-0)	25	0	5	PG64-22 H	Type 1:1%	12.5
16	2016	SPS10-3 (25-25-0)	25	0	5	PG64-22H	Type 1:1% + Type 2:0.5%	9.5
17	2016	SPS10-6 (17-0-17)	0	17	5.4	PG58-28	Type 1:1%	9.5
18	2016	SPS10-9 (46-16-30)	16	30	5.3	PG46-34	Type 1:2%	12.5

Note: Type 1 = anti-stripping agent ("Morelife T280"); Type 2 = warm-mix additive ("Evotherm"); Type 3 = rejuvenator additive ("EvoFlex CA"); Type 4 = anti-stripping agent ("IPC-70"); Type 5 = warm-mix additive ("PC 2106"); Type 6 = bag house fines; Type 7 = anti-stripping agent ("AD-here HP Plus"); Type 8 = anti-stripping agent ("LOF 65-00LS1"); MO13_1 (17, 17, 0) = total ABR: 17%, ABR by RAP: 17%, ABR by RAS: 0%; ABR = asphalt binder replacement.

[a]By total mass of binder, including neat and recycled.

[b]Heavy traffic designation (from multiple stress creep recovery [MSCR] test).

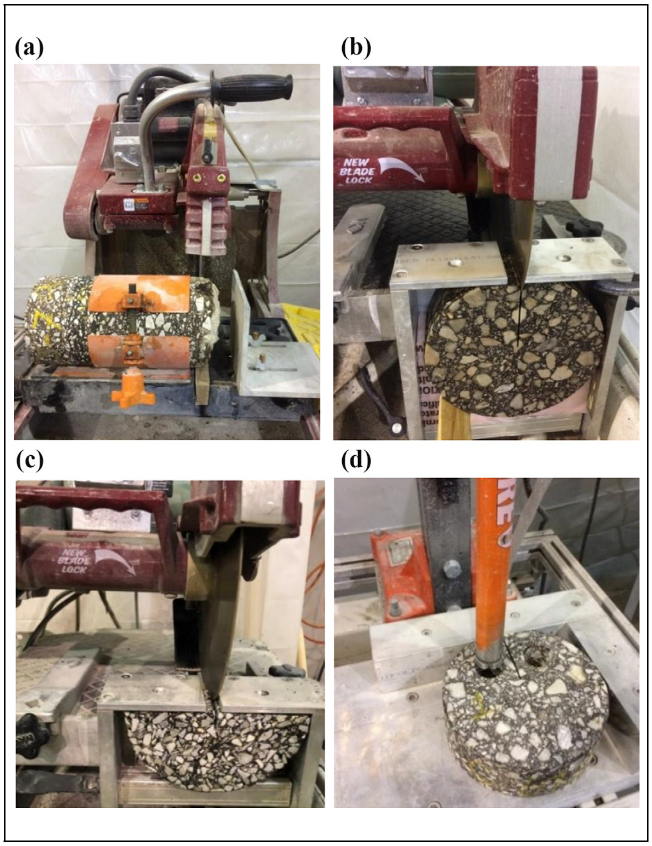

Figure 1. Fabrication equipment: (*a*) full depth field core mounted in block saw; (*b*) DC(T) sample in tile saw; (*c*) SCB sample in tile saw; (*d*) DC(T) sample in coring rig.

As shown, 11 out of 18 sections had a Superpave performance grade low-temperature (PGLT) grade of -22, which is the low-temperature binder grade currently specified in Missouri.

Sample Preparation

For mixture testing, a majority of tests conducted in this study were performed on 150-mm diameter by 50-mm thick specimens. This includes disk-shaped compact tension (DC[T]) testing, SCB tests, and HWTT. In addition, further sample fabrication cuts and coring operations were required, depending on the test requirements. A block saw was used to cut field cores (Figure 1*a*), and a tile saw and small coring rig were used to meet additional fabrication requirements for mixture mechanical tests. The fabrication equipment used was obtained from Test Quip, LLC, and featured a 16-in. diameter block saw blade (stiffer than the more common 20-in. diameter blade), and purpose-built masonry (tile) saw (Figure 1, *b* and *c*) and coring rig (Figure 1*d*), which were equipped with adjustable fixtures to facilitate proper sample dimensioning. All cutting devices were water-cooled to avoid overheating of samples.

