


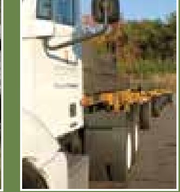

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National Center for Asphalt Technology
NCAT
at AUBURN UNIVERSITY

Enhanced Compaction to Improve Durability and Extend Pavement Service Life

As stated by R. Bruce Noel at the 1977 annual meeting of the Association of Asphalt Paving Technologists (AAPT), "The single most important construction control that will provide for long-term serviceability is compaction." When compaction is enhanced, in-place density can be optimized, thereby improving durability and extending pavement service life.

A review of several past studies clearly shows the effect of reduced air voids on improved fatigue and rutting performance of asphalt mixtures, both in the laboratory and field. Depending on mix type and experiment (which included constant-stress bending beam fatigue testing and field results from WesTrack), fatigue performance improved between 8.2 and 43.8% with a 1% decrease in air voids. Likewise, based on rutting results from WesTrack and laboratory flow number testing, rutting resistance improved by 7.3 to 66.3% with a 1% decrease in air voids depending on mix type and analysis. The results of a recent study conducted for the New Jersey Department of Transportation, which included data from 55 pavement sections, also suggest that a 1% decrease in in-place air voids increases asphalt mixture service life by conservatively 10%.

For illustration purposes, consider the effect of increasing the in-place density of an asphalt overlay from 92 to 93%. The service life will increase by conservatively 10%, meaning that the overlay will be expected to last 20 years instead of 18. Figure 1 shows a life cycle cost analysis (LCCA) of the two alternatives with an analysis period of 20 years.

For alternative 1, the initial construction cost of the overlay compacted to 93% density is assumed to be $1,000,000. The cost of providing a 1% increase in in-place density is assumed to be negligible, so the same initial cost is used for both alternatives. At year 20, the overlay in alternative 1 will be replaced, thus, the salvage value is 0 and the net present value (NPV) is $1,000,000. For alternative 2, in which the overlay is compacted to 92% density, the overlay will be replaced at year 18 at a future cost of $1,000,000. The new overlay (also expected to last 18 years) will have a remaining life of 16 years at the end of the 20-year analysis period and a prorated salvage value of $889,000. The replacement cost and salvage value is discounted to year 0 at a real discount rate of 4%, resulting in an NPV of $1,088,000 for alternative 2. Thus, on a $1,000,000 paving project, an NPV cost savings of $88,000 (or 8.8%) will be realized simply by increasing the in-place density requirement by 1%.

Because in-place density is such an important factor affecting long-term pavement performance, it is a key pay factor for acceptance in most states. The most common minimum in-place density criterion is 92 percent of theoretical maximum specific gravity (G_{mm}). Most density criteria are based on what could be achieved in the past and are not necessarily optimal in-place densities. Significant advances have been made in asphalt design and construction technology in recent years, making it possible to achieve greater densities.

Since higher in-place densities can result in significant cost savings due to extended pavement service life, highway agencies may consider increasing minimum density requirements. Higher in-place densities can be achievable by following best practices and implementing new technologies and techniques. For guidance on best practices, a recently presented webinar entitled "Best Practices in Compaction" is available for download (free for NAPA members) at store.asphaltpavement.org.

Warm-mix asphalt (WMA) technologies, including asphalt foaming technologies and chemical and organic additives, can aid in compaction. WMA improves the workability of the asphalt binder and allows for mix production at temperatures 25 to 90°F lower than HMA. A review of several regionally diverse studies comparing the quality of WMA and HMA mixtures showed that similar in-place densities can be achieved with WMA at considerably lower compaction temperatures. Using WMA can result in improved in-place densities for projects with extended haul times as well as cold-weather projects.

Intelligent compaction (IC) is another technology that can help improve in-place densities. With the IC system, vibratory rollers are equipped with GPS-based mapping along with instrumentation that provides real-time monitoring of compaction and material stiffness. This allows adjustments to be made as needed to reach optimum density and uniform coverage while eliminating unnecessary roller passes and making the process more efficient. Optional feedback controls can also make continuous adjustments to the force and frequency of the roller drum. IC is a potentially useful tool for quality control, but it is not a replacement for acceptance testing, as the relationship between IC stiffness measurements and in-place density is inconsistent.

Many premature pavement failures are caused by low density at longitudinal joints. Well-constructed longitudinal joints require very close attention to detail during the paving and compaction process. During compaction of the first lane, the roller should overhang the unconfined edge by six inches. When placing the adjacent lane, there should be an overlap of one to one-and-a-half inches, and it is crucial that there is sufficient material (about 20% additional thickness) to allow for proper compaction. This lane should be compacted from the hot side with the roller drum extending six inches over the joint. In recent years, several other techniques have been used to improve longitudinal joints, including the use of a tapered or wedge joint, which also eases the vertical transition between lanes

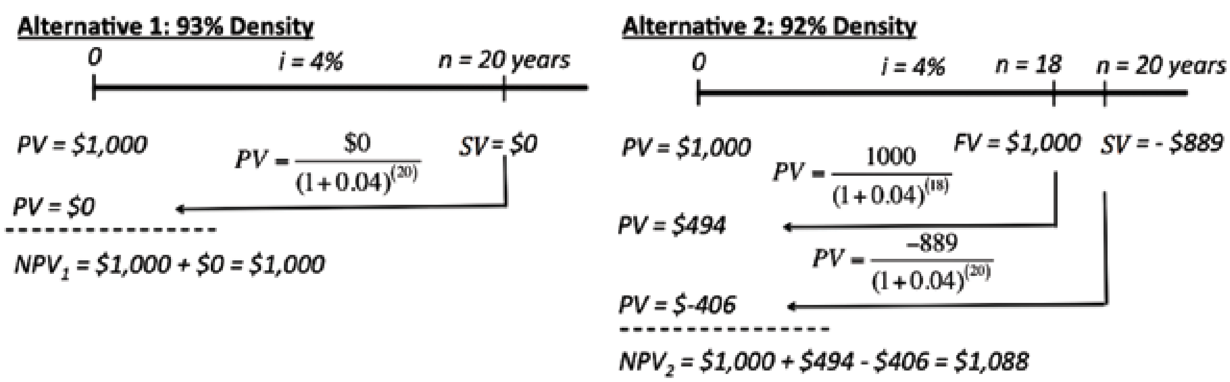


Figure 1: Life Cycle Cost Analysis (per $1,000)

Pneumatic tire rollers are used on the NCAT Test Track to achieve proper compaction and in-place density.

during construction. Also, two methods that may help bonding occur between the cold and hot edges of the joint are tack coat application on the joint and the use of infrared heaters to reheat the cold edge.

