Asphalt Technology News

National Center for Asphalt Technology
NCAT
at AUBURN UNIVERSITY

High Friction Surfaces Gain Traction at NCAT

Each year, more than 25 percent of highway fatalities in the United States occur at or near horizontal curves, according to the Federal Highway Administration (FHWA). In an attempt to save lives and prevent injuries, some 30 states across the U.S. are applying high friction surfaces (HFS) to improve skid resistance at locations with a high incident of vehicle crashes such as severe horizontal curves, steep grades, roadway intersections, railroad crossings, and crosswalks. As roadway surfaces polish, available pavement friction is reduced. High friction surfaces provide improved skid resistance, and maintaining an appropriate amount of pavement friction is critical for safe driving.

High friction surfaces are pavement treatments with high skid-resistant properties not typically provided by conventional materials. Through site-specific application, a thin layer of high quality, durable aggregates is bonded to the existing pavement surface with a liquid polymer resin binder, restoring and maintaining pavement friction in areas where safe pavement surface properties are critical. These aggregate systems provide a life-saving technology customizable to specific state and local safety needs.

The most commonly specified aggregate component of HFS is calcined bauxite, an aluminum ore that increases in physical hardness and stability when heated to a high temperature. Used primarily because it resists polishing, calcined bauxite retains its friction values and properties after years of wear. This aggregate is narrowly graded, passing #4 sieve and retained on #16 sieve. Aggregates must be clean and uniform in size for best results.

Flint HFS Placed on Laboratory Slab

Although multi-component polymer resin binders are most typical, several varieties are used in HFS depending upon pavement surface type and condition. A good specification will always address the mechanical properties of the HFS

High Friction Surfaces on the Test Track
1. Taconite
2. Emery
3. Steel Slag
4. Silica
5. Basalt
6. Flint
7. Calcined Bauxite
8. Granite

The dynamic friction tester (DFT) is a lightweight and portable device that calculates surface frictional properties

binder along with limitations such as mixing, storing, and placement temperatures. Proper binder preparation is critical.

Surface preparation begins with cleaning the pavement to remove oils, grease, and other matter to ensure an effective bond. Insufficient cleaning and drying may result in delamination of the binder from the pavement surface. HFS should not be applied to pavements with certain types of distress, and any cracking in the existing pavement will reflect through the surface. High friction surfaces are purely a treatment for increasing skid resistance, not a pavement preservation treatment.

High friction surfaces can be applied manually or by automated equipment. Regardless of installation method, it is critical that a precise amount of binding agent is applied to achieve the proper film thickness and ensure successful adhesion to both the pavement surface and the aggregate. Mechanical or hand spreading of the aggregate occurs immediately following the binder placement and must ensure complete surface coverage. After installation and initial curing, any excess aggregate is swept or vacuumed for disposal or reuse in another project.
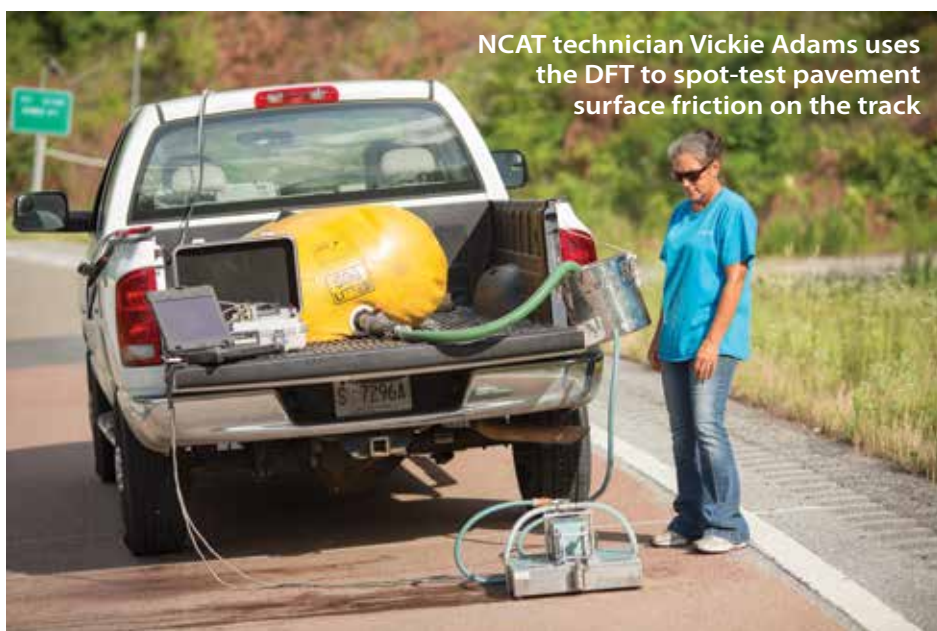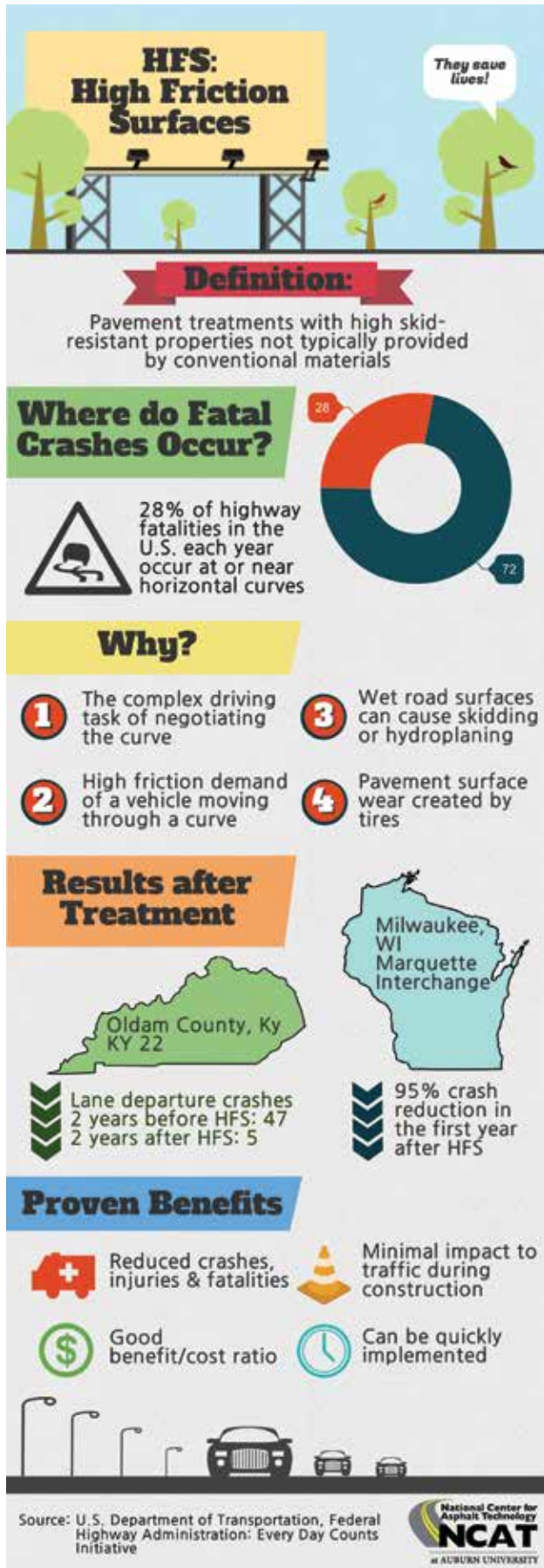

NCAT technician Vickie Adams uses the DFT to spot-test pavement surface friction on the track

The benefits of HFS outweigh the cost when compared to the value of preventing crashes, injuries, and fatalities. Generally ranging from $25 to $35 per square yard, their limited use as spot treatments in dangerous locations make HFS a relatively low cost option compared to expensive and time-consuming roadway reconstruction alternatives. Unlike reconstruction, HFS can be installed immediately. Project lengths are short, with treatments often being applied in just a few hours. Typically lasting between six and ten years, their performance ultimately depends on the initial installation process, condition of underlying pavement, amount of traffic, and winter maintenance practices.

Although the major cost factor in HFS is the bonding agent, calcined bauxite must be imported from overseas, making it an expensive aggregate. In April of 2011, FHWA funded the installation of eight experimental HFS sections on the NCAT Pavement Test Track to evaluate the durability of regionally available, less costly friction aggregate alternatives. All eight sections were installed on the same day using a two-part polymer binder. Calcined bauxite was tested alongside granite, flint, taconite, emery, basalt, steel slag, and silica.

A full report of this study will be released soon. Although several aggregates showed good performance, tests in the laboratory and on the NCAT Pavement Test Track confirmed that calcined bauxite is the superior HFS aggregate. Aggregates used in HFS must be highly resistant to polishing and abrasion. While the NCAT study showed that calcined bauxite is the premium aggregate, these results may provide agencies with more cost-effective, regionally available alternatives for projects where calcined bauxite is not necessary. Safety studies are needed to determine if these alternatives provide an acceptable level of pavement surface friction properties for HFS locations.

As the use of HFS increases for critical crash locations, so does the need to better understand the dynamics of the tire-pavement interface. For these critical pavement locations, what is the relative importance of macrotexture, which reduces hydroplaning potential, and microtexture, the friction characteristic of the surface aggregate-tire interface? High friction surfaces provide a solution to pavements where loss of friction is a significant factor in crashes. Investigating these issues will ensure a cost-effective use of this life-saving technology.

HFS: High Friction Surfaces

They save lives!

Definition:
Pavement treatments with high skid-resistant properties not typically provided by conventional materials

Where do Fatal Crashes Occur?