Performance Tests on Field Cores

Disk-Shaped Compact Tension (DC[T]) Testing

The DC(T) test was developed to characterize the fracture behavior of asphalt concrete mixtures at low temperatures. The testing temperature is 10°C warmer than the PGLT grade of the mixture, per (*16*). Thermal cracking in asphalt pavements can be considered as occurring in pure tensile opening or fracture mode I, as the cracks propagate perpendicular to the direction of the thermally induced stresses in the pavement, that is, transverse to the direction of traffic (*17*). The fracture energy is computed as follows:

$$G_f = \frac{AREA}{B \cdot L} \tag{1}$$

where

G_f denotes fracture energy in J/m^2,

AREA is the area under the load–CMOD$_{FIT}$ curve, until the terminal load of 0.1 kN is reached,

B is specimen thickness in m, generally 0.050 m (except for field cores), and

L is ligament length, usually around 0.083 m.

The DC(T) test procedure used in this study includes conditioning of the fabricated specimen at the selected test temperature in a temperature-controlled chamber for a minimum of 2 h. After the conditioning, the specimens are suspended on loading pins in the DC(T) machine. A portable Test Quip DC(T) device was used, which is housed at the Missouri Asphalt Pavement and Innovation Lab (MAPIL). The test is performed at a constant crack mouth opening displacement (CMOD) rate, which is controlled by a CMOD clip-on gauge mounted at the crack mouth. The CMOD rate specified in ASTM D7313-13 is 0.017 mm/s (1 mm/min). To begin the testing sequence, a seating load no greater than 0.2 kN (typically about 0.1 kN) is applied to "seat" the specimen. The test finishes when a crack has propagated such that the post-peak load level is reduced to 0.1 kN. The fracture energy can be obtained by measuring the area under the load-CMOD curve and dividing it by the fractured area (ligament length times thickness).

Illinois Flexibility Index Testing (I-FIT)

In 2016, Ozer et al. introduced the IL-SCB method for cracking resistance characterization in asphalt mixes (*18*). The goal of the research was to develop an inexpensive, rapid test as a means to limit general pavement cracking. It was decided to simplify the test procedure by testing at room temperature, or 25°C. It was observed that fracture energy obtained in this compact, arched bending mode at this temperature did not uniquely characterize mixture cracking properties. Rather than abandoning the test, the researchers observed that the post-peak slope of the load–displacement curve from the SCB test was sensitive to the changes in the asphalt mixture specimen composition, and they subsequently used this to develop the flexibility index (FI). The FI has been proposed to provide a means to identify brittle mixtures that are prone to premature cracking, and was specifically developed to be sensitive to recycled material content. The FI is an empirical index parameter that is computed as the total fracture energy divided by the absolute value of the slope of the post-peak softening curve following AASHTO TP-124 (*18*):

$$\mathrm{FI} = \frac{G_f}{|m|}(0.01) \tag{2}$$

where G_f is computed in a similar manner as in the DC(T) test, and m represents the slope of the post-peak softening curve. There are countless ways to estimate the slope of a curve resulting from a material test, and this became a challenge for test standardization early in the development of the I-FIT. In this study, three terms of the exponential function suggested in AASHTO-TP124 were implemented to fit the post-peak load-deflection curve.

To fabricate samples, a notch is cut along the axis of symmetry of an SCB specimen to a depth of 15 ± 1 mm. Test specimens are then conditioned in the environmental chamber at 25°C for 2 h ± 10 min. After a contact load of 0.1 kN is reached, the test is carried out at a rate of 50 mm/min with a data sampling rate of 40 samples per second. The test is considered to be complete when the load drops below 0.1 kN, which is identical to the DC(T) test termination definition.

Hamburg Wheel Track Testing (HWTT)

Permanent deformation (rutting) in an asphalt pavement is a result of consolidation and shear flow caused by traffic loading in hot weather. This results in gradual accumulation of volumetric and shear strains in the HMA layers. The measured deformation of different layers of flexible pavement revealed that the upper 100 mm (4 in.) serves the main portion of the pavement rut depth such that the asphalt layer accumulates the majority of total permanent deformation (*19*). Wheel load tracking (WLT) tests are the most common performance tests for measuring the rutting potential of HMA mixes. The WLT methods simulate traffic by passing over standardized wheels simulating real-life traffic loads on an HMA specimen at a given temperature. The HWTT was performed in accordance to the AASHTO-T324 standard (*20*). A loaded steel wheel, weighing approximately 71.7 kg tracks over the samples placed in a water bath at 50°C. The vertical deformation of the specimen is recorded along with the number of wheel passes. A Cooper Hamburg device was used in this study. The output file of this device contains the rut depth at 220 points (designated by −110 to 110 with 0 being at the center) with 1 mm intervals along the tested sample. The presented rut depth is calculated as the average of three point readings including the point at which the maximum rut depth at the end of desired number of passes is measured and the two readings with 15 mm distance around it. For example, if the maximum rut depth at 20,000 passes occurs at the center of the tested sample, the −15, 0, and 15 reading points were averaged out and reported.