Minor increases in density are feasible and provide a cost-effective means of improving pavement service life. For increased density specifications to have maximum benefit and effectiveness, agencies should consider the following:

- *Appropriate project selection*. When choosing projects for implementing higher in-place density requirements, evaluate the underlying base. If the base is weak, optimal compaction will be difficult to achieve.
- *Lift thickness*. Increased lift thickness can improve compaction effectiveness because the mat cools more slowly, allowing more time for compaction. Also, lift thickness should be sufficient to allow for the re-orientation of aggregate particles during compaction. Fine-graded mixes should have a minimum lift thickness of three times the nominal maximum aggregate size (NMAS). For coarse-graded mixes, minimum lift thickness should be four times the NMAS.
- *Mix design*. Appropriate binder content should be selected for the desired in-place air voids. Gradation, binder content, and volumetric properties such as air voids and voids in mineral aggregate (VMA) can influence compactability of a mix and should be carefully controlled during production to minimize variability.
- *Criteria*. Appropriate measures of field compaction should be used for acceptance requirements. Comparing in-place density with theoretical maximum density allows less potential for variability in field compaction than comparison with density of laboratory samples.
- *Performance incentives*. As shown in the LCCA example, in-place density has a significant economic impact on highway agencies. Thus, performance incentives should be established to encourage enhanced compaction and achieving higher in-place density.

This literature review, which supports the FHWA Asphalt Pavement Technology Program strategic directive on extending pavement service life through enhanced compaction, will soon be available as an NCAT report.

Summary of ME Design Software Local Calibration Efforts

Many state highway agencies have considered adopting the Mechanistic-Empirical Pavement Design Guide (MEPDG) and the accompanying AASHTOWare Pavement ME Design software to replace the empirical AASHTO Pavement Design Guides and the widely used DARWin pavement design software. Based on the inputs and trial design information, the ME Design software "mechanistically" calculates pavement responses (stresses and strains) and uses those responses to compute incremental damage over time. It then utilizes the cumulative damage to "empirically" predict pavement distresses for each trial pavement structure.

Although the MEPDG was nationally calibrated using representative pavement test sites across North America, the application of the models to the full range of construction methods, materials, pavement preservation and maintenance practices, and climatic conditions is likely to significantly affect distress and performance. These local effects should be addressed through local calibration studies to adjust, if necessary, the calibration coefficients of transfer functions in the ME Design software.

Without properly conducted local calibration efforts, implementation of the MEPDG will not improve the pavement design process and may yield thicker, overdesigned asphalt pavements. Recognizing the importance of local calibration, NCAT engineers have been reviewing the general approach undertaken for state highway agencies, the results of those efforts, and recommendations for implementing the nationally or locally calibrated models.

While it is often referred to as "local calibration", the process may include local verification, calibration, and validation of the MEPDG. As a minimum, the local verification process is needed to determine if state practices, policies, and conditions significantly affect design results. In this process, the distresses predicted by the Pavement ME Design software using the nationally calibrated coefficients are compared with measured distresses for selected pavement sections. If the difference between the predicted and measured distresses is not significant, the Pavement ME Design can be adopted using the default models and coefficients; otherwise, it should be calibrated to local conditions.

If local calibration is warranted, it is important that it addresses both the potential bias and precision of each transfer function in the Pavement ME Design software. Figure 1 illustrates how the bias and precision terms can be improved during the local calibration process. In practical terms, bias is the difference between the 50% reliability predicted distress and the measured distress. Precision dictates how far the predicted values at a specified design reliability level would be from the corresponding predicted values at the 50% reliability prediction. The locally calibrated models are then validated using an independent set of data. The models are considered successfully validated to local conditions if the bias and precision statistics of the models are similar to those obtained from model calibration when applied to a new dataset.

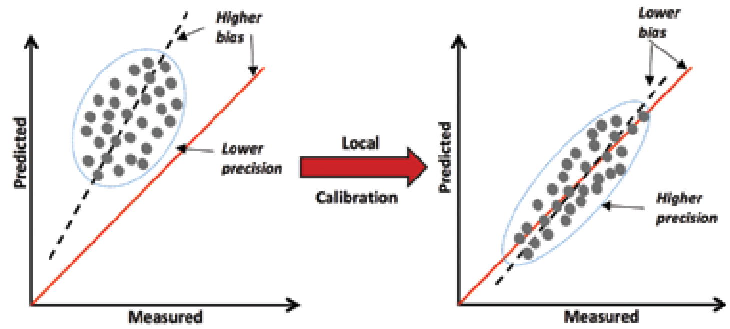


Figure 1: Improvement of Bias and Precision through Local Calibration

While the AASHTO calibration guide details a step-by-step procedure for conducting local calibration, researchers have found that the actual procedures utilized vary from agency to agency. This is partially due to the timing of the publication relative to the initiation of such efforts and the release of new versions of the software. This presents challenges for state agencies, as local calibration is a cumbersome and intensive process and the software and embedded distress models are evolving faster than local calibrations can be completed.

Researchers found that calibration was typically attempted by looking at the predicted measured distress for a set of roadway segments and reducing the error between measured and predicted values by optimizing the local calibration coefficients. However, other approaches were taken. Although a minimum number of roadway segments necessary to conduct the local calibration for each distress model is provided in the AASHTO calibration guide, the step for estimating sample size for assessing the distress models was not always reported. For those efforts that did report a sample size, some were smaller than the minimum amount recommended.

The AASHTO calibration guide recommends conducting statistical analyses to determine goodness of fit, spread of the data, and the presence of bias in the model predictions. Three hypothesis tests are recommended: 1) to assess the slope, 2) to assess the intercept of the measured versus predicted plot, and 3) a paired t-test to determine if the measured and predictions populations are statistically different. Although some local calibration efforts included all three hypothesis tests, many only evaluated some of the statistical tests and others relied only on qualitative comparisons of measured versus predicted distresses.

As such, it is difficult to assess the quality of the predictions or effectiveness of local calibration for each model. For asphalt pavements, the rutting model was the most commonly calibrated model. Transverse and longitudinal cracking models were calibrated the least, with the longitudinal cracking model having the poorest precision. Given the significant spread reported for the default longitudinal cracking model, it should not be used for design. It is anticipated that a new model will be developed under the ongoing NCHRP 1-52 project.

