TRANSPORTATION RESEARCH

CIRCULAR

Number E-C237 October 2018

Innovations in Asphalt Mixture Design Procedures

The National Academies of
SCIENCES · ENGINEERING · MEDICINE
TRB
TRANSPORTATION RESEARCH BOARD

Transportation Research Circular E-C237

Innovations in Asphalt Mixture Design Procedures

Prepared by
Frank Fee
Shane Buchanan
Samuel Cooper III
Louay Mohammad
Hasan Ozer
Imad Al-Qadi
Fujie Zhou
Thomas Bennert
John Haddock
Tim Aschenbrener

For the
Standing Committee on Critical Issues and Emerging Technologies in Asphalt

Transportation Research Board
500 Fifth Street, NW
Washington, D.C.
www.TRB.org

TRANSPORTATION RESEARCH CIRCULAR E-C237
ISSN 0097-8515

The **Transportation Research Board** is one of seven programs of the National Academies of Sciences, Engineering, and Medicine. The mission of the Transportation Research Board is to provide leadership in transportation innovation and progress through research and information exchange, conducted within a setting that is objective, interdisciplinary, and multimodal.

The **Transportation Research Board** is distributing this E-Circular to make the information contained herein available for use by individual practitioners in state and local transportation agencies, researchers in academic institutions, and other members of the transportation research community. The information in this circular was taken directly from the submission of the authors. This document is not a report of the National Academies of Sciences, Engineering, and Medicine.

Transportation Research Board
500 Fifth Street, NW
Washington, D.C.
www.TRB.org

Foreword

This E-Circular was developed from presentations made during the 95th Annual Meeting of the Transportation Research Board in a workshop titled Innovations in Asphalt Mix Design Procedures. Audrey Copeland of the National Asphalt Pavement Association guided the session, which was sponsored by the Standing Committee on Critical Issues and Emerging Technologies in Asphalt.

The fundamentals of asphalt mix design are examined for improved durability and performance along with recent advancements in specifications and construction. Common themes include strategies that yield more binder into the mix with trials that modify the air void and void in mineral aggregate (VMA) requirements that complement the Superpave system. Methodologies to incorporate rejuvenators for increased reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS) are presented. Performance testing ties together these approaches the concepts shown for balanced mix designs. The benefits of achieving adequate field compaction and comparisons of these innovative mix design and performance evaluations from field test sections are explored.

ACKNOWLEDGMENTS

The Committee recognizes former Members who contributed while serving during the workshop and development of this e-Circular: Timothy Aschenbrener, Dean Maurer, Rebecca McDaniel, Louay Mohammad, Marshall Shackelford, and Fred Hejl (TRB).

PUBLISHER'S NOTE

Contents

Optimized Mix Design Approach
Contractor's Perspective

Shane Buchanan
Oldcastle Materials

Asphalt mix design is a critical initial step in achieving a long-lasting asphalt pavement. While a good mix design will not guarantee long-lasting field performance, a poor mix design will be more likely to have performance-related issues. With regards to performance, an asphalt mix (pavement) should possess adequate stability (i.e., resistance to permanent deformation or rutting) and durability (i.e., resistance to cracking) for the intended design application.

In basic terms, the mix design process consists of combining aggregates, asphalt binder, and, most likely, recycled materials to meet project requirements. Many mix designs can be developed for a given application, but the focus should be to have a design which best utilizes available (local) materials, which may be more readily available and potentially reduce mix cost, while providing the necessary performance.

Most commonly, the binder content is the main item of interest obtained from the mix design. Mix binder content is often referred to as either the "design" or "optimum" binder content, but these are different. There can be many design binder contents, based on varying specification requirements, but the ultimate mix design approach objective is to determine the optimum mix binder content for the specific mix design application (e.g., design traffic, pavement layer location, and climate).

Binder content is important since it is a primary driver of mix performance. With all other mix constituents held constant, mix performance will be mainly dictated by the binder content. Binder contents that are "drier" than optimum can lead to inadequate mix durability, while contents "wetter" than optimum can lead to inadequate mix stability. This fundamental concept is illustrated in Figure 1 (*1*).

The significance of a properly conducted mix design should not be overlooked. As an example to illustrate the importance, consider that during 2014, approximately 360 million tons of asphalt mix was produced in the United States. Assuming a Monday-to-Friday production basis, approximately 1.4 million tons of mix were placed daily, which yields a quantity sufficient to pave a 12-ft wide, 1.5-in. thick pavement from New York to Las Vegas (~2,500 lane miles) each day. This example serves to reinforce the critical need for good mix design development and the need to address any inadequacies in current mix design approaches to help ensure long-lasting pavement performance.

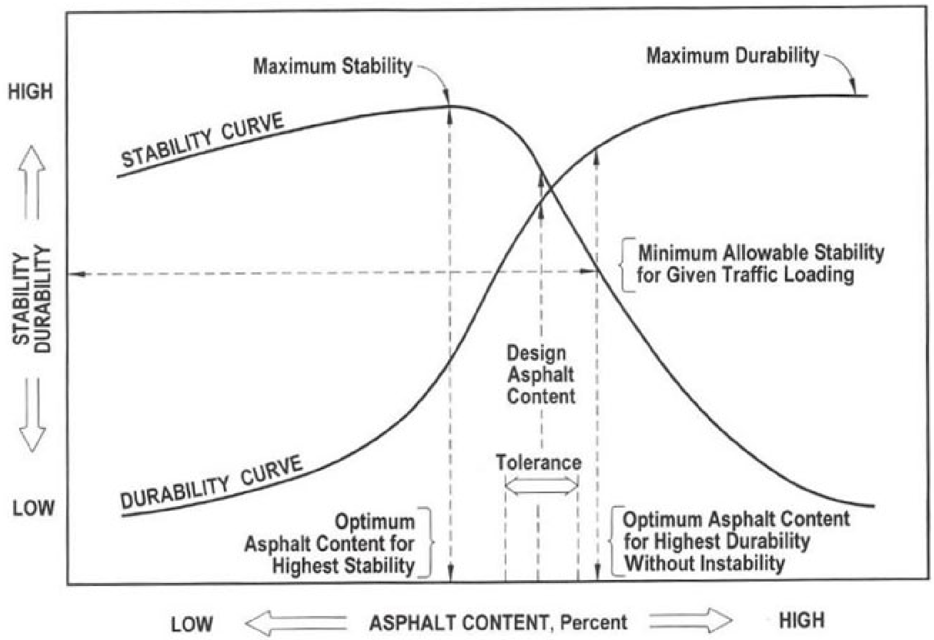


FIGURE 1 Asphalt mix design performance illustration (*1*).

CURRENT PERFORMANCE ISSUES

In recent years, there have been observations and reports of mix durability-related performance problems. While the true cause(s) of these problems have not been fully investigated, it is thought that "dry" mixes are the general cause, with contributing factors being (1) too high a design gyration level (N_{design}); (2) excessive recycle [i.e., recycled asphalt pavement or recycled asphalt shingles (RAS)]; (3) inadequate mix specifications; or (4) inappropriate mix type selection, etc. In many cases the cause is likely a combination of several factors.

A recent survey (*2*) conducted of Oldcastle Materials operating companies, found that most reported pavement distresses that were observed within the last 5 years, were durability related, as illustrated in Figure 2. In Figure 2, rutting was only reported by 7% of the respondents, which supports the widely held belief that rutting is not a major performance issue with today's mixes. While rutting still may occur, it is probably the case of a mix production issue, construction-related issue, or an inadequate structural design, and not one stemming from the mix design itself.

SPECIFICATION CHANGES TO ADDRESS PERFORMANCE ISSUES

State departments of transportation (DOTs) have recognized these performance-related issues and, as a result, have implemented a variety of specification changes. Many of these changes are focused on increasing the mix binder content. Figure 3 illustrates the specification changes within the last 5 years obtained from the Oldcastle survey (*2*).

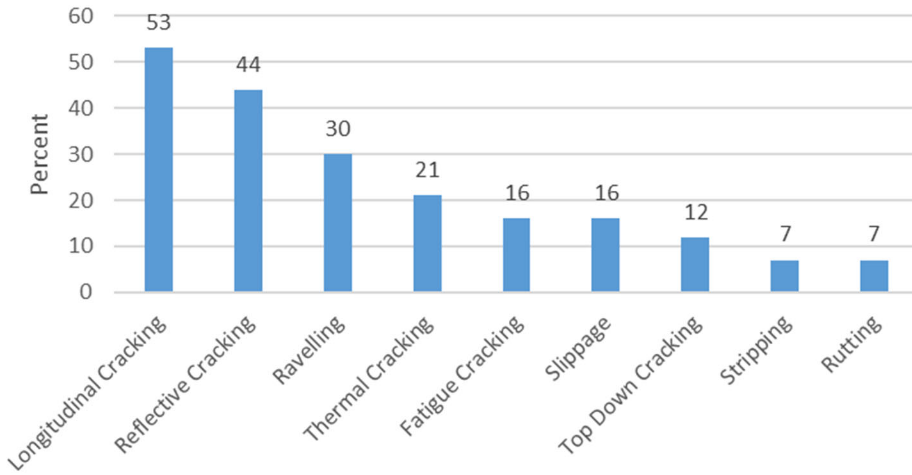


FIGURE 2 Oldcastle survey results: observed pavement distress for the past 5 years (*2*).

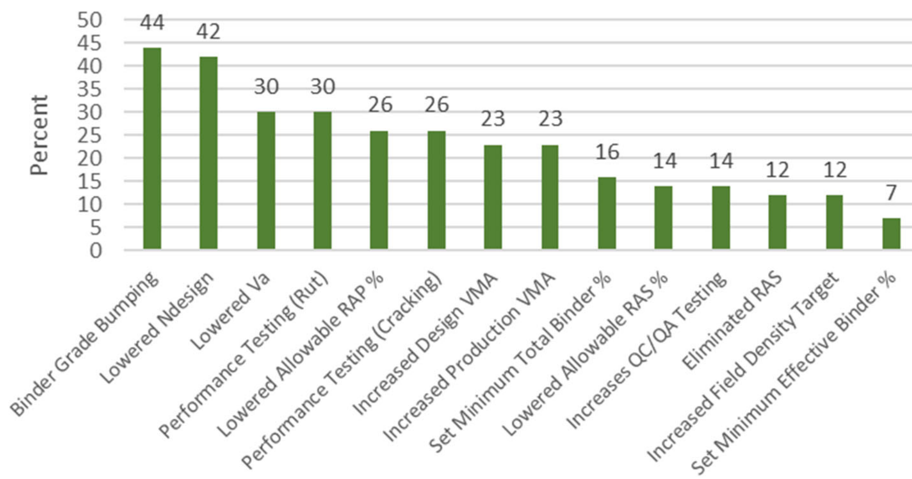


FIGURE 3 Oldcastle survey: agency specification changes for the past 5 years (*2*).

One concerning observation is that some states are implementing multiple specification changes simultaneously. While doing so may yield satisfactory results, it ultimately makes it difficult to impossible determine true cause and effect, resulting from the changes. One example of this is with the Alabama DOT, where (1) an N_{design} of 60 is specified for all mixes; (2) the minimum design void in mineral aggregate (VMA) is increased by 0.5%; (3) a minimum total binder content established for non-RAS and RAS mixes (RAS mixes with 0.2% higher binder content); and (4) the design air voids for RAS mixes is set at 3.5%.

One item that highlights the varying DOT specifications is the specified N_{design} level. Table 1 shows the commonly utilized N_{design} levels for various DOTs (as of April 2015). A couple items of interest can be observed from Table 1. First, several DOTs are specifying one N_{design} level for all mixes; these states include Alabama (60), Ohio (65), and Virginia (65). Second, some states are utilizing many N_{design} levels with some levels being high (e.g., 125 gyrations) and levels, in some cases, only differing by 1 to 5 gyrations.

TABLE 1 State DOT N_{design} Levels (as of April 2015)

State	Gyration Level[1,2]
Alabama	**60**
Arkansas	50, 75, 100, 125
Colorado	75, 100
Connecticut	75, 100
Florida	50,65,**75,100**
Idaho	50,75,100, 125
Iowa	50, 60, 65, 68, 76, 86, 96, 109, 126
Kansas	**75**, 100
Kentucky	50, 75, 100
Maine	**50,** 75
Massachusetts	50, **75, 100**
Michigan	45, 50, 76, 86, 96, 109, 126
Minnesota	40, 60, 90, 100
Mississippi	50, **65**, 85
Missouri	50, 75, **80**, 100, 125
Montana	**75**
Nebraska	40, 65, 95
Nevada	Use Hveem
New Hampshire	50, 75
New Jersey	50, **75**
New Mexico	75, **100**, 125
New York	50, **75**, 100
North Carolina	50, **65, 75**, 100
Ohio	**65**
Oklahoma	64-22 (50), 70-28 (60) , and 76-28 (80)
Oregon	65, 80, 100
Pennsylvania	50, 75, 100
Rhode Island	**50**
Tennessee	**65** or 75 Marshall
Texas	**50**
Utah	50, **75**, 100, 125
Vermont	50, **65**, 80
Virginia	**65**
Washington	50,75,100, 125
West Virginia	50, 65, 80, 100

Notes: 1) Gyration level highlighted in "Bold" indicates main level used.

2) AASHTO R35 Gyration Levels are 50, 75, 100, 125

Many states have lowered the N_{design} levels in recent years in an attempt to increase the mix binder content. However, it is critical to understand that reducing the N_{design} level alone will not automatically result in a long-term increase in the mix binder content. Over time, mix designers could find methods of adjusting the mix design (e.g., alternate aggregate materials and gradings) to lower the binder content in order to be competitive in the marketplace.

MIX PERFORMANCE KEY

There are many factors that contribute to good mix performance, however, the volume of effective binder (V_{be}) in an asphalt mixture has been identified as the primary mix design factor affecting both durability and fatigue cracking resistance (*3*). V_{be} is the difference between the mix VMA and air voids (V_a). Increasing the V_{be} results in a higher effective binder content which will aid the mix in providing acceptable durability performance. Higher V_{be} values can be obtained by either increasing the VMA with a constant V_a or lowering the V_a with a constant VMA. Unfortunately, most specifications reference a total binder content in lieu of an effective binder content, which can be misleading, due to not accounting for the absorbed asphalt binder (P_{ba}), and potentially result in dry mixes.

HISTORY OF MIX DESIGN

The history of asphalt mix design is illustrated in Figure 4 (*4*). The first asphalt mixes (1890s) had much higher binder contents and consisted of finer aggregate gradings (i.e., sand type mixes) with higher minus No. 200 contents. Mixes were designed in this manner because the driving performance needed was long-term durability, not stability. Resistance against rutting was not a primary concern since high traffic volumes or loadings were not present.

Through the years with the various mix design development, binder contents became progressively lower, as traffic volumes and loadings increased. Another item to observe with the development history is the advent of performance testing. In the 1920s a compression test (stability) was first utilized with the Hubbard Field Method and was the first performance test associated with mix design. With the development of the Hveem and Marshall design procedures, stability and durability performance tests were implemented in an effort to balance the mix design similar to the concept illustrated in Figure 1.

The Superpave mix design system was developed in the early 1990s and, as originally developed, was to have three hierarchical levels of design, based on the intended mix application and traffic level. Level 1 was to be volumetric mix design only with Levels 2 and 3 being varying levels of volumetric mix design plus associated mix analysis utilizing performance testing. However, Levels 2 and 3 were not implemented and the Superpave system became a volumetric mix design system only. While Superpave has provided improvements to the design process, the lack of defined performance testing is considered a deficiency

Since Superpave implementation, state DOTs and other owner agencies have recognized the need for performance testing and have worked, at varying levels of effort, to occupy the performance testing void with various tests (empirical and/or fundamental) developed and utilized to help ensure needed pavement performance.