Performance Testing Results

Disk-Shaped Compact Tension (DC[T]) Test

Figure 2 shows the average DC(T) fracture energy of at least three and normally four replicates tested at −12°C. In addition to the fracture energy, the PGLT grade of the virgin binder used in the mixture is also plotted. Six sections had only one grade bump (i.e., use of one PGLT grade softer), which was done to compensate for the brittleness associated with the presence of RAP or RAS. Considering the suggested 460 J/m^2 fracture energy threshold for short-term aged, moderate traffic volume facilities, US54_5 and US54_1 were found to have been designed with sufficient fracture energy. Furthermore, considering a recommended minimum of 400 J/m^2 for aged field sections and moderate traffic, US54_7, which was paved in 2003, meets fracture energy criteria. Although beyond the scope of this paper, field performance results reported in a recent report to the Missouri Department of Transportation (*21*) indicate that this section did not in fact experience thermal cracking distress. The following general observations can be made based on DC(T) fracture energy:

- Higher ABR without grade bumping was generally detrimental to DC(T) fracture energy. Yielding DC(T) fracture energy higher than 450 J/m^2, US54_5, US54_1, and US54_7 were the best three performers in terms of fracture resistance at low temperature. US54_5 and US54_7 did not contain any RAP and RAS with a binder PGLT of -22. Whereas their age, base binder, NMAS, and additive are the same, US54_5 with zero ABR has more than 100 J/m^2 higher fracture energy comparing with SPS10-1 and SPS10-2 with 24% and 25% ABR, respectively.
- When a high ABR was used, grade bumping appeared to help restore ductile behavior and as a result, led to higher fracture energy. SPS10-9 which had the highest ABR (46%) resulting from

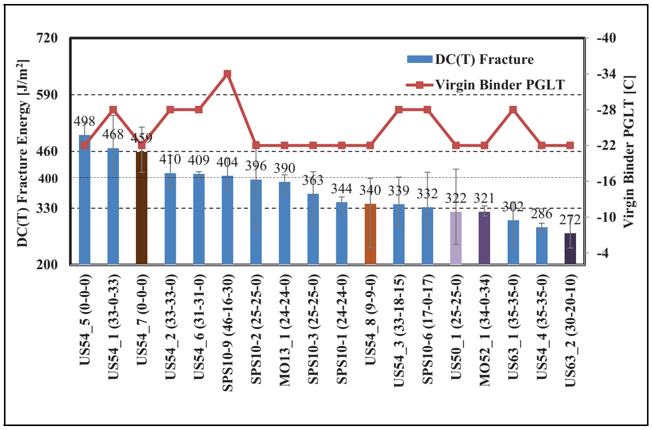

Figure 2. DC(T) fracture testing results of field cores at −12°C.

both RAP and RAS, benefited from two grade bumps on the low PG side. As expected, grade bumping of the virgin binder on the low-temperature side helped US54_6, US54_1, and US54_2 yield fracture energies higher than 400 J/m^2, whereas in the case of US63_1 and SPS10-6, it was not enough to compensate for the 35% and 17% ABR, respectively, in these mixes. The lack of a softer virgin binder in US63_1, US54_5, US50_1, and US63_2 with ABRs ranging from 24% to 35% ABR led to relatively poor performance in the DC(T) fracture test, as one might expect. In addition, a combination of softer base binder and lower ABR by 33% in US54_5 resulted in 159 J/m^2 of additional fracture energy as compared with US54_3.