Twelve studies conducted for state highway agencies were reviewed. Of the twelve studies, eleven aimed to calibrate performance models to state-specific conditions and one calibrated to conditions specific to a region. The table below denotes the number of verification, calibration, and validation efforts conducted for each performance model. Additionally, the results of the local calibration efforts are summarized. General improvements in predictions were realized with local calibration, however, the degree of improvement varied. Not all of the studies evaluated the presence of bias, but for those that did the results varied by model and study.

It is recommended that the statistical analyses outlined in the Guide for the Local Calibration of the MEPDG be utilized, as they enable a quantitative assessment of the calibration results. Specifically, such parameters help to determine if local calibration has reduced bias and improved precision and if implementation is warranted. This will also help identifying any weaknesses that may exist in the model that must be considered during the design process. The MEPDG and the AASHTOWare® Pavement ME software have the potential to improve pavement design. Local calibration and validation of the performance models are essential to the implementation of this design framework, however it is an ongoing effort as models continue to be refined, and unconventional materials utilized.

Table 1: Number of Verification/Calibration Studies and Summary of Calibration Results

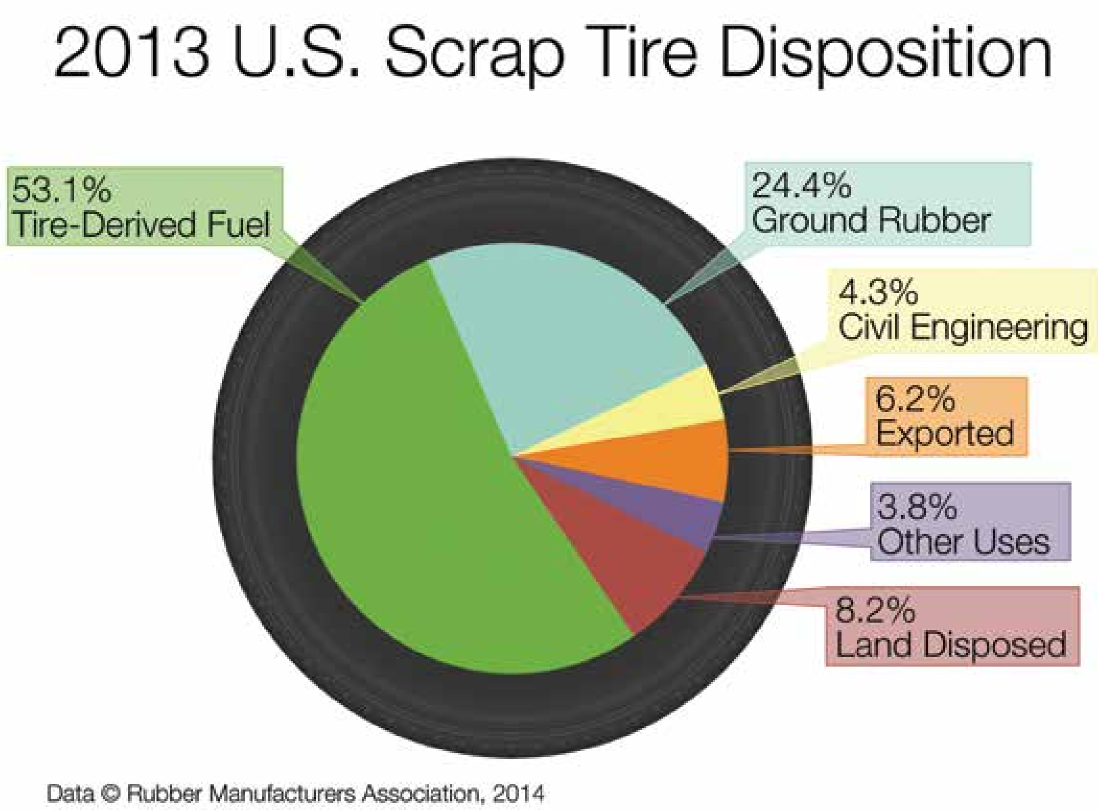
MODEL	Fatigue Cracking	Total Rutting	Transverse Cracking	IRI	Longitudinal Cracking
RESULTS	Two studies showed improvements in predictions through a higher coefficient of determination, R^2. Both R^2 values were greater than the R^2 for the national model (27.5%). Two calibration efforts resulted in a reduction in bias and standard error. One study reported only the Sum of the Squared Error (SSE), which showed an improvement in predictions with the calibrated model. Two efforts were qualitative analyses, which showed that calibration resulted in predictions closely matching measured data.	Generally, improvements in predictions were reported. Three efforts resulted in an increase in R^2, although R^2 remained low with only one greater than the R^2 for the default model (57.7%). Eight studies resulted in improvements in standard error of the estimate, S_e. Five of these were greater than 0.107, the S_e for the default model. Four efforts resulted in a reduction in bias. One effort showed bias remained despite calibration.	Two studies resulted in improvements in R^2 with both values greater than the R^2 reported for the default model. For one study, predictions were reasonable, while the other resulted in excellent predictions with slight bias. One study found the calibration resulted in an improvement in S_e and reasonable predictions, but the predictions were not an improvement over the national model.	Four efforts resulted in an improvement in R^2, all of which were greater than the R^2 for the national model. Only SSE was reported for one study, which indicated an improvement in predictions with the calibrated model.	For two studies, the predictions and bias were reportedly improved, but the S_e remained large in both calibrated models. One study only reported SSE, which was reduced with calibration, resulting in improved predictions. One qualitative analysis was conducted and resulted in reasonable predictions.
VERIFICATION ATTEMPTS	10	12	9	7	4
CALIBRATION ATTEMPTS	7	12	3	6	4
VALIDATION ATTEMPTS	1	2	0	1	1

Field Trials and Lab Tests Show Excellent Performance of GTR Modified Asphalt Mixes

Current reports estimate that approximately 233 million used tires are generated annually in the United States. Of these 233 million tires, approximately 91.8% are recycled or reused in fuel, agricultural or civil engineering markets, leaving 20 million tires that are disposed of in landfills or stockpiles. About 59.5 million scrap tires each year are being converted into ground tire rubber and used in rubber modified asphalt technologies.

The utilization of scrap tire rubber in asphalt pavements started in the mid-1960s when ground rubber was used in chip seal applications for roadway surfaces. Later on in the 1970s, ground tire rubber (GTR) modified asphalt chip seals were used as a stress-absorbing membrane interlayer. The use of GTR extended to asphalt mixtures and has continued to evolve due to its enhancement of mixture performance, including improved rutting and cracking.

