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# Asphalt Technology News

IN THIS ISSUE



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at AUBURN UNIVERSITY

## Moving Towards Balanced Mix Design for Asphalt Mixtures

### What is Balanced Mix Design?

In September 2015, the FHWA Expert Task Group on Mixtures and Construction formed a Balanced Mix Design Task Force. This group defined balanced mix design (BMD) as "asphalt mix design using performance tests on appropriately conditioned specimens that address multiple modes of distress taking into consideration mix aging, traffic, climate and location within the pavement structure." In short, BMD incorporates two or more mechanical tests such a rutting test and a cracking test to assess how well the mixture resists common forms of distress. The Task Force identified three potential approaches to the use of BMD: volumetric design with performance verification, performance-modified volumetric mix design, and performance design. These approaches are illustrated in Figure 1 and briefly discussed as follows:

1. **Volumetric Design with Performance Verification.** Basically, this is the straight Superpave volumetric mix design approach with performance tests conducted at the end. If the mixture does not pass performance tests, the entire mix design process is repeated. This approach is currently used in Illinois, Louisiana, New Jersey, Texas, and Wisconsin.
2. **Performance-Modified Volumetric Mix Design.** This approach begins with the Superpave mix design method to establish an initial aggregate blend and asphalt content. Adjustments in the mix proportions are then permitted to meet the performance tests. The final design may not be required to meet all of the traditional Superpave criteria. California currently uses this approach.
3. **Performance Design.** This approach skips the volumetric mix design and starts with evaluation of mix trials (possibly multiple gradation trial blends and asphalt contents) using the performance tests. Minimum requirements may be set for asphalt binder and aggregate properties. Traditional volumetric criteria may be used as non-mandatory guides but not as design criteria. This approach is not currently used but could be a viable option.

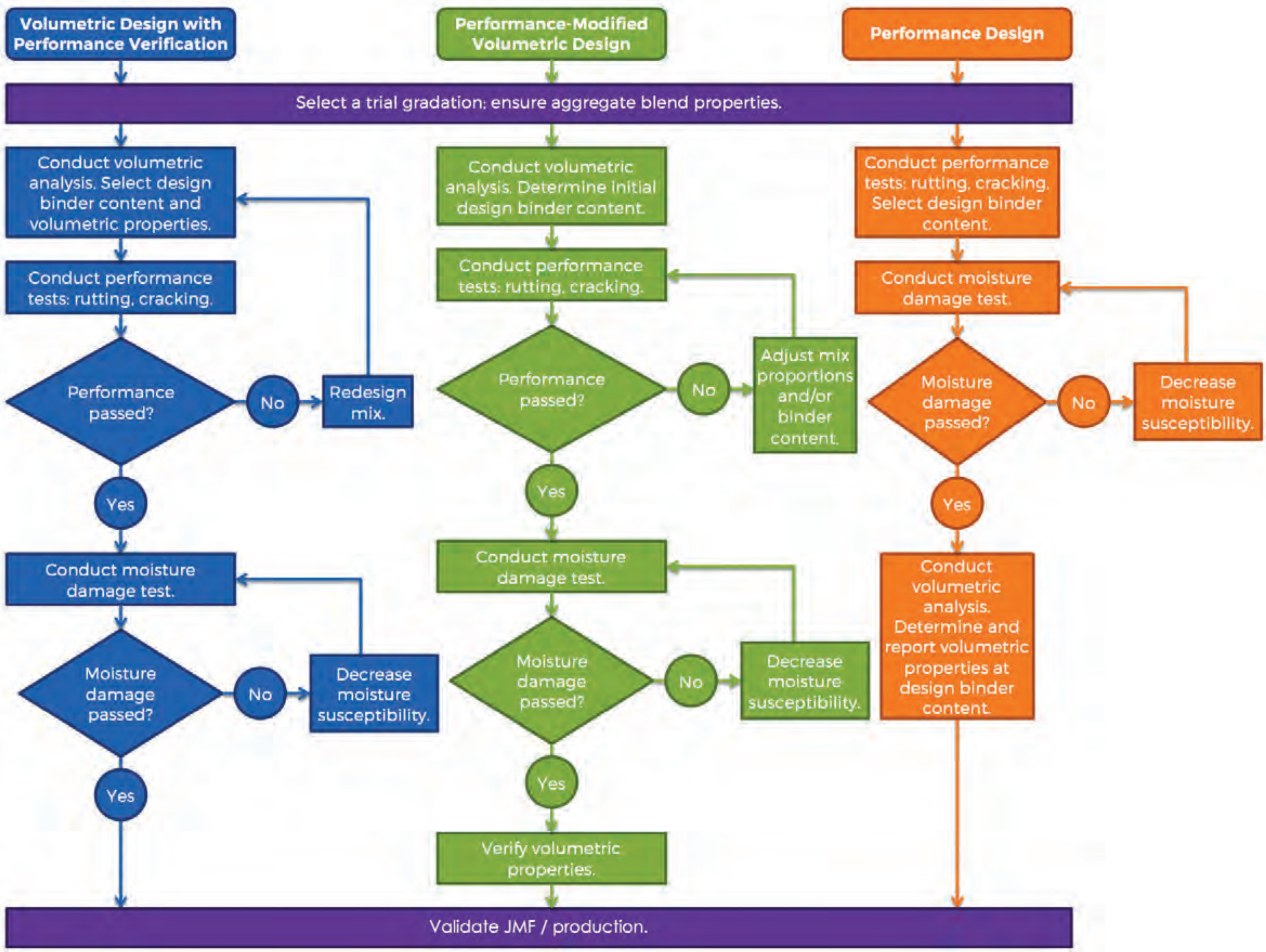



*Figure 1: Flowchart of Three Balanced Mix Design Approaches*

## Why Is Balanced Mix Design Needed?

Today, asphalt mixtures are primarily designed under the Superpave system, where proportioning of the aggregates and the asphalt binder relies primarily on empirical aggregate quality characteristics and mixture volumetric properties such as air voids, voids in the mineral aggregate (VMA), and voids filled with asphalt (VFA). Calculation of the volumetric properties is highly dependent on an accurate determination of the specific gravity of the mix components. However, there are well-known issues with the accuracy and variability of aggregate bulk specific gravity testing. This raises serious concerns about whether or not the correct amount of asphalt is selected in the mix design. Mixtures designed with too much asphalt may be susceptible to rutting, while those with too little asphalt are prone to cracking, raveling, and other durability related pavement distresses. Concerns about the accuracy of aggregate specific gravity determinations increase with the incorporation of reclaimed asphalt pavements (RAP) and recycled asphalt shingles (RAS). Moreover, the effects of the binders in these recycled materials are not captured in volumetric properties. It is still not well understood how recycled binders interact with virgin binders, which ultimately creates more doubt about how these materials affect field performance. Furthermore, the effects of warm-mix asphalt additives, polymers, rejuvenators, and fibers cannot be assessed in the current volumetric mix design method. Therefore, performance tests need to be included as part of the mix design procedure to help ensure desirable pavement performance in the field.