28% of highway fatalities in the U.S. each year occur at or near horizontal curves

Why?
1. The complex driving task of negotiating the curve
2. High friction demand of a vehicle moving through a curve
3. Wet road surfaces can cause skidding or hydroplaning
4. Pavement surface wear created by tires

Results after Treatment

Milwaukee, WI Marquette Interchange

Oldam County, Ky KY 22

Lane departure crashes 2 years before HFS: 47 2 years after HFS: 5

95% crash reduction in the first year after HFS

Proven Benefits

Reduced crashes, injuries & fatalities

Good benefit/cost ratio

Minimal impact to traffic during construction

Can be quickly implemented

Source: U.S. Department of Transportation, Federal Highway Administration: Every Day Counts Initiative

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How Should We Express RAP and RAS Contents?

A growing number of highway agencies are revising their asphalt specifications to establish limits or trigger points for reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) based on how much the recycled binders contribute to the total binder content of mixes. Some agencies use the term "binder replacement" or "binder ratio" to convey this idea, which changes the emphasis to how the RAP and RAS binders influence mix characteristics and performance.

Historically, most people have expressed RAP and RAS contents as the percentage of the RAP or RAS by weight of total mix. For example, when someone said the mix had 10% RAP and 5% RAS, most people interpreted that to mean that the mix contained 10% RAP and 5% RAS by weight of total mix. From that interpretation, we could then determine how much RAP binder and RAS binder would be in the mix and how much RAP aggregate and RAS aggregate would be in the total aggregate blend.

To calculate "binder replacement" or "recycled binder ratio" we need to know the asphalt contents of the recycled materials, the percentages of the RAP and/or RAS by weight of mix, and the total asphalt content for the mix. Equation 1 shows the math.

Equation 1

$$Recycled\ Binder\ Ratio\ (RBR) = \frac{(Pb_{RAP} \times P_{RAP}) + (Pb_{RAS} \times P_{RAS})}{100 \times Pb_{total}}$$

One concern with this approach is that it appears to equate RAP binder and RAS binder. Although they are both recycled asphalt materials and are stiffer than virgin paving grade binders, RAP and RAS binders are very different. RAS binders, whether from post-consumer or manufacturer's waste, are *much* harder than RAP binders. Performance Grading (PG) of recovered RAS binders is very challenging because they are so stiff. On the high temperature end, RAS binders from shingle manufacturer waste are typically in the range of 125 to 135°C and post-consumer RAS binders typically grade at 150 to 170°C. RAP binders typically grade in the range of 85 to 95°C on the high temperature end and -20 and -5°C on the low end. Making and testing Bending Beam Rheometer (BBR) specimens with RAS binders is extremely challenging, so low temperature grading of pure RAS binders is questionable. Therefore, NCAT now suggests that RAP and RAS binder ratios be kept as separate quantities as shown in equations 2 and 3.

Equation 2

$$RAP\ Binder\ Ratio\ (RAPBR) = \frac{(Pb_{RAP} \times P_{RAP})}{100 \times Pb_{total}}$$

Equation 3

$$RAS\ Binder\ Ratio\ (RASBR) = \frac{(Pb_{RAS} \times P_{RAS})}{100 \times Pb_{total}}$$

Another challenge with the binder ratio equations is that we don't know the optimum asphalt content of the mix (Pb_{total}) until we've completed the mix design. Therefore, we can't determine the recycled binder ratios and we can't know exactly how much RAP or RAS to use in the aggregate blending calculations. However, we can use the formulas and the specification limits to help us get started with the mix design.

Let's say that the specification limits surface mixes to a maximum RAS binder ratio of 0.20. If our RAS material has 22.0% asphalt and we want to use 5% RAS in the mix, then by rearranging equation 3 we can calculate the minimum total binder content to get a RASBR of 0.20.

Example 1

$$min.\,Pb_{total} = \frac{Pb_{RAS} \times P_{RAS}}{100 \times RASBR} = \frac{22.0 \times 5}{100 \times 0.20} = 5.5\%$$

Therefore, your mix design should start out with a trial asphalt content of no less than 5.5%.

For an example using RAP, let's say the specification allows a maximum RAP binder ratio of 0.25, the RAP has an asphalt content of 4.6%, and we would like to design a mix with 30% RAP by weight of mix. Rearranging equation 2, we can calculate the minimum total binder content to get a RAPBR of 0.25.

Example 2

$$min.\,Pb_{total} = \frac{Pb_{RAP} \times P_{RAP}}{100 \times RAPBR} = \frac{4.6 \times 30}{100 \times 0.25} = 5.5\%$$

For either of these examples, if the trial mix design samples with 5.5% asphalt have air voids below 4.0% (or the mix design target air void content), then another aggregate blend or a lower RAP or RAS content will have to be tried.

Another benefit of calculating RAP and RAS binder ratios separately is for estimating the properties of the composite binder when recycled binders are combined with a virgin binder. In the past, we used blending charts to estimate the combined properties of RAP and virgin binders. In effect, blending charts simply determine the weighted average of the properties of the two binders being blended. Unfortunately, blending charts are not helpful when more than two binders are combined. However, a weighted average can be easily calculated for any number of components. Table 1 shows the estimated true grade of composite binders for three example combinations of recycled and virgin binders. This approach, like blending charts, assumes complete blending of the component binders, which may not be the case. Research has yet to find a way to determine the amount of blending that occurs in an actual mix.

For mix designs, we also need to determine how much the RAP and/or RAS contribute to the total aggregate blend. For RAP only mixes, "by weight of mix" and "by weight of total aggregate" differ little when the asphalt content of the RAP and the mix are similar. However, for mixes containing RAS, the differences between "by weight of mix" and "by weight of total aggregate" is more significant.

First, let's consider a mix with 20% RAP by weight of total mix. The asphalt content of the RAP is 5.1% and the asphalt content of the mix design is 5.6%.

The percentage of total aggregate by weight of total mix is…
$100\text{-}Pb_{total}$ = 100-5.6 = 94.4%

The percentage of RAP aggregate by weight of total mix is…
$[P_{RAP} \times (100\text{-}Pb_{RAP})]/100$ = [20×(100-5.1)]/100= 19.0%

Therefore, the percentage of RAP aggregate by weight of total blend is…
19.0%/94.4% = 0.201 or 20.1%

So saying "20% RAP by weight of mix" is almost the same as "20% RAP aggregate by weight of total aggregate."

Now let's consider an example with RAS. Let's say the mix contains 5% RAS by weight of total mix. The asphalt content of the RAS is 21.3% and the asphalt content of the mix design is 5.6%. The fiber content of the RAS was determined to be 1.2% by weight of total RAS.

The percentage of total aggregate by weight of total mix is…
$100\text{-}Pb_{total}$ = 100-5.6 = 94.4

The percentage of RAS aggregate by weight of total mix is…
$[P_{RAS} \times (100\text{-}(Pb_{RAS} + Pf_{RAS}))]/100$ = [5.0×(100-(21.3+1.2))]/100=3.9%

where Pf_{RAS} is the fiber content of the RAS. Although the literature on shingles indicates that the fiber content of RAS may range from 2 to 15%, NCAT test results on RAS from various sources in the USA have found the fiber contents to range from 1 to 4%.

Therefore, the percentage of RAS aggregate in the total aggregate blend is…
3.9%/94.4% = 0.042 or 4.2%

NCAT has begun to use a shorthand notation for expressing RAP and RAS contents in asphalt paving mixtures. Since we want to express both RAP and RAS contents as percentages of total mix and their contributions to the total binder content, NCAT offers the following notation.

A/B | D/E

where:

A = RAP content as a percentage of total mix (%)
B = RAS content as a percentage of total mix (%)
C = RAP binder ratio (decimal)
D = RAS binder ratio (decimal)

For example, 20/5 | .18/.19 indicates that the mix contains 20% RAP and 5% RAS by weight of mix, respectively, and the RAP and RAS binder ratios are 0.18 and 0.19, respectively.

Table 1: Example of Calculating Weighted Average Composite Properties for Different RAP and RAS Binder Ratios

Binder Components				Example 1			Example 2			Example 3		
	True Grade			Binder Ratios (proportion of the total binder)								
Source	High	Int.	Low									
Virgin binder	68	21	-23	0.70			0.80			0.70		
RAP binder	88	31	-15	0.30			--			0.10		
RAS binder	138	32	5	--			0.20			0.20		
Estimated Composite Binder True Grade				High	Int.	Low	High	Int.	Low	High	Int.	Low
				75.2	24.0	-20.6	82.8	23.2	-17.4	84.0	24.2	-16.6

Sampling for Quality: A Comparison of Sampling Locations for Asphalt Paving Mixtures

Minimizing variability in sampling and testing asphalt paving mixtures is essential for quality assurance and acceptance. However, opinions differ regarding where samples should be taken, and practices vary from state to state. Some agencies take samples from a loaded truck at the plant, while others specify sampling from the roadway. Is there a preference for one location over the other, or does it really matter?

NCAT recently conducted a survey of all 50 states, plus the District of Columbia and Commonwealth of Puerto Rico, to compare how highway agencies specify sample location. Survey results indicate that about as many agencies sample from the roadway as from the plant. The survey results showed 22 agencies sample from a truck at the plant, while 22 sample from the roadway. The remaining eight agencies allow samples at either location. Two of those (Hawaii and Montana) routinely sample from the roadway; in the other six agencies that allow either location, sampling at the plant is preferred. The map below shows that plant sampling is most prevalent throughout the Southeast. In the midwest and western states, roadway sampling is more commonly used.