In the early 1990s, a federal mandate required the use of rubber in asphalt mixtures; however, this mandate was later repealed and interest in the use of recycled tire rubber declined in most states. Some states, such as California, Arizona, Texas, and Florida, continued to use rubber modified asphalt technologies in many highway projects.



In 2008, a polymer shortage caused a significant increase in the price of polymer modified asphalt binders, and interest in rubber modified asphalt technologies was renewed. In August of 2010, the Alabama Department of Transportation (ALDOT) placed its first GTR modified asphalt mixture in over a decade outside of Dothan, Alabama on US-231 South. Blacklidge Emulsions terminally blended the GTR modified binder, which contained approximately 11% GTR and a crosslinking agent, before it was delivered to APAC Midsouth in Dothan. This binder met the requirements for a PG 76-22.

An experiment was developed by NCAT and ALDOT to compare the performance of the GTR modified asphalt mixture to that of a polymer modified asphalt mixture in both the laboratory and the field. The 19.0 mm maximum aggregate size surface mixtures contained similar aggregate structures and consisted of granite, limestone, screenings, shot gravel, sand, and 20% RAP. Due to the differences in the GTR and polymer modified binders, the asphalt contents and gradations differed slightly between the two mixtures. The GTR modified mixture had an asphalt content of 5.5%, while the polymer modified mixtures had 5.1% asphalt. The GTR modified mixture also had 0.5% more dust than the polymer modified mixture. Consequently, there were also volumetric differences between the two mixtures. The target thickness for each mixture was 1.5 inches and the target density was 93% of G_{mm}.

Mixtures and binders were sampled during construction so they could be tested at NCAT while trafficking occurred in the field. The binder was tested using the performance grade (PG) specification and the multiple stress creep recovery (MSCR) standard. The polymer modified binder had a true grade of PG 79.2 – 24.4, and the GTR modified binder had a true grade of PG

80.4 – 21.9. The *m*-value of the GTR binder did not meet the criterion for a low temperature grade of -22°C. Therefore, the binder was graded one performance grade higher than expected for the critical low temperature. Despite the properties of the GTR modified binder, results from the low temperature indirect tension creep compliance and strength tests indicated that both mixtures had critical low temperatures below -22°C.

GTR modified asphalt mixture placed outside of Dothan, Alabama on US-231 South.

It was also important to characterize the rutting and cracking resistance of both mixtures in the laboratory. The Asphalt Pavement Analyzer, Hamburg Wheel Tracking Test, and Flow Number were all used to assess the rutting susceptibility of these mixtures. In every case, the rubber modified mixture had equivalent or better performance than the polymer modified mixture.

The energy ratio (ER) method developed by the University of Florida and the semi-circular bend (SCB) test developed by the Louisiana Transportation Research Center (LTRC) were used to evaluate the potential for surface cracking in the mixtures. The energy ratio test uses two parameters to determine if the mixture is brittle ($DSCE_{HMA}$ < 0.75) or susceptible to surface cracking (ER < 1.95). Both mixtures easily exceeded the minimum $DSCE_{HMA}$ and minimum ER criteria. The rubber modified mixture had a slightly higher ER. For the SCB test, the J-integral is used to determine cracking susceptibility. The higher the J-integral, the less susceptible the mixture should be to cracking. The J-integral for the rubber mixture was 31% greater than that of the polymer modified mixture, suggesting greater resistance to surface cracking.

While laboratory test results are useful in assessing a mixture's resistance to common distresses, the ultimate proof of a technology's appropriateness is field performance. The field performance of each test section was evaluated to assess rutting, cracking, texture, and smoothness. The rut depths, texture, and smoothness of each test section were evaluated using an inertial profiler, while cracking was assessed by monitoring three 100-foot test strips in each test section.

After five years of trafficking, both test sections are free of cracking, and rutting in the sections is less than 4 mm. Additionally, both test sections were built smooth (IRI < 45 in/mile), and there has been no deterioration in pavement smoothness or change in texture.

A similar experiment also began in 2009 at the NCAT Test Track, where the Missouri Department of Transportation sponsored two test sections comparing a GTR modified mixture to a polymer modified mixture. After 10 million equivalent single axle loads (ESALs), there was no difference in the performance of these two test sections. The polymer modified test section was removed in the summer of 2012; however, the GTR test section remained in place for another 10 million ESALs. After a total of 20 million ESALs of traffic, there was minimal rutting and no cracking found in the test section. Laboratory experiments also showed equivalent performance between the two mixtures.

Given the comparable lab results and field performance for both the NCAT and Dothan test sections, which indicate equivalent performance of GTR and polymer modified mixtures, it appears that the asphalt pavement industry has several feasible options for providing the riding public with a smooth wearing surface that will perform well over time.

Evaluation of Mixes Containing REOB/VTAE with I-FIT Procedure

Re-refined engine oil bottoms (REOB), also known as vacuum tower asphalt extenders (VTAE), are a product of the engine oil recycling process. After recovered engine oils have been tested to insure the oil is free of hazardous contaminants and water is removed they enter the re-refining process. The oil is subjected to atmospheric distillation followed by vacuum distillation. The nondistillable fraction from this process is the REOB/VTAE. Although REOB/VTAE have been used to modify asphalt binders since the 1980s, concerns have recently been expressed in the asphalt paving industry due to a lack of comprehensive research. In a study sponsored by Heritage Research Group (HRG), NCAT recently performed a limited analysis on the impact of REOB/VTAE containing binders on the cracking resistance of asphalt paving mixtures.

The first phase of the study involved laboratory preparation and testing of seven asphalt mixture samples using the Illinois Flexibility Index Test (I-FIT). The I-FIT test setup used for this study and an example I-FIT data file are shown in Figure 1. The I-FIT test determines two parameters, the Fracture Energy and the Flexibility Index. The Fracture Energy is the area under the load-displacement curve and represents the energy required to fracture the asphalt specimen. The Flexibility Index is calculated by dividing the Fracture Energy by the slope of the load-displacement curve at the post-peak inflection point. In general, a steeper post-peak slope is indicative of a more brittle mix.

I-FIT samples were prepared for each mix using both short-term and long-term oven aging in accordance with AASHTO R30. The base PG (Performance Based) binder for this study was a PG 58-28. One mix contained the base binder without REOB/VTAE, and the remaining mixes contained one of two REOB/VTAE products (Source A and Source B) at dosage rates of 3%, 6%, and 9% by weight of virgin binder. All materials were formulated to meet PG 58-28 specifications and a mix design was provided by HRG using Illinois DOT materials.