## What Performance Tests Are Available, and How Are These Used in Balanced Mix Design?

Over the past few decades, numerous performance tests have been developed by different researchers to evaluate the rutting resistance, cracking resistance, and moisture susceptibility of asphalt mixtures. Considering the different mechanisms in crack initiation and propagation, mixture cracking tests can be further categorized into thermal cracking, reflection cracking, bottom-up fatigue cracking, and top-down fatigue cracking. Table 1 provides a list of mixture performance tests that are commonly used in asphalt research and are being considered by highway agencies for use in mix design approval. In order to include any mixture performance test in the BMD procedure, criteria for the test result must be established based on a strong relationship to field performance.

Performance space diagrams (PSDs) may be used in BMD to simultaneously chart the results of multiple performance tests. PSDs allow the mix designer to engineer the mix to provide the desired performance and to illustrate the impact of varying mix factors on performance. An example of PSD is shown in Figure 2. This example shows Hamburg wheel tracking test (HWTT) versus disk-shaped compact tension test (DCT) results and criteria to identify performance and applicability zones. Other test results such as stability versus durability/cracking can be substituted with the same application.

*Table 1: Commonly Used Asphalt Mixture Performance Tests*

| Mixture Property | Laboratory Test | Test Standard |
|---|---|---|
| Thermal Cracking | Disk-Shaped Compact Tension Test | ASTM D7313-13 |
| | Indirect Tensile (IDT) Test | AASHTO T 322-07 |
| | Semi-Circular Bend (SCB) Test | AASHTO TP 105-13 |
| | Thermal Stress Restrained Specimen Test | BS EN12697-4 |
| Reflection Cracking | Disk-Shaped Compact Tension Test | ASTM D7313-13 |
| | Texas Overlay Test | TxDOT Tex-248-F<br>NJDOT B-10 |
| Bottom-Up Fatigue Cracking | Direct Tension Cyclic Fatigue Test | AASHTO TP 107-14 |
| | Flexural Bending Beam Fatigue Test | AASHTO T 321<br>ASTM D7460 |
| | IDT Fracture Energy Test | N/A |
| | Illinois Flexibility Index Test | AASHTO TP 124-16 |
| | SCB at Intermediate Temperature | LaDOTD TR 330-14<br>ASTM D8044-16 |
| | Texas Overlay Test | TxDOT Tex-248-F |
| Top-Down Fatigue Cracking | Direct Tension Test | N/A |
| | IDT Energy Ratio Test | N/A |
| Rutting | Asphalt Pavement Analyzer | AASHTO T 340 |
| | Flow Number | AASHTO TP 79-15 |
| | Hamburg Wheel Tracking Test | AASHTO T 324 |
| | Superpave Shear Tester | AASHTO T 320-07 |
| | Triaxial Stress Sweep Test | AASHTO TP 116-15 |
| Moisture Susceptibility | Hamburg Wheel Tracking Test | AASHTO T 324 |
| | Tensile Strength Ratio | AASHTO T 283 |

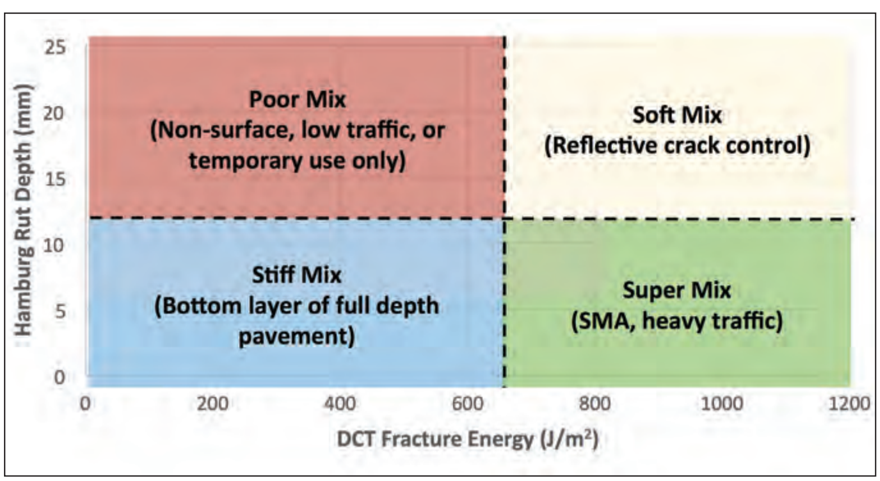



*Figure 2: Example of Performance Space Diagram*

## What Is Being Done to Move toward Balanced Mix Design?

A few highway agencies have started to either explore or adopt approaches to BMD. Others are evaluating how their current mixtures fare with some of the performance tests. In Texas, where the BMD concept originated, the HWTT is used to evaluate resistance to rutting and the overlay test (OT) is used to evaluate cracking resistance. Volumetric criteria are used to select the asphalt binder content. Performance tests are conducted at three asphalt contents: optimum and optimum ±0.5%. The Louisiana Department of Transportation and Development uses a BMD approach that also uses the volumetric mix design process with the HWTT but uses the semi-circular bend (SCB) test and a parameter called J-integral to assess the cracking potential of mix designs. The Illinois approach is similar but uses a simpler SCB test that yields a parameter called Flexibility Index to assess cracking resistance.

The California Department of Transportation (Caltrans) has a pavement design framework that includes performance-based specifications and the CalME (Caltrans Mechanistic Empirical Design Program) to perform mix designs. Performance testing consists of repeated shear and frequency sweep tests using the Superpave shear tester, flexural bending beam fatigue tests, and HWTT.

Rutgers University recently proposed a performance-based balanced mix design procedure for the New Jersey Department of Transportation (NJDOT). The proposed method sets the asphalt content at the midpoint between the maximum asphalt content to meet the Asphalt Pavement Analyzer (APA) test criteria and the minimum asphalt content to satisfy the OT criteria. The performance criteria for both tests were established based on the field performance of existing NJDOT projects.

Minnesota, Ohio, Utah, Maryland, Wisconsin, and Florida are at various stages of research, development, and implementation of BMD. Despite all of these efforts, there are still some gaps in the knowledge needed for future development of a detailed, comprehensive standard practice. Some of the gaps include:

- The relationship of test results to pavement performance,
- Sensitivity of the test to changes in key properties (e.g. air voids),
- Suitability of the tests to assess the effects of non-traditional materials,
- Testing issues including precision, bias, and ruggedness,
- Failure criteria for rutting and cracking tests appropriate for different regions,
- Conditioning protocols for aging and moisture,
- Tests for assessing workability, compactability, and segregation susceptibility,
- Applicability of the tests to quality assurance and acceptance testing,
- The role and use of volumetric properties,
- The use of cores in the performance test, and
- A lack of training materials and courses for implementation.