Minnesota, for instance, has recently changed from requiring only roadway samples to allowing truck sampling as well in order to obtain timely test results. Most contractors also prefer plant sampling because it significantly reduces the lag time between when the sample is taken and when test results are known, eliminates the logistical problem of getting samples from the roadway back to a laboratory for testing, and eliminates potential smoothness issues when samples are taken from the finished mat.

The reasons that different agencies provided for sampling at either location were sometimes in contrast with one another. For example, safety was often mentioned as a reason for sampling from the roadway. Agencies are reluctant to have technicians climbing onto, into, or around trucks loaded with hot asphalt mixture due to concerns that a technician may get burned. However, other agencies also cited safety concerns as a reason

NATIONWIDE REVIEW OF ASPHALT PAVING MIXTURE SAMPLING LOCATIONS

PLANT ONLY · ROADWAY ONLY · PLANT AND/OR ROADWAY

for sampling at the plant. Those agencies expressed concern with having technicians stopping the paving operation and walking on hot asphalt material next to live traffic and construction equipment while obtaining roadway samples. One agency that had previously sampled only from the roadway stated they switched to sampling at the plant just to avoid the safety concern of technicians working next to live traffic.

One agency pointed out a potential problem with roadway sampling on contracts involving multiple contractors. In New York, for example, it is common to have one contractor produce the mixture and another contractor place the mix. In those situations, it is challenging to decide which contractor is responsible for repairs, replacement, or reduced pay factors based on roadway samples.

A couple of agencies reported that it was more economical to sample from the roadway because it eliminated the need to have a technician at the plant. However, there were more agencies that believed it actually cost more to sample from the roadway due to logistical problems in getting samples to a laboratory for testing and for the increased risk to the contractor from delays in getting sample results.

Even for agencies that typically sample from the roadway, there are some exceptions for certain mix types or layer thickness. Exceptions reported by several agencies were:
- Open Graded Friction Course (OGFC)
- Mix for leveling courses
- Mix for thin lifts (thickness < 2 x NMAS)
- Mix for low volume projects/small quantities

The required sample size impacts where to take samples. Some agencies require samples of 100 lbs. or more in order to have sufficient material to determine Gmb, Gmm, extraction, tensile strength, etc. Often material is needed for three or four comparisons (contractor, DOT, referee sample, and independent assurance). Samples with this much mass disrupt a large area of the mat, especially when thin layers are being placed.

A number of agencies use metal plates or templates to sample behind the paver, which is where roadway samples are most often taken. However, metal plates are prone to sliding on the pavement under the paving screed as mix is being placed. To avoid this, some agencies nail down the plates. In order to find the plate after it is covered, a thin wire is typically tied to the plate and extended out onto the shoulder or adjacent lane. After the mixture is placed, the wire is pulled up through the mat to locate the plate.

Roadway sampling using metal plates is limited to certain types of paving equipment. When windrows are used, the plates can interfere with the windrow pickup device. In order to resolve the problem, the paver and windrow pickup machine must be stopped for the plate to be placed between the two machines. However, this presents safety concerns in having technicians go between two pieces of roadway equipment to place the plates, as well as smoothness concerns from having to stop the paver.

Sampling from a material transfer vehicle (MTV) offers a safer way to sample. In this case, the MTV is advanced well ahead of the paver, the discharge conveyor is swung over to the shoulder, and mixture is discharged directly into sample cans for transport. There are several obvious benefits from sampling from the MTV:
- Safety is improved because technicians can take samples on the shoulder of the roadway away from live vehicle traffic.
- The paving operation does not have to be stopped while samples are taken.
- Roadway smoothness is not affected.
- Remixing through the MTV is accounted for.
- Certain applications such as leveling and thin lifts do not need to be excluded.

In a recent study, Florida DOT (FDOT) technicians took a sample from a loaded truck at the plant at the same time the contractor's technician also took a comparison sample. The truck was then followed to the roadway so that roadway samples could be taken from the same load of material as the truck samples were taken. Three roadway samples were taken randomly at transverse locations across the mat behind the

Samples were taken at transverse locations behind the paver in a recent FDOT study

paver, as shown on page 6. A single shovel full of mix was taken at each location and combined. Sampling within one foot of the longitudinal edges was avoided. The sampled areas were then filled with mix and raked smooth. In addition, a sample was also taken from the auger chamber of the paver. For safety reasons, the auger samples were taken on the side that was away from traffic. The sampling comparisons were made on 13 projects across the state.

An Analysis of Variance (ANOVA) was conducted of the test results to determine whether project site and sample location were significant factors. Analysis included mixture properties such as the No. 4, No. 8, and No. 200 sieves, asphalt content, and lab compacted air voids. The statistical analysis showed a significant difference in results for percent passing the No. 4 sieve for projects 4 and 7 which were OGFC mixes. Therefore, those two projects were excluded from the analysis.

With projects 4 and 7 removed, another ANOVA was conducted for percent passing the No. 4 sieve. Table 1 shows that the only significant factor was the projects. Sample location was not a significant factor at a 95 percent confidence level (p-value < 0.05). Similar results were found for the other mixture properties evaluated.

Table 2 shows a summary of the ANOVA results for all the mix properties evaluated after the OGFC projects were removed. At the 95 percent confidence level, only the percent passing the No. 200 (P200) sieve was statistically affected by sample location. However, the range of mean values for P200 was only from 5.0 to 5.3 percent. The contractor and DOT samples taken from a truck at the plant ranged from 5.0 to 5.2 percent and were not statistically different. Both mat and auger samples from the roadway averaged 5.3 percent for P200, and this difference was determined to be statistically different from the plant sample taken by the contractor. This difference is not believed to be practically significant and is well within the mix control tolerances used for production. The difference in sequential sum of squares (Seq SS) for P200 also shows that almost all the error is from the difference caused by projects (109.66 vs 1.647).

A final analysis examined the impact of the P200 variations on FDOT Percent Within Limits (PWL) specification tolerances. This was done to determine whether the differences for P200 (as well as for other mixture properties) were great enough to affect a contractor's payment. The PWL was determined by multiplying the mean standard deviation by a Z-factor of 1.645, which is the number of standard deviations needed to encompass 90 percent of the test data and receive a 1.0 pay factor. The results in Table 3 show that all samples were within the current specification except for the auger samples. The auger samples exceeded the PWL specification for P8 and slightly exceeded the limit for percent asphalt content. Based on this comparison, auger samples are not

recommended for mixture acceptance testing. The current specification limits for percent lab air voids are ± 1.2 for fine-graded mixes and ± 1.4 for coarse-graded mixes. FDOT does not have a PWL limit for P4.

Based on these results, either plant or roadway sampling from the mat can be used with equal confidence. However, auger samples are not recommended for use in the acceptance decision.

TABLE 1: General Linear Model: y = % Pass No. 4 Sieve versus Projects, Locations (Projects 4 & 7 Removed)

Factor	Type	Levels	Values
Projects	Fixed	11	1, 2, 3, 5, 6, 8, 9, 10, 11, 13, 14
Location	Fixed	4	Auger, Contractor, Mat, Plant

Analysis of Variance for y, Using Adjusted SS for Tests

Source	DF	Seq SS	Adj SS	Adj MS	F	P
Projects	10	1955.994	1955.994	195.599	46.28	0.000
Location	3	25.297	24.802	8.267	1.96	0.124
Projects*Location	30	143.064	143.064	4.769	1.13	0.314
Error	128	541.033	541.033	4.227		
Total	171	2665.388				

S = 2.05592   R-Sq = 79.70%   R-Sq(adj) = 72.88%

TABLE 2: Summary of ANOVA Results

Property	Projects			Location				
	Seq SS	F-Value	P-Value	Seq SS	F-Value	P-Value	S	R², %
P4	1955.994	46.28	0.000	25.297	1.96	0.124	2.056	79.7
P8	810.373	23.25	0.000	21.847	2.01	0.115	1.863	67.1
P200	109.66	104.33	0.000	1.647	5.03	0.002	0.324	89.75
AC, %	61.298	84.97	0.000	0.209	0.28	0.420	0.268	87.06
Lab Air Voids	56.5817	9.23	0.000	2.047	1.11	0.348	0.785	39.16

TABLE 3: Comparison of PWL Specification Limits

Property	Contractor - Plant		DOT - Plant		Mat		Auger		
	Mean Std Dev	Calc. Spec Limits	Mean Std Dev	Calc. Spec Limits	Mean Std Dev	Calc. Spec Limits	Mean Std Dev	Calc. Spec Limits	Current Spec Limits
P4	1.747	2.87	1.701	2.80	1.708	2.81	2.312	3.80	***
P8	1.536	2.53	1.630	2.68	1.427	2.35	2.068	3.40	± 3.1
P200	0.252	0.41	0.267	0.44	0.319	0.52	0.284	0.47	± 1.0
% AC	0.197	0.32	0.197	0.32	0.167	0.27	0.254	0.42	± 0.4
% Air Voids	0.488	0.80	0.689	1.13	0.59	0.97	0.621	1.02	± 1.2/± 1.4

Demystifying the Dynamic Modulus Master Curve

Dynamic modulus, abbreviated as |E*|, is used to describe how an asphalt mixture's stiffness changes over a range of temperatures and loading rates. It represents asphalt's viscoelastic nature – that is, how its stiffness changes from winter to summer and how it changes for fast or slow moving traffic. In engineering terms, modulus is the applied stress divided by the measured recoverable strain. In this case, stress is defined as the load, or force, divided by the area to which the load is applied. Strain is the deformation of the specimen divided by its unloaded dimension. Shown in the figure below, when cyclic loading is applied, strain is induced in the asphalt concrete, such that the induced strain lags behind the applied stress. This lag in time represents the phase angle, φ, and is an indication of how viscous the material is. A phase angle of 90° represents an entirely viscous material, whereas, 0° represents an entirely elastic material. A small amount of permanent strain, ε_a, can accumulate during testing; however, the recoverable strain, ε_o, is used in defining dynamic modulus.