A statistical analysis was performed to determine which factors (REOB/VTAE dosage, REOB/VTAE source, and aging process) had a significant impact on the two I-FIT result parameters (Fracture Energy and Flexibility Index). The analysis showed that none of the experimental factors had a significant impact on the Fracture Energy results. Although long-term oven aging and the presence of REOB/VTAE statistically reduced the Flexibility Index, increasing the REOB/VTAE dosage above 3% did not have an impact. The average and standard deviation of the I-FIT Fracture Energy and Flexibility Index for the seven test mixes are shown in Figures 2 and 3, respectively.

An additional analysis was conducted on the Flexibility Index results using a one-way ANOVA to determine the impact of the seven mixes at both the short-term

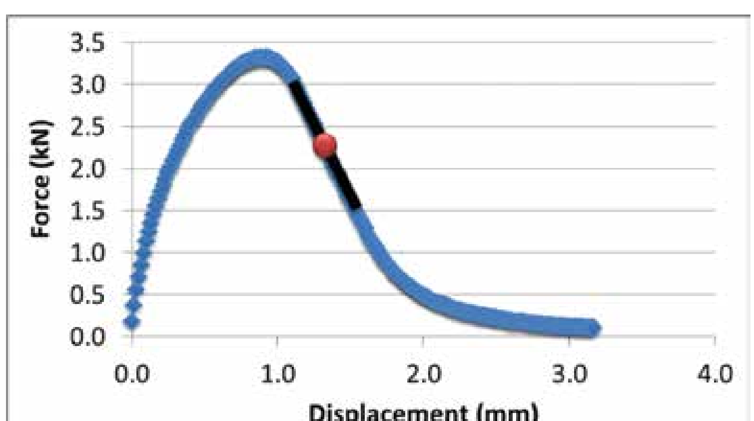


Figure 1: I-FIT Test Setup (left) and Example I-FIT Data (right) with Post-Peak Slope Highlighted

and long-term aged conditions. The results showed that at both aging conditions, the mix without REOB/VTAE had the highest mean Flexibility Index value. At the short-term aged condition, the mix without REOB/VTAE and 3% Source B REOB/VTAE had statistically different results, while the rest of the mixes were statistically equivalent. At the long-term aged condition, all of the mixes had statistically equivalent Flexibility Index results.

The second phase of the study involved verifying the REOB/VTAE content in each binder using inductively coupled plasma (ICP) analysis. Other studies have indicated that zinc and calcium detected in asphalt binders with similar techniques can be correlated to REOB/VTAE contents. The results of the ICP analysis indicate that the asphalt binders used in this study contained approximately 3%, 6%, and 9% REOB/VTAE as reported.

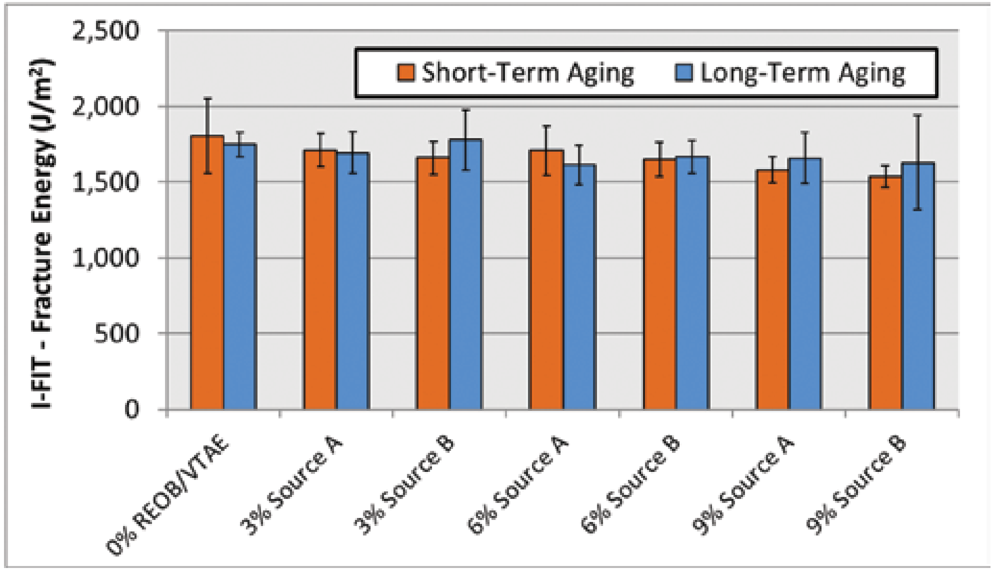


Figure 2: I-FIT Average and Standard Deviation of Fracture Energy

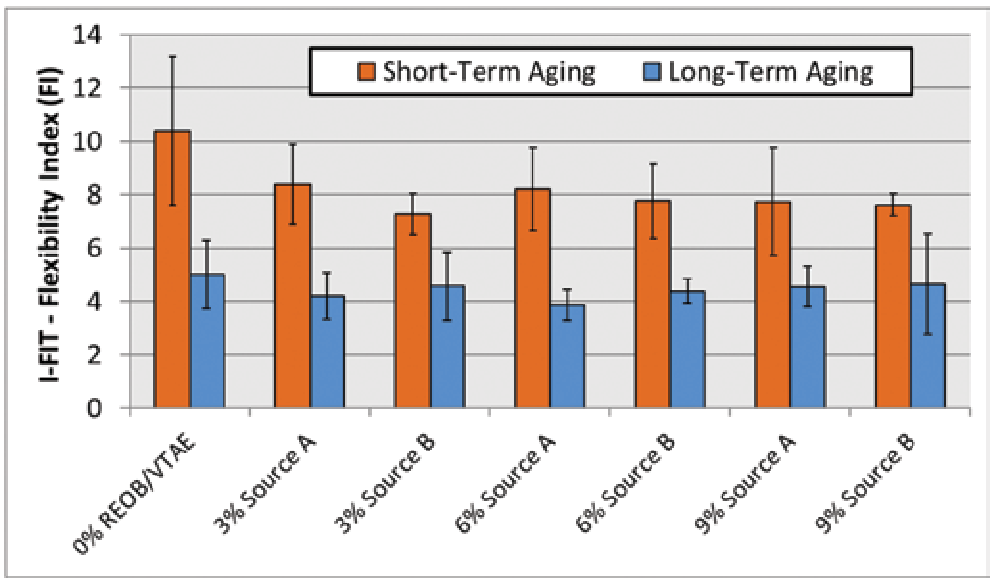


Figure 3: I-FIT Average and Standard Deviation of Flexibility Index

The results of this limited study showed that while REOB/VTAE reduced the average Flexibility Index for short-term aged mixes, long-term aged mixes were not affected by REOB/VTAE. Results also indicate that the source of the REOB/VTAE or dosage rates between 3 and 9% did not statistically affect the Flexibility Index. However, this study is limited in that it only evaluated one set of materials (one mix design and one base binder grade). An expanded evaluation with additional materials is needed. Additional evaluations of this type would be helpful in quantifying the impact of REOB/VTAE on the cracking susceptibility of asphalt mixtures and alleviating industry concerns regarding its use.