- Mixtures with the same ABR but from different sources did not necessarily show similar low-temperature cracking resistance. Among the tested sections, US54_6 (31-31-0), US54_2 (33-33-0), US54_3 (33-18-15), and SPS10-6 (17-0-17) were all paved in 2016 and have similar additive and base binder (1% Antistrip and PG58-28, respectively). In this set of four sections, SPS10_6, with 17% ABR by RAS, had the lowest DC(T) fracture energy (332 J/m^2). In addition, compared with US54_2, US54_3 had 15% less RAP (which was replaced by RAS). This combination of RAP and RAS reduced DC(T) fracture energy by 70 J/m^2. US50_2 and US63_2 had similar ABR levels but different RAP sources, and had 50 J/m^2 difference in fracture energy. On the other hand, US54_6 and US54_2 had almost the same ABR by RAP, similar aggregates, and were constructed at the same time by the same contractor and yielded nearly identical DC(T) fracture energy levels.
- Fracture energy was generally reduced as the mixtures aged and lost their ductility. However, for the US54_7 section, which was constructed 13 years ago, it appears that the asphalt mixture used in this section had sufficient fracture energy to tolerate environmental and traffic loads during service. In comparison, US50_1, MO52_1, US63_2, and US54_8, which were aged for 5, 6, 8, and 10 years, respectively, and which contained recycled materials, were among the poorest performers, with DC(T) fracture energy levels below 350 J/m^2. In the future, a softer binder grade or rejuvenator should be considered to improve the cracking resistance of mixes with significant RAP content.
- The type of additive generally played a significant role in low-temperature performance. Comparing the US54_1 and US54_2 sections, which have the same age, base binder, NMAS, and total ABR, it can be observed that the mix with 33% ABR by RAS plus three additives (US54_1) resulted in a better low-temperature fracture resistance than the 33% ABR by RAP mix with 1% antistrip agent (US54_2). In addition, although US63_1 has only 4% higher ABR than US54_6, the difference in additive resulted in more than a 100 J/m^2 difference in fracture energy. On the other hand, the additives used in MO13_1 and SPS10-3 had similar effects on mixture fracture energy.

Illinois Flexibility Index Test (I-FIT)

The I-FIT test was performed to evaluate the general cracking potential of the Missouri asphalt mixtures. As indicated before, the test is conducted at 25°C and uses a bending configuration to drive a crack in the SCB configuration. Figure 3 shows the averaged FI obtained from four replicates. All of the eighteen tested sections yielded FI values below 8, generally indicating that the Missouri mixtures exhibited brittle behavior as per the I-FIT test. Similar to DC(T) testing results, US54_5 with 0% ABR was the best performer with FI = 5.4. However, the other virgin mix (US54_7) which was considerably aged (13 years under service) did not yield a favorable FI (1.8), even though it was found to stay relatively crack-free in the field (*21*). This might suggest a high sensitivity of FI to field aging, and perhaps a much lower FI threshold should be applied when evaluating the fracture resistance of specimens from aged field cores. This is supported by the fact that three out of the five worst performers in the I-FIT were more aged compared with the other mixtures. In addition, binder bumping on the low side of the virgin binder PG did not appear to help with the FI, which contradicts contemporary thinking. Data did not show a significant increase in FI by using PGLT of -28 instead of PGLT of -22. More to this point, despite two downward grade bumps on the PG low-temperature side, SPS10-9 is among the four

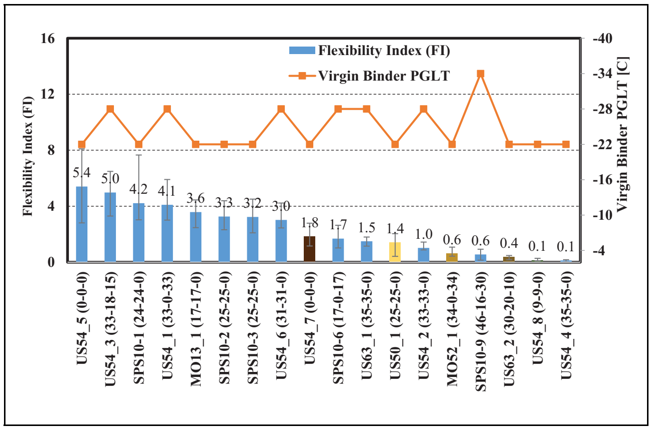

Figure 3. I-FIT testing results of field cores at 25°C.

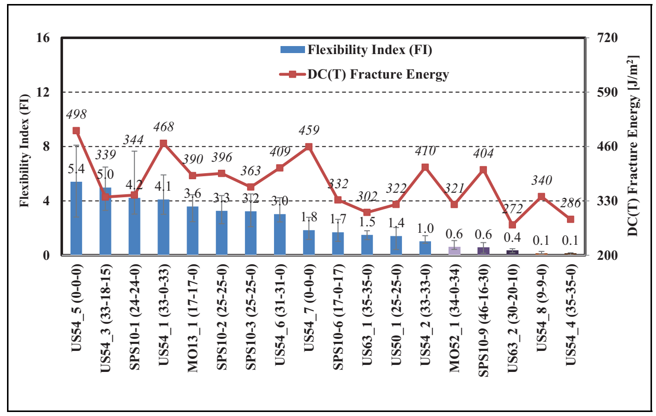

Figure 4. DC(T) versus I-FIT.