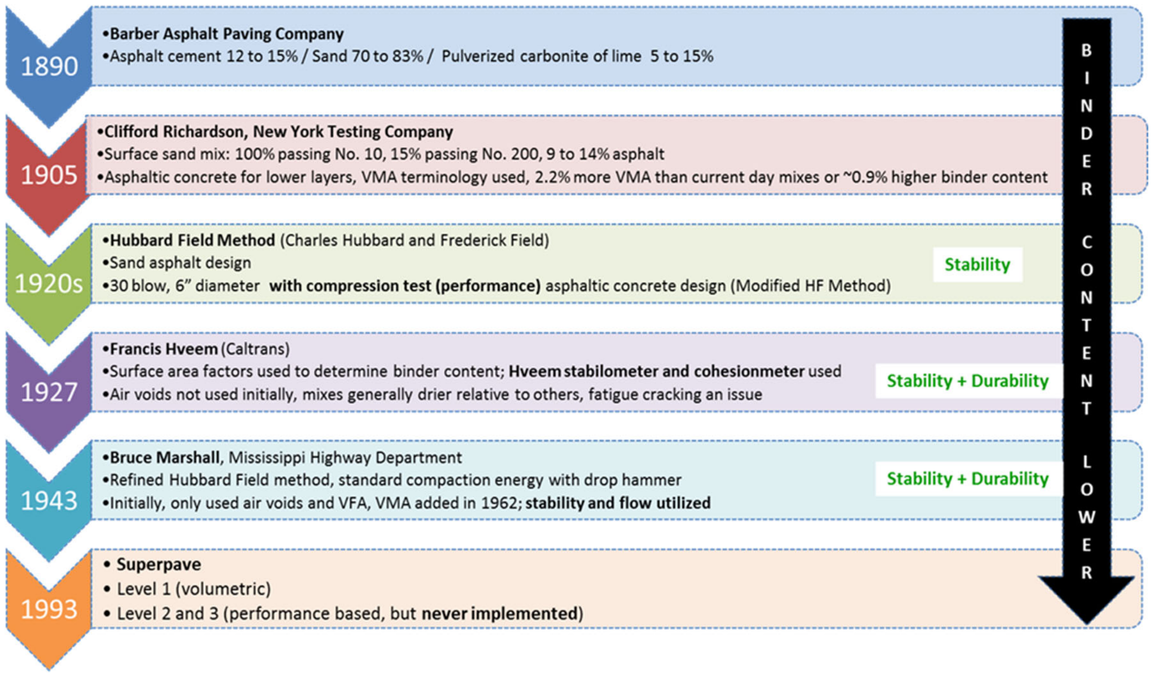


FIGURE 4 History of mix design (*4*).

CURRENT MIX DESIGN SPECIFICATIONS

Most of today's mix design specifications are heavily scripted and follow a "recipe" or standardized approach. Specifications will typically have set requirements for volumetric property requirements, aggregate type/properties, aggregate blend grading, binder type, recycle content, additives, etc. While these recipes may work, specifications have become convoluted and confounded over time with included specifications items often competing against each other in achieving the ultimate mix performance goal. An example of this would be changing the air void and/or VMA requirements, but not adjusting the allowable voids filled with asphalt (VFA) range. Additionally, over time, new items typically get added to the specifications, but rarely do older specification items get removed. All these issues combine to stifle the potential for innovation during the mix design process and potentially drive mix costs higher due to the inability to utilize locally available materials.

OPTIMIZED MIX DESIGN: A BETTER APPROACH

Ultimately, the goal of "recipe" type specifications is good mix performance. A better approach for mix design lies with an optimized mix design methodology. An optimized mix design is one where the appropriate binder content and other mix items (aggregate type, aggregate blend grading, recycle type/content, binder grade, etc.) are selected and optimized to provide needed performance for the specific application. In an optimized mix design approach the desired mix performance is defined and the mix design specification opened up to allow for innovation on part of the mix designer to achieve the needed performance. The freedom and ability to innovate is key to transition mix design and ultimately mix performance to the next level.

The key foundation points of optimized mix design are (1) use what works, (2) eliminate what doesn't, and (3) be simple, practical and correct. The last point is key because "good doesn't have to be complicated" in order to develop a quality mix design and lasting performance.

An optimized mix design approach will likely request changes in processes and procedures by specifying agencies and producers in order to allow for a greater emphasis on allowing innovation and engineering within the mix design development. An optimized approach should be embraced by innovative and proactive producers. These producers are typically those who have invested in personnel, training, equipment, and processes and thus understand and produce the highest quality, most consistent asphalt mix possible.

Achieving mix design innovation requires the number of "rules and restrictions" during mix design to be greatly limited. As previously stated, any designed mix must meet the needed performance characteristics for the given mix application and not just meet "historical" established specification requirements.

OPTIMIZED MIX DESIGN APPROACH FRAMEWORK

One framework for a design approach developed by the authors' company is an Optimized Mix Design Approach (OMEGA) in which innovation and mix engineering efforts are emphasized. The OMEGA approach consists of (1) materials evaluation and selection, (2) mixture stability performance evaluation, and (3) mixture durability/cracking performance evaluation.

Materials Evaluation and Selection

One key to success within OMEGA is to improve the knowledge level of the materials that comprise the asphalt mixture. A more extensive understanding and control of material properties (e.g., virgin aggregate–recycle grading consistency, virgin–recycle aggregate specific gravity, and virgin–recycle binder continuous grading) must be obtained. Improved understanding and control of materials, as well thoroughly understanding plant production capabilities/limitations will yield a more-consistent asphalt mixture and one that is more likely to meet and maintain the needed performance.

One potential benefit from an optimized design approach is the ability to select and evaluate a wider range of mix materials with properties that may differ from those specified in the historic controlling specification. Examples of this may include using an alternate performance grade (PG) binder, local aggregate sources, mix performance additives, etc. Again, in the optimized mix design framework, the design will be ultimately decided by performance.

Laboratory Compaction

The laboratory compactive effort (N_{design}) of Superpave mixes is a key item in the design procedure. Sufficient mix compaction should be completed to lock the aggregate blend structure into place, but not to the point of breaking or crushing the aggregate. Using an N_{design} that is too high will result in a low optimum binder content, an unrealistically high lab density and a mix that is overly difficult to compact (i.e., stiff) in the field. These factors will work together to negatively impact the mix durability performance.

As previously mentioned, several states (e.g., Alabama, Ohio, and Virginia) have selected one N_{design} level (60 to 65 gyrations) as the compaction level for all Superpave mixes. This level of N_{design} typically matches up well with the compaction "locking point" for most asphalt mixes. The locking point concept, regardless of the method used to calculate it, identifies the gyration level where the mix aggregate structure "locks" together and resists further consolidation under applied compaction.

Stability Performance Evaluation

Once the mix design materials are selected, mix stability performance should be evaluated at a range of binder contents to define the stability—binder content performance curve. Today, the most commonly used stability performance tests are the Asphalt Pavement Analyzer (APA) and the Hamburg Wheel Tracker, with the Oldcastle survey (*2*) indicating approximately equal use, as shown in Figure 5. Furthermore, 33% of the responses indicated volumetrics only were used to provide the necessary stability performance.

Other stability-related tests are available and should be considered for use, provided there are sufficient supporting data to meaningfully and correctly evaluate performance.

Regardless of the test utilized, the critical item to consider is the selection of the appropriate performance threshold. The threshold should be based on the actual field performance of locally produced mixes correlated to the stability performance test being utilized. Survey results, shown in Figure 6, show that the stability performance thresholds used by state DOTs are predominantly based on design traffic and PG binder. One finding was that some thresholds are based on "opinion" or adopted from "other states requirements," instead of being internally developed by the agency, based on local mixes and their performance.

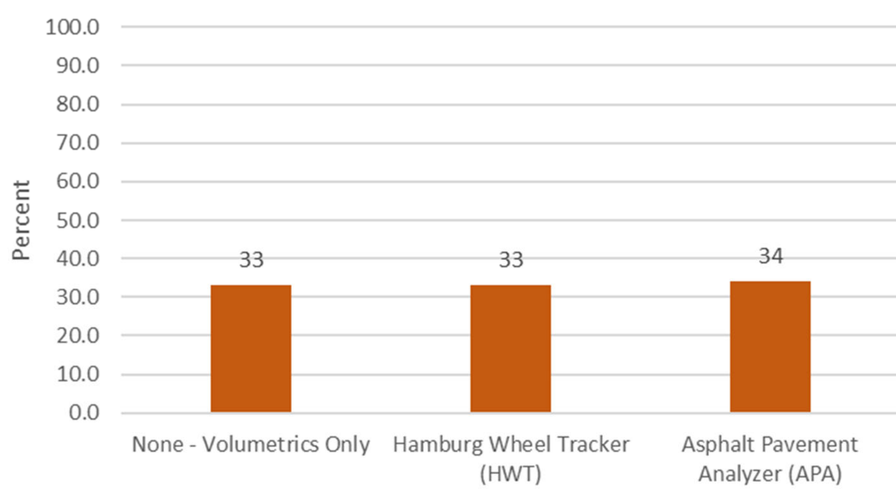


FIGURE 5 Oldcastle survey: agency-utilized stability performance test results (*2*).

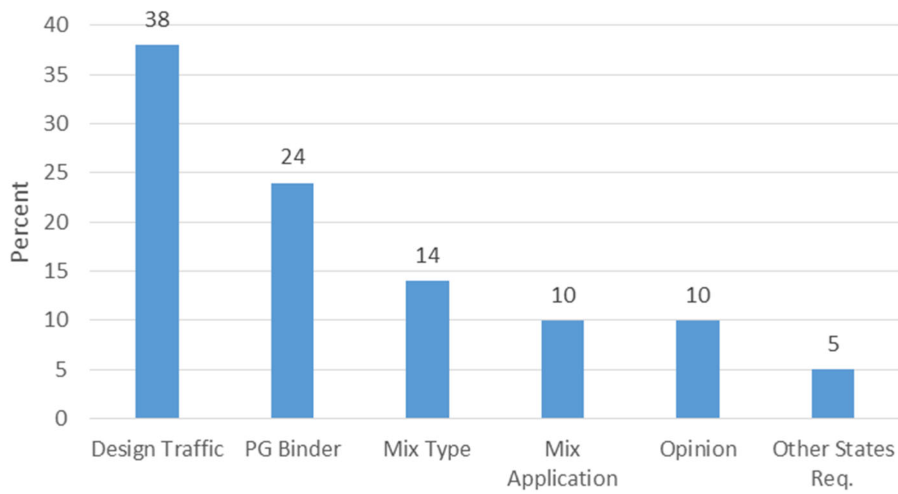


FIGURE 6 Oldcastle survey: stability performance threshold selection basis (*2*).

Durability Performance Evaluation

In a similar manner as with stability testing, mix durability (cracking) performance should be evaluated at the same range of binder contents. Durability evaluation is far more complicated than stability due to (1) mix performance sensitivity to aging and (2) needing to accurately understand the mode of durability related distress in order to properly test the mixture. With the stability evaluation, the most conservative test condition is "unaged," while the opposite holds true with a durability evaluation. Furthermore, durability related testing is a known weak link in performance testing with no general consensus as to the best performance test or the appropriate performance thresholds.

Pavement durability issues can manifest themselves in several modes. These include top-down fatigue cracking, low-temperature (thermal) cracking, bottom-up fatigue cracking, reflection cracking, etc. Today, a multitude of cracking tests are available with each being developed to evaluate a targeted mode of distress. Currently utilized durability related performance tests include bending beam fatigue (BBF), disc-shaped compact tension (DCT), semicircular bend (SCB) test, Texas overlay tester (OT), and indirect tension (IDT).

Two general objectives when selecting the durability performance test are as follows: (1) match the test to the anticipated pavement distress and (2) set appropriate performance thresholds. For example, if the anticipated mode of distress was top-down cracking, a test developed for bottom-up fatigue or reflection cracking evaluation may not be appropriate for use and may yield misleading results as to expected mix performance. NCHRP Project 9-57 is being currently being conducted to develop an experimental design for field validation of selected laboratory tests to assess cracking potential of asphalt mixtures. As part of this research effort, a workshop was conducted in early 2015 to obtain feedback from 31 industry experts (agency, industry, and academia) on the appropriate performance tests for each cracking mode. Selection criteria included (1) test method availability, (2) simplicity, (3) variability, (4) sensitivity to mix parameters, (5) complexity of data analysis, (6) availability/cost of test equipment, and (7) lab-to-field correlation. The results from that workshop are shown in Table 2 (*5*).

TABLE 2 NCHRP 9-57 Highest Ranked Cracking Performance Tests (*5*)

Items	Thermal Cracking	Reflection Cracking	Bottom-up Fatigue Cracking	Top-down Fatigue Cracking
Selected cracking tests	1. DCT 2. SCB-IL 3. SCB at low temp.	1. OT 2. SCB at intermediate temp. 3. BBF	1. BBF 2. SCB at intermediate temp.	1. SCB at intermediate temp. 2. IDT-UF

NOTE: SCB-IL is now referred to as the Illinois Flexibility Index Test (I-FIT).

The Oldcastle survey (2) found the use of durability performance testing is lagging behind stability testing, with 68% relying on volumetrics only to ensure durability performance, as provided in Figure 7. Small percentages of the states are using IDT, OT, DCT and BBF. Failure criteria, for those states utilizing durability tests, are predominately based on mix type and design traffic, as shown in Figure 8.

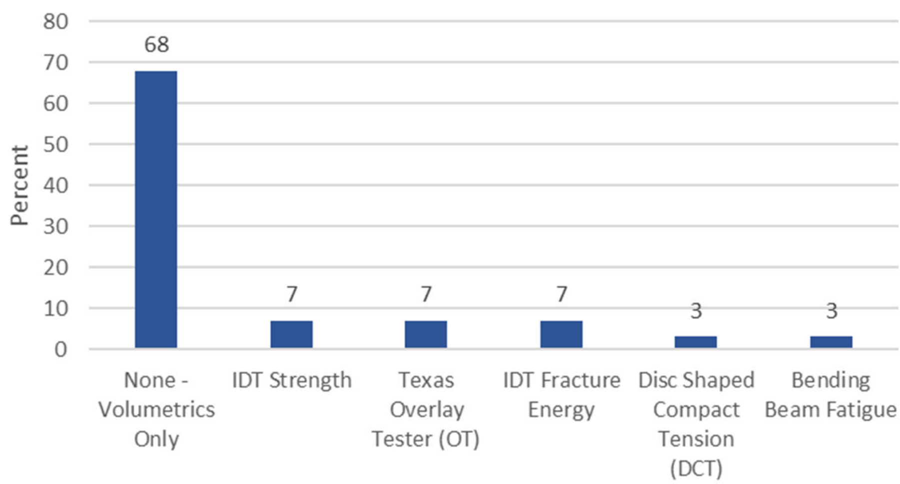


FIGURE 7 Oldcastle Survey: agency-utilized durability–cracking performance test results (*2*).

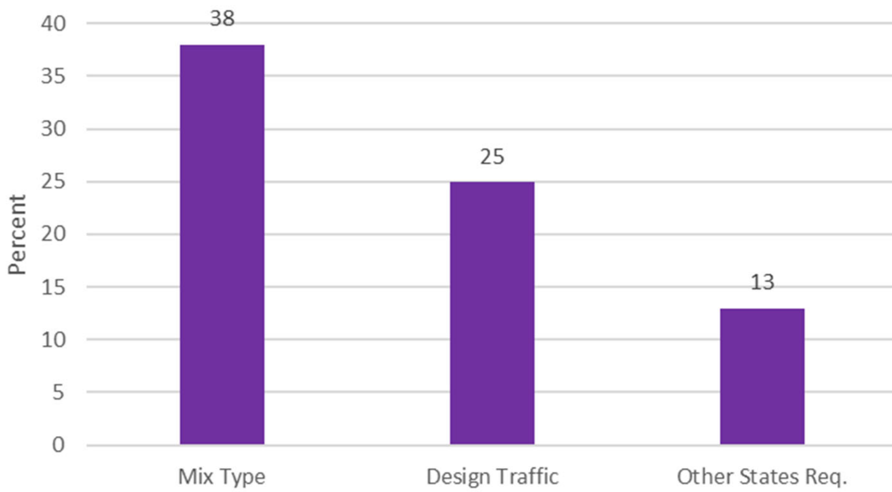


FIGURE 8 Oldcastle Survey: durability–cracking performance threshold selection basis (*2*).