The most recent effort to move toward BMD is NCHRP Project 20-07/Task 406. The objective of this research is to develop a framework that addresses alternate approaches to devise and implement balanced mix design procedures incorporating performance testing and criteria. The project is expected to begin in spring 2017.

Also this spring, NCAT engineers began offering a Balanced Mix Design Course consisting of interactive lectures, demonstrations, and hands-on training with performance-based mix design procedures. This course is specifically designed to provide asphalt industry and agency personnel with a better understanding of the BMD concept based on results of laboratory tests that reflect how the mix will perform in regard to rutting resistance, resistance to cracking, and long-term durability.

# Is Your Gsb Correct?

Aggregate bulk specific gravity may be one of the least appreciated factors affecting the performance of asphalt pavements. Although high quality mixtures can be made with aggregates having a wide range in bulk specific gravity values, precisely determining the bulk specific gravity (Gsb) is critical to setting the optimum asphalt content of every mix design. Volumetric mix design criteria for air void content (Va) and voids in the mineral aggregate (VMA) set the minimum volume of effective binder (Vbe) for a mixture:

$$Vbe = VMA - Va$$

However, calculation of a mixture's correct VMA is highly dependent on an accurate determination of the Gsb. An error in Gsb can result in too much or too little asphalt in the mix design and ultimately lead to a variety of asphalt pavement performance problems such as rutting, raveling, bleeding, and cracking.

Unfortunately, there are considerable issues of accuracy and variability associated with Gsb determinations. A relatively small error in the blend's Gsb has a significant effect. As illustrated below, a difference in Gsb of 0.030 (a difference that is well within the repeatability of Gsb results) can change the VMA by 1.0% for a mixture, easily changing a failing mix design into a passing mix design.

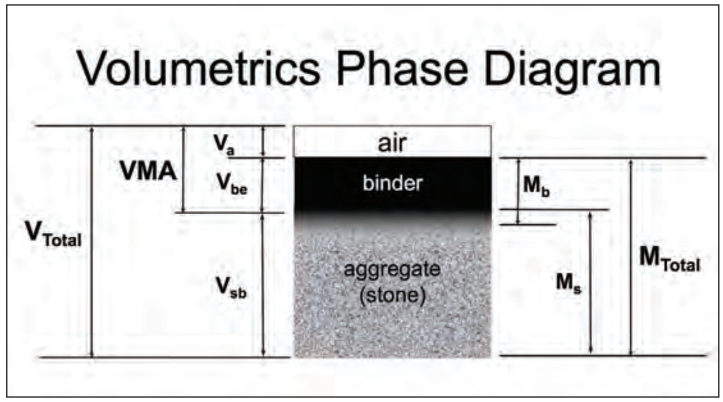
Given:

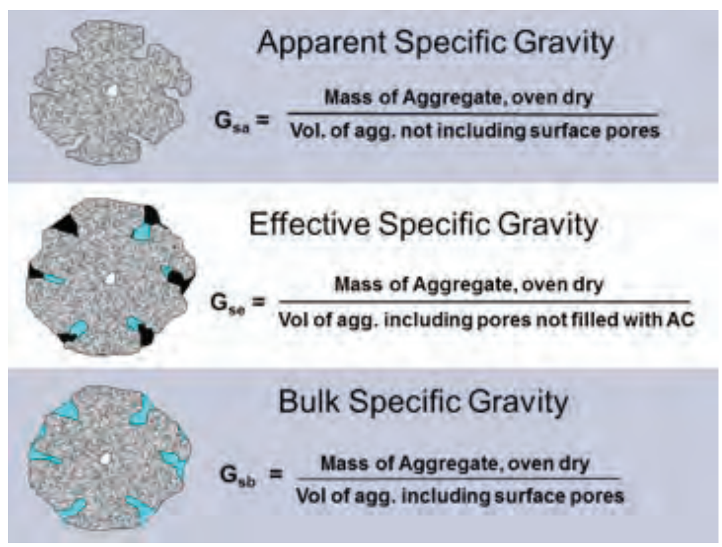
| | |
|---|---|
| Gsb = 2.640 | Gsb = 2.670 |
| Pb = 5.2% | Pb = 5.2% |
| Gmm = 2.531 | Gmm = 2.531 |
| Gmb = 2.431 | Gmb = 2.431 |

Calculated:

| | |
|---|---|
| Air Voids = 4.0% | Air Voids = 4.0% |
| VMA = 12.7% | VMA = 13.7% |
| VFA = 69% | VFA = 71% |

A fundamental check that should be part of every mix design is making sure that the relationships Gsa > Gse > Gsb are correct. The apparent specific gravity, Gsa, of each aggregate component is determined from the same measurements used to determine Gsb, so no extra effort is involved. Gsa of the aggregate blend is calculated in the same way as Gsb for the blend. Gse is determined independently, calculated from the mixture's maximum theoretical specific gravity, asphalt content, and the specific gravity of the asphalt binder. Ensuring that these three aggregate specific gravity values are in the right order is a reality check on the testing. If Gsb > Gse, then it would mean that the asphalt absportion is negative; if Gse > Gsa, then it would mean that the aggregate absorbs more asphalt than water – both non-sensical results.




One of the most important policies an agency makes is determining who sets the Gsb for the aggregates used in mix designs. There are three basic approaches:

1. Gsb set by the aggregate suppliers,
2. Gsb set by the mix designers, and
3. Gsb set by the agency.

Each of these approaches has advantages and disadvantages. Allowing mix designers to set Gsb values is probably the most common and the most efficient approach, but a process must be established to independently verify the Gsb values. Even then, given the wide range of allowable differences in Gsb values between two labs, there are opportunities to abuse the system. When the aggregate suppliers set the Gsb, they provide that third-party independence, except for those cases where the aggregate supplier and the asphalt producer are the same company. Aggregate supplier set values also establishes Gsb consistency for each aggregate product. For example, multiple contractors using the same aggregate material from a source have to use the same value in their mix designs. Having the highway agency set the Gsb values for every aggregate product is the most fair and reliable approach but puts a tremendous burden on the agency to test so many products. The frequency of checking Gsb should be based on historical data for how much Gsb values change over time.