poorest performers in the I-FIT. Therefore, in addition to aging, ABR plays a significant role in the measured FI. Using the mixture characterization in Table 1, the following observations were made:

- The source of ABR was found to be very important. Considering US54_2 and US54_3, which have the same ABR but different combinations of RAP and RAS, US54_2 showed 400% higher FI than US54_3.
- US54_6 and US63_1, with different additives, had a difference of 50% in FI. The significant effect of additive was also noticeable in DC(T) fracture energy.
- Comparing US54_2 and SPS10-1, one grade bump could not compensate for the presence of 9% ABR—the FI of SPS10-1 (24-24-0) is 320% higher than US54_2 (33-33-0).
- Comparing with US54_3, one grade bump and 1% more of Type 1 additive did not help SPS10-9 to compensate for 15% additional ABR by RAS.

Comparing Cracking Resistance with Respect to DC(T) and I-FIT Results

Figure 4 shows the FI and DC(T) fracture energy of the Missouri mixtures. Although not perfectly correlated, a general trend can be observed in both DC(T) and FI results. Section US54_5, a virgin mix paved in 2016, yielded both the highest DC(T) fracture energy and the highest FI. In addition, US54_4 and US63_2, with relatively high amount of recycled materials (35% and 30% ABR, respectively) and with no PGLT bump, were among the poorest performers in these two tests. Notwithstanding some of the similarities in ranking, there appear to be fundamental differences between these two tests. The plotting ranges selected for both vertical scales in Figure 4 represent typical ranges for dense-graded Superpave mixes, based on experience. DC(T) fracture energy rarely drops below 200 J/m^2, and values over 700 J/m^2 are usually only observed on stone-matrix asphalt (SMA) mixtures, and stress absorbing membrane interlayers (SAMIs). The role of aggregates in resisting crack propagation even when the mastic system is very brittle probably explains the lower threshold of 200 J/m^2 in the DC(T). A plotting range of 0 to 16 was used for the FI scale. Using these plotting ranges, both tests appear to have a similar spread in test results; however, unlike DC(T) fracture energy, FI's tend to bottom out near zero. This may be because of the bending configuration and high loading rate used in the I-FIT, leading to unstable fracture in specimens with higher brittleness. These mixtures were observed to exhibit brittle failure in the I-FIT, with snap-back behavior (unstable transfer of stored strain energy in bending to the propagating crack after peak load is reached) resulting in nearly infinite slope.

To further investigate the DC(T) and I-FIT performance tests, the coefficient of variation (COV) of each main test parameter was computed (Table 2). As mentioned in reference to Equation 1, the DC(T) test computes the work of fracture and divides it by the fractured surface to yield fracture energy. The I-FIT FI uses the fracture energy as well as the post-peak slope (Equation 2) to determine the FI. It was found that the COV of the DC(T) fracture energy obtained from three replicates for each section, ranged from 3.4% to 27.6% with an average of 13.2% (Table 2). On the other hand, the FI parameter had a much higher COV level of 43.7%, as summarized in Table 2. This was mainly because of the high variability associated with the average computed slope in the I-FIT, which had a similarly high COV level.