Binder Content Selection

Once performance testing has been completed, stability and durability performance curves can be developed to determine the range of binder content that satisfies both performance requirements. A generic illustration of this approach is found in Figure 9. In this case, a range of acceptable V_{be} is defined to provide performance.

One question to consider is "Where should the binder content be selected within the acceptable performance range?" A commonly stated first answer is to select the midpoint binder content range. However, the binder selection should ultimately be the producer's decision based on their ability to produce a mix with a consistent binder content which meets the performance requirements. For example, a producer that can maintain a mix binder content with a standard deviation of 0.1% may chose (based on accepted risk level) to produce closer to the lower acceptable binder content level, than a producer whose binder content standard deviation is 0.3%. Such a situation could result from a low variability producer being more proactive in (1) evaluating the recycle binder asphalt content and grading, (2) calibrating the plant (e.g., asphalt pumps, weigh bridges), and (3) monitoring/controlling moisture content. This situation is similar to a percent within limit specification in which the producer sets a production target based on the known or expected variability and the acceptable level of risk. Regardless, the mix performance must be maintained.

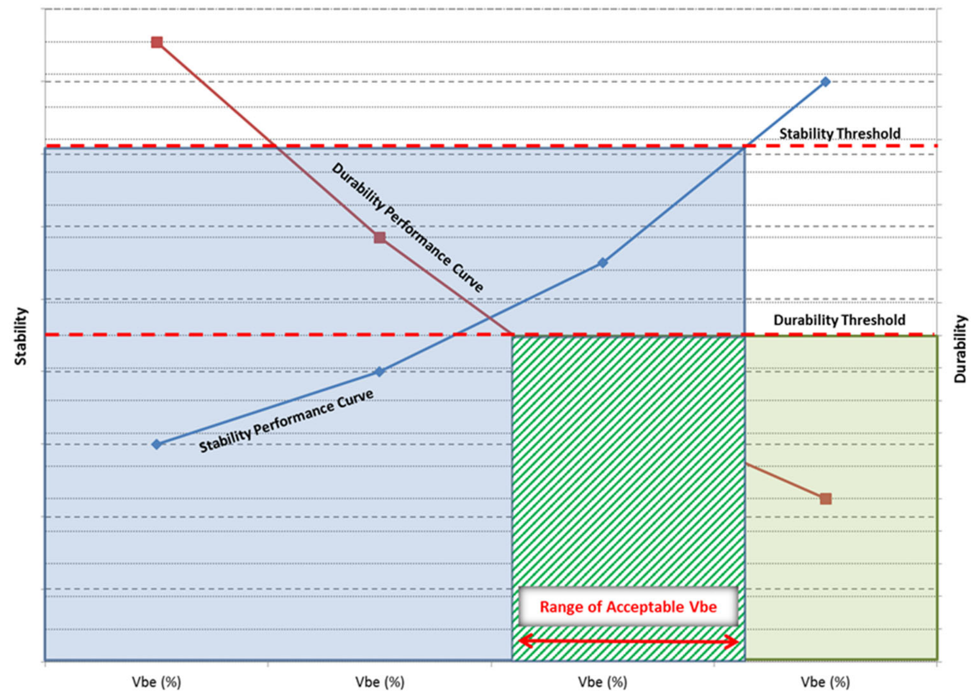


FIGURE 9 Binder content selection from performance curves.

MIX PRODUCTION CONTROL

Performance Testing Considerations

With regards to performance testing, new tests and test variations continue to be developed. In many cases, the repeatability (i.e., variability within lab) and reproducibility (i.e., variability between labs) of these tests have yet to be fully established. This has implications for performance testing use for acceptance purposes. A test without a properly developed precision statement should not be utilized for payment, only as a "go or no-go" decision or screening type test. The lack of a developed precision statement and/or excessively high test variability is an issue to be addressed with some of the performance tests utilized today. Work should continue with development of these precision statements for all developed performance tests.

Mix design, regardless of the level of effort or approach utilized, is only a first step in the process of achieving a long lasting pavement. During production, measures must be in place to ensure the produced mix matches the design mix and that performance requirements are maintained. There are several ways that may work in helping to achieve this goal. Below are some options for consideration.

Control with Established Volumetrics

In the OMEGA approach, performance will dictate design with mix volumetrics calculated at the selected binder content, which provides the necessary performance. This may result in air voids that are different than the conventional 4% level or a VMA that is different than that typically specified for a given nominal maximum aggregate size (NMAS). Differing volumetrics from historical requirements are not negative or detrimental to performance. Volumetric property requirements were initially developed to help ensure performance, but now performance is being directly measured with volumetrics calculated at the required performance level.

Control with Surrogate Performance Testing and Established Volumetrics

Some performance tests, especially the durability/cracking tests, may not be directly suitable for production related testing due to a lack of widespread equipment availability, test specimen fabrication time, extended testing time, etc. With most of the monotonic (i.e., consistent loading) performance tests, the specimen preparation time is the factor that drives the testing time. Other repeated load or cyclic tests (e.g., BBF) can often have much longer testing times than the monotonic tests.

Producers need to know the mix acceptance decision early on so that any needed corrective action can be completed in a prompt manner. Long performance testing times are highly undesirable during production operations. In this case, there is potential for the use of a surrogate test to quickly evaluate the produced mix acceptance. There are many possibilities for a surrogate test (e.g., indirect tensile strength, SCB, Cantabro, etc.), but the key is to conduct enough testing with the selected surrogate test to establish a good correlation (confidence) to the performance test used in design.

Additionally, with this approach, the production mix volumetrics would be evaluated against the baseline volumetric properties determined during design. The design volumetrics may not be the same as historical requirements. For example, if the optimum binder content was selected and the air voids and VMA were 3.5% and 15.2%, respectively, these values would become the “target.”

Control with Mix Design Performance Testing and Established Volumetrics

In some cases for larger tonnage jobs, it may be possible to test the produced mixture with the same performance testing used during design at a defined frequency. Currently this approach is used, in limited projects, with a typical frequency being 1 test per 10,000 tons. This approach is based on a “go/no-go” decision basis with volumetric properties calculated and compared against established baseline volumetric properties developed during mix design, as discussed previously.

CONCLUSIONS

The asphalt industry must continue with theoretical research/modeling efforts, but, at the same time, not be afraid to utilize practical approaches to find much needed, more immediate solutions. The foremost goal should be to move incrementally in the appropriate direction to limit risk of mix performance issues. Advancements toward an optimized mix design approach should be undertaken to help ensure long lasting asphalt pavements. The design approach presented highlights the need for specifications which allow innovation by producers but places mix performance as the forefront requirement.

REFERENCES

1. U.S. Army Corps of Engineers. Appendix 1. *Hot-Mix Asphalt Pavement Handbook 2000*. AC 150/5370-14A. National Research Council, 2000.
2. Oldcastle Materials Company. Internal Company Survey (unpublished), 2015.
3. Bonaquist, R. Impact of Mix Design on Asphalt Pavement Durability. *Transportation Research Circular E-C186*, Transportation Research Board of the National Academies, Washington, D.C., 2014, pp. 1 - 17.
4. Huber, G. History of Asphalt Mix Design in North America, Asphalt Institute. http://asphaltmagazine.com/history-of-asphalt-mix-design-in-north-america-part-1/. Accessed September 1, 2015.
5. Zhou, F., and D. Newbomb. NCHRP Project 9-57: Laboratory Tests to Assess Cracking Resistance of Asphalt Mixtures. Presentated at FHWA Asphalt Mixture Expert Task Group Meeting, April 2016, Salt Lake City, Utah. http://www.asphaltpavement.org/PDFs/Engineering_ETGs/Mix_201604/04%20Newcomb%209-57%20Presentation.pdf

Application of Performance Tests During Asphalt Mixture Design

Louisiana's Approach

Samuel B. Cooper, III
Louisiana Transportation and Research Center

Louay N. Mohammad
Louisiana State University

Conventional asphalt mixture design methodologies such as Superpave, Marshall, and Hveem are used to determine the optimum asphalt content by means of empirical laboratory measurements (Zhou et al., 2006). Marshall and Hveem mixture design procedures utilize both volumetric computation and stability measurements, while Superpave requires a volumetric and densification criteria evaluation of the mixture. Superpave was implemented to address the inadequacies of the Marshall and Hveem procedures. However, there is a need to develop laboratory tests to complement the Superpave procedure (Pellinen, 2004).

Fatigue cracking, permanent deformation (rutting), and thermal cracking are three major modes of distress to consider in asphalt concrete pavements (Monismith, 1992). A proper mixture design should consider these distresses where applicable. This may be accomplished through mechanistic laboratory evaluation of the mixture (Monismith et al.). The concept of mixture performance evaluation as a part of mixture design is not a new concept (Monismith, 1992; Brown, 1980; Brown et al., 1985). However, much of the consideration in the 1980s and 1990s was given to rutting resistance of the asphalt pavement layers. To address the rutting concern, asphalt mixtures were produced with less asphalt content, stiffer binder, and coarser aggregate structures. These changes led to increased cracking, reduced durability, and workability issues of asphalt mixtures (Zhou et al., 2006). In addition, the recent use of recycled materials and sustainable practices have further strained the capabilities of volumetric mixture design, thus increasing the importance of laboratory evaluation during the design of asphalt mixtures (Elseifi et al., 2011).

An important component to successful mixture design is the balance between volumetric composition and material compatibility (Pellinen, 2004). Laboratory testing, capable of ascertaining an asphalt mixture's internal compatibility is necessary to complement current design methodologies. To accomplish this, mechanistic laboratory testing that can determine a mixture's resistance to common distresses should be conducted.

The Louisiana Department of Transportation and Development (LADOTD) contracted 10.4 million tons of Superpave asphalt mixture from April 2009 to June 2013 which cost a total of $780 million, or nearly $200 million per year. With significant financial and temporal investment in asphalt pavement systems, it is critical to ensure the pavement will meet performance expectations and provide years of service. To address this concern, LADOTD has made efforts to improve conventional asphalt mixture procedures through specification modification.

For Louisiana mixtures, which are typically rut-resistant, balanced-mixture design commonly results in increased asphalt content. In 2016, LADOTD has implemented new specification requirements to increase the asphalt content of asphalt mixtures. This was accomplished by reducing the number of gyrations at N_{design}, as well as increasing the minimum

VMA and voids filled with asphalt (VFA) requirements. This paper documents Louisiana's experience with the development of a balanced mixture design by complementing volumetric criteria with the Hamburg loaded wheel tester (HLWT) and SCB tests for high-temperature and intermediate-temperature performance, respectively.

OBJECTIVES AND SCOPE

The objective of this study was to evaluate the effects of the 2016 LADOTD specification modification on the laboratory performance of asphalt mixtures. Mixtures were produced in accordance with newly implemented specifications to achieve a balance with respect to rutting and fatigue cracking. Eleven plant-produced mixtures were collected from six field projects using the newly implemented balanced specification criteria. HLWT and SCB data were compared between mixtures produced under the new specification with that of mixtures produced using the previous specification criteria. Mixture details are provided in the methodology section of this report.

BACKGROUND

Balanced Mixture Design

Studies have shown achieving mixture designs that satisfy rutting, cracking, and volumetric criteria is possible (Zhou et al., 2006; Zamhari et al., 1998; Walubita et al., 2013; Zhou et al., 2007; Scullion, 2010; Blankenship et al., 2010; Cooper et al., 2014). Walibuta et al. (2013) conducted extensive laboratory and field testing of asphalt mixtures constructed in accordance with Texas Department of Transportation (TxDOT) specifications. The research included the development of specification criteria modification to generate more-balanced mixtures. The HLWT was used to evaluate rutting potential while the Texas OT was used to evaluate resistance to fatigue cracking. Accelerated testing was conducted to evaluate field performance of the mixtures. Results of the experimental program indicate the balanced mix design (BMD) method resulted in mixtures with superior cracking resistance and constructability when compared to conventionally designed mixtures (Walubita et al., 2013).

Zhou et al. (2007) evaluated the effects of BMD procedures on 11 commonly used TxDOT mixtures. The mixtures were designed to meet HLWT and OT in addition to TxDOT volumetric criteria. The study found BMD methodologies typically resulted in higher optimum asphalt content as compared to volumetric analysis alone. Overall, the research stated balanced mixtures are achievable provided acceptable materials (i.e., aggregates, and asphalt cement) are used in the mixture design process (Zhou et al., 2007).

Scullion (Scullion, 2010) further evaluated the use of BMD methodologies for crack attenuating mixtures (CAM). The research concluded a CAM with asphalt content of 8.3% under conventional design methodologies experienced a reduction in optimum asphalt content (7.5%) under BMD methodology. The research also noted a balanced mixture was not achieved when using a PG 70- 22 binder. However, a balanced mixture was achieved utilizing a PG 76-22 binder (Scullion, 2010).

Blankenship (2010) evaluated the effect of increasing the density of a mixture to improve laboratory performance by increasing the design asphalt content. The mixture was evaluated using beam fatigue, dynamic modulus, and flow number. The research concluded a more balanced mixture could be achieved through increase density and asphalt content (Blankenship et al., 2010).

Cooper et al. (2014) conducted preliminary research evaluating the impacts of specification modification for an improved BMD. Cooper et al. conducted laboratory evaluation using pilot specifications for LADOTD to determine whether the mixtures designed would be balanced. The research showed that the adjustments to the volumetric requirements resulted in an increase of balanced mixture, when compared to previous specification criteria.

A balance of both rut and crack resistance in response to the traffic loads and environment conditions is required by the pavement to perform well in the field. Controlling volumetric properties of asphalt mixture is not enough to ensure good pavement performance, as often pavements do not perform as designed. A possible solution would be the development of laboratory test procedures to evaluate the as-built pavement qualities to predict pavement performance and life.

Selection of Mechanical Tests

There are several factors to consider when determining a suitable mechanical test for distress mitigation. The following factors were used by LADOTD for laboratory performance test evaluation:

- Measure/relate to fundamental properties,
- Simple, repeatable, easily calibrated,
- quick, not requiring highly trained personnel,
- Can utilize low-cost equipment,
- Sensitive to subtle changes in mixture properties, and
- Relate to pavement performance, criteria

Rutting Resistance

Numerous state transportation agencies use a version of the Loaded Wheel Test (LWT) to evaluate rutting potential and moisture susceptibility of asphalt mixtures (Izzo et al., 1999; Cooley Jr. et al., 2000). This test has shown potential as a verification tool for mixture design as well as quality control–quality assurance (QC/QA) practices. Since 2004, TxDOT has successfully included the LWT (Hamburg type) in their standard specification for hot-mix asphalt (HMA) pavement (TxDOT, 2004). TxDOT specifications allow a maximum rutting value of 12.5 mm at 20,000, 15,000, and 10,000 passes for mixtures containing PG 76-22, PG 70-22, and PG 64-22 binders respectively (TxDOT, 2004).

Additionally, LADOTD has implemented the use of HLWT test during mixture design approval, validation and quality control. Mohammad et al conducted research regarding performance-based specification implementation for LADOTD (Mohammad et al., 2016). The research found a suitable correlation between LWT rut depth and field performance. Mohammad et al. recommended maximum HLWT rut depths of 10 and 6 mm at 20,000 passes for medium traffic and high traffic respectively (Mohammad et al., 2016).

Intermediate Temperature Cracking Resistance

Similar to rutting, fatigue cracking of HMA pavement is another major concern. The fatigue cracking process includes two phases: (1) crack initiation in which micro-cracks grow from microscopic size until a critical length is obtained and (2) crack propagation, where a single crack or a few cracks grow until the crack(s) reach the pavement surface. Both micro-cracks and macro-cracks can be propagated by tensile or shear stresses or their combinations. However, there is a lack of rapid, simple, practical, and performance-related test procedure to characterize the crack resistance of asphalt mixtures.