Gsb for reclaimed asphalt pavement (RAP) aggregate should be based on a method proven to provide the most suitable values for the aggregates used in the state. Gse should not be used for RAP. Doing so will inflate the calculated Gsb for the aggregate blend and make the VMA appear higher than it actually is. For mixtures containing RAP, more caution should be used in the approach of determining optimum asphalt content. NCAT can provide an *Experimental Guide for Identifying the Best Method for Determining the RAP Aggregate Bulk Specific Gravity for a State or Region.*

Another Gsb issue to consider is dealing with natural variations that occur in aggregate materials over time. Some aggregate sources may have very consistent Gsb values over decades, whereas others have significant variations within a single year due to the site's geology and mining operations. In cases where mix designs are valid for a year or two and the Gsb changes by more than 0.020 over that time, then there is a risk that the VMA calculated with the original Gsb values is not valid. Since some agencies use VMA as part of acceptance testing, there is a strong chance of accepting a poor mix or rejecting a good quality mix due to the unknown changes in Gsb. Regularly checking Gse during mix production and comparing the values to the Gse from the mix design is a good way to catch variations that occur in the natural materials.

# Asphalt Film Thickness Debunked

## Background

Asphalt film thickness is often mentioned in the literature on asphalt paving technology. A few highway agencies have used minimum asphalt film thickness as a mix design and quality assurance criteria. Although the concept of asphalt film thickness sort of makes sense, the reality is a different story.

The concept of asphalt film thickness was originally proposed by Francis Hveem to estimate a starting point of asphalt content for mix designs. It is defined as a ratio of the effective asphalt volume to the surface area of aggregate, as shown in the following equation, where $T_F$ is the average film thickness (unit: μm), $V_{asp}$ is the volume of effective asphalt binder (unit: L), *SA* is the aggregate surface area (unit: m2/kg), and *W* is the aggregate weight (unit: kg).

$$T_F = \frac{V_{asp}}{SA \times W} \times 1000$$

There are a number of questionable aspects about the concept. One debated aspect is the assumption that each aggregate particle is covered with a uniform thickness of asphalt. This notion has been questioned for decades. In a compacted mixture, the coating may be very thin at the points of contact between aggregate particles, but where the aggregates are close together the asphalt film is shared. Fine aggregate particles may have a much thicker coating than coarse aggregate particles. In fact, extremely fine parts of the mineral filler might simply be embedded in the asphalt coating.

## Calculation of Asphalt Film Thickness

There are two critical steps involved in calculating asphalt film thickness: determination of effective volume of asphalt binder and calculation of the total surface area of aggregate. Hveem used a set of surface area factors to relate an aggregate gradation to a total surface area of aggregate. Table 1 lists these surface area factors that are commonly multiplied by the percent passing of the gradation.

These surface area factors were based on the assumption that all of the particles are spherical, the representative particle size for aggregates passing sieve No. 200 is 0.03 mm, and the bulk specific gravity of the aggregate is 2.650. The first assumption is an obvious flaw. Secondly, it is not appropriate to specify a single representative particle size (i.e., 0.03 mm) for all kinds

*Table 1: Commonly Used Asphalt Mixture Performance Tests*

| Sieve Size (mm) | Surface Area Factor, $m^2/kg$ ($ft^2/lb$) |
|---|---|
| Maximum aggregate size | 0.41 (2) |
| 4.75 | 0.41 (2) |
| 2.36 | 0.82 (4) |
| 1.18 | 1.64 (8) |
| 0.60 | 2.87 (14) |
| 0.30 | 6.14 (30) |
| 0.15 | 12.29 (60) |
| 0.075 | 32.77 (160) |

of aggregate passing the No. 200 sieve. Figure 1 shows the particle size distributions of the baghouse fines from various plants. As illustrated, particles smaller than 0.075 mm have a wide size distribution. Using one representative size for fines passing the No. 200 sieve grossly misrepresents the surface area contribution for most materials.

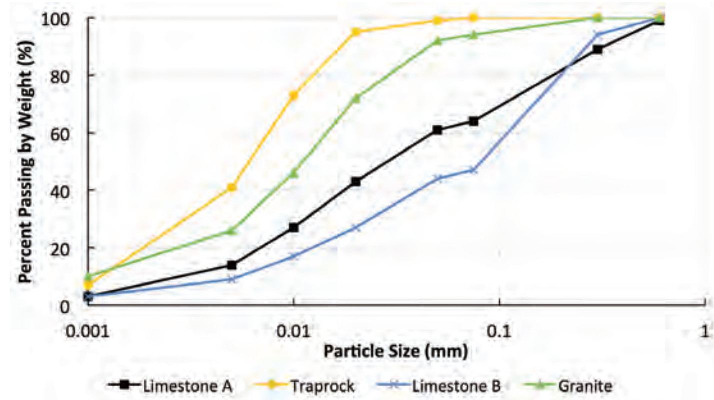



*Figure 1: Particle Size Distributions of Baghouse Fines from Various Plants as documented by Anderson and Tarris*

Thirdly, many aggregates used in asphalt mixes have bulk specific gravity values different from 2.65. Even within a given aggregate blend, coarse and fine aggregates typically have different Gsb values, so the surface area factors require different Gsb adjustments for the coarse and fine portions.

## Measured Surface Areas of Aggregate Fractions

Several decades ago, a technique was developed to actually measure the surface areas of particulate materials. The Brunauer-Emmett-Teller (BET) gas adsorption technique has been widely used to measure the specific surface area (SSA) of a range of materials ranging from soils to nano materials. Recently, this technique has been applied to measure the SSA of aggregates in order to determine their surface free energy. Table 2 presents the results of the measured SSA of aggregates of individual sizes from several studies. It can be seen for each aggregate size range that measured SSA results have a very wide range. For example, the surface area of the basalt aggregates between 4.75 mm and 2.36 mm (i.e., 7.06 m2/g) is nearly 90 times larger than that of the granite aggregates (i.e., 0.08 m2/g).

*Table 2: Measured Specific Surface Area of Aggregates*

| Mineral Type | Specific Surface Area ($m^2/g$) | Aggregate Size | References |
|---|---|---|---|
| Granite | 0.67 | Between 4.75 mm and 2.36 mm | Lytton et al., 2005 |
| Quartzite | 1.35 | | |
| Light Sandstone | 0.83 | | |
| Dark Stone | 1.00 | | |
| Gravel | 0.80 | | |
| Limestone Screenings | 0.49 | | |
| Limestone | 0.53 | | |
| Gravel | 4.76 | | |
| Limestone | 0.21 | | Bhasin and Little, 2007 |
| Gravel | 1.28 | | |
| Basalt | 7.06 | | |
| Granite | 0.08 | | |
| Sandstone | 0.97 | | Cheng, 2002 |
| Georgia Granite 1 | 0.10 | | |
| Georgia Granite 2 | 0.11 | | |
| Texas Limestone 1 | 0.44 | | |
| Texas Limestone 2 | 0.43 | | |
| Colorado Limestone 1 | 0.31 | | |
| Colorado Limestone 2 | 0.26 | | Howson et al., 2009 |
| Brownwood Limestone | 1.01 | | |
| Fordyce River Gravel | 0.45 | | |
| Knippa Traprock | 1.25 | | |
| Mill Creek Granite | 0.53 | | |
| MM Sandstone | 0.79 | | |
| Limestone | 0.62 | Between 2.36 mm and 1.18 mm | Apeagyei et al., 2015 |
| Basalt | 0.65 | | |
| Greywacke | 1.40 | | |
| Granite | 2.28 | | |
| Idaho River Gravel | 2.30 | Between 0.85 mm and 0.5 mm | Petersen et al., 1982 |
| FHWA Region 10 Sand | 1.92 | | |
| Montana Bench Gravel | 0.47 | | |
| Virginia Granite | 2.61 | | |
| Colorado Crushed Stone | 1.95 | | |
| Arizona River Gravel | 3.00 | | |
| Georgia Granite Gneiss | 0.10 | | |
| Granite | 4.58 | Mineral Filler | Tan and Guo, 2013 |
| Andesite | 3.28 | | |
| Limestone | 2.72 | | |