Figure 1: Illustration of the Phase Angle
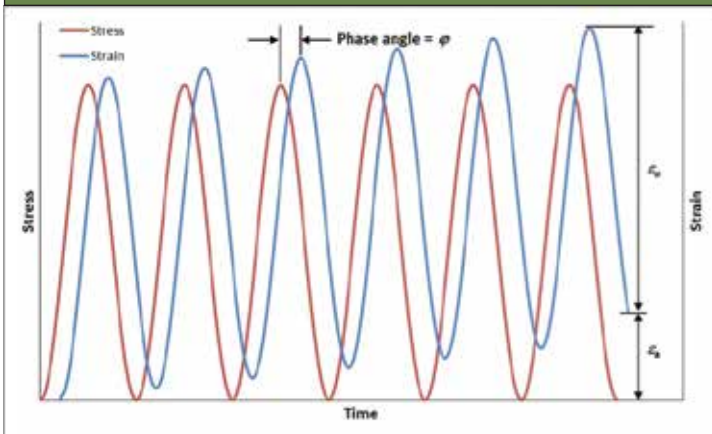
The Asphalt Mixture Performance Tester (AMPT) is commonly used to perform dynamic modulus testing on laboratory-prepared specimens from either lab-produced or plant-produced mix according to AASHTO TP 79-13. Dynamic modulus data is primarily used for two purposes. First, it is used in pavement design to predict pavement responses (stress and strain) for a given vehicle's speed and a pavement temperature. This is important as we move forward with mechanistic-empirical (M-E) pavement design, which relies on engineering properties of the pavement layers to predict pavement performance. Second, it is used to compare the relative stiffness of different asphalt mixtures. For example, it can be used to compare mixtures with different grades of binder, varying RAP or RAS contents, or to compare warm mix to hot mix asphalt.

The dynamic modulus test quantifies the stiffness of an asphalt mixture by applying cyclic loads of a known magnitude on a specimen and measuring its deformation. The loading frequency is adjusted and the process is repeated. After three or four frequency levels, the test temperature is increased and the process is repeated again. Temperature is controlled using an environmental chamber. Frequency, or rate of loading, is intended to represent vehicle speed. Given the viscoelastic nature of asphalt concrete, the magnitude of the induced stress or strain within the material as well as the time the material will experience stress or strain is dependent on the rate of loading. Figure 2 is an example of three strain pulses measured at the bottom of asphalt concrete under the same axle at varying speeds and similar temperatures. The length of the pulse (shown as time in seconds) varies by speed, where the slowest speed (25 mph) has the longest pulse and highest magnitude of strain. A frequency of 10 Hz (the highest E* test frequency) is believed to be equivalent to highway speed.

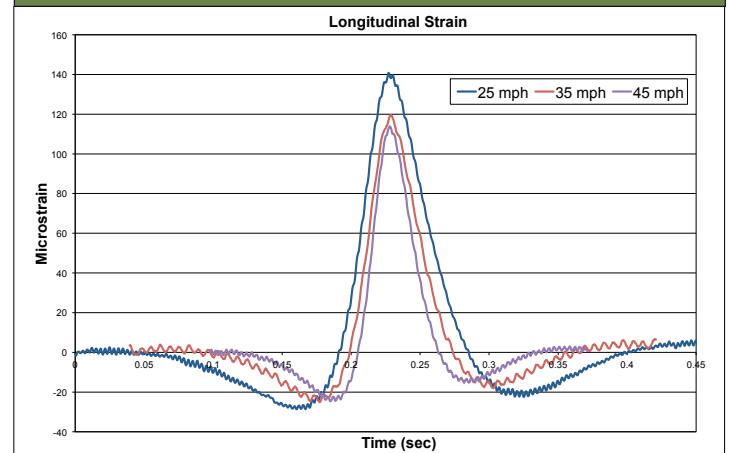## Figure 2: Load Pulse Width as a Function of Vehicle Speed

Figure 3 shows typical test data from the AMPT where the dynamic modulus is plotted against the testing frequency. Another way to illustrate the change in modulus is to plot

Asphalt Mixture Performance Tester (AMPT)

it versus temperature, with families of curves for different speeds (Figure 4). Figures 3 and 4 represent the same set of raw data. Both figures show that material stiffness increases as temperature decreases. In other words, colder temperatures = stiffer pavement. At the same time, as the rate of loading increases the pavement also becomes stiffer. The tested data points can be used to make a dynamic modulus master curve in accordance with the procedure given in AASHTO PP 61-13.

Figure 3: Typical Dynamic Modulus Test Data – Modulus versus Frequecy
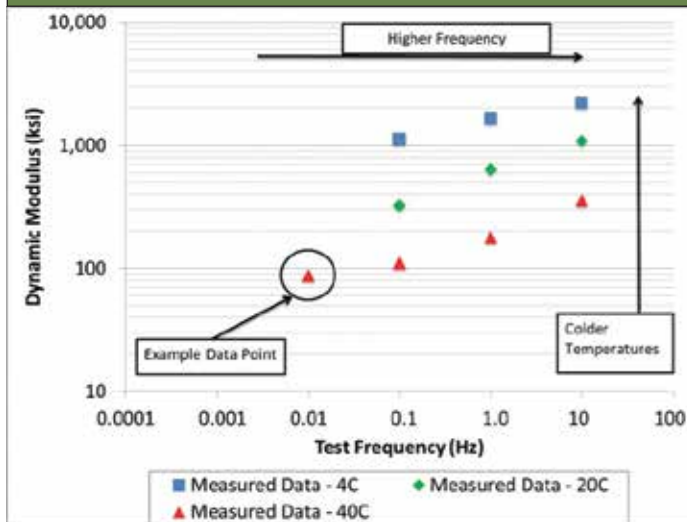
Figure 4: Typical Dynamic Modulus Test Data – Modulus versus Temperature
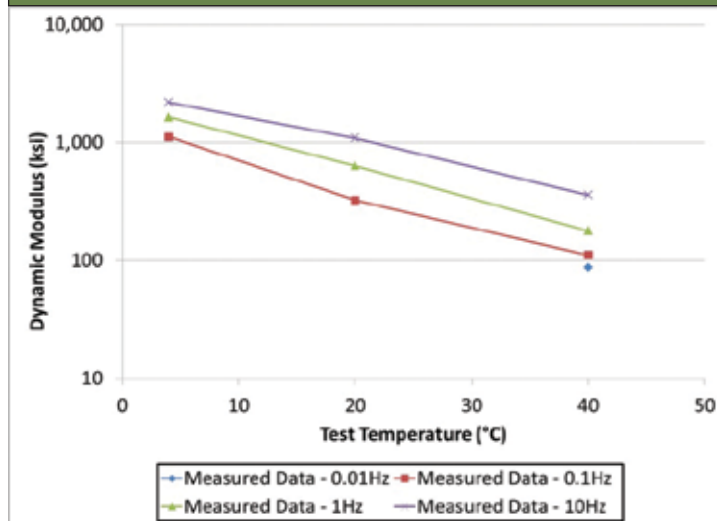
After testing is complete, a dynamic modulus master curve is developed using the time-temperature superposition principle. The time-temperature superposition principle essentially states that time and temperature are equivalent, in the sense that modulus measured at a given temperature under a slow rate of loading (longer time) is equivalent to the modulus that is measured at a high temperature for a fast rate of loading (shorter period of time). The time-temperature superposition principle is a way to relate modulus values of a material obtained at different temperatures and frequencies.

Once a reference temperature has been selected, data collected at various temperatures can be "shifted" with respect to the time of loading so that the different curves can be aligned to form a single smooth master curve. The shift factor, log (a(T)), is the mechanism that allows for the horizontal shifting of the data on the dynamic modulus master curve. However, this requires that the x-axis be scaled to reflect this (see equations 1 and 2) and the concept of the reduced frequency must be employed. To obtain the reduced frequency, f_r, the actual frequency is multiplied by the shift factor for the master curve. The reduced frequency accounts for both frequency and temperature of the original data point, while the temperature is now held constant at the reference temperature.

Equation 1

$$\log(f_r) = \log(f) + \log[a(T)]$$

Equation 2

$$f_r = f \times a(T)$$

where:
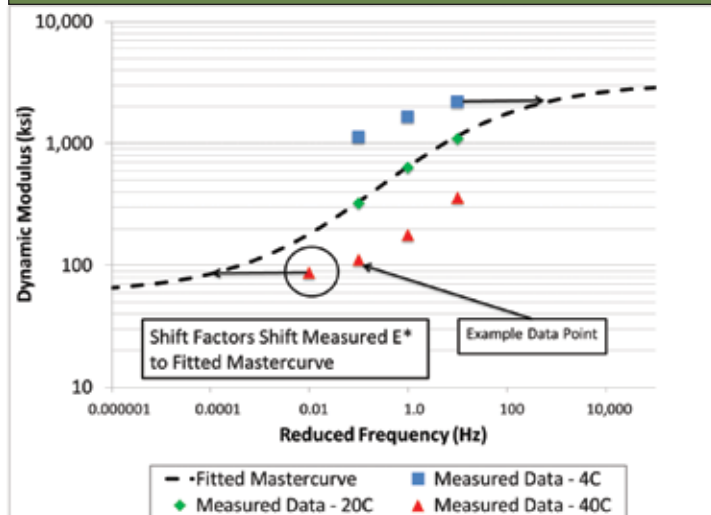f_r = reduced frequency (Hz)
f = testing frequency (Hz)
$a(T)$ = shift factor (T = given temperature)

Through the use of shift factors, moduli measured at different temperatures can be superimposed on a plot at one reference temperature. The common reference temperature for laboratory |E*| data is 20°C. The shift factor is temperature dependent and allows moduli measured at other temperatures to be shifted to a reference temperature (on a log-log plot), creating a singular curve (Figure 5). The left side of the dynamic modulus master curve represents stiffness at high testing temperatures, or slow rates of loading. The right side of the dynamic modulus master curve represents stiffness at low testing temperatures, or high rates of loading.