The SCB test, however, adopted by Mohammad et al. (2004), has shown promise to predict the fracture resistance of asphalt pavements. This test is a traditional strength of materials approach that accounts for flaws as represented by a notch of a certain depth that in turn reveals the resistance of the material to crack propagation. The fracture resistance of a material is represented by the term critical value of J-integral (J_c). Greater J_c values represent a better fracture resistance of the material. Note that, previous fracture resistance data from other studies (Mohammad et al., 2004; Mull et al., 2002) indicated that mixtures achieving J_c values of greater than 0.50 kJ/m^2 – 0.65 kJ/m^2 are expected to exhibit good fracture resistance in the field, Figure 1 (Kim et al., 2012).

LADOTD has implemented the use of HLWT and SCB tests to evaluate the balance of mixture designed with conventional volumetric criteria (Figure 2).

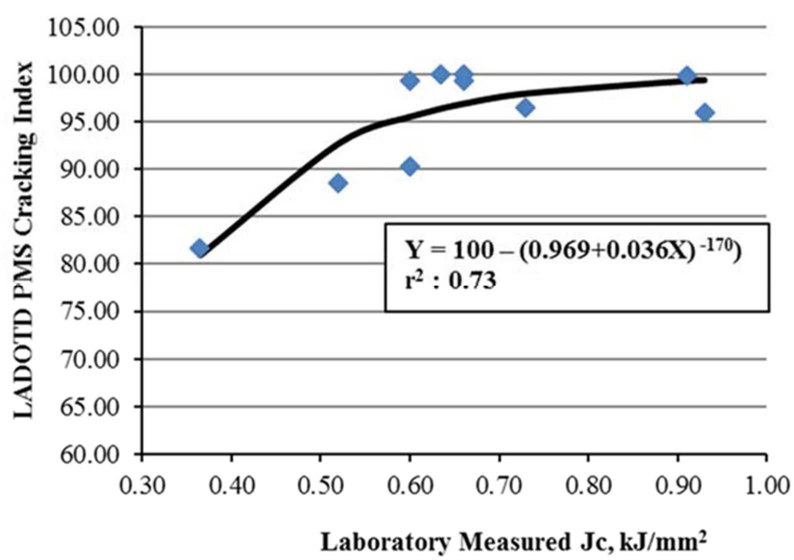


FIGURE 1 Measured J_c vs. field performance (Kim et al., 2012).

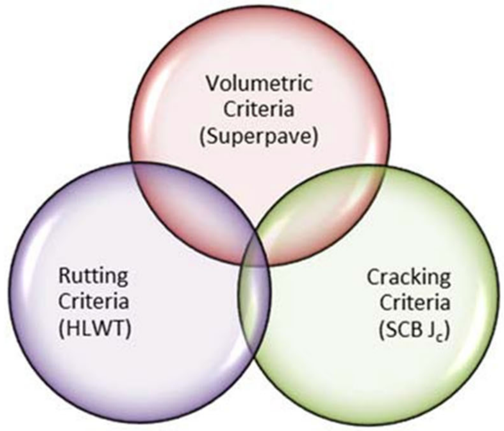


FIGURE 2 Balanced mixture objectives.

LADOTD Volumetric Mixture Design

The mixtures evaluated in this study were designed according to AASHTO TP 28: Standard Practice for Designing Superpave HMA and Section 502 of the 2006 Louisiana Standard Specifications for Roads and Bridges (Louisiana, 2006). The optimum asphalt cement content was determined based on volumetric (VTM = 2.5% – 4.5%, VMA ≥ 12%, VFA = 68% – 78%) and densification (%G_{mm} at $N_{initial}$ ≤ 89, %G_{mm} at N_{final} ≤ 98) requirements. Aggregates commonly used in Louisiana (siliceous limestone, granite, sandstone, river gravel, and coarse natural sand) were used in mix preparation. In addition, aggregate testing was conducted to verify their aggregate consensus properties. Consensus properties included coarse aggregate angularity, fine aggregate angularity, flat and elongated particles, and sand equivalency.

A new specification criterion implemented by LADOTD in 2016 was evaluated. Table 1 presents modifications to the LADOTD volumetric mixture design specifications. The required specifications are based on the type of mixture and its intended use (i.e., binder or wearing course, traffic level, etc.). LADOTD newly implemented specification changes increase the effective binder content of the mixtures to address cracking potential while considering possible impacts to rutting.

Project Description

The laboratory performance of 51 mixtures was evaluated using the HLWT and SCB test. Both laboratory- and plant-produced mixtures were evaluated. Of the 51 mixtures, 11 projects were selected to utilize mixtures designed to meet the criteria of Louisiana BMD methodologies as per 2016 LADOTD balanced mixture specifications. The remaining 40 mixtures were designed using conventional volumetric mixture design methodologies as per 2006 LADOTD specifications. Table 2 presents the 11 mixtures, from six field projects, designed under the 2016 LADOTD specification guidelines.

Figure 3 shows the locations of the six field projects. Five of the projects provided both binder and wearing courses, while the sixth project only consisted of wearing course.

TABLE 1 LADOTD Volumetric Specifications

Property	2016 LADOTD Specifications
N_{design}, gyrations	65 – 75a
Minimum VMA, %	10.5 – 13.0
VFA, %	69 – 80
Air voids, %	2.5 – 4.5
LWT required	Yes
SCB required	Yes

TABLE 2 Field Project Descriptions (Cooper et al., 2014)

Mixture Designation	Route	Mixture Level	NMAS, mm
LA3235BC	LA 3235	Binder	19.0
LA3235WC		Wearing	12.5
LA93BC	LA 93	Binder	19.0
LA93WC		Wearing	12.5
LA113BC	LA 113	Binder	25.0
LA113WC		Wearing	12.5
LA519WC	LA 519	Wearing	12.5
US80BC	US 80	Binder	19.0
US80WC		Wearing	12.5
LA16BC	LA 16	Binder	19.0
LA16WC		Wearing	12.5

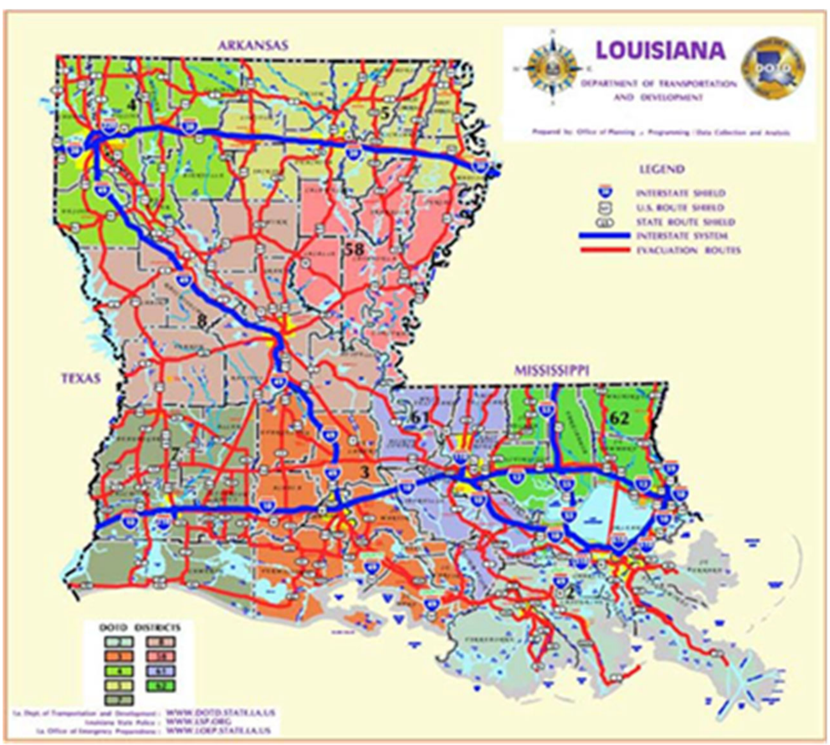


FIGURE 3 Field project locations (Cooper et al., 2014).

LADOTD Balanced Mixtures

Figure 4 presents the design gradations of the 11 mixtures formulated under the 2016 LADOTD specification. As shown in the figure, there were six 12.5-mm mixtures, four 19-mm mixtures, and one 25-mm mixture. In general, the mixtures were designed in the fine side of the maximum density line. Table 3 presents the design job mix formulas. There was an increase in the values of VMA (+0.5%) and VFA (+2%). In addition, the film thickness and asphalt content is greater than that of mixtures meeting the 2006 LADOTD specification criteria. Also, the LA 113 mixtures did not contain RAP.

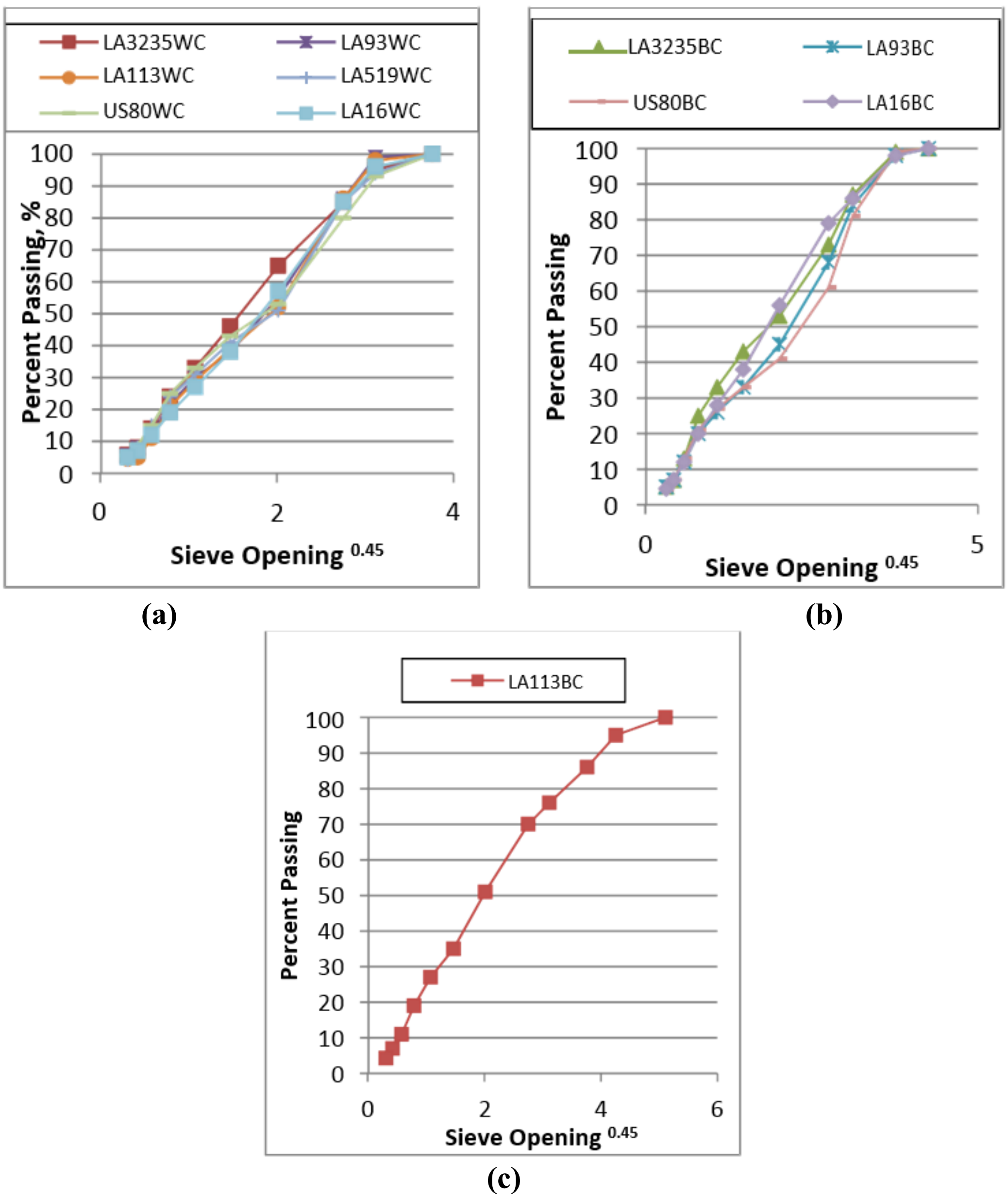


FIGURE 4 Field project gradations (Cooper et al., 2014): (*a*) 12.5 mm NMAS; (*b*) 19.0 mm NMAS; and (*c*) 25.0 mm NMAS.

TABLE 3 Job Mix Formula (Cooper et al., 2014)

Mixture Designation	LA3235 BC	LA3235 WC	LA93 BC	LA93 WC	LA113 BC	LA113 WC
Mix type (mm)	19.0	12.5	19.0	12.5	25.0	12.5
Binder type	PG 70-22 M	PG 70-22 M	PG 64-22	PG 70-22 M	PG 70-22 M	PG 70-22 M
Binder content (%)	4.4	5.2	4.2	4.6	3.7	4.6
G_{mm}	2.447	2.416	2.505	2.481	2.532	2.501
% G_{mm} at N_{Ini}	90.5	89.6	88.5	88.5	87.6	88.6
% G_{mm} at N_{Max}	96.5	97.2	97.3	97.5	97.5	97.7
Design air void (%)	3.4	3.5	3.5	3.5	3.5	3.5
VMA (%)	13.0	14.2	13.0	14.0	11.8	13.7
VFA (%)	74	75	73	75	70	74
Sieve Size	**Composite Gradation Blend**					
37.5 mm	100	100	100	100	100	100
25.0 mm	100	100	100	100	95	100
19.0 mm	99	100	98	100	86	100
12.5 mm	87	95	84	99	76	98
9.5 mm	73	85	68	86	70	86
4.75 mm	53	65	45	55	51	52
2.36 mm	43	46	33	38	35	39
1.18 mm	33	33	26	30	27	29
0.600 mm	25	24	20	22	19	21
0.300 mm	13	14	12	12	11	11
0.150 mm	7	8	7	8	7	5
0.075 mm	5.1	5.8	5.1	5.6	4.3	4.6
D:A	1.2	1.2	1.3	1.2	1.2	1.1
Tf, micron	7.6	8.0	8.0	8.2	7.4	9.2

continued on next page

TABLE 3 (*continued*) **Job Mix Formula (Cooper et al., 2014)**

Mixture Designation	LA519 WC	US80 BC	US80 WC	LA16 BC	LA16 WC
Mix type (mm)	12.5	19.0	12.5	19.0	12.5
Binder type	PG 70-22 M	PG 70-22 M	PG 70-22 M	PG 82-22 CRM	PG 82-2 CRM
Binder content (%)	5.2	4.4	5.1	4.9	5.5
G_{mm}	2.456	2.493	2.467	2.376	2.371
% G_{mm} at N_{Ini}	89.2	89.7	89.1	88.9	55.2
% G_{mm} at N_{Max}	97.7	97.4	97.4	97.2	97.3
Design air void (%)	3.2	3.5	3.5	3.5	3.5
VMA (%)	14.9	13.5	14.8	14.0	15.6
VFA (%)	79	74	76	75	78
Sieve Size	**Composite Gradation Blend**				
37. 5 mm	100	100	100	100	100
25.0 mm	100	100	100	100	100
19.0 mm	100	99	100	98	100
12. 5 mm	94	81	93	86	96
9. 5 mm	85	61	80	79	85
4. 75 mm	51	41	53	56	57
2. 36 mm	41	33	43	38	38
1. 18 mm	31	27	33	28	27
0.600 mm	24	21	25	20	19
0.300 mm	15	13	15	12	12
0.150 mm	8	7	9	7	7
0.075 mm	4.3	4.4	5.7	4.6	5.0
D:A	0.8	1.0	1.2	1.0	0.9

NOTE: BC = binder course; WC = wearing course; M = elastomeric polymer modified; CRM = crumb rubber modified; D:A = dust to effective asphalt ratio; Tf = film thickness.