Table 2. Average and COV of DC(T) and I-FIT Testing Parameters

	DC(T)		I-FIT					
Section	Avg. FE (J/m^2)	COV (%)	Avg. FE (J/m^2)	COV (%)	Avg. Slope (kN/mm)	COV (%)	Avg. FI	COV (%)
MO13_1 (17-17-0)	390.3	5.4	1,623.6	4.4	4.73	25.5	3.59	23.5
US63_1 (35-35-0)	302.4	18.4	1,092.6	15.0	7.59	25.1	1.50	23.3
US54_6 (31-31-0)	408.8	1.4	1,883.6	7.8	6.56	27.2	3.03	32.1
US54_1 (33-0-33)	467.6	14.5	1,788.4	15.2	4.53	17.4	4.11	32.2
US54_2 (33-33-0)	410.4	9.1	1,410.1	9.1	14.41	19.7	1.02	29.9
US54_3 (33-18-15)	339.2	16.5	2,227.9	18.8	4.57	16.4	4.98	32.2
US54_4 (35-35-0)	285.8	3.4	1,208.7	21.2	301.18	124.0	0.13	77.8
US54_5 (0-0-0)	497.9	5.7	2,229.1	14.7	4.62	33.2	5.41	43.7
SPS10-1 (24-24-0)	343.9	5.8	2,183.6	20.0	6.04	37.7	4.22	57.6
SPS10-2 (25-25-0)	395.6	26.1	1,864.6	10.2	5.39	16.7	3.27	28.7
SPS10-3 (25-25-0)	363.0	17.2	2,152.2	10.7	7.14	30.5	3.24	31.8
SPS10-6 (17-0-17)	332.3	22.7	1,837.5	13.0	11.97	32.2	1.69	41.0
SPS10-9 (46-16-30)	404.4	8.8	1,142.7	12.5	34.80	86.6	0.56	76.1
US50_1 (25-25-0)	321.5	27.6	759.4	7.6	7.68	81.7	1.41	51.6
MO52_1 (34-0-34)	321.2	3.8	546.2	13.1	10.67	43.3	0.61	51.4
US63_2 (30-20-10)	272.4	13.7	634.7	3.0	20.14	49.9	0.36	35.4
US54_8 (9-9-0)	340.2	25.5	1,170.7	35.4	262.08	141.2	0.14	80.7
US54_7 (0-0-0)	459.3	11.4	1,221.2	7.4	7.19	30.1	1.85	37.6
Avg. COV (%)		13.2		13.3		46.6		43.7

Note: Avg. = average; COV = coefficient of variation; FE = fracture energy.

Hamburg Wheel Tracking Test (HWTT)

HWTT was performed for 20,000 passes in a water bath controlled at 50°C. Two 62-mm cylinders were cut from two field cores from each section to produce Hamburg samples (Figure 5). Figure 6 shows the rut depth accumulated under HWTT test for each of the 18 tested sections. The solid bars show the rut depth at 10,000 wheel passes, whereas the diagonally striped columns indicate the rut depth after 20,000 passes. Furthermore, the high PG grade of the virgin binder (PGHT) was plotted to study the effect of binder grade. Considering the current maximum recommended rut depth of 12.5 mm at 10,000 passes in Missouri, all of the mixtures passed the Hamburg test. The majority of the mixes accumulated rut depths of less than 5 mm at 10,000 passes, which shows that there is not currently a rutting problem in Missouri Superpave mixes, which agrees with field observations. More specifically, the virgin mixes, namely, US54_5 and US54_7, with PGHT grades of 64, exhibited 2.0 and 3.0 mm rut depths at 10,000 passes, respectively. In addition, MO13_1 and SPS10-1 had the highest rut depths at 7.3 and 10.4 mm, respectively. However, US63_1 and US54_7 exhibited notable differences in the rut depths between 10,000 and 20,000 passes. Rut depths at 20,000 passes were at least two times higher than those at 10,000 passes. This indicates that these mixtures may suffer from moisture sensitivity, which will be investigated in more detail in a future study.

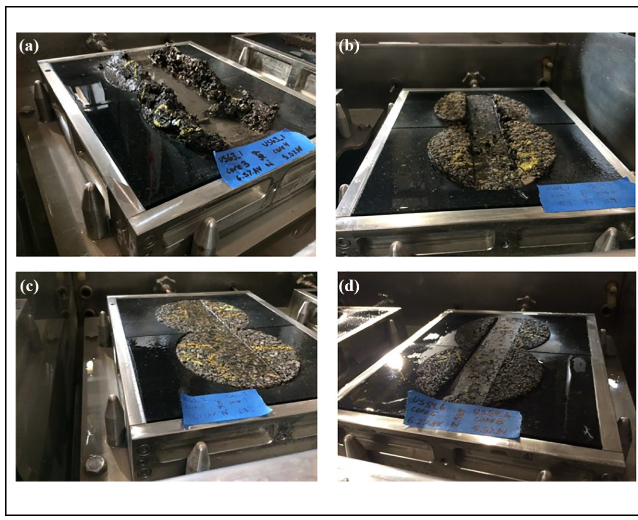

Figure 5. Tested samples in HWTT: (*a*) US63_1; (*b*) US54_7; (*c*) US63_2; (*d*) US54_6.