DID YOU KNOW? Dynamic Modulus (|E*|) is one of the key material inputs in the new AASHTOWare® Pavement ME software, formerly the Mechanistic Empirical Pavement Design Guide (MEPDG), used to predict field fatigue cracking and rutting of asphalt pavements.

Figure 5: Use of Time-Temperature Superposition and Shift Factors to make the E* Master Curve

Figure 6: Using Master Curves to Compare Relative Mixture Stiffness – Logarithmic Scale (above) and Arithmetic Scale (below)

Using the example data in Figures 3 and 5 and following the modulus data point at frequency = 0.01 Hz and T = 40.0°C, the reduced frequency at the reference temperature of 20.0°C (equivalent to this combination of frequency and temperature) is approximately 0.0001 Hz. Since frequency is the inverse of time, a smaller frequency is equivalent to a longer period of time. This reflects the time-temperature position, as the frequency is adjusted to reflect a longer period of time (0.0001Hz frequency or 10,000 seconds loading time) for a cooler temperature (20.0°C) to achieve the equivalency of a higher temperature (40.0°C) but shorter period of time (0.01 Hz or 100 seconds). The shift factor is zero for the dynamic modulus data tested at the reference temperature, since no shifting of the data is required in order for those points to fall on the dynamic modulus master curve.

Dynamic modulus master curves are a useful tool for comparing the stiffness of multiple asphalt mixtures. From the example shown in Figure 6, it can be seen that an additional 15% RAP increases the stiffness of the asphalt material over the majority of the master curve. Typically, master curves are shown with the y-axis in logarithmic scale to illustrate the differences in mixture stiffness at lower stiffness levels. Master curves can also be plotted with dynamic modulus on the arithmetic scale to illustrate the difference in mixture stiffness at the higher stiffness levels. For the example shown in Figure 6, where a control mixture with 20% RAP is compared to a mixture with 35% RAP, comparing the master curves at the lowest reduced frequency (0.000001 Hz at 20°C) shows the addition of 15% RAP increased the dynamic modulus from 12.0 to 15.9 ksi (a 33% increase in stiffness). This difference is easier to see on the logarithmic scale. At 1 Hz (reference temperature of 20°C), the additional 15% RAP increased the mixture stiffness from 789.8 to 1169.3 ksi (a 48.1% increase in stiffness). This difference is easier to visualize on the arithmetic scale.
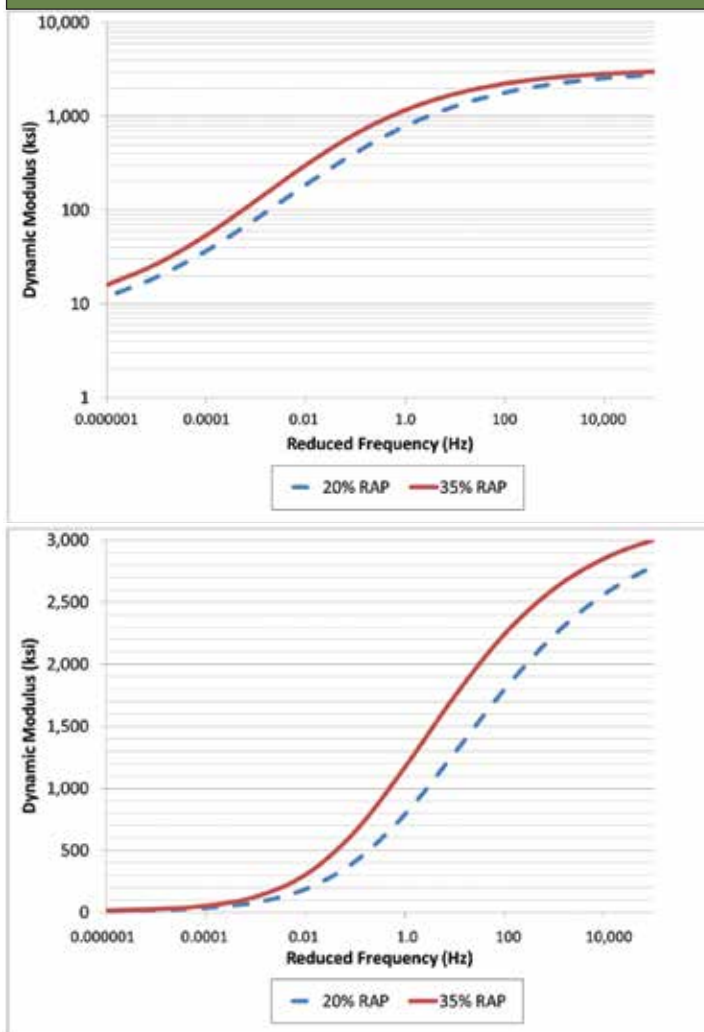
In Search of Longer Life PFC's

Some people go through life searching for that magic potion that will enable them to live longer. Researchers at NCAT and other organizations are working to extend life as well—the life of asphalt pavements, that is. There is an urgent need to increase the lifespan of porous friction courses (PFCs), which allow water to flow directly through the pavement surface. Pavement engineers and traveling motorists alike realize there are many benefits of PFC surfaces, such as reduced hydroplaning, reduced splash and spray, improved visibility of traffic stripes in wet weather, and reduced noise levels. But unless ways can be found to extend the service life of PFCs, pavement policy makers may decide to discontinue their use. For this reason, NCAT is involved in research studies that will help improve the cost/benefit ratio of PFC pavements by extending functionality and service life.

Although the safety benefits of PFCs (formerly referred to as open-graded friction courses, or OGFCs) have been well known for decades, it is not easy to put a dollar value on the safety enhancements. The National Highway Traffic Safety Administration reported that in 2010 32,999 people were killed, 3.9 million were injured, and 24 million vehicles were damaged in traffic accidents, resulting in a total cost of $277 billion to the United States economy. This equates to $897 for every person in the United States. The lifetime economic cost for each fatality was found to be $1.4 million; and when quality-of-life factors are included, the total cost to society is $9.1 million per fatality. Therefore, any reduction in accidents—especially a reduction in traffic fatalities—can have a dramatic impact on our society as a whole. Accident data reported by Texas DOT showed that a route in Travis County had 44 accidents, resulting in five fatalities in 2003. After a PFC surface was added to the roadway in 2004, there were only 17 accidents and no fatalities. For the potential lives that might be saved, as well as the economic benefit to our society, it is imperative that ways be found for PFC mixtures to have long and successful lives.

The two typical distresses that result in reduced performance life of PFCs are raveling and top-down cracking, shown below. There are several possible causes of these types of distresses, and researchers at NCAT believe that these distresses may even be inter-related. For example, thin layer placement, inadequate tack coat, and low asphalt content may result in both raveling and top-down cracking.

Sections at the NCAT Pavement Test Track have provided a clue about how layer thickness affects performance based on the principle of lift thickness to nominal maximum aggregate size (NMAS). In 2009, identical OGFC mixtures were placed on S8 and N2 with the main difference being layer thickness. S8 was placed at 1.3 inches thick (about 135 lb/sy) and performed extremely well, while N2 was placed at 0.8 inches thick (90 lb/sy) and failed within the test cycle so that it had to be replaced. The S8 success is likely related to its thickness/NMAS ratio of 2.5. When the coarse-graded OGFC was placed at 90 lb/sy, the thickness/NMAS was only 1.6, or just over half what was used on S8.

Researchers believe that the mixture must have enough internal cohesion to provide tensile strength values that will withstand the stresses that can tear it apart. Agencies typically specify the use of modified asphalt and depend on the binder to provide the necessary cohesive strength, but binder alone may be insufficient. Dense-graded mixtures demonstrate high tensile strength due to the cohesive nature and interaction of aggregate particles. When PFC is placed at barely more than the thickness of one aggregate particle, the mix does not have enough aggregate overlap to create internal strength, thus premature raveling develops. For these reasons, NCAT recommends a thickness/NMAS ratio of 2.5 for PFC mixtures.