Even for the same aggregate type, the SSA results vary significantly (e.g., the SSA of gravel aggregates between 4.75 mm and 2.36 mm ranges from 0.45 m2/g to 4.76 m2/g). In addition to aggregate particle sizes, their shapes and textures have remarkable effects on SSA. Figure 2 shows the morphology difference of aggregate particles. As illustrated, the aggregate particles have significantly different shapes and textures.

*Figure 2a: Angularity of Granite Aggregates*

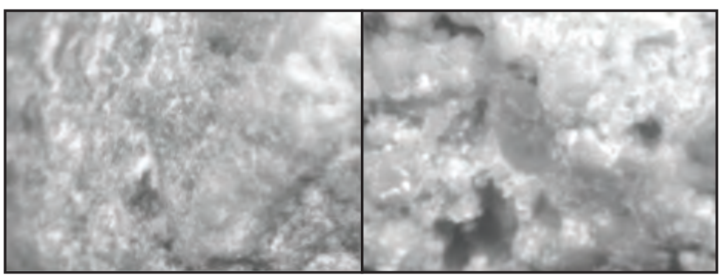


*Figure 2b: Textures of Two Granite Aggregates*

## Conclusion

Asphalt film thickness is a flawed concept primarily because the surface area factors used to estimate the specific surface area of an aggregate solely from its gradation are not valid. Measurements of specific surface areas using the gas adsorption technique have shown that the specific surface area of the particles from the same sieve size vary by nearly two orders of magnitude.

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# The Delta Tc Parameter: What Is It and How Do We Use It?

You may have heard of a new asphalt binder parameter being discussed recently for evaluating age related cracking potential. $\Delta T_c$ (pronounced *delta tea see*) is defined as the numerical difference between the low continuous grade temperature determined from the Bending Beam Rheometer (BBR) stiffness criteria (the temperature where stiffness, S, equals 300 MPa) and the low continuous grade temperature determined from the BBR m-value (the temperature where m equals 0.300). If you aren't sure how to determine low temperature continuous grades, you can find instructions in ASTM D7643, *Standard Practice for Determining the Continuous Grading Temperature and Continuous Grades for PG Graded Asphalt Binders*. Don't forget to subtract 10°C from your BBR test temperatures when determining low temperature continuous grades!

For example, let's say a set of BBR tests yielded the following results at two test temperatures:

**-18°C** : Stiffness = 243 MPa and m-value = 0.309

**-24°C** : Stiffness = 400 MPa and m-value = 0.256

Using these results, the low continuous grade temperature for the stiffness criteria ($T_{cont}$, S) equals -30.5°C and the low continuous grade temperature for the m-value criteria ($T_{cont}$, m) equals -29.0°C. Once you have the temperatures, just subtract $T_{cont}$, m from $T_{cont}$, S to get the value of $\Delta T_c$.

For this example:

$\Delta T_c$ = -30.5 - (-29.0) = -1.5.

It's that simple! The $\Delta T_c$ parameter can be measured on any asphalt binder, whether it's a virgin asphalt binder or binder that's been extracted and recovered from a sample of asphalt mixture.

Now that you know what $\Delta T_c$ is and how it's calculated, let's discuss how it relates to asphalt pavement performance and what it shows us that we can't get from BBR stiffness and m-value results alone. $\Delta T_c$ was first proposed by Asphalt Institute engineer Mike Anderson in 2011 to measure the ductility loss of aged asphalt binder as part of a study examining relationships between asphalt binder properties and non-load related cracking. In particular, the study focused on finding a parameter to explain block cracking in airport pavements.

Block cracking is a non-load related cracking phenomenon similar to thermal cracking that causes cracks to develop in both longitudinal and transverse directions. This results in a square or "block" pattern. Block cracking is most often seen in significantly aged pavements with low traffic volumes. The lack of traffic allows the asphalt binder to develop a type of internal structure (thixotropic hardening) that will exhibit brittle behavior when exposed to thermal stresses. Although it's similar to thermal cracking, studies have shown that block cracking may be more dependent on the age of the asphalt pavement than on the environmental conditions. In other words, an older pavement that does not experience environmental conditions that would cause thermal cracking may still experience block cracking.

Ductility is defined as the ability of a material to be stretched without breaking. This is important in flexible pavements because the thin films of asphalt binder between the aggregate particles must have a certain amount of ductility to withstand stresses in the pavement due to traffic or temperature changes. Prior to the Superpave Performance Grading (PG) system,

*Figure 1: Block Cracking on the NCAT Test Track*

ductility was used as a surrogate relaxation parameter for asphalt binders and was considered a way to distinguish cracking performance. In general, research has shown that ductility may be an important factor in cracking performance as asphalt pavements age. In particular, pavements with low ductility binders tend to exhibit poor cracking performance, even when the overall asphalt binder stiffness values is similar to that of pavements that perform well. These observations imply that asphalt binder stiffness and relaxation may not change at the same rate due to aging and that the loss in relaxation (or ductility) may have a more significant effect on cracking performance than the increase in stiffness.

Our current Superpave PG system does not include a direct measure of ductility. Instead, it relies on relaxation parameters such as the phase angle measured by the Dynamic Shear Rheometer (DSR) at intermediate temperatures and the BBR m-value at low temperatures to predict cracking performance. While these parameters are suitable for determining the relaxation properties needed for other types of cracking, they do not provide a relationship between stiffness and ductility that may be needed to control block cracking.

Other studies have shown that as some asphalt binders age, their low temperature relaxation properties, as measured by the BBR m-value, deteriorate significantly faster than their low temperature stiffness increases. This leads us back to the possible use of $\Delta T_c$. The next version of AASHTO PP78 *Design Considerations When Using Reclaimed Asphalt Shingles (RAS) in Asphalt Mixtures*, to be released later this year, includes a criteria for $\Delta T_c$ as part of the mix design process. Extracted and recovered binders from mixtures containing RAS are to be aged in a pressure aging vessel (PAV) for 40 hours before BBR testing and determining $\Delta T_c$. The minimum $\Delta T_c$ criteria is -5.0°C, meaning that a recovered and aged binder with a $\Delta T_c$ of -6.0°C is not acceptable. The revised PP 78 contains notes that allow the $\Delta T_c$ criteria to be adjusted based on local experience or the use of a mixture cracking test implemented by the agency in lieu of the criteria for $\Delta T_c$.