New Criteria Recommended for Designing Perpetual Pavements

Long-life or perpetual pavements are designed and built to last longer than 50 years, requiring only a periodic mill and inlay of the surface layer. The pavement structure is designed using appropriate materials and layer thicknesses to prevent structural distresses that begin at the bottom of the pavement structure, such as bottom-up fatigue cracking and subgrade rutting. To eliminate deep structural distresses, horizontal tensile strains at the bottom of the asphalt layer and vertical compressive stresses/strains at the top of the subgrade must be less than critical thresholds at which damage begins to occur. Additional asphalt thickness above what is required to keep stresses/strains below the critical thresholds is unnecessary to ensure long life. Thus, the goal of perpetual pavement design is to optimize layer thicknesses to sustain heavy loads indefinitely without being overly conservative. Asphalt thicknesses for perpetual pavements have typically ranged from 9 to 16 in. depending on traffic, materials, and site conditions.

Refining Limiting Strain Criteria for Perpetual Pavement Design

NCAT recently completed a NAPA-sponsored study to improve the cost-effectiveness of long-life pavements by establishing critical design thresholds and approximate ranges of maximum thickness. Existing literature was reviewed to determine historical parameters used in perpetual pavement design. To prevent bottom-up fatigue cracking, perpetual pavements have typically been designed so that tensile strains at the bottom of the asphalt structure are less than the fatigue endurance limit (FEL) of the asphalt mixture in the bottom layer. Typical values for the FEL range from early, conservative estimates of 70 microstrain up to more recent values of 200 microstrain. To prevent structural rutting, a vertical strain limit of 200 microstrain at the top of the subgrade has been proposed.

A previous analysis of pavement response data from instrumented sections at the NCAT Test Track showed poor correlations between laboratory FELs and field fatigue cracking performance or field-measured strains. Instead, cumulative field-measured strain distributions and fatigue ratios (cumulative strains divided by the FEL of the asphalt base layer) were found to better predict bottom-up fatigue cracking. Due to these promising results, another detailed analysis was conducted to refine the approach using PerRoad, a mechanistic-based pavement design and analysis program.

Six test sections built in the 2003 and 2006 Test Track research cycles were simulated in PerRoad to predict strain values at the bottom of the asphalt structures. Three sections were deemed perpetual without any bottom-up cracking, and the other three sections experienced bottom-up fatigue cracking. The predicted strain values were then used to develop a limiting cumulative strain distribution and maximum fatigue ratios, which clearly separated the perpetual pavement sections from the others.

The refined limiting strain distribution and maximum fatigue ratios were then validated using data from six sections constructed for the 2009 Test Track research cycle. As before, cumulative strain distributions and fatigue ratios determined by PerRoad simulations failed both limiting cumulative strain distribution and maximum fatigue ratios. All six sections experienced bottom-up fatigue cracking, and comparisons with the design limiting strain distribution accurately characterized all six sections.

Table 1 presents the limiting cumulative strain distribution and fatigue ratios refined in this study for future use in perpetual pavement design instead of a conservative limiting strain of 70 microstrain or the laboratory FEL of the asphalt base layer.

Estimating Ranges of Maximum Pavement Thicknesses

Based on the refined limiting strain criteria, an analysis was conducted using PerRoad simulations to determine ranges of maximum pavement thickness. The analysis included three climatic conditions and incorporated a conservative traffic level within legal load limits for various subgrade and base moduli combinations. The procedure was similar to that conducted in SHRP 2 Project R23 (although this used the new limited strain criteria) and included entering structural information and loading conditions, running the PerRoad simulation, and extracting the output data to compare against the limiting criteria in Table 1. The full procedure is listed in NCAT Report 15-05.

For the pavement to pass, the cumulative tensile strain distribution at the bottom of the asphalt needed to be less than that presented in Table 1, and the vertical compressive strain distribution at the top of the subgrade needed to have at least 50% of values below 200 microstrain. If the pavement failed either of these criteria, the AC thickness was adjusted and the design was repeated. The resulting approximate ranges of maximum design thicknesses for asphalt pavements are shown in Table 2. Depending on site-specific conditions, the range of approximate maximum asphalt thickness was between 5 and 15.5 in.

Recommendations for Implementation

The limiting cumulative strain distribution was found to be the best indicator of how the structural test sections resisted bottom-up fatigue cracking at the NCAT Test Track. Based on these findings, the limiting cumulative strain distribution and fatigue ratios in Table 1 are recommended for use in perpetual pavement design instead of a conservative limiting strain of 70 microstrain or the laboratory FEL of the asphalt base layer.

Approximate ranges of maximum AC thicknesses for flexible pavement design are provided in Table 2. A similar table can be developed on a state-by-state basis based on the methodology presented in this study (detailed in NCAT Report 15-05) to represent specific climate, material, and subgrade conditions.

When the thickness of a new or rehabilitated pavement designed based on agency-specific methodology is greater than the maximum thickness, the perpetual pavement design approach may be used to optimize a design that can sustain the heaviest loads and provide an indefinite structural life without being overly conservative.