Severe Top-Down Cracking
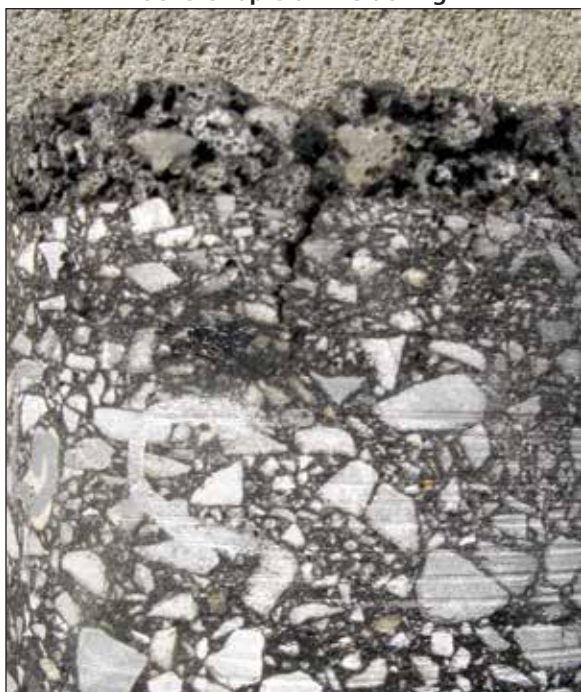
Severe Raveling

During lean economic times, agencies are looking for ways to reduce layer thickness in order to stretch their construction budget further. In order to maintain a reasonable thickness/NMAS ratio when placed at 90 lb/sy, a finer OGFC mixture must be used. Instead of the usual 12.5 mm NMAS, a finer mix such as the 9.5 mm should be considered. Although the 9.5 mm mix was used during the 1980s and 1990s with less than satisfactory performance, it was typically placed too thin (60-70 lb/sy) and did not use modified binder. The 9.5 mm mix has performed exceptionally well on the NCAT Pavement Test Track, where it was placed as the final riding surface on section N13 in 2006. In that section, the fine-graded OGFC was placed at 0.8 inches thick (90 lb/sy) and still had less than 5 mm of rutting after 20 million equivalent single axle loads (ESALs) of traffic. The IRI smoothness results stayed constant at 50-60 inches/mile, and mean texture depth also remained constant at about 1.25 mm, indicating that raveling was not occurring.

In preparing for the 2012 Test Track construction, NCAT conducted two PFC mix designs. At 6.0 percent asphalt, the 9.5 mm PFC had 5 percent more air voids than the coarser 12.5 mm PFC. Therefore, the 9.5 mm mix should provide more void space for additional asphalt, improving its durability without losing its effectiveness in dissipating noise at the tire/pavement interface or in maintaining drainage capacity when PFC is placed less than one inch thick.

The Cantabro test has been recommended for inclusion in the PFC mix design procedure as a means to detect the potential of a mix for stone loss in the form of raveling. When performed on the two mixes mentioned above, the 12.5 mm mix needed 6.0 percent asphalt content to meet the desired maximum Cantabro loss of 15 percent, but at that point, the air voids barely met the minimum agency requirement of 15 percent (Figure 1). Based on these results, we still have much to learn about the importance of gradation and aggregate particle interaction to provide the cohesiveness necessary for long-life PFC mixtures.

Construction-related issues are another common cause of poor PFC performance. Cold mix used at start-up transition joints often leads to raveling problems (as shown on page 13), which may also be caused by placing PFC with an insufficient number of haul trucks. As the paver stops and has to wait for the next truck
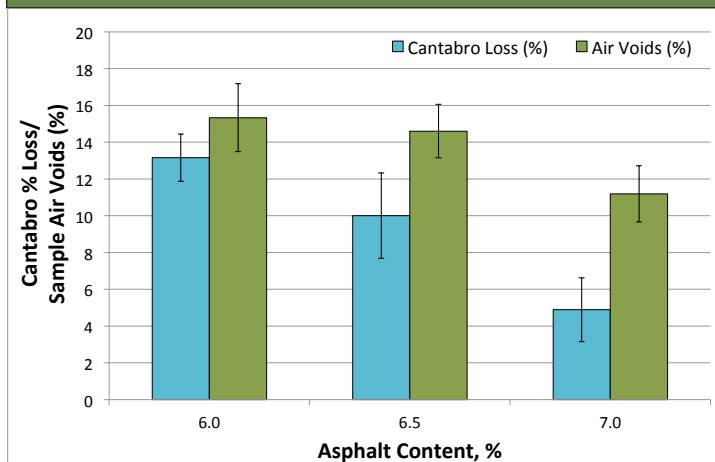

Raveling at Paver Streak

Streaked Tack Application

Figure 1: Effect of Asphalt Content on Cantabro Loss and Air Voids

Raveling at Transverse Joint

to arrive at the project, the mixture cools and generates even less cohesive strength. In some cases, agencies have specified the use of material remixing devices to ensure that PFC mixes are placed at a more uniform temperature.

Streaks in the mat caused by improper adjustment of the screed may also increase the potential for raveling. The center streak shown on the left appears to be caused by inadequate lead crown on the paver screed. This may have resulted in the mat being thinner at the center and therefore more susceptible to raveling, even though there is very little traffic at the center of the mat. Streaks from improperly aligned screed extensions will have a similar effect.

More attention must also be given to tack application. Non-uniform tack shots (as shown on page 12) and low application rates reduce the bond between asphalt layers. A review of 69 tack shots for several projects showed that average tack application rates used for PFCs were less than what is typically expected for dense-graded mixtures (Figure 2). Generally, around 0.06-0.08 gal/sy residual asphalt is recommended when placing PFC mixtures. In contrast, the average rate for tack coat using a paver-laid process (where the tack coat is applied by the paver) is about 0.13 gal/sy (Figure 3). If non-tracking tack is used, the residual application rate should be similar to that of the paver-laid process (about 0.13 gal/sy).

Currently, NCAT is conducting a laboratory investigation of aggregate quality requirements for PFC mixes and a long-term field study that will document and quantify the safety
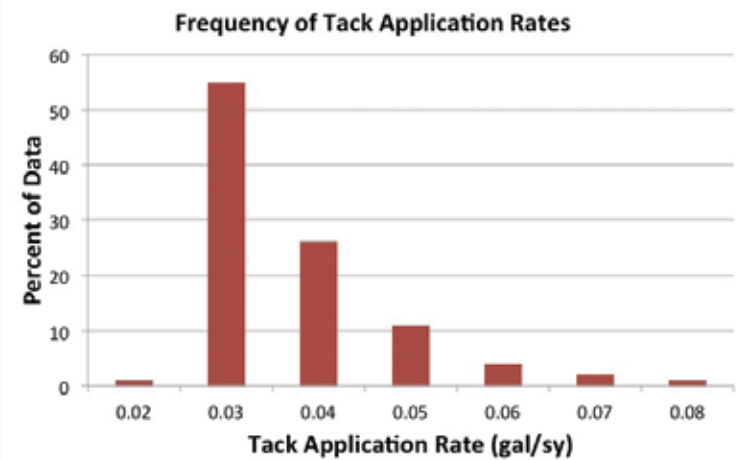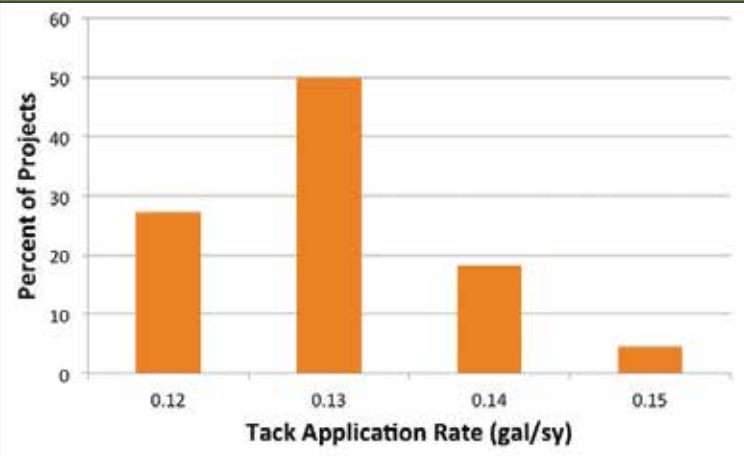
Figure 3: Paver-Laid Tack Coat Application Rate

benefits of PFC mixes over time. NCAT is also beginning work on NCHRP 1-55 to develop a performance-based mix design procedure. This research is much needed, as the current standard PFC mix design procedure, ASTM D 7064-04, does not incorporate performance-based testing that could evaluate the potential for commonly seen distresses. With tests such as the Cantabro to help indicate potential loss of stone and other tests to address top-down cracking, PFC mixes can be designed with a much better chance for long-term success. With improvements in consistency of placement and tack application rates and procedures, long life is very possible indeed.

Improving Aggregate Specific Gravity Tests

Accurate specific gravity measurements for coarse and fine aggregates are essential in designing and producing quality asphalt mixtures. Aggregate bulk specific gravity (Gsb) in particular has a significant impact on voids in mineral aggregate (VMA), which is a key criterion in asphalt mix design and quality assurance.

AASHTO T85 (ASTM C 127) and AASHTO T84 (ASTM C 128) are the standard test methods used to determine the specific gravity of coarse and fine aggregate, respectively. Although these test methods can be performed at a reasonable cost, there are certain drawbacks that have potential for improvement. For instance, both test methods require that samples be soaked 15-19 hours, which is inefficient for quality control testing. Other limitations include the methods of determining the saturated surface-dry (SSD) condition. For coarse aggregate, SSD is determined visually, which can be subjective, particularly when the aggregate is highly absorptive. For fine aggregate, the cone and tamp method is used to determine SSD, but this technique does not work well with aggregate that is angular, highly absorptive, or has high dust content.

NCHRP Project 4-35 evaluated existing test methods, including modified versions of the AASHTO standards, in order to develop improved methods for testing aggregate specific gravity and absorption. Accuracy, precision, and ruggedness were primary considerations, as were ease of use and the total time required for conditioning and testing. Equipment and testing cost were also taken into account, as well as the ability to effectively handle a broad range of aggregate materials.

The aggregate specific gravity test methods selected for the initial laboratory evaluation are shown on page 15. The Gilson SG-5 Specific Gravity and Absorption System was originally included as one of the selected test methods; however, a production unit was not available at the time of the study.

Experiment 1 evaluated these test methods for several aggregate sources with a range of absorptions, particle shapes, and textures. One trained operator conducted all of the testing. Based on the findings, six test methods were selected for further evaluation with a wider variety of aggregate types. The following three test methods were dropped from the study for these reasons:

- Rapid AASHTO T85 with CoreLok—added cost of equipment and bags without significant improvement in the test variability
- Modification to Materials Tested in SSDetect System—repeatability was not improved by removing the P200 material
- Volumetric Immersion using Phunque Flasks for Combined Aggregate—poor precision

Five coarse aggregate and five fine aggregate sources were tested in Experiment 2, encompassing natural, crushed, recycled, and manufactured materials. To evaluate precision, three operators used three different test devices for each method.