Experimental Program

Triplicate specimens were prepared for testing, except for the LWT where two specimens were tested. All specimens were compacted to an air void level of 7.0% ± 0.50%. Results of the tests had a coefficient of variation (COV) of 20% or less. A brief description of each of the test methods considered are presented in the following sections.

Hamburg Loaded Wheel Tester

Rutting performance of the mix was assessed using an HLWT, manufactured by PMW, Inc., of Salina, Kansas. This test was conducted in accordance with AASHTO T 324: Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot Mix Asphalt (HMA). This test is considered a torture test that produces damage by rolling a 703-N (158-lb) steel wheel across the surface of a specimen that is submerged in 50°C water for 20,000 passes at 56 passes a minute. A maximum allowable rut depth of 6 mm at 20,000 passes at 50°C was used. The rut depth at 20,000 cycles was measured and used in the analysis (AASHTO T 324).

The HLWT may also be used to evaluate the moisture sensitivity of the mixture. The Stripping Inflection Point (SIP), calculated from LWT test results can be used to determine the stripping potential of HMA mixtures. SIP is the number of wheel passes at which a sudden increase in rut depth occurs, (e.g., tertiary flow occurs). The SIP is related to the mechanical energy required to produce stripping; therefore, a higher stripping inflection point indicates that a mixture is less likely to strip.

Semi-Circular Bend Test

Fracture resistance potential was assessed using the SCB approach proposed by Wu et al. (Louisiana, 2006). This test characterizes the fracture resistance of asphalt mixtures based on fracture mechanics principals, the critical strain energy release rate, also called the critical value of J-integral, or J_c. Figure 5 presents the three-point bend load configuration and typical test result outputs from the SCB test. To determine the critical value of J-integral (J_c), semicircular specimens with at least two different notch depths need to be tested for each mixture. In this study, three notch depths of 25.4, 31.8, and 38 mm were selected based on an a/rd ratio (the notch depth to the radius of the specimen) between 0.50 and 0.65. Test temperature was selected to be 25°C. The semicircular specimen is loaded monotonically until fracture failure under a constant cross-head deformation rate of 0.5 mm/min in a three-point bending load configuration. The load and deformation are continuously recorded and the critical value of J-integral (J_c) is determined using the Equation 1 (Wu et al., 2005):

$$J_c = \left(\frac{U_1}{b_1} - \frac{U_2}{b_2}\right)\frac{1}{a_2 - a_1} \tag{1}$$

where

b = sample thickness, mm;
a = the notch depth, mm; and
U = the strain energy to failure, K_n mm.

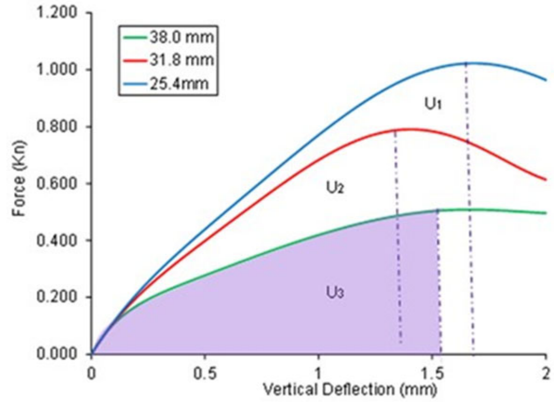


FIGURE 5 The semicircular bending test.

RESULTS AND ANALYSIS

Rutting Resistance

Figure 6*a* presents the results of the HLWT test results at 50°C for the mixtures evaluated in this study. Mixtures designed according to the new LADOTD specifications are indicated by star symbols. In general, mixtures designed according to the 2006 and the new LADOTD specifications performed well in the HWLT test with a mean rut depth of less than 6.0 and 10.1 mm at 20,000 passes. The 10.0-mm criterion is used for mixtures containing unmodified PG 64-22 binder, while the 6.0-mm criterion is used for modified binders. The 11 mixtures that were designed according to the new specifications (indicated by star symbols) exhibited improved or similar performance with respect to rut resistance as measured by the HLWT. In addition, the 11 mixtures produced under the new specification criteria did not exhibit tertiary flow, thus do not exhibit moisture susceptibility as indicated by the HLWT. Therefore, the newly implemented LADOTD specification modifications do not appear to have adversely affected the rutting resistance of the mixtures. In addition, mixtures containing polymer-modified binders (i.e., PG 70-22M and PG 76-22M) resulted in the improved performance when compared to unmodified binders (i.e., PG 64-22). Figure 7 presents the average rut depths by binder grade. The figure shows a decrease in rut depth with increase in high temperature grade of the binder. This is to be expected as the HLWT was conducted at a single temperature (50°C) regardless of binder grade.

Fatigue Cracking Resistance

Figure 6*c* presents the SCB test data generated for this study. The minimum passing criterion used in this analysis is 0.5 kJ/m2 (Kim et al., 2012). Mixtures designed according to the new LADOTD specifications are indicated by star symbols. This figure shows nearly 50% of the pilot mixtures met or exceeded the cracking criteria. However, historically mixture containing PG 70-22M binder met the criteria at the same percentage (50%). In general, mixtures containing elastomeric type of polymer modified binder (PG 76-22M) outperformed mixtures containing other binders. In addition, mixtures containing crumb rubber modifiers should be monitored

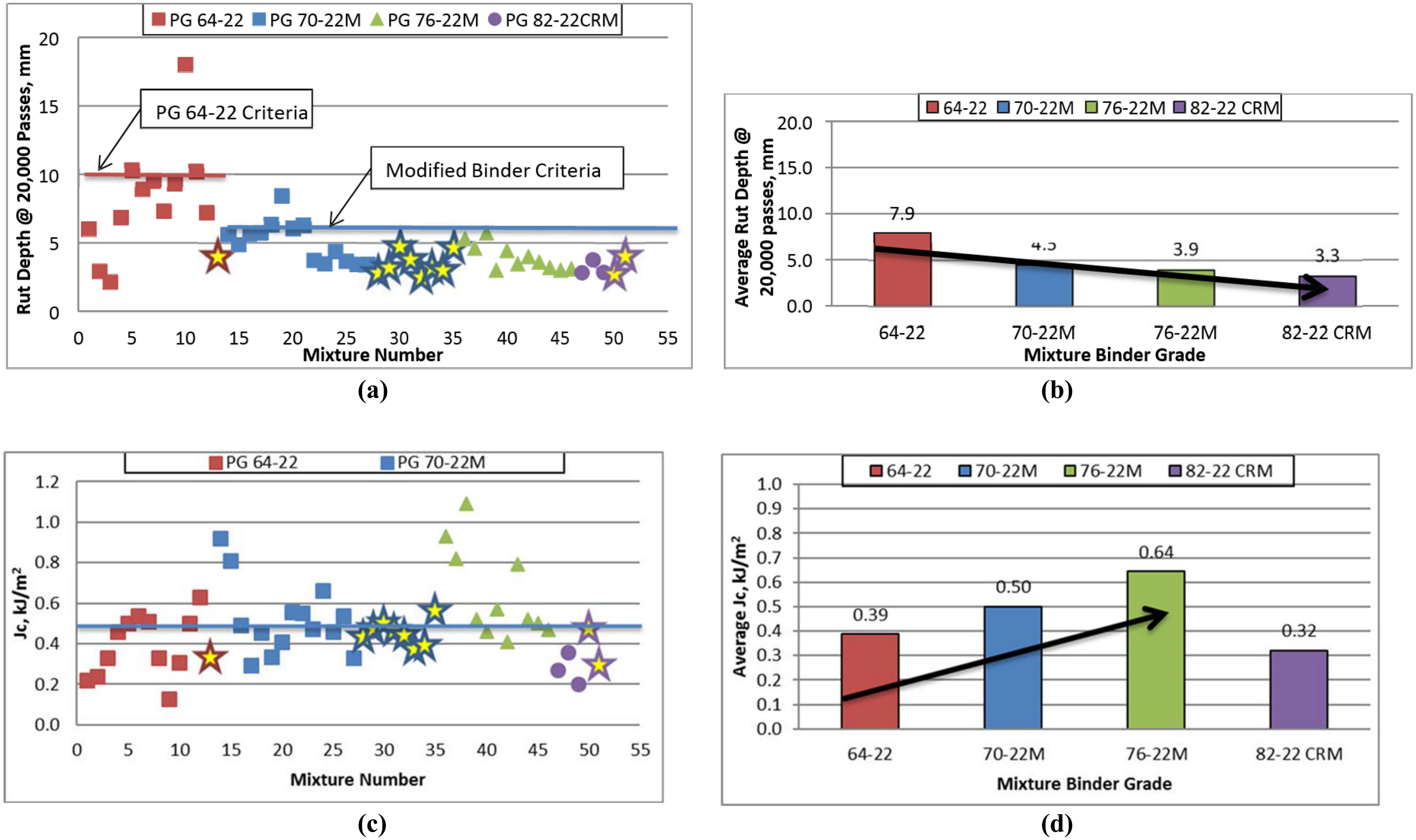


FIGURE 6 (*a*) **HLWT test results;** (*b*) **HLWT test results: binder grade comparison;** (*c*) **SCB test results; and** (*d*) **SCB test binder comparison (Cooper et al., 2014).**

closely as the base binder is a PG 64-22. Figure 6*d* presents the J_c values comparison with respect to binder grade. This figure identifies the effect of binder grade on cracking resistance as measured by the SCB test. The improved cracking resistance may be attributed to the elastomeric polymer modifiers used in the PG 70-22M and PG 76-22M binders. In general, mixtures containing no RAP exhibited improved J_c.

Balanced Mixture Analysis

Figure 7 presents the balanced mixture analysis for the 51 mixtures evaluated in this paper. The balanced region highlighted indicates mixtures that satisfied both rutting and fracture criteria. As shown in the figure, the mixtures designed using the newly implemented specification balanced 50% of the time. The mixtures produced under the new specification containing PG 64-22 binder did not balance. Mixtures designed according to the 2006 LADOTD specifications were balanced 52% of the time (PG 64-22-36%; PG 70-22M-50%; PG 76-22M-92%; PG 82-22CRM-0%). The percentage of PG 82-22CRM mixtures that balanced increased from 0% to 50%. However, the sample size for PG 82-22 CRM mixtures was limited.

SUMMARY AND CONCLUSIONS

The objective of this study was to evaluate the effects of LADOTD specification modification on the laboratory performance of asphalt mixtures. Mixtures were produced in accordance with newly implemented specifications to achieve a balance with respect to rutting and fatigue cracking. Eleven plant-produced mixtures were collected from six field projects using balanced specification criteria. HLWT and SCB data were compared between mixtures produced under

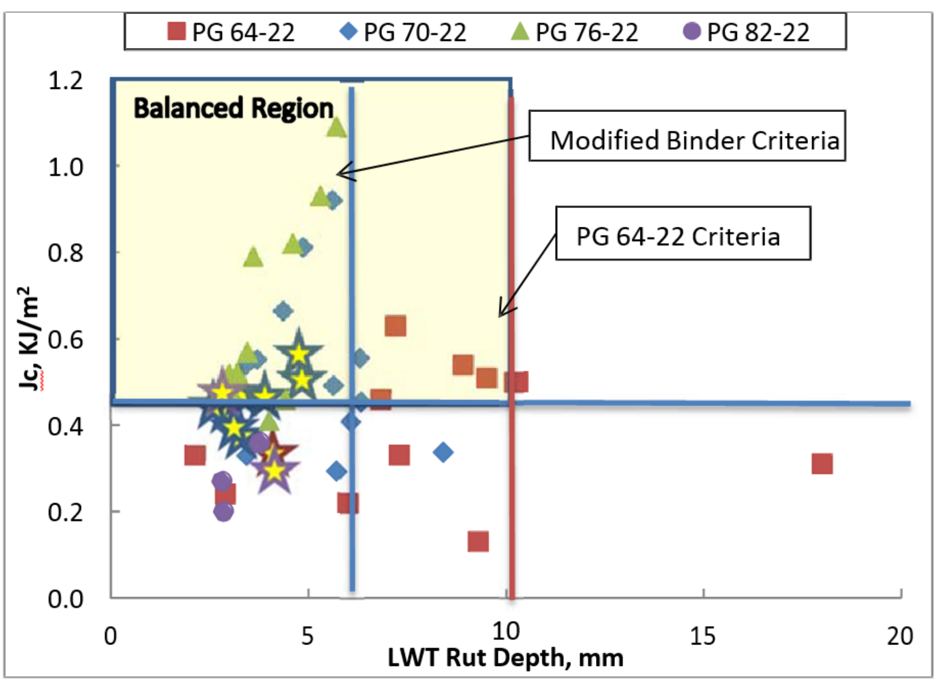


FIGURE 7 Balance mixture analysis (Cooper et al., 2014).

the new specification with that of mixtures produced using the 2006 specification criteria. Based on the results of the analysis, the following findings and conclusions may be drawn:

- With respect to rut resistance, the 11 mixtures produced using the 2016 LADOTD specifications exhibited improved or similar performance to mixtures produced using the 2006 LADOTD specification.
- Mixtures containing polymer modified binders (i.e., PG 70-22M and PG 76-22M) resulted in improved rutting performance when compared to unmodified binders (i.e., PG 64-22).
- Fifty percent of the mixtures designed according to the 2016 LADOTD specifications met or exceeded the cracking criteria of 0.5 kJ/m^2 as determined by the SCB test.
- Mixtures containing PG 76-22M modified binder outperformed the mixtures containing other binders (e.g., PG 64-22, PG 70-22M, and PG 82-22CRM).
- In general, mixtures containing no RAP exhibited improved J_c.

ACKNOWLEDGMENTS

This work was supported by the Louisiana Transportation Research Center in cooperation with the Louisiana Department of Transportation and Development and the Federal Highway Administration. The authors acknowledge the efforts of William Gueho, Patrick Frazier, and Jeremy Icenogle at Louisiana Transportation Research Center (LTRC) Asphalt Laboratory as well as the contributions of Engineering Material Characterization and Research Facility staff.

REFERENCES

Blankenship, P. B., and R. M. Anderson. Laboratory Investigation of HMA Modulus, Flow Number and Flexural Fatigue on Samples of Varying Density. *Journal of the Association of Asphalt Paving Technologists*, Vol 79, 2010, pp. 497–518.

Brown, S. F. *An Introduction to the Analytical Design of Bituminous Pavements*. Nottingham University, England, 1980, p. 51.

Brown, S. F., J. M. Brunton, and A. F. Stock. The Analytical Design of Bituminous Pavements. *Proc., Institution of Civil Engineers*, Vol. 79, Part 2, 1985, pp. 1–31.

Cooley Jr., L. A., P. S. Kandhal, M. S. Buchanan, F. Fee, and A. Epps. Loaded Wheel Testers in the United States: State of the Practice. *Transportation Research Circular E-C016*, 2000.

Cooper, S., L. Mohammad, S. Kabir, and W. King. Balanced Asphalt Mixture Design Through Specification Modification: Louisiana's Experience. *Transportation Research Record: Journal of the Transportation Research Board, No. 2447*, 2014, pp. 92–100.

Elseifi, M. A., Mohammad, L.N., and Cooper III, S. B. "Laboratory Evaluation of Asphalt Mixtures Containing Sustainable Technologies." *Journal of the Association of Asphalt Paving Technologists*, Volume 80, 2011

Izzo, R. P., and M. Tahmoressi. Testing Repeatability of the Hamburg Wheel-Tracking Device and Replicating Wheel-Tracking Devices among Different Laboratories." *Journal of the Association of Asphalt Paving Technologists*, Vol. 68, 1999, pp. 589-612

Kim, M., L. M. Mohammad, and M. A. Elseifi. Characterization of Fracture Properties of Asphalt Mixtures as Measured by Semicircular Bend Test and Indirect Tension Test. *Transportation Research Record: Journal of the Transportation Research Board, No. 2296*, 2012, pp. 115–124.

Louisiana Standard Specifications for Roads and Bridges. Louisiana Department of Transportation and Development, Baton Rouge, La., 2006.