Hamburg-DC(T) Performance-Space Diagram Results

Figure 7 presents an *x-y* plotting form known as the "performance-space diagram" (*19*, *20*), or more specifically in this case, a "Hamburg-DC(T)" plot. This plot allows the simultaneous and convenient evaluation of rutting and cracking behavior, and can serve as a mixture-centric analogy to the evaluation and control of high- and low-temperature performance in the Superpave

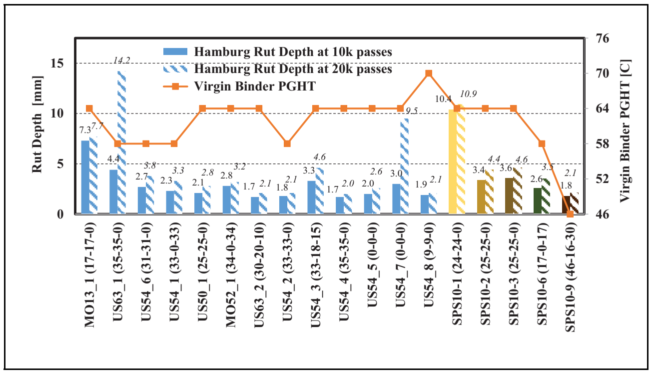

Figure 6. HWTT results at 50°C.

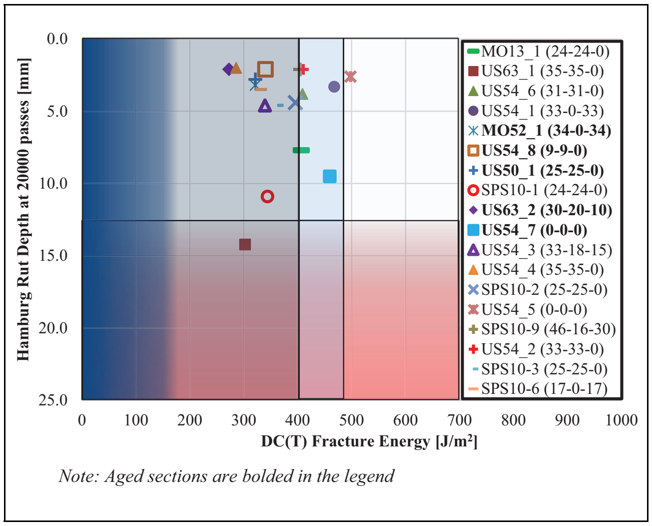

Figure 7. Performance-space diagram.

PG binder specification. This is particularly useful in evaluating the net behavior of mixtures containing recycled materials and other additives, as their behavior cannot be adequately predicted from the virgin PG binder used because of the presence of recycled materials and additives such as RAP, RAS, ground-tire rubber, rejuvenators, fibers, and so forth. Some useful trends that can often be observed when viewing data in this form are that:

- The best overall performing mixtures will appear in the upper-right corner of the diagram (low rutting depth, high fracture energy). These can be considered as high "total energy" mixtures, that is, rut and crack (or damage) resistant. These are high toughness mixtures, and the best candidates for surfacing materials especially in demanding climates and for high traffic volumes.
- Mix variables that increase total net energy in the mix and thus "move" mixtures in the direction of the upper-right corner of the plot include:
  - Higher quality binder (low temperature susceptibility, higher UTI, degree of polymer modification);
  - Higher quality aggregate (stronger, more angular, better bond with asphalt); and
  - The presence of crack interceptors or rut mitigators, such as fibers, rubber particles, and even RAS (but only if properly used).
- Other salient features of the plot include:
  - Binders with different grades but similar UTI tending to move a mixture along a "binder tradeoff axis," or, roughly speaking, diagonal lines moving in the upwards-left or downwards-right directions, for stiffening and softening, respectively;
  - Pure stiffening elements, such as RAP, tending to move points upwards and to the left;
  - Pure softening elements, such as rejuvenators, tending to move points downwards and to the right;
  - Binders with higher UTI, for which the grade bump is on the high-temperature grade, tending to move points mainly upwards, but also slightly to the right because of the benefits of polymer or crumb rubber in intercepting cracks;
  - Binders with higher UTI, for which the grade bump is on the low-temperature grade, tending to move points mainly to the right, but also slightly upwards, again, because of the benefits of polymer in intercepting cracks; and
  - Data points that appear in the undesirable middle-to-lower-left portion of the plot sometimes being those that contain RAP and insufficient binder bumping, and possibly poor bond, for which the RAP tended to cause lower DC(T) values, and the nature of the RAP-virgin material combination led to a moisture-susceptible mix with high Hamburg rut depth value.