For coarse aggregate, AASHTO T85 provided more repeatable and reproducible results than the Phunque method. AASHTO T85 also yielded more accurate results for absorption, Gsb, and Gssd. This was especially true for absorptive aggregate. Another drawback to the Phunque method is that the flasks are fragile. Based on these findings, AASHTO T85 was still the best method for testing coarse aggregate specific gravity and absorption.

For fine aggregate, the modified AASHTO T84 method (removal of P200 material) and SSDetect yielded the most accurate, repeatable, and reproducible results for absorption, Gsb, and Gssd. One drawback of the modified AASHTO T84 procedure is that another test is required to measure the specific gravity of the P200 material. Advantages of the SSDetect system are that it measures the full gradation of fine aggregate, it removes subjectivity by using infrared light to determine the SSD condition, and it can be completed in substantially less time than the modified AASHTO T84 method (one day instead of three). However, the equipment cost for the SSDetect (approximately $7000) is considerably higher. For fine aggregate with less than 10 percent P200 material, the standard AASHTO T84 method yielded acceptable results. The Phunque method yielded less accurate measurements, and as noted before, the flasks were not very durable. Based on these findings, the Modified AASHTO T84 and SSDetect methods were recommended for testing fine aggregate specific gravity and absorption.

In order to reduce testing time for AASHTO T84 and T85, Experiment 3 evaluated other methods for drying and soaking aggregate samples. Alternatives to oven-drying include vacuum-drying and beginning the testing with the sample in its in-situ moisture condition. The results for aggregate tested in the in-situ moisture condition were closer to the results of the oven-dried aggregate, but some materials still yielded statistically significant differences. The vacuum-drying method, however, was unable to completely dry highly absorptive aggregate.

The standard 15-hour soaking method was also compared with vacuum-soaking for 5, 10, and 15 minutes (similar to the vacuum saturation method in AASHTO T 209). Vacuum-soaking for 10 minutes yielded Gsb results that were comparable to those measured using the 15-hour soaking method for both coarse and fine aggregate. This modification considerably reduces testing time, which is especially important for QA testing.

Experiment 4 investigated the impact of P200 materials both with and without clay on AASHTO T84 test results. Results showed that if the material does not contain absorptive clays, including the P200 fraction does not significantly affect AASHTO T84 results. However, when P200 is more than 10 percent, including the P200 fraction may result in lower absorption due to over-drying of the sample to reach SSD condition. When highly absorptive clays are present, erroneous AASHTO T84 results can occur. Thus, it was recommended that the P200 fraction be tested separately (ASTM C 110 or AASHTO T 133) when the sand equivalent value is less than 75 percent. However, this sand equivalent value was based on limited data and needs further verification.

Proposed changes based on the findings of these experiments were incorporated into the procedures for AASHTO T85 and T84, and a ruggedness study then evaluated the sensitivity of the test methods to potential variations in operating and environmental parameters. Seven sources of variation that could affect test results were evaluated at minimum and maximum

anticipated levels using multiple coarse and fine aggregates with low and high absorption. The variables included parameters such as soak time, water temperature, and distribution of blows in the cone tamp test. Three laboratories participated in the ruggedness study: NCAT, AASHTO Advanced Pavement Research Laboratory (AAPRL), and the National Institute for Science and Technology (NIST). The findings were used to establish suitable ranges for operating parameters and minimize sources of variation within the test procedures, ensuring more precise test results.

The following modifications were proposed for AASHTO T85:

- Agency and contractor testing should be conducted on the same size aggregate, either retained on the 4.75-mm (No. 4) or 2.36-mm (No. 8) sieve.
- For low-absorption aggregate (<2 percent), aggregate can be tested in its in-situ moisture condition.
- Oven-dried samples should be cooled to approximately 50°C (comfortable to handle) prior to submersion in room-temperature water.
- For low-absorption aggregate (<2 percent), samples can be vacuum soaked for 10±0.5 minutes.
- For high-absorption aggregate (>2 percent), samples should be hydrostatically soaked for 15-16 hours.
- The tolerance for the water bath temperature should be changed from 23.0±1.7°C to 23.0±1.0°C to improve test result variability.
- Soaked aggregate samples should be dried using the same method (dry or damp cloth) for both agency and contractor testing.

The following modifications were proposed for AASHTO T84:

- Testing should not include P200 material.
- Fine aggregate can be tested in its in-situ moisture condition.
- For low-absorption aggregate (<2 percent), samples can be vacuum-soaked for 10±0.5 minutes.
- For high-absorption aggregate (>2 percent), samples should be hydrostatically soaked for 15-16 hours.
- The tolerance for the water bath temperature should be changed from 23.0±1.7°C to 23.0±1.0°C to improve test result variability.
- To further improve variability, consistent methods should be used for distributing the blows in the cone test (either 25 times in succession or in four sets) and for eliminating air bubbles (either by manual or mechanical agitation).

TEST METHODS EVALUATED

COARSE AGGREGATE
- AASHTO T 85 and ASTM C127
- Rapid AASHTO T 85 Using CoreLok
- Volumetric Immersion Using Phunque Flasks

FINE AGGREGATE
- AASHTO T 84 and ASTM C128
- Modified AASHTO T 84 (Removal of P200 Material*)
- SSDetect System (ASTM D7172)
- Modified SSDetect (Removal of P200 Material*)
- Volumetric Immersion Using Phunque Flasks

COMBINED COARSE AND FINE AGGREGATE
- Volumetric Immersion Using Phunque Flasks

*AASHTO T133 Used to Test P200 Material

Can Recycled Asphalt Shingles and Ground Tire Rubber Eliminate the Need for Fibers in Open-Graded and Stone Matrix Asphalt Mixtures?

Open-graded friction course (OGFC) and stone matrix asphalt (SMA) mixes typically contain some type of fiber to prevent binder drain down while the asphalt mixes are hot. When binder drain down occurs, it results in small flushed areas of the pavement with excess asphalt (sometimes called "fat spots"). The rest of the mix is lean, which results in premature raveling and cracking. After the introduction of SMA to the U.S., the use of fibers in OGFC became common. Prior to the use of fibers, OGFC mixes were typically produced at cooler temperatures (240 - 260°F) and had lower binder contents. Currently, OGFC and SMA mixtures also contain polymer-modified binders to improve mixture durability.

The two types of fibers commonly used in OFGC and SMA mixes are mineral and cellulose, with cellulose fibers more commonly used. The cost of fibers is around $0.25 per pound and adds about $1.50 per ton to the cost of an asphalt mix.

Other fiber types and additives have been evaluated over the past several decades to provide drain down resistance including waste carpet fibers, waste cotton fibers, paper mill sludge, polyester fibers, and aramid fibers. Other additives evaluated for drain down resistance have included ground tire rubber (GTR) and some chemical warm mix asphalt additives.

Studies in the United States and abroad have shown that GTR can be effectively used for drain down resistance and as a binder modifier in SMA and OGFCs. It has been well understood since the 1990s that GTR can be used to modify asphalt binders to improve their rutting and cracking performance. A recent NCAT study (Report 13-11) examined how using different GTR-modified binders affected the performance of OGFCs in comparison to an OGFC that used a polymer-modified binder and fibers. The study showed that using 10% GTR by weight of the binder eliminated the need for fibers in the drain down test. Additionally, the GTR-modified OGFC maintained commonly accepted lab test criteria for durability, moisture damage, and rutting. GTR-modification increases the viscosity of the binder and allows for slightly higher total binder contents, increasing the film thickness of the binder on the aggregate needed for specialty mixture performance. Thus, the increased binder viscosity prevents drain down while the increased film thickness ensures mixture durability.

While laboratory tests have shown that GTR is a viable replacement for both fibers and polymer-modified asphalt, there have been few documented field studies where fibers were eliminated from the mixtures. Two case study OGFCs were conducted as part of the 2012 NCAT Pavement Test Track, which used GTR-modified binders and no fibers. Section S1 was sponsored by the Virginia Department of Transportation, and Section E9b was an OGFC sponsored by the Alabama Department of Transportation. At the time of construction, both of the mixtures met standard drain down requirements. After approximately eight million equivalent single axle loads (ESALs), both test sections show excellent rutting and durability performance. Cracking or raveling are not evident in either test section to date.

A few highway agencies and researchers are beginning to assess the possibility of using RAS as a substitute for fibers in specialty mixtures. Like GTR, RAS binder has a high viscosity that can reduce drain down potential. Additionally, RAS commonly has a fiber content between 2% and 3%. When released from the asphalt mastic, the RAS fibers can prevent drain down.

The Illinois Tollway, the City of Chicago, and Illinois DOT have used 5% post-consumer RAS in SMA mixtures without added fibers on many projects. Testing during construction showed no signs of drain down and the projects have performed well through very harsh weather. A couple of the projects were part of the National Pooled Fund Study on RAS. Similarly, a test section was built on the NCAT Test Track in 2012 with an SMA surface layer containing 5% post-consumer RAS and no added fiber. Under the severe loading on the track, the SMA with PC-RAS has outperformed a dense-graded Superpave mix with 20% RAP.

The Tennessee DOT constructed an OGFC on the test track with only 3% RAS and no additional fibers. At the time of construction, the mixture barely failed the drain down test; however, the DOT elected to keep the mixture in place. To date, no cracking, raveling, or flushing has been witnessed in this section.