Mohammad, L. N., M. Kim, and H. Challa. *LTRC Final Report 558: Development of Performance-Based Specification for Louisiana Asphalt Mixtures*. Louisiana Transportation Research Center, 2016, p. 129.

Mohammad, L. N., Z. Wu, and M. A. Aglan. Characterization of Fracture and Fatigue Resistance on Recycled Polymer–Modified Asphalt Pavements. *Proc.*, *5th International Conference*, Limoges, France, 2004, pp. 375–382.

Development and Implementation of the Illinois Flexibility Index Test

A Protocol to Evaluate the Cracking Resistance of Asphalt Mixtures

HASAN OZER
IMAD L. AL-QADI
University of Illinois at Urbana–Champaign

The types of materials used in flexible pavement construction has increased over the years, with the overall goal to reduce costs of construction. Recycled materials and a variety of modifiers and additives have been introduced at various stages of asphalt concrete (AC) material production. The most commonly used recycled materials in AC mixtures are reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS). However, the increased use of recycled materials or binder modifiers has raised concerns about the long-term durability of flexible pavements. For example, replacing virgin asphalt binder in AC mixtures with aged and stiffer binders poses numerous technical challenges in terms of AC mixture volumetrics and resistance to thermal (low-temperature) cracking, fatigue cracking, and other types of AC pavement deterioration.

Cracking in AC pavements is among the major driving modes of pavement deterioration. Even though the extent to which material properties can affect crack initiation and growth varies, material properties are always considered critical in determining the resistance of pavement to different modes of cracking. The two critical material characteristics that define the role of material in the two cracking stages are modulus and fracture, or damage-resistance properties.

With the introduction of a rutting performance test for AC acceptance a few years ago, cracking of AC pavements is now the most cause of pavement rehabilitation in Illinois. With existing pavement structures continuing to deteriorate and the occasional cold spikes in winter temperatures [–27°C (–16.6°F) in January 2014], fatigue, reflective, and thermal cracking resistance of AC are of paramount concern. Changing the material sources, especially stemming from the desire to be more sustainable, could increase the uncertainty in the value of historical performance, thereby hindering the estimation reliability of future pavement life cycles. Therefore, a distinct need existed for a comprehensive study to assess the impact of high RAP and/or RAS contents on critical AC performance criteria, such as thermal and fatigue cracking. In addition, a practical test suite and proven procedures are needed for screening AC mixes contained increased amounts of RAP and/or RAS to ensure that performance expectations are met.

The Illinois Center for Transportation (ICT) and Illinois DOT (IDOT) partnered in a series of research projects beginning in 2012 with a specific objective of identifying, developing, and evaluating protocols, procedures, and specifications for testing engineering properties of AC mixtures with varying amounts of asphalt binder replacement (up to 60%) using RAP and RAS, as well as a number of other AC mixtures with various field performance and mixture volumetrics. One of the major outcomes of this study was the development of the Illinois Flexibility Index Test (I-FIT) method and protocol that can rank AC mixtures based on their cracking resistance (Al-Qadi et al. 2015).

DEVELOPMENT OF THE ILLINOIS FLEXIBILITY INDEX TEST PROTOCOL

The SCB fracture testing protocol was developed to evaluate an AC mixture's overall resistance to cracking-related damage (Al-Qadi et al., 2015; Ozer et al., 2016a). The test was intended to be used at the AC mix design and production levels. In addition to the availability of off-the-shelf equipment, the following four criteria: (1) a statistically significant and meaningful spread in test outcome to distinguish between AC mixes based on their cracking resistance ability; (2) repeatability, practicality, low cost, and easy implementation by technicians; (3) correlation to other independent test methods and engineering intuition; and (4) correlation to field performance.

TEST METHOD DESCRIPTION

The I-FIT protocol, also known as Illinois SCB (IL-SCB), is conducted at an intermediate temperature [25°C (77°F)] using a custom-designed SCB fixture placed in a servo-hydraulic or pneumatic AC testing machine (AASHTO TP 124). The test is conducted using load-line displacement control at a displacement rate of 50 mm/min (Al-Qadi et al., 2015; Ozer et al., 2016a, 2016b). The main outcome of the test procedure is a parameter called the flexibility index (FI), along with fracture energy. Figure 1 shows a typical SCB specimen during the fracture and specimen geometry.

According to the work-of-fracture method (Hillerborg, 1985; Bazant, 1996), fracture energy is the area under the load-displacement curve until the specimen is broken. The area corresponds to the work done by load (P) on the load-point deflection (u). Assuming that all of the work of load P is dissipated by crack formation and propagation, this work would correspond to fracture energy. The method determines fracture energy, or more accurately, apparent fracture energy, because not all energy may be dissipated at the crack tip, as follows:

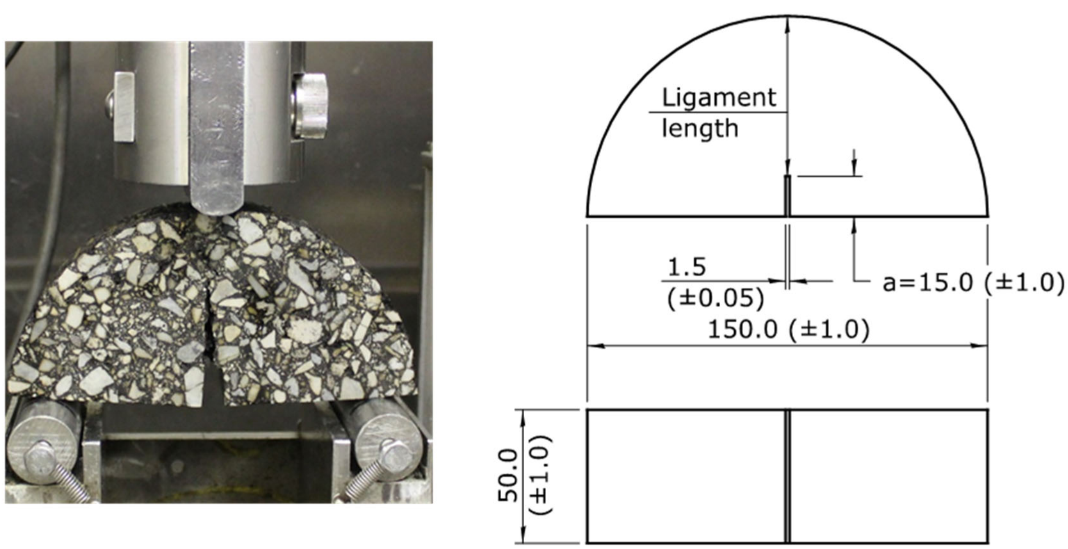


FIGURE 1 I-FIT test and specimen configuration (dimensions in millimeters).

$$G_{fa} = \frac{1}{b(D-a)}\left[\int_{0}^{u_o} P_1(u)du + \int_{u_o}^{u_{final}} P_2(u)du\right]$$

where $P_1(u)$ and $P_2(u)$ are the fitting equations before and after the peak, respectively; u_o is the displacement at the peak; and u_{final} is the final displacement that can be selected as the displacement at a cut-off load value where the test is considered at an end (usually taken as 0.1 kN). If desired, the load-displacement curve can also be extrapolated to calculate the remaining area under the tail part of the curve, which is generally less than 5% of the total area. The load-displacement curves for four AC mixes with varying degrees of recycled content (L3 to L6) are shown in Figure 2.

Displacement Measurements

The I-FIT method was developed to calculate significant parameters from the SCB test method and later drafted as a provisional AASHTO specification. In the development of the test method, the following additional considerations were taken into account. Additional details for each consideration can be found elsewhere (Al-Qadi et al., 2015, Ozer et al., 2016a, Doll et al., 2017).

- Robustness of fixture and compliance in recording actual specimen deformations (using digital image correlation).
- Existence of other dissipation mechanisms that might affect calculated fracture energy (using digital image correlation).
- Effect of specimen geometry on the load-displacement curve.
- Specimen conditioning method (water bath vs. oven conditioning).
- Repeatability of the selected displacement rate (variable rate and temperature testing).

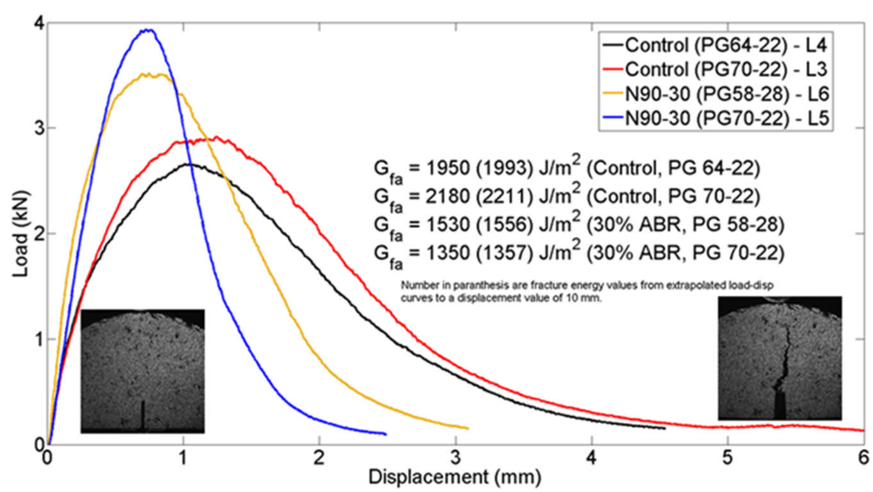


FIGURE 2 Typical load-displacement curves for AC mixes with varying asphalt binder replacement (ABR) (L3 and L4: control; L5 and L6: 30% ABR) and corresponding fracture energy calculated at cut-off displacement (short) and extrapolated (long) from the I-FIT tests (Al-Qadi et al., 2015).

The digital image correlation (DIC) technique was used to compare the two methods by measuring the displacement of DIC gauges at the surface of the specimen. The digital DIC gauges are representing the zones at the surface of the specimen where the displacement is averaged. The DIC gauge measuring the AASHTO TP 105-13 displacement is positioned as suggested by that method, while the DIC gauge for the loading head displacement is positioned directly under the loading head (Figure 3). The displacements measured through DIC were used to obtain the load-displacement curves and compare them to a load displacement obtained directly from the machine. The curves in Figure 3 show that the two measurements with the DIC are almost exactly the same. There are some minor differences from the measurements recorded with the load frame; those differences are probably due to the machine compliance. The results show that the loading head displacement method provides results similar to the AASHTO method. However, the AASHTO method has some drawbacks: it requires gauge points that can be on the crack path and may affect measurements, which can be avoided when using loading head measurements.

Flexibility Index Calculation

Fracture energy is not sufficient as the sole parameter to distinguish between AC mixtures. For example, Figure 4 illustrates a comparison of two AC mixes (control with no recycled materials and the same mix with 30% ABR using 7% RAS) tested at 50 mm/min (2 in./min) at a temperature of 25°C (77°F). The fracture energy values of the two AC mixes were nearly identical; however, the mixes had distinctive load-displacement characteristics that may significantly differentiate their cracking response. Hence, it is evident that fracture energy alone cannot be used to discriminate between the two AC mixes.

This conclusion can be attributed to the nature of the fracture energy parameter. Fracture energy is a function of both the strength (defined by peak load) and ductility (defined as the maximum displacement at the end of the test) of the material. If the material displays a high peak

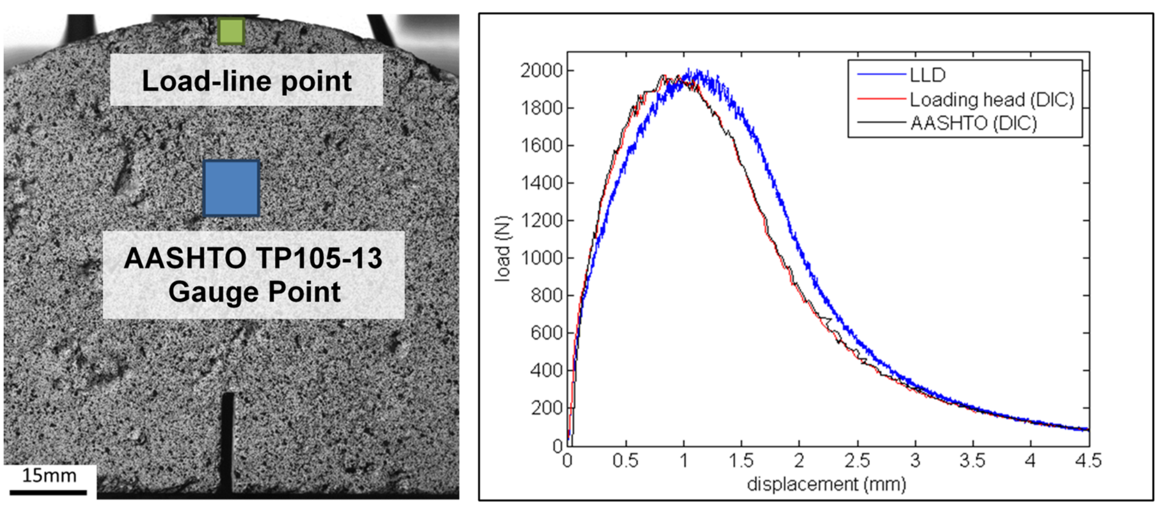


FIGURE 3 Comparison of load displacement using the two different displacement recording location on the SCB specimen (Al-Qadi et al., 2015).

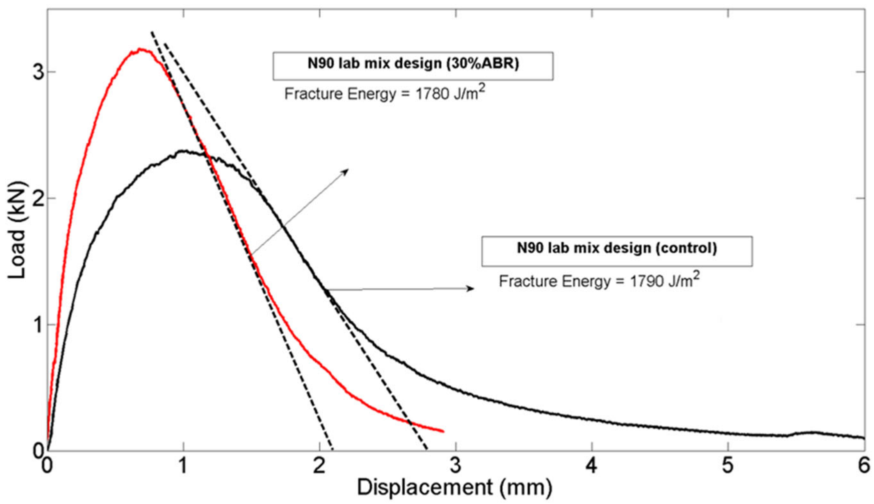


FIGURE 4 Major characteristics derived from load-displacement curves from the tests conducted at 25°C (77°F) and at 50 mm/min (2 in./min) displacement rate illustrating the potential effects of ABR (Al-Qadi et al., 2015).

load, it may compensate for the lack of ductility in the post-peak region of the load-displacement curve when fracture energy is calculated. This may explain the high fracture energy of brittle AC mixtures with high amounts of recycled content, compared with their counterparts with no recycled materials.

Therefore, the FI was introduced to capture the cracking resistance of AC mixes in a more robust and consistent way. Derived from the load-displacement curves obtained from the I-FIT method with parameters of fracture energy and slope at the post-peak inflection point, the FI describes the fundamental fracture processes consistent with the size of the crack tip process zone. The FI can capture the effects caused by various changes in the materials and volumetric design of AC mixes (Al-Qadi et al., 2015; Ozer et al., 2016a, 2016b). Plant-produced, laboratory-produced, and field core specimens were used in validating the I-FIT and the ability of FI to predict cracking resistance of AC mixes. The effects of increasing the RAP and RAS content were shown, with a reduction in the FI, indicating a more brittle behavior. The FI values varied from 15 to 1 for the best- and poorest-performing laboratory-produced AC mixtures, respectively. A typical load-displacement curve obtained from the I-FIT is shown in Figure 5 with the parameters calculated from the test.