Table 1: Refined Limiting Distribution and Maximum Fatigue Ratios for Predicted Tensile Strain

Percentile	Limiting Design Distribution for Predicted Strain (Microstrain)	Maximum Fatigue Ratio for Predicted Strain
1%	29	
5%	41	
10%	48	
15%	54	
20%	60	
25%	66	
30%	71	
35%	78	
40%	84	
45%	91	
50%	100	0.68
55%	110	0.74
60%	120	0.81
65%	131	0.88
70%	143	0.96
75%	158	1.06
80%	175	1.18
85%	194	1.31
90%	221	1.49
95%	257	1.73
99%	326	2.19

Table 2: Ranges of Maximum AC Thicknesses

Subgrade Mr (ksi)	Base Mr (ksi)	Maximum Asphalt Thicknesses (in.)					
		6-in. Aggregate Base		8-in. Aggregate Base		10-in. Aggregate Base	
		Average	Range	Average	Range	Average	Range
5	30	14.0	12.5-15.5	13.8	12.5-15.0	13.3	12.0-14.5
5	50	13.7	12.0-15.0	13.2	11.5-14.5	12.3	11.0-13.5
5	100	13.2	12.0-14.0	12.2	11.0-13.0	11.2	10.0-12.0
5	250	10.2	8.5-12.0	9.0	7.5-10.5	7.7	6.5-9.0
5	500	9.3	8.0-11.0	8.3	7.0-10.0	7.2	6.0-8.5
10	30	12.2	10.5-14.0	11.7	10.5-13.0	11.0	10.0-12.0
10	50	11.8	10.5-13.0	11.2	10.0-12.0	10.0	9.0-11.0
10	100	11.0	10.0-12.0	10.2	9.0-11.0	9.0	8.0-10.0
10	250	9.7	8.5-11.0	8.7	7.5-10.0	7.3	6.5-8.5
10	500	8.8	7.5-10.5	7.8	6.5-9.5	6.7	5.5-8.0
20	30	10.5	9.0-12.5	10.7	9.0-12.5	10.0	8.5-11.0
20	50	10.2	8.5-12.5	10.2	8.5-12.0	9.0	7.5-10.0
20	100	9.7	8.0-12.0	9.0	7.5-10.5	8.0	6.5-9.0
20	250	8.8	7.5-10.5	8.0	7.0-9.5	6.8	6.0-8.0
20	500	8.3	12.5-15.5	7.8	6.5-9.5	6.5	5.0-8.0

ALDOT Field Evaluation of Rejuvenators

Asphalt pavements have been known as America's most recycled product. As asphalt plants use more reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) in the process of producing asphalt mixtures, there is concern that mixtures with high percentages of these recycled materials may be more susceptible to cracking, as recycled binders are aged and stiffer than virgin asphalt. Thus, most states limit the recycled binder content of asphalt mixtures with percentages varying by pavement layer. To address the concern, rejuvenators, or recycling agents, have been developed to potentially improve the cracking resistance of recycled asphalt mixtures and to more effectively use recycled materials in asphalt mixtures.

Rejuvenators have been used since the 1960's for pavement preservation treatments (such as fog seals) and have also been used in cold and hot in-place recycling. Recent laboratory studies have shown that rejuvenators or recycling agents can improve the laboratory properties of recycled mixtures, but as yet, the U.S. has little documented experience in their use with hot mix asphalt (HMA) or warm mix asphalt (WMA). A 2014 NAPA study tour of Japan, however, showed that recycling agents are used successfully there to produce mixtures with high recycled contents.

This study, conducted for the Alabama Department of Transportation (ALDOT), is evaluating the results of laboratory tests and the field performance of recycled mixtures with and without rejuvenators. Three 12.5-mm nominal maximum aggregate size (NMAS) mixes, each produced with PG 67-22 virgin binder, were placed on U.S. 31 near Prattville, Alabama. The control mix (no rejuvenator) contained 20% RAP, while the mixes with rejuvenators contained 25% RAP and 5% RAS. Two rejuvenators were evaluated in this study. The respective suppliers recommended the dosages used in the experimental mixes. The first experimental mix used 4.8% of Rejuvenator 1 by weight of total binder while the second experimental mix used 8% of Rejuvenator 2. Both rejuvenators were blended with the virgin binder prior to mixing with the aggregate. Each mix was sampled during production for laboratory testing (binder and mix). Field performance of the test sections continues to be monitored.

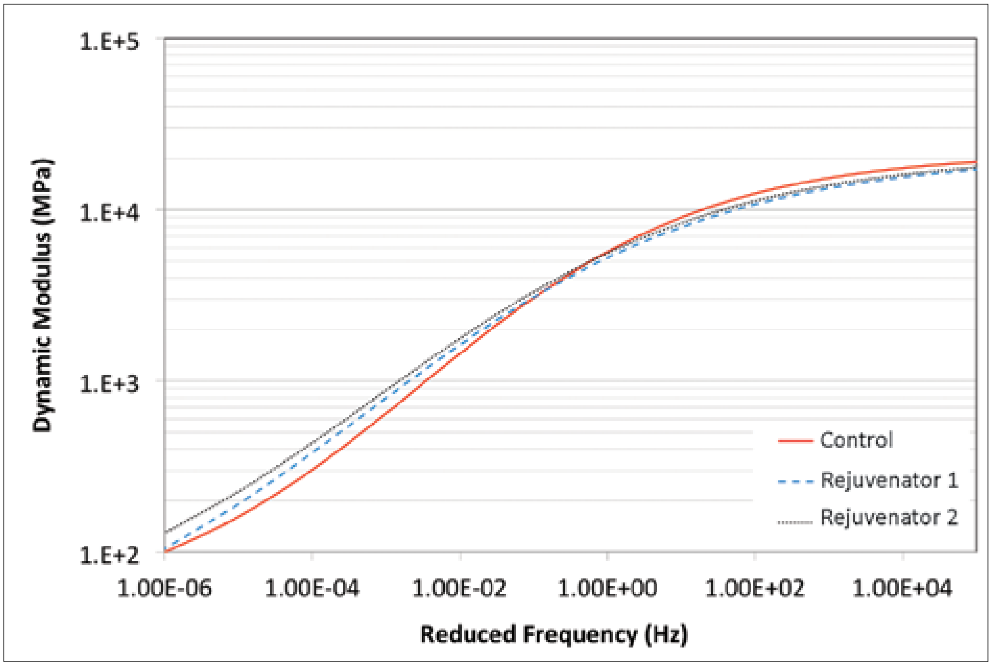


Figure 1: Dynamic Modulus Master Curves

To evaluate the effect of the rejuvenators on asphalt binder, the following asphalt blends were graded: virgin binder with and without each rejuvenator (at the recommended amounts used for mix production), recovered binder from each plant-produced mix, and recovered RAP and RAS binders with and without each rejuvenator (at a rate of 10% by weight of RAP/RAS binder). For the recovered RAP/RAS binders, both rejuvenators decreased the high- and low-temperature grades. While Rejuvenator 2 improved the PG grade of RAP binder more effectively than Rejuvenator 1, both rejuvenators had a similar effect on decreasing the high-temperature grade of RAS binder. Compared to RAP, RAS was less sensitive to both rejuvenators. Performance grading of the recovered binders from the plant-produced mixes showed that both rejuvenated 25% RAP/5% RAS mixes were PG 94-10 and the control 20% RAP mix was PG 88-10.