In summary, the $\Delta T_c$ parameter has been proposed as a relatively simple method for measuring the loss of relaxation properties of asphalt binders. Although data on this parameter as an indicator of binder brittleness continues to be collected, some agencies may begin to use it as a criteria for RAS mixes and are considering expanding its use to evaluate the impact of REOB and the effectiveness of rejuvenators and soft asphalts with mixtures containing RAP.

# Where Are They Now?

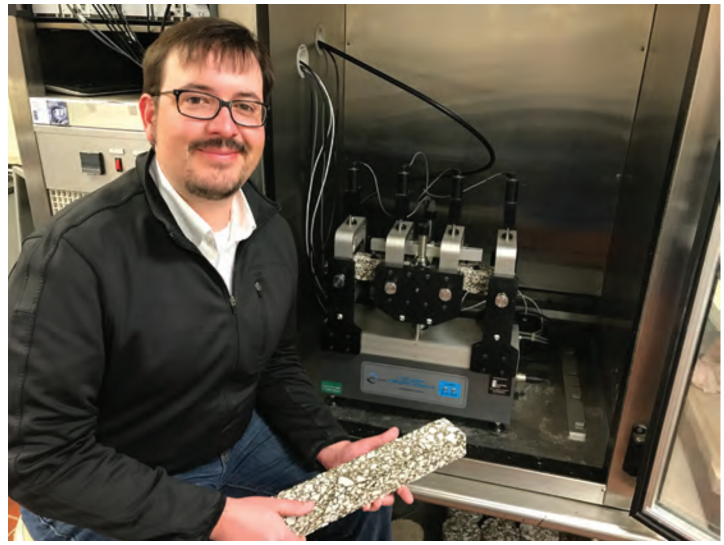
NCAT opened its doors three decades ago with a mission to provide innovative, relevant and implementable research, technology development and education that advances safe, durable and sustainable asphalt pavements. Along the way, more than 100 graduate students and post-doctoral researchers have been involved in NCAT research. Three such individuals have attributed much of their career success to their time spent at NCAT: Nick Murphy, James Winford, Jr., and Jason Wielinski.


## Nick Murphy

Nick Murphy began his NCAT work as an undergraduate lab technician. Upon graduating, he was offered a graduate research assistant position under Dr. Ray Brown, Director of NCAT at that time. Mr. Murphy's research was devoted to the early development of what would become the ignition method for determining the asphalt content of asphalt mixtures, standardized as ASTM D6307 and AASHTO T 308. In 1994, he received a Master of Science from Auburn University. As a Professional Engineer, he is currently the senior vice president of E.R. Snell Contractor, Inc. in Snellville, Georgia. His company includes 11 asphalt plants, an asphalt cement terminal, and a quality control lab. The company also performs grading, paving, and bridge installation. He and his wife Jennifer have three children, Taylor Kate, Cole, and Logan, ages 13, 11, and 8 respectively. He attributes his knowledge of materials to his time at NCAT and considers it to be the foundation of the career. Nick is grateful for the opportunities that his NCAT experience has afforded him and is thankful that Mr. Robert Brown, the lab manager at the time, and Dr. Ray Brown saw his potential.

## James Winford, Jr.

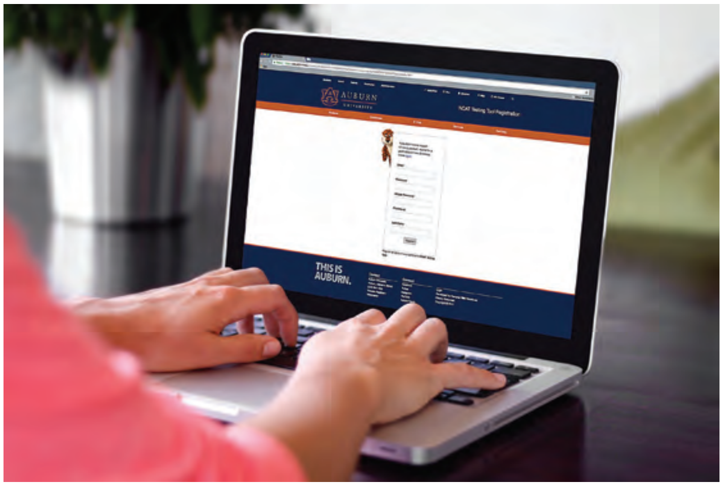
Dr. James (Jay) Winford is a Professional Engineer who comes from a family of asphalt enthusiasts, with his grandfather and father both being civil engineers. In the late 1980's, he taught undergraduate materials classes while he worked on his PhD and conducted research on the influence of aggregate shapes and texture on permanent deformation of asphalt mixtures. He completed his doctorate in 1991 from Auburn University under the mentorship of Dr. Freddy Roberts, Dr. Ray Brown, and Mr. Ken Kandhal. Currently, Jay is the president and co-owner of Prairie Contractors in Louisiana. They operate three fixed base asphalt plants and have grown the company to include aggregate distribution, among other things. He and his wife Laura have two children, James and Katherine. While his children have grown and moved away, he and his wife reside in Opelousas, Louisiana where his company is headquartered. They also love spending time at their second home in New Orleans. Jay continued to contribute to NCAT over the years as past chairs of the Applications Steering Committee and Board of Directors, and he still serves as an Emeritus Member on the NCAT Board. Jay attributes much of his career growth to surrounding himself with peers in the asphalt industry who have challenged him along the way. He hopes to spend his retirement continuing to teach materials classes in civil engineering just as he did over 15 years ago while at NCAT.



## Jason Wielinski

In 2005, Jason began his research at NCAT investigating ways to improve the stability of permeable asphalt base courses during construction. He graduated with a Master of Science from Auburn University in 2007. He and Alicia, his wife of eight years, now reside in Brownsburg, Indiana with their two children. His oldest child, Ava, is five years old and Alexander just turned four. Jason is a Professional Engineer and currently works as an asphalt research engineer at Heritage Research Group. He and his team have spent most of their time researching, designing, and constructing in-place recycling projects across the Midwest. The team has a passion for full depth reclamation, cold in-place recycling, and cold central plant recycling with asphalt emulsion. He has also served as the technical committee chair on the Asphalt Recycling and Reclamation Association Cold Recycling Subcommittee. His passion for the asphalt industry and its research began while he was a student at NCAT. Jason learned much from his peers in the lab and is especially thankful to Dr. David Timm for investing his time in him. Jason strives to emulate Dr. Timm's professionalism, patience, and coaching style by mentoring the engineers and technicians on his team at Heritage Research Group.