The FI is calculated using the overall fracture energy normalized by the slope at the inflection point of the post-peak part of the load displacement curve. The inflection point indicates that crack propagation is slowing down.

$$\mathrm{FI} = \frac{G_{fa}}{|\mathrm{m}|} \times A$$

where |m| is the absolute value of the post-peak slope at the inflection point (reported as kN/mm); G_{fa} is fracture energy reported in joules/m^2 and represents the area under the load-displacement curve normalized by fractured area; and coefficient A is a unit conversion factor and scaling coefficient (0.01 is taken as the default).

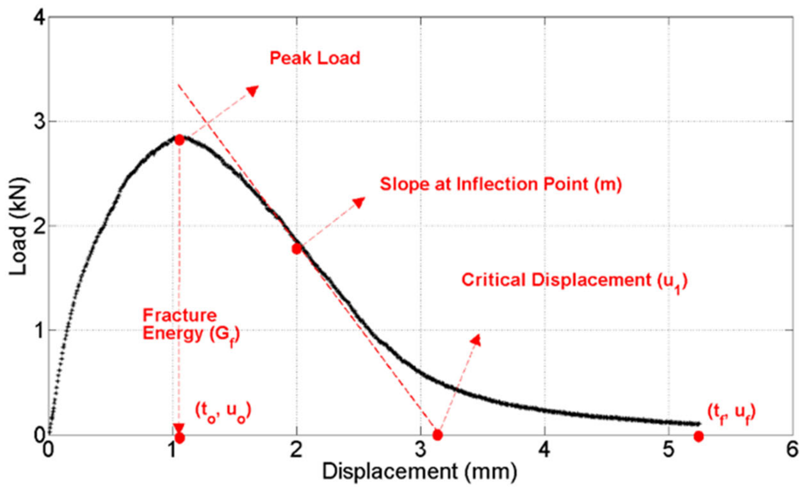


FIGURE 5 A typical outcome of the I-FIT illustrating the parameters derived from the load-displacement curve including peak load (can be related to tensile strength), critical displacement, slope at inflection point, displacement at peak load, and fracture energy.

FLEXIBILITY EVALUATION OF VARIOUS MIXES

Mixes with Varying Asphalt Binder Replacement

The effect of varying degrees of ABR was assessed using the I-FIT protocol and FI. Various mixes were developed from a parent control mix design with the addition of RAP and RAS increasing up to 60% (Table 1). The objective was to evaluate the flexibility of the AC mixes with increasing ABR with RAP and RAS together or RAS only, as well as to determine the effectiveness of binder grade bumping to compensate for the presence of recycled stiff binder. An N90 wearing surface dense-grade AC mix design was used.

The FI and fracture energy values are shown in Figure 6 for the tested AC mixes. The values were normalized with respect to the control AC mix with PG 70-22. The overall pattern with the FI was consistent reduction with increasing ABR. The reduction was much more pronounced when it was compared with fracture energy values obtained at the same temperature. Some of the key findings from the comparison of FI values for various AC mixes are as follows (see Table 2 for details):

- The worst FI values is that of L5 (N90, 30% ABR with PG 70-22) and L10 (N90, 60% ABR with PG 52-34).
- Mixes with similar ABR content and same binder type but different proportions of RAP and RAS (L6, L9, L12, and L13) had similar FI values, also indicating that RAS source does not have a significant impact.
- The changes in binder grade had significant impact on the FI values. For example, AC mixes with the same ABR and stiffer binder [L5 (N90, 30% ABR with PG 70-22)] had significantly lower FI values compared with AC mixes having a softer binder [L6 (N90, 30% ABR with PG 58-22)].

TABLE 1 Laboratory AC Mix Designs Used in This Study for I-FIT (Al-Qadi et al., 2015).

Mix ID	Mix Name	Binder Grade	RAP (%)	RAS (%)	ABR (%)	AC (%)	VMA (%)
L3	N90-0[1] CG[2]	70-22	—	—	—	6.0	15.3
L4	N90-0 CG	64-22	—	—	—	6.0	15.3
L5	N90-30 CG S1[3]	70-22	—	7	29.8	6.0	15.3
L6	N90-30 CG S1	58-28	—	7	29.8	6.0	15.3
L7	N90-20 CG S1	58-28	—	5	21.2	6.0	15.3
L8	N90-10 CG S1	64-22	—	2.5	10.5	6.0	15.3
L9	N90-30 CG S2[4] AS[5]	58-28	11	5	30.5	6.0	15.2
L10	N90-60 CG S2 AS	52-34	40	7	60.8	6.1	15.2
L11	N90-0 CG AS	64-22	—	—	—	6.0	15.3
L12	N90-30 CG S2[4] AS[1]	58-28	—	7	30.6	6.0	15.2
L13	N90-30 CG S1[3] AS[1]	58-28	—	7	29.8	6.0	15.3

[1] N90-0, N90-20, N90-30, and N90-60 indicate N_{design} and ABR percentage.
[2] CG = coarse graded.
[3] S1 = RAS source.
[4] S2 = RAS source.
[5] AS = Mixture with 1% anti-strip added to virgin binder.

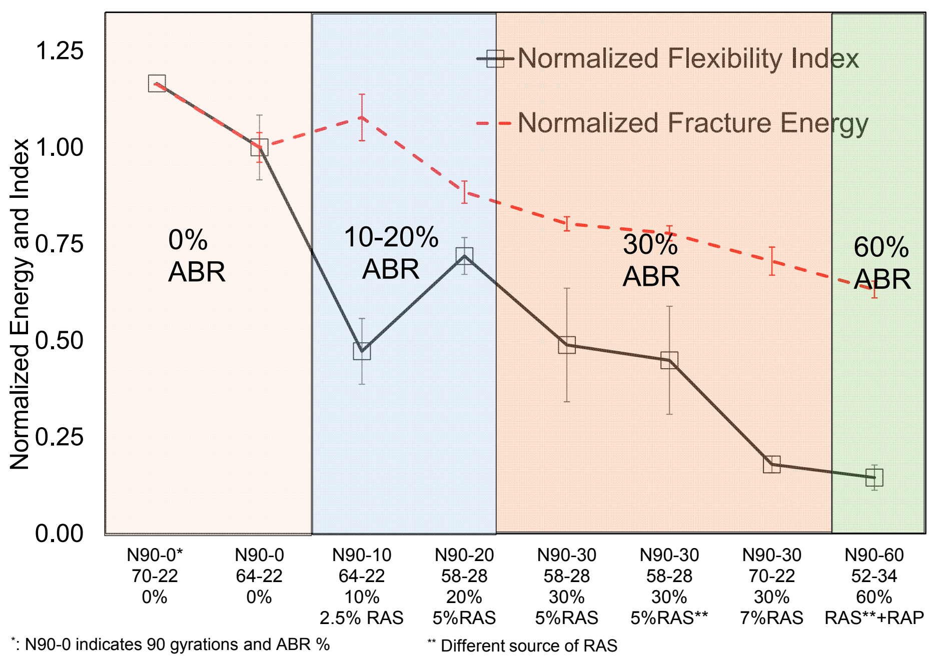


FIGURE 6 Comparison of normalized fracture energy with normalized FI for N90 design AC mixes with varying ABR (in percent) obtained using various combinations of RAS and RAP (Al-Qadi et al., 2015).

TABLE 2 FI and Fracture Energy for the Laboratory-Produced AC Mixtures, Illustrating the Effect of ABR (Al-Qadi et al., 2015)

Mix ID	Binder Grade	ABR (%)	RAP (%)	RAS (%)	G_{fa} (J/m^2)	COV (%)	FI	COV (%)
L3	70-22	—	—	—	2307	3	16	5
L4	64-22	—	—	—	1944	8	13	14
L5	70-22	29.8	—	7	1418	4	2	11
L6	58-28	29.8	—	7	1503	5	5	20
L7	58-28	21.2	—	5	1718	4	9	4
L8	64-22	10.5	—	2.5	2019	6	6	20
L9	58-28	30.5	11	5	1642	4	4	15
L10	52-34	60.8	40	7	1374	16	2	18
L11	64-22	—	—	—	1465	11	13	5
L12	58-28	30.6	—	7	1442	5	5	12
L13	58-28	29.8	—	7	1541	15	3	8

I-FIT Variability and Discrimination Potential Between AC Mixes

The discrimination potential of the FI between AC mixes is shown in Figure 7. This illustrates the results of comparisons with the fracture test parameters obtained from I-FIT conducted at different temperatures. Laboratory-produced AC mixes were used in the comparison of low-temperature fracture energy, intermediate-temperature fracture energy, and FI. The differences in each fracture parameter's discrimination potential is shown by the contrasts in the overlap between each probability curve. Accordingly, the FI has the greatest discrimination potential between the laboratory-produced AC mixes when ABR level and/or binder grade changes. There is low to no overlap between the AC mixes illustrated in the figure for the FI. Another consideration is the COV for the FI. It was observed that the FI generally has a somewhat higher COV, which is expected because the FI is derived from the shape of the post-peak segment of the load-displacement curve characteristics, whereas fracture energy represents an average value derived from the same area under the same curve (e.g., an average integrated quantity). It is expected that the FI would be sensitive to density changes in the specimen, operator variability, and other random material or equipment variability. The COV values for FI are within the range of 10% to 20% at an average of 8.8% for the plant-produced mixes and 12% for the laboratory-produced mixes (Al-Qadi et al., 2015). The ability of FI to distinguish between AC mixes is evident as shown in the least overlap, in spite of the relatively higher COV. On the other hand fracture energy could not distinguish between AC mixes having various levels of ABR; hence, the COV becomes irrelevant factor for comparison.

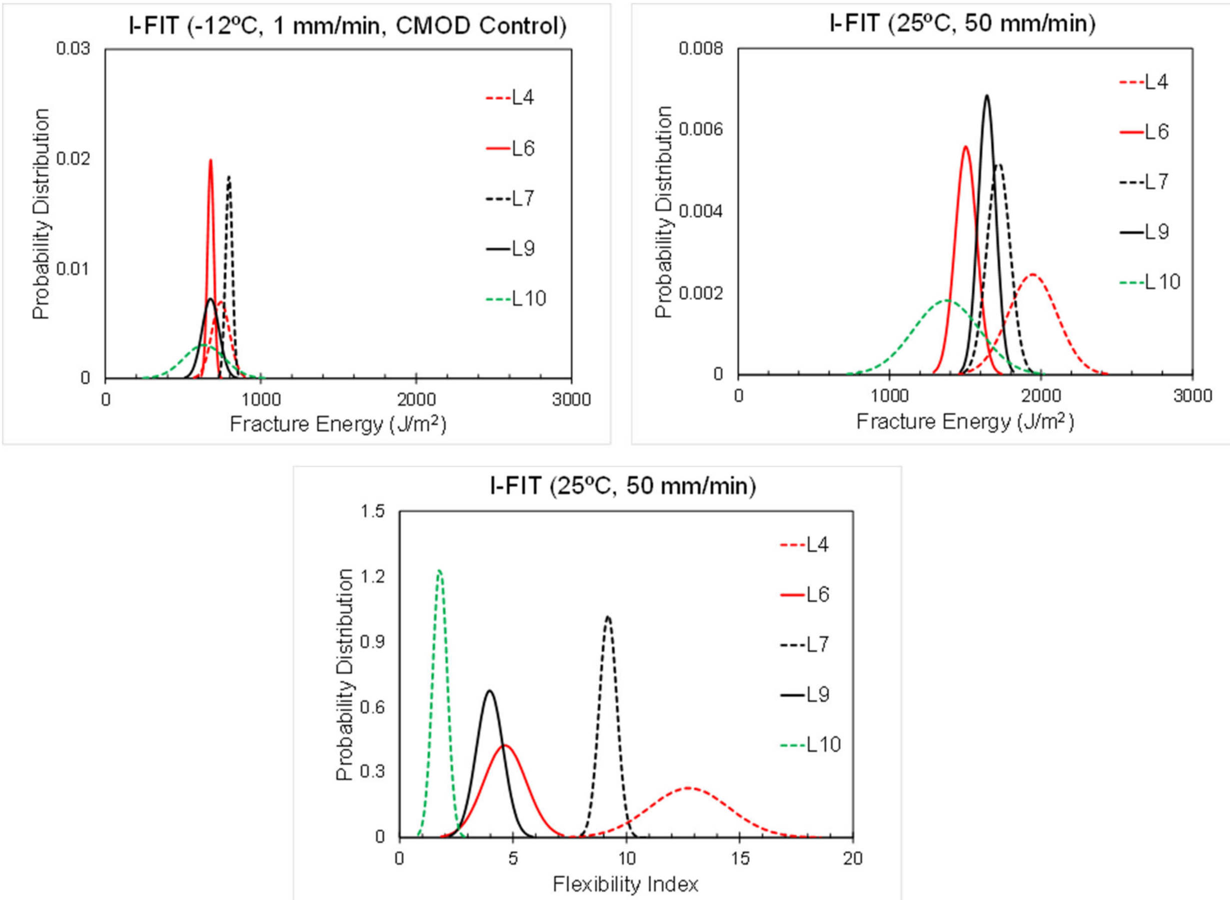


FIGURE 7 Comparison of I-FIT fracture energy parameters for discrimination potential (after Al-Qadi et al., 2015).

Balanced Mix Design Using I-FIT and Hamburg Tests

The FI is a measure of overall potential for cracking-related damage in AC mixtures. However, to more comprehensively evaluate AC mixture performance, it is necessary to have a performance measure criterion that includes not only indicators of cracking potential but also of the potential for high-temperature rutting. Progress has been made in developing a BMD approach by integrating several laboratory-level tests to control permanent deformations and cracking. The BMD approach suggest integrating performance-related laboratory tests such as Hamburg wheel tracking (for rutting), and a cracking test (e.g., Texas overlay, I-FIT) into the existing AC mixture volumetric design specifications. The ultimate goal is obtaining a balance between volumetrics and resistance to rutting and cracking.

The Hamburg wheel tracking test is a widely accepted test to evaluate rutting potential and is already part of the Illinois AC mixture design specifications. The FI was proposed to be added (along with the Hamburg test results) into the AC mix design specifications and preliminary thresholds were identified (Al-Qadi et al., 2015). A conceptual illustration of the BMD is shown in Figure 8. According to the distribution of AC mixes in the diagram, the AC mixes in each quadrant have the following characteristics:

- Stiff and flexible: mixes with good cracking (flexible) and rutting resistance (stiff). This quadrant can be further subdivided into high performance (I) and standard performance (II), depending on the cracking resistance thresholds (e.g., control mix without any RAP or RAS).
- Soft and flexible: mixes with sufficient cracking resistance (flexible) but with high rutting potential (soft) (e.g., 20% ABR with double grade bumping may indicate an unjustified grade bumping making the mix too weak).
- Stiff and brittle: low cracking resistance (brittle) and high rutting resistance (stiff) (e.g., mix with 60 % ABR).
- Soft and unstable: extremely low cracking and rutting resistance with insufficient load carrying capacity at all temperatures.

FIELD PERFORMANCE VALIDATION

Correlation to Accelerated Pavement Testing Results

The accelerated test sections built in the FHWA's Accelerated Loading Facility (ALF) were used to correlate with the results obtained from the I-FIT. The AC mixes used in the study contained various levels of RAP and RAS (up to 40% ABR), along with different binder grades commonly used to compensate for the presence of aged and stiff recycled binder. Details of the AC mix designs in the ALF experiment is shown in Table 3.