Results of laboratory testing of the three plant-produced mixes were as follows:

- All three mixes met ALDOT specifications for gradation and asphalt content (determined using both solvent extraction and ignition oven).
- Moisture susceptibility was assessed according to AASHTO T 283. While the control mix had a

satisfactory tensile strength ratio (TSR) greater than 80%, both rejuvenated mixes had TSR values less than the recommended minimum of 80%. Thus, both rejuvenated mixes may show greater susceptibility to moisture damage than the control mix.

- Rutting resistance was evaluated using the Asphalt Pavement Analyzer (APA). Numerically, the control mix had greater rut depths than the rejuvenated mixes, which both had similar rut depths. Based on a statistical analysis, the control mix had statistically greater APA rut depth results than either rejuvenated mix. Thus, both rejuvenated mixes showed better rutting resistance than the control mix.
- To evaluate mix stiffness, dynamic modulus (E*) was determined at three temperatures (4°C, 20°C, and 40°C) and four frequencies (0.01, 0.1, 1, and 10 Hz). Master curves at a reference temperature of 20°C were constructed for all three mixes. As shown in the E* master curves (Figure 1), both rejuvenated mixes had slightly lower moduli than the control mix at higher reduced frequencies (which correspond to lower temperatures) but were stiffer than the control mix at lower reduced frequencies (which correspond to higher temperatures). Thus, the two rejuvenated mixes show higher stiffness at high temperatures, concurring with the APA test results.
- The semi-circular bending (SCB) test developed at the Louisiana Transportation Research Center was used to evaluate resistance to intermediate temperature cracking or fatigue cracking. For fracture-resistant mixes, a higher critical value of J-integral (Jc) is desired. The experimental mix with Rejuvenator 2 exhibited the highest Jc value (0.479 kJ/m2), followed by the control mix (0.458 kJ/m2) and the experimental mix with Rejuvenator 1 (0.376 kJ/m2). Thus, the experimental mix with Rejuvenator 2 would be expected to have the best fatigue cracking resistance.
- Resistance to reflective cracking was evaluated using the Overlay Tester (OT) in accordance with the Tex-248-F test method used by Texas DOT. The control mix had significantly higher cycles to failure than the rejuvenated mixes, which had statistically equivalent cycles to failure. These results indicate that both of the experimental mixes with 25% RAP/5% RAS could show less resistance to reflective cracking than the 20% RAP control mix.

The U.S. 31 test sections have been monitored for field performance for approximately two years. No distresses (rutting, cracking, moisture damage, or bleeding) were visually evident in any of the three test sections. Field performance of these test sections continues to be monitored to validate the laboratory test results.

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.

Randy West, NCAT
Are the G_{sb} values used in mix designs in your state determined by the highway department or by the contractor (or mix design lab)? How often are the G_{sb} values tested?

Jerry Geib, Minnesota DOT
In 2016, eight of the HMA cells at MnROAD will be reconstructed as part of the NCAT-MnDOT cracking study. The MnROAD experiment will focus on thermal cracking.

MnDOT will be using a 4.75 mm mix for a thin overlay of PCC on the Interstate in Duluth.

The ultrathin bonded wearing course (UTBWC) reacts differently than our dense graded HMA in rural areas during snow and drifting events. We are discussing a future effort to look into this matter. This is not an issue in our metro area.

Asphalt Forum Responses

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

MnDOT is moving toward implementation of the disk-shaped compact tension (DCT) test. This test measures fracture energy. High fracture energy provides greater resistance to thermal cracking. Are other DOT's looking at new performance tests?

Michael Stanford, Colorado DOT
Colorado DOT is currently performing the DCT test for information only. We test at a frequency of one test per mix for each project.

Eric Biehl, Ohio DOT
In Ohio, we are looking at mix fatigue tests for high RAP and RAS mixes through a research project that just kicked off. The Illinois Flexibility Index Test (I-FIT or also known as SCB-IL) and Texas Overlay Test are probably the front-runners for us. We have not looked at low temperature mix tests yet, but could see that coming at some point in the next few years. For PG binders, we're hoping to see what comes out of NCHRP 9-59 and 9-60 in a few years.

James Williams, Mississippi DOT
Currently, Mississippi DOT is not looking at any particular performance test for cracking. Of particular interest is research into various cracking tests and their relationship with field performance. These relationships and the associated performance models are key to ME design.

Specification Corner

Colorado DOT
We revised our chip seal, microsurfacing, slurry seal, and emulsion specifications to get them more in line with the national ISSA standards.

Florida DOT
Effective January 2015, specifications were modified to require non tracking tack for all FDOT projects. Though there were some initial issues, this is slowly being sorted out and is working well. FDOT implemented the MSCR J_{nr} requirements for modified binders in July 2013 and will implement the J_{nr} requirements for unmodified binders beginning in July 2016.

Minnesota DOT
No significant changes to our Superpave specifications. We are implementing the MSCR for binders, and most 2017 projects will use the MSCR specification.

Mississippi DOT
In January 2016, MDOT revised the specification for SMA to allow up to 10% RAP by weight of aggregate. The RAP is required to be crushed and processed.

Upcoming Training Sessions

NCAT training courses are designed for the asphalt pavement industry, and we can also customize workshops designed to meet your specific training needs. For more information or to register for a course, visit ncat.us/education/training.

Aggregate Technology
August 15-19, 2016
This course provides classroom instruction, laboratory demonstrations, and hands-on training relating to the specifications and testing of fine and course aggregate, particularly ALDOT 249: Procedure for the Acceptance of Fine and Coarse Aggregates.

Asphalt Level I
June 13-17, 2016; September 12-16, 2016
This course provides classroom instruction, laboratory demonstrations, and hands-on training on sampling, testing, and analysis to control the quality of hot mix asphalt (HMA) pavements.

Earthwork / Base Technology
August 29-31, 2016
This course provides classroom instruction and case studies on soils delivery, placement and compaction, and use of nuclear gauges to determine densities.

Radiation Safety
April 11-12, 2016; July 18-19, 2016; August 22-23, 2016
The objective of this course is to ensure that individuals who will use nuclear density gauges and nuclear asphalt content gauges understand the appropriate safety procedures.

Roadway Technology
April 12-15, 2016; July 19-22, 2016; August 23-26, 2016
This is a classroom and laboratory course that covers areas of asphalt pavement construction; delivery, placement, and compaction of HMA mixtures; random sampling; and computation of pay factors used for HMA pavements.

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