# Using the NCAT Online Asphalt Mix Design Program




The Superpave mix design procedure has been adopted by transportation agencies across the United States for designing dense-graded asphalt mixtures. Many highway agencies have developed customized mix design spreadsheets to use in designing and approving asphalt mix designs.

With advancements in computer programing languages, NCAT engineers have developed the NCAT Testing Tool, a web-based Superpave mix design program. The tool is freely accessible on the Internet, which makes it easy to store data online. It can be updated with changes in AASHTO specifications, providing users immediate access to the latest version without downloading and re-installing software. Data saved by a user, such as stockpile data, is securely stored on the Auburn server so that only the single user ID can access it.

The program can be accessed at: https://cwstest.auburn.edu/ncat. Users without an Auburn University account will be able to register for a guest account on the main page.

After logging in, users can create a new mix design, share, remove, or copy an existing mix design, or download the user manual by clicking on the corresponding buttons. Users cannot share, remove, or copy a mix design until one has been created.

An asphalt mix design can be created using the following 10 steps.

**Step 1: Setup.** The user inputs basic mix design information, binder information, consensus and specific gravity test results, and the number of trial aggregate blends (at least one) to be evaluated.

**Step 2: Aggregate Gradations and Consensus Properties.** The user inputs the aggregate gradations and consensus properties.

**Step 3: Gmm & Gmb Results for Trial Blend(s).** The user inputs the maximum theoretical specific gravity (Gmm) and bulk specific gravity (Gmb) for each trial blend.

**Step 4: Volumetric Results for Trial Blend(s).** Once the required information is entered, the volumetric properties for trial blends are calculated and the optimum binder contents are estimated. The user can review the volumetric properties of each trial aggregate blend and select the blend that best meets the mix design requirements. The user also selects an estimated optimum binder content for further evaluation.

**Step 5: Selected Design Aggregate Structure.** The design aggregate gradation and its combined aggregate properties are summarized.

**Step 6: Gmm & Gmb Results for Three Asphalt Contents.** The user enters Gmm and Gmb test results and sample heights at $N_{ini}$ and $N_{des}$ for three trial asphalt contents (optimum and optimum ± 0.5%).

**Step 7: Volumetric Results for Three Asphalt Contents.** The volumetric properties are automatically calculated and plotted for the three trial asphalt contents. The detailed volumetric properties are also included on this page. Based on the volumetric results, the user can select the optimum asphalt content and determine other volumetric properties for the design aggregate blend.

**Step 8: Gmm & Gmb Results at Optimum Asphalt Content Compacted to Nmax (Optional).** If testing is also conducted on samples mixed at the optimum asphalt content and compacted to the Nmax, the Gmm and Gmb test results as well as specimen heights at $N_{ini}$, $N_{des}$, and $N_{max}$ can be entered on this page. This step is not required in some state agencies' specifications.

**Step 9: Moisture Susceptibility (TSR) Inputs.** The user enters tensile strength ratio (TSR) test data, which are then used to calculate the indirect tensile strengths and tensile strength ratio for reporting moisture susceptibility.

**Step 10: Job Mix Formula Report.** All of the mix design information and test results are summarized on this page. This includes general information, design gradation, optimum binder content, and other volumetric properties for the final mix design.

# Specification Corner

**Colorado DOT**
We have developed Project Special Provisions for cape seal and double chip seal as statewide specifications options for our low volume roads. We have also developed a polyphosphoric acid (PPA) Pilot Project Special Provision to be used at the discretion of the Regional Materials Engineer.

**Florida DOT**
Effective with projects bid starting January 2017, PG 76-22 (PMA) and PG 76-22 (ARB) are considered equivalents, and it is the contractor's choice as to which binder to use. Both binders must meet the same testing requirements with the addition of the separation test for the ARB binder. Effective July 2017, PG 82-22 (PMA) will be replaced by PG 76-22 (HP) binder, often referred to as HiMa binder. This will be our premium binder and will be used in locations with the potential for severe rutting or cracking. We will now refer to the binder as "HP" binder.

We are now allowing cellulose fibers to be used in OGFC mixtures with the same binder content as required for mineral fibers. Previously, the OGFC designs with cellulose fibers had higher AC contents than those designs with mineral fibers. Cantabro testing confirmed that the mixtures are equally as durable with the same AC content and different fiber types. Contractors using cellulose fibers are experiencing better laydown without any draindown, cleaner trucks, and better conditions for asphalt plant workers who handle the fibers.

**Minnesota DOT**
Effective 2017, all projects are using the MSCR specification for binders.

**Montana DOT**
We have moved to limits on binder replacement rather than limits on RAP as a percent by weight of mix. We are also using the Hamburg wheel tracking test more often for field acceptance when there are volumetric issues. For example, if a sample fails volumetric test criteria, Hamburg testing is performed to determine if the material should be left in place. We have also instituted a longitudinal joint specification.

**Ohio DOT**
We are looking to fine up the gradations of our 19 mm mixtures to help resolve segregation issues and to add more virgin PG binder. Numerous lab mixes were made by ODOT and contractors to determine minimum virgin and total AC %. We are also planning on revising our micro-surfacing specification to increase contractor QC testing and controls and to update to the current ISSA and AASHTO requirements.

**South Carolina DOT**
We have revised our SC-M-402 specification to get back to a finer surface mixture for use beneath our OGFC. We now specify a 9.5mm mix with 75 gyrations versus the 12.5mm mix with 100 gyrations used previously. We are also getting more binder to be added to RAP/RAS mixtures by using a 75% availability ratio for the percentage of aged binder (similar to Georgia DOT).

**Tennessee DOT**
We now allow 10% RAP regardless of processing/fractionation for all of our AS-mix (open-graded base mix) as long as the final gradation meets the master gradation range. This had previously ranged from 0 -15% depending on processing/fractionation and had restricted RAP particle size. We also require all AS-mix to be covered prior to winter or refrain from paving with AS-mix until spring.

The minimum placement temperature for OGFC mixes has been lowered to 280° F.

We now require the use of 75% skid-resistant aggregate in our 'scratch' mixes when used as a riding surface.

Our MSCR specification has been modified as follows:

1. % Difference (Jnr 3.2 - Jnr 0.1) is not applicable for PG 76-22 and will be waived for PG 70-22 if the Jnr (3.2) is less than 0.5.
2. Minimum % Recovery for PG 70-22 is now a flat 29%. The recovery curve in AASHTO M332 Appendix X1 still applies to PG 76-22 and PG 82-22.

# Asphalt Forum

NCAT invites your comments and questions, which may be submitted to Christine Hall at christine@auburn.edu. Questions and responses are published with editing for consistency and space limitations.