In summary, lab experiments and field trials have shown GTR and RAS as possible substitutes for fibers in open and gap-graded specialty asphalt mixtures. Further evaluation of the long-term performance of such mixtures continues. As the paving community shows continued interest in the triple bottom line of sustainability, the use of these recycled materials to enhance the performance of certain asphalt mixes appears to be a positive step forward.

NCAT had the pleasure this spring of hosting members of the Board of Directors and staff of Flexible Pavements of Ohio

Asphalt Forum

Greg Sholar, Florida DOT
Does anyone have success with micro-surfacing on medium to higher traffic volume roads (greater than 5 million ESALs for a 20 year design)? If so, what is the life expectancy in years?

Oak Metcalfe, Montana DOT
We continue to have issues with CRS-2P as described in the Spring 2014 newsletter (shown below).

Mark Woods, Tennessee DOT
What preliminary requirements are states using to select projects for HMA thin lifts or micro-surface projects? Are there any quantitative limitations such as IRI or crack thickness?

Asphalt Forum Responses

The following responses have been received to questions shared in the Spring 2014 Asphalt Forum.

1. We appear to be having an issue in Montana with our CRS-2P emulsions "reactivating" at higher mat temperatures. The material doesn't fail our distillation and penetration tests, but if the ambient temperature rises after application the emulsion allows for chips to roll and causes all sorts of splash. Ultimately, the emulsion cures and the chip seal performs. Do you believe this is a crude source issue? We allow the use of latex for modification, which could also be a problem. (Oak Metcalfe, Montana DOT)

   Eric Biehl, Ohio DOT
   Ohio DOT allows latex modification and to my knowledge hasn't seen this issue. We do not follow AASHTO M 316 and have a softening point requirement minimum of 140° F.

2. We are concerned with the use of RAS in Ohio, especially in surface mixes and the lack of free asphalt available. What courses do you allow RAS? What is the maximum amount of RAS allowed? Do you have a correction factor for RAS binder in mix designs? Any data available on durability with mixes with RAS in them? Any other special requirements such as mixing temperatures, use of WMA, etc.? (Eric Biehl, Ohio DOT)

   Michael Stanford, Colorado DOT
   Our use of RAS is project specific. To date, we have not had much success due to premature cracking on the top mat. The maximum amount of RAS allowed is 5% by weight, not to exceed 30% effective binder replacement. We have a correction factor for RAS binder, and only allow RAS from manufactured shingle waste or post-consumer shingles as defined by AASHTO MP 15.

   Greg Sholar, Florida DOT
   FDOT is only conducting pilot projects at this time. For mainline, we are restricting it to lower structural layers. We are allowing RAS in the surface layer for 10" wide shoulders. We allow a 20% maximum amount of RAS by binder replacement, and when using RAS and RAP the combined total must not exceed 40% binder replacement. FDOT currently uses a 0.85 binder availability factor. FDOT does not allow WMA with the use of RAS. All mixtures must use a PG 52-28. 100% must pass the 3/8" sieve and 95% must pass the 4.75 mm sieve.

   Thomas Clements, Kentucky Transportation Cabinet
   We allow RAS in surface and base courses, but it cannot be used with PG 76-22. All percentages represent effective binder replacement. With PG 64-22: ≤13% surface, ≤15% base. With PG 58-28: 14-20% surface, 17-24% base. We have a correction factor of .75 for RAS binder. We haven't been using RAS long enough to really evaluate it.

   Chris Abadie, Louisiana DOT
   Louisiana does not allow RAS.

   Bryan Engstrom, Massachusetts DOT
   MassDOT will be piloting RAS in the near future.

   Kevin McCaskill, Mississippi DOT
   MDOT does not currently allow/use RAS.

   Oak Metcalfe, Montana DOT
   We are silent at this point but are considering its use.

   Denis M. Boisvert, New Hampshire DOT
   We allow intermediate and base use of RAS, but are currently proposing not to allow it until more is known about performance. We allow 0.6% based on weight of the mix. Composite binders must be tested to determine needed virgin grade and verify that the specified grade is met.

   Timothy L. Ramirez, Pennsylvania DOT
   We currently allow the use of RAS in dense graded base, binder, and wearing courses. The maximum amount allowed is 5% by weight of total mixture.

   Mark E. Woods, Tennessee DOT
   Tennessee currently allows RAS in all dense-graded mixtures at a maximum of 5% by weight. The maximum allowable ratio of recycled binder is 35% for base and binder courses

and 20% for surface courses. The maximum amount of RAS allowed is 5%. We assume a blending efficiency of 75%. We do not yet have any data available on mixture durability.

Howard Anderson, Utah DOT
So far we have not used RAS. We do not allow RAP in any of our surface mixes.

3. Have any states had experience using 9.5mm in lieu of 12.5mm OGFC on an interstate or other high ADT routes? Do you feel like there is a need to use PMA and fibers in those finer OGFC mixes, especially since we have WMA and other means to reduce drain-down? (Cliff Selkinghaus, South Carolina DOT)

Michael Stanford, Colorado DOT
Colorado does not use OGFC.

Chris Abadie, Louisiana DOT
There is a need to use PMA in those finer OGFC mixes, but maybe not fibers.

Kevin McCaskill, Mississippi DOT
To date, all of our OGFC mixes have been 9.5mm. OGFC mixes in Mississippi are still relatively new, so there is limited experience. Future field and/or lab data may allow the fiber requirement to be reduced and/or eliminated from specifications.

Eric Biehl, Ohio DOT
Ohio DOT rarely uses OGFC, and when we do they are 12.5mm.

Mark E. Woods, Tennessee DOT
Tennessee does not have experience with any OGFCs finer than our standard 12.5mm. We specify PG76-22 and cellulose in all OGFCs, with or without WMA.

4. What states use RAP and RAS together? If so, in what maximum percentages? If not, do you have any plans for trial projects to determine feasibility as a cost-saving measure? (Chris Jones, Wiregrass Construction Company)

Michael Stanford, Colorado DOT
Total binder replaced by RAP and RAS shall not exceed 30% of the effective binder content.

Greg Sholar, Florida DOT
There is a need to use PMA in those finer OGFC mixes, but maybe not fibers.

Thomas Clements, Kentucky Transportation Cabinet
All percentages represent effective binder replacement. With PG 64-22: ≤15% surface, ≤18% base. With PG 58-28: 16-25% surface, 19-30% base.

Kevin McCaskill, Mississippi DOT
MDOT does not currently allow the use of RAP and RAS together.

Denis M. Boisvert, New Hampshire DOT
New Hampshire allows a combined replacement total of 1.5%, of which 0.6% can be RAS binder. We are planning to discontinue allowing RAS.

Eric Biehl, Ohio DOT
Ohio DOT allows both. When we permit RAS, we only allow 5.0% by dry weight of mix. Our RAP amounts vary depending on the course and how the RAP was processed, which we go all the way up to 55% RAP & 5% RAS. We also have a minimum total virgin binder content.

Timothy L. Ramirez, Pennsylvania DOT
Pennsylvania allows the use of RAP and RAS together. Maximum amount of RAS is 5% by total weight of mixture. Any combination requires the submission of RAP and RAS materials to the department's central lab for asphalt binder analysis.

Mark E. Woods, Tennessee DOT
Tennessee permits the use of RAP and RAS together within the limitations of recycled binder ratios defined previously.

Florida
FDOT will be switching to trackless tack coats exclusively with awarded contracts starting January 2015. Currently, five approved products have had test sections placed and bond strength cores tested.

Kentucky
We are in the process of changing from elastic recovery to multiple stress creep recovery testing for binders. This is being done so that we can better evaluate binders that are using more modern production procedures. In addition, we will now be using a national standard as opposed to a state specific one. This will make binder research easier for us and be more convenient for suppliers.

Massachusetts
MassDOT is currently in the second year of Hamburg testing for mix approval. The passing rate has increased since the first year.

Mississippi
MDOT is revising the crumb rubber specification to allow 30-40 mesh material versus the current 80 mesh requirement. We are developing a specification allowing RAS use and a specification allowing the use of RAP in SMA.

Montana
We are doing some internal research to determine if and/or how we can adopt the MSCR test for asphalt binders, and we have altered our VMA specs for 3/8" and 1/2" HMA. VMA will now be 14.5 for 1/2" mixes and 15.5 for 3/8" mixes.

New Hampshire
Use of RAS has been proposed for removal from our specification, and it will likely be finalized in September 2014. New Hampshire has issued a ban on the use of re-refined engine oil bottoms in its binders until additional performance data is available to show that it is not harmful to our pavements.

Tennessee
We are developing a fully new specification book, which we expect to be published in either late 2014 or early 2015. Many administrative and formatting changes are being made, but no functional developments come to mind.

Utah
We are working on new specifications for asphalt longitudinal joint density, intelligent compaction, and cold in-place recycling (CIR). For intelligent compaction, we are using only the mapping of the rollers and the temperature of the mat. We are not using the stiffness at this time. For CIR we have conducted research determining field performance tests to show if the material is ready to open to traffic, and we are also coming up with a shorter and simpler mix design procedure.

Professor Training Course
June 16-20, 2014
National Center for Asphalt Technology

Front Row (L-R): Nam Tran (Instructor), Buzz Powell (Instructor), Jason Moore (Instructor), Don Watson (Instructor), Mike Heitzman (Instructor), Randy West (Instructor), Jason Nelson (Instructor)
Middle Row (L-R): Thomas Korman, Ashok Julaganti, Mohammad Hossain, Rajan Saha, Ashraf Rahim, Zhen Liu, Shu Yang, Xiaoyang Jia, Roger Subramanian
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