Several interesting findings can be extracted from the results of Missouri field cores, including the following:

- The best performing mix overall was US54_5, which is a virgin mix with a relatively high UTI binder (PG 64-22 H). It has a gold-colored asterisk symbol, and is the furthest data point to the upper right.
- The next three best performing mixtures were:

  - US54_1: Consisting of 33% ABR, all coming from RAS, and PG58-28 binder with a UTI of 89.6;
  - US54_2: Relatively high ABR by RAP benefiting from one grade bump and additive Type 1; and
  - SPS10_9: An innovative mix, containing 46% ABR, with 16% binder replacement from RAP, and 30% from RAS, along with PG46-34 binder, with a relatively moderate UTI (86.4).
- Two poor-performing mixes in the Hamburg-DC(T) space contained RAS and exhibited stripping and/or rutting potential in the Hamburg test, including:
  - US63_1 (Figure 7, "Sample 2"), with 35% RAS and PG 58-28 binder, for which the results suggest that the mixture could have benefited from a second binder bump (such as PG XX-34 binder) to improve DC(T) fracture energy, along with more effective measures such as change of aggregate type and gradation to improve moisture sensitivity;
  - SPS10_1, with 24% RAP and PG64-22 H binder. The Hamburg results for this mix are shown in Figure 6. The results also suggest that the mixture could have benefited from binder bumping (softening) to improve DC(T) fracture energy, along with more effective measures to improve moisture sensitivity.
- A large cloud of mixtures had similar "total energy" on the Hamburg-DC(T) plot, but with a range of "mix stiffness." These mixes fell along a similar diagonal contour, spanning from the US54_7 mix (light blue square) to the US63_2 mix (purple diamond). The results suggest that these mixes have similar overall total performance, with some performing better on the Hamburg relative to the DC(T) and vice-versa. In general, material and mix design changes that move mixtures along this diagonal can be achieved at little-to-no cost. For instance, changing from a stiffer to a softer binder grade with a similar UTI, such as PG 64-22 and PG 58-28, would tend to shift the mixture down and to the right along this contour. The mix designer could potentially use this plotting technique to make mix adjustments to achieve passing performance test results, while retaining mix economy.
- The effect of the type of additive can be observed by comparing the US63_1 and US54_6 sections. Additive type 1, used in US54_6, considerably improved both rutting resistance and fracture energy in the US54_6 mix. As a result, US54_6 plots further upwards and to the right of US63_1.

Summary and Conclusion

A comprehensive lab investigation was carried out to evaluate the performance of recycled asphalt mixtures in Missouri. Eighteen field sections were evaluated, including several sections from the recent Long-Term Pavement Performance (LTPP), Special Pavement Sections (SPS-10) project in Osage Beach, Missouri, which was constructed in 2016. Well and poorly performing sections dating back as far as 2003 construction were sampled and tested. Some key observations made in the study are that:

- The national rules-of-thumb, suggesting the use of softer virgin binder grades for binder replacement levels over 20% appear to be validated based on lab and field test results. For ABR values above 30%, it appears that double grade bumping may often be beneficial.
- A very good performing field section was identified and tested in this study (US54_7). This zero ABR mix has performed very well after 13 years in service.
- The DC(T)-Hamburg plot showed that Missouri recycled mixes are generally on the brittle side, although three of 11 newer mixes investigated met recommended Hamburg and DC(T) criteria.
- Most mixes exhibit sufficient "total" energy to pass Hamburg and DC(T) recommended criteria without major changes in the aggregate structure, recycling level, or binder cost. Adjusting these mixes would simply involve the selection of a softer virgin binder grade, with a similar UTI, and therefore similar cost to the existing binder (which is normally PG 64-22 or PG 64-22H).
- The differences in test range for the DC(T) and I-FIT are accompanied by significant differences in test repeatability for these two cracking tests. The DC(T) and I-FIT showed an average COV for all study mixes of 13.2% and 43.7%, respectively.
- The DC(T) and I-FIT were both able to identify the best and worst performing section among the eighteen field sections. However, the relative rankings suggest that the tests may be best suited for the control of different crack modes for the pavements tested.

Ultimately, comparison of mixture performance tests with field performance data will lead to the eventual selection and adoption of tests by mix designers and owner agencies. Preliminary comparisons between the cracking tests evaluated here to five field sections were reported in (*21*). However, detailed review of these results is beyond the scope of this paper. Moreover, additional field sections in Missouri and Illinois are currently under investigation, along with the evaluation of additional parameters from the DC(T) and I-FIT tests, and the evaluation of additional tests such as the IDEAL test (*22*), and optimization of the Hamburg test with consideration to climate and traffic level. These results will lead to new performance-based asphalt mixture specifications to be used in Missouri, and at the Illinois Tollway.

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Author Contributions

Each of the four authors made equal contribution to: conducting of experiments, analysis of the experimental data, and writing the manuscript.

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