**Greg Sholar, Florida DOT**
Are any other agencies having difficulties with trackless tack materials?

**Jerry Geib, Minnesota DOT**
In 2017, as part of the NRRA pooled fund, we will build different overlay combinations on the old PCC of I-94 (MnROAD bypass). These will be the first HMA overlays placed on the old PCC.

# Asphalt Forum Responses

The following responses have been received to questions shared in the previous issue.

***For states that require fractionated RAP, at what RAP percentage do you start to require fractionation, and how are the sizes handled? Have you noticed if this has helped eliminate segregation due to too much RAP? -Eric Biehl, Ohio DOT***

**Michael Stanford, Colorado DOT**
We do not require fractionated RAP.

**Jerry Geib, Minnesota DOT**
We do not require fractionated RAP.

**Cliff Selkinghaus, South Carolina DOT**
We allow up to 10% more aged binder when the contractors choose to fractionate. We require a RAP/RAS QC plan by contractors to show how they are fractionating the recycled materials. We require a high frequency fractionating device and frequency of testing, and our specification is SC-M-407. This has not completely eliminated segregation; it also depends on stockpile management and the time between crushing, screening, and actually feeding the materials into the plant.

I would also like to point out an issue that should be looked at on all milling jobs. Be sure the milling contractor is aware that the material is not a waste product; it is recycled material, and the road should be broomed prior to milling to prevent pick up of unwanted debris and trash.

**Matthew Chandler, Tennessee DOT**
Binder layers: 30%-35%, maximum 3/4”; Scratch layers: 15%-25%, maximum 5/16”; Unmodified surface layers: 15%-20%, maximum 1/2” (5/16” for thinlifts); Modified surface layers: 10%-15%, maximum 1/2” (5/16” for thinlifts); Shoulders: 30%-35%, maximum 1/2”.

The contractor may use the higher percentage of fractionated RAP specified only if individual fractions of two different maximum particles size are introduced into the plant as separate material sources for increased control.

***How many states are using WMA as a tool to assist with placement issues that sometimes occur when paving over pavements that have previously been crack sealed? Do you have a minimum curing time in your specifications for paving over crack sealants? Do you use any other sealants besides ASTM D6690 Type 1 material to seal cracks? -Cliff Selkinghaus, South Carolina DOT***

**Michael Stanford, Colorado DOT**
We have specified WMA on two projects to assist with this issue. We do not have a minimum curing time, and we use ASTM D6690 Type II and Type IV materials. Our approved products list is online at:
http://apps.coloradodot.info/apl/SearchRpt.cfm?cid=Sealant+%5BJoint+and+Crack%5D

**Eric Biehl, Ohio DOT**
We are not currently using WMA in this type of situation. We are mainly a water-injection foamed WMA state but are looking at using WMA in this application and possibly others including thin, highly modified overlays. We do not have a minimum curing time, but the longer it can cure, the better. We have four types of material under ODOT Construction and Materials Specification 702.17: ODOT Type 1 is the same as ASTM D6690 Type 2. ODOT Type 2 and 3 use PG 64-22 with either polyester or polypropylene fibers. ODOT Type 4 uses a modified binder (similar to D6690 Type 2) with polyester fibers.

**Matthew Chandler, Tennessee DOT**
We permissively allow the use of WMA but do not call for it in this situation. Generally, we recommend to the districts to crack seal about a year before resurfacing. We use ASTM D6690 type II.

**Jerry Geib, Minnesota DOT**
We have used WMA for this situation. We have three types of sealants. Type I, II and IV, all of which are modifications of the ASTM D6690 specification. We do not have a minimum curing time.

***What experience do other states have with adhesives for longitudinal joints during construction? If successful, was the application pre-applied, post-applied, or both? -Matthew Chandler, Tennessee DOT***

**Michael Stanford, Colorado DOT**
We do not have any experience with this.

**Eric Biehl, Ohio DOT**
We have a Supplemental Specification 875 for the joint adhesive material and placement. In the past few years, we have required contractors to use a certified PG binder or joint adhesive that's pre-applied to the cold vertical face prior to the hot mat being placed for our longitudinal joints. We've been successful overall, with one of our twelve districts using this method for a decade or more. The only issue we've had is over application and pooling in the corner of the existing surface and vertical face.

**Jerry Geib, Minnesota DOT**
We use Crafco joint adhesive during the construction of the joint.

***Have any states successfully implemented a post-milling sweeping/vacuum performance specification? -Matthew Chandler, Tennessee DOT***

**Michael Stanford, Colorado DOT**
Yes, our specification language is: All milled surfaces shall be broomed with a pick-up broom, unless otherwise specified, before being opened to traffic. A sufficient number of brooms shall be used immediately after planing to remove all milled material remaining in the roadway. If the Contractor fails to adequately clean the roadway, work shall cease until the Engineer has approved the Contractor's revised written proposal to adequately clean the roadway.

**Eric Biehl, Ohio DOT**
We require the surface to be clean after milling and prior to tacking and placing the new mix. Most use sweepers.

**Jerry Geib, Minnesota DOT**
We have not implemented any specifications for post-milling sweeping or vacuuming.

***Have any states had trouble with their AC suppliers meeting the % recovery curve for a single grade bumped binder (i.e. H grades under MSCR)? -Matthew Chandler, Tennessee DOT***

**Michael Stanford, Colorado DOT**
We do not perform MSCR testing.

**Jason Davis, Louisiana DOTD**
Louisiana has had trouble with our PG 70-22m (basically an "H" grade) meeting the MSCR curve. Our research group is conducting a study to determine if the materials not meeting the curve are providing adequate mix performance. At least one contractor has reported positive results on mix tests by limiting the lower and upper Jnr values on the base asphalt. Part of the research may be to determine the Jnr limits to place on the base grade, particularly with plant-blended materials (i.e. latex-modified). The crude stock may have significant influence on the potential for modification.

**Eric Biehl, Ohio DOT**
We do not use MSCR and still follow ODOT's version of AASHTO T 301. We also still follow AASHTO M320 PG grading. If we were to use MSCR to replace elastic recovery, we would not follow the curve, but would set a minimum value for each PG binder grade. We have concerns with repeatability of the MSCR test.

National Center for Asphalt Technology
NCAT
at AUBURN UNIVERSITY
1161 W. Samford Ave., Building 8
Auburn, AL 36830-9989

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**National Center for Asphalt Technology**
277 Technology Parkway, Auburn, AL 36830-5354
ncat.us | Phone: 334.844.6228 | Fax: 334.844.6248

Randy West, Director, NCAT
Christine Hall, Editor, Writer, NCAT
Ann Moore, Writer, NCAT

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NCAT Engineer Pamela Turner (left photo) and NCAT Lab Technician Vickie Adams (right photo) are shown teaching a Superpave binder training and certification course in Qatar.