The test sections provided a unique opportunity to compare the performance of AC mixture tests because the accelerated test sections were intended to have identical pavement structures. ALF performance was grouped into three major categories (good, intermediate, and poor) for a distinct number of load repetitions to a threshold of fatigue cracking. The control AC

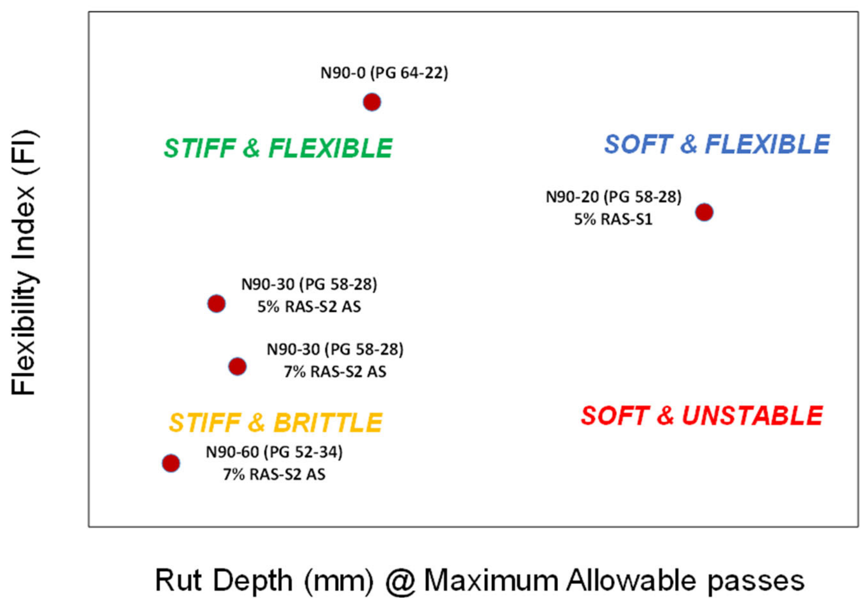


FIGURE 8 Interaction plot between FI and rut depth for balanced AC mix design (preliminary quadrants for conceptual illustration) (Al-Qadi et al., 2015).

TABLE 3 Summary of the AC Mixture Designs from the FHWA's ALF Sections and Production Mixture Volumetrics (Al-Qadi et al., 2015)

	Mixture Design Properties					Production Volumetrics		
	Binder Grade	ABR (%)	RAP (%)	RAS (%)	WMA Technology	Production VMA (%)	Production Air Voids (%)	Average Layer Thickness (in.)
Lane 1	PG 64-22	—	—	—	—	16.1	4.3	5.0
Lane 2	PG 58-28	40	44	—	Foaming	16.1	4.3	4.8
Lane 3	PG 64-22	20	—	6.3	None	14.6	3.3	4.5
Lane 4	PG 64-22	20	23	—	Chemical	15.6	4.4	4.9
Lane 5	PG 64-22	40	44	—	—	15.9	5.2	3.9
Lane 6	PG 64-22	20	23	—	—	14.9	3.6	5.0
Lane 7	PG 58-28	20	—	6.3	—	15.3	4.1	4.3
Lane 8	PG 58-28	40	44	—	—	16.4	4.9	4.6
Lane 9	PG 64-22	20	23	—	Foaming	15.1	3.7	4.0
Lane 11	PG 58-28	40	44	—	Chemical	16.5	4.9	3.9

mix with no recycled content (Lane 1) and AC mixes with low levels of RAP (Lanes 6 and 9) were among the good-performing group, whereas mixes with RAS (Lanes 3 and 7) and the highest recycled binder ratio without binder grade bumping to a softer grade (Lane 5) were in the poor-performing group. The I-FIT results with fracture energy and FI were evaluated, and the summary of FI results is shown in Figure 9. A clear reduction in flexibility of mixes with increasing recycled material content and presence of RAS was observed. A correlation exists between the number of cycles to failure from the ALF tests and the laboratory test FI values.

Field Performance and I-FIT Results of Field Core

A comparison was made between FI and observed field performance of selected field sections. Field cores were acquired from 35 sections from nine IDOT districts. Six to eight cores were extracted from each section. The top layer of each field core was carefully trimmed and fabricated to obtain the I-FIT geometry. The diameter of the field cores ranged from 143 to 147 mm (5.63 to 5.79 in.), and the thickness ranged from 30 to 50 mm (1.2 to 2 in.), the result of variation in section surface layer thickness. The notch length was 14 ± 1 mm (0.55 ± 0.04 in.) to match the 0.1 diameter-to-notch ratio of the laboratory specimens.

From the data obtained on distress severity, condition rating survey rating, and field observations, the pavement sections were subjectively divided into three categories: poor, fair, and good. The FI values corresponding to field performance data are presented in Figure 10. To reduce the effect of age on performance, the sections compared are only the ones constructed in 2013–2014. It was observed that the FI value of most of the sections correlated well with field performance except Section 1-13 (District 1), Section 867S1 (District 8), and Section 5US136-1

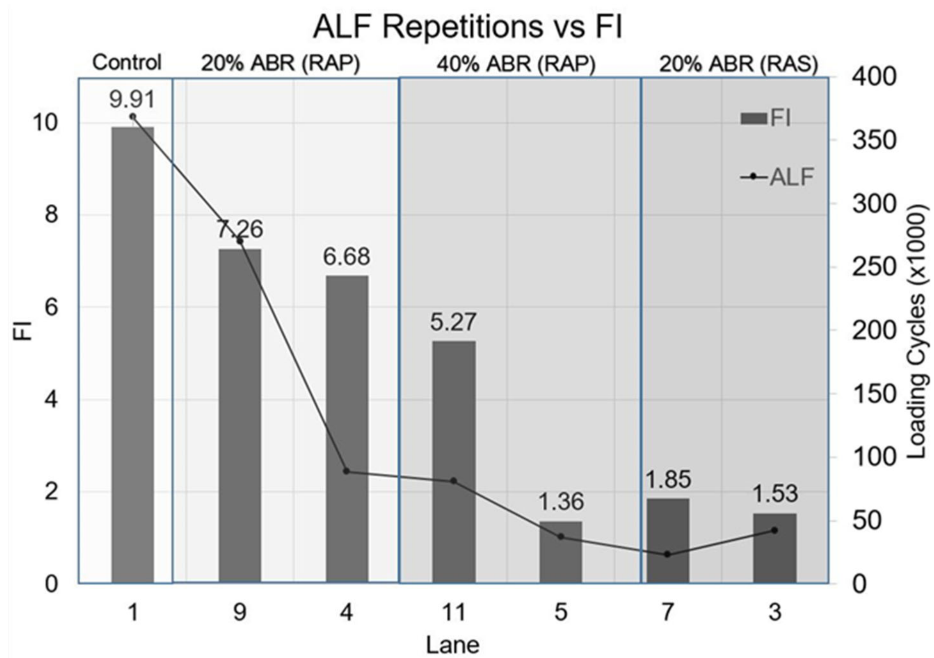


FIGURE 9 Summary of the I-FIT and its correlation to ALF cycles to failure performed on the plant-produced AC mixtures collected from the ALF experiment sections (Al-Qadi et al., 2015).

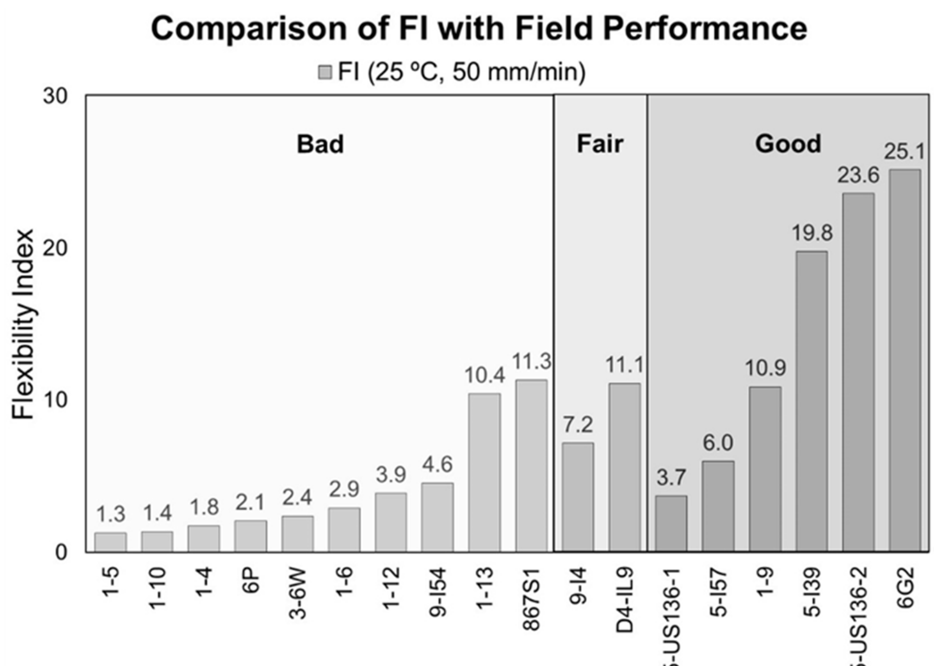


FIGURE 10 Correlation of FI with field performance using the field cores collected from IDOT districts.

(District 5). In a later investigation, it was reported that Section 1-13 had moderate to severe frost heave that could have caused bad performance. The worst-performing sections had FI values ranging from 1.3 to 3.9, whereas good-performing sections generally had values greater than 10.

CONCLUSIONS

A practical test method was developed that can be readily implemented to quantify an AC mixture's cracking potential. The I-FIT method is performed at 25°C (77°F). A 50 mm/min (2 in./min) loading rate is proposed for screening AC mixtures to control potential premature cracking. Ultimately, the introduced test method is coupled with an index parameter, the FI, to characterize the fracture potential of AC mixes. The FI is derived from the load-displacement response incorporating fracture energy and slope of the load-displacement curve after a crack begins to propagate.

Some of the findings during the development and implementation of the I-FIT (also known as IL-SCB) test method are the following:

- The developed FI provides greater separation between AC mixes to capture some of the changes that could not be captured by fracture energy alone. The effects of binder grade bumping and ABR levels as low as 10% can be captured by the FI. A consistent reduction in FI values with increasing ABR was observed, whereby FI values were greater than 10 for the control AC mix and reduced to as low as 2 with high ABR mixes (30% to 60% ABR).
- The correlation between field performance and the FI values was validated using field cores from sections with varying age and performance as well as cores from accelerated pavement test sections at FHWA's ALF. The FI values obtained for the field cores showed the effects of aging. A clear reduction in FI values was noted for the sections constructed more than 10 years ago compared with AC cores of relatively new construction. In general, the FI values were consistent with field performance data provided by the districts. The data obtained from FHWA's accelerated pavement test sections indicated agreement between the FI and performance ranking based on number of loading repetitions to failure. The three poor-performing sections had FI values less than 2, whereas the control section (among the best performing in the accelerated testing) had an FI value of 10.
- Implementation of BMD concept can be facilitated by integrating commonly available Hamburg testing and the practical I-FIT method.
- The I-FIT method provides an opportunity for agencies and contractors to implement a performance related test method during production and construction stages and allow contactor to better control the AC mixes and, hence, ability to improve the quality of pavements.

REFERENCES

AASHTO TP 105-13. 2013. Standard Method for Determining the Fracture Energy of Asphalt Mixtures Using the Semi Circular Bend Geometry (SCB). *American Association of State Highway and Transportation Officials, Washington, DC.*

AASHTO TP124-16 (2016). "Determining the fracture potential of asphalt mixtures using semicircular bend geometry (SCB) at intermediate temperature." *American Association of State Highway and Transportation Officials, Washington, DC.*

Al-Qadi, I. L., Ozer, H., Lambros, J., El Khatib, A., Singhvi, P., Khan, T., and Doll, B. (2015). "Testing protocols to ensure performance of high asphalt binder replacement mixes using RAP and RAS." Report No. *FHWA ICT-15-07*. Rantoul, IL: Illinois Center for Transportation.

Bazant, Z. P. (1996). "Analysis of Work-of-Fracture Method for Measuring Fracture Energy of Concrete." *Journal of Engineering Mechanics*, Vol. 122, No.2, pp.138-144.

Doll, B., Ozer, H., Rivera, J., Al-Qadi, I.L., and Lambros, J. (2017). "Investigation of Viscoelastic Fracture Fields in Asphalt Mixtures using Digital Image Correlation." *International Journal of Fracture*, Vol. 205, pp. 37-56

Hillerborg, A. (1985). "The Theoretical Basis of a Method to Determine the Fracture Energy of Concrete." *Materials and Structures*, Vol 18, No.4, pp. 291-296.

Ozer, H., Al-Qadi, I. L., Lambros, J., El-Khatib, A., Singhvi, P., & Doll, B. (2016a). "Development of the fracture-based flexibility index for asphalt concrete cracking potential using modified semi-circle bending test parameters." *Construction and Building Materials*, 115, 390-401. doi:10.1016/j.conbuildmat.2016.03.144

Ozer, H., Singhvi, P., Kahn, T., Rivera, J., & Al-Qadi, I. L. (2016b). "Fracture Characterization of Asphalt Mixtures with High Recycled Content Using Illinois Semicircular Bending Test Method and Flexibility Inde." *Transportation Research Record: Journal of the Transportation Research Board*, No. 2575, pp. 130-137.

Ozer, H., Al-Qadi, I. L., Singhvi, P., Bausano, J., Carvalho, R., Li, X., and Gibson, N. (2018). "Prediction of pavement fatigue cracking at an accelerated testing section using asphalt mixture performance tests" *International Journal of Pavement Engineering*, Vol. 19, pp. 264-278.

Balanced RAP–RAS–Recycling Agent Mix Design for Project-Specific Conditions

Fujie Zhou
Texas A&M Transportation Institute

The use of RAP and RAS in pavements has already become a norm in the United States because of the substantial reduction in construction cost, energy consumption, and greenhouse gas emissions. A recent study revealed that, in 2013 alone, asphalt pavement industry consumed nearly 68.8 million tons of RAP and nearly 1.6 million tons of RAS that would have otherwise gone to landfill, indicating a wide acceptance of such materials in asphalt pavements (*1*). The study also estimated that asphalt pavement industry saved more than $2 billion worth of taxpayers' money by substituting nearly 19 million barrels of virgin binders with the binders already present in RAP materials. However, recycled binders from both RAP and RAS are often severely aged and substantially stiffer than the virgin binders. As the percentage of recycled binder increases in asphalt mixes, the ratio of the aged binder to the total binder increases, resulting in a stiffer mix that often has a lower resistance to cracking, which is one of the major concerns with RAP–RAS mixes. It is critical to address the premature cracking problem in the mix design phase in order to most effectively use these recycled materials.

Generally, four approaches have been utilized to address the potentially premature cracking issue of RAP–RAS mixes: (1) limiting RAP–RAS usage; many states DOTs in the United States set an upper limit for RAP–RAS usage, such as maximum allowable binder replacement of say for example 20% for surface mixes; (2) using soft virgin binders especially on the low temperature grade (i.e., PG XX-28, PG XX-34); (3) increasing design density (lowering design air voids) or reducing N_{design}; and (4) rejuvenating RAP–RAS binder with recycling agents (or rejuvenators) in the mix design process.

The Texas DOT (TxDOT) has implemented the first three approaches in its new asphalt mix specifications. In last several years, TxDOT has been working the Texas A&M Transportation Institute (TTI) to investigate the best use of recycling agents to improve RAP–RAS mixes performance. This paper presents the work and findings from the research done at TTI. The characteristics of RAP, RAS, and recycling agents are discussed first, followed by the characteristics of their blends. This paper also presents the BMD method for mixes containing RAP–RAS–recycling agent. To demonstrate the proposed BMD, a field project designed with the BMD with RAP–RAS–recycling agent is described. Finally, a summary and conclusions are presented at the end of this paper.

RAP–RAS BINDER AND RECYCLING AGENT

In the last 10 years, TTI has extracted and characterized the binders from a variety of RAP and RAS sources (*2–5*). Figure 1 shows the overall rankings of PG high-temperature grades of RAP binders, manufacture waste asphalt shingles (MWAS) binders, and tear-off asphalt shingles (TOAS) binders. For comparison purpose, the virgin binders most often used in Texas are also presented in Figure 1. The binders in RAP and especially in RAS are much stiffer than the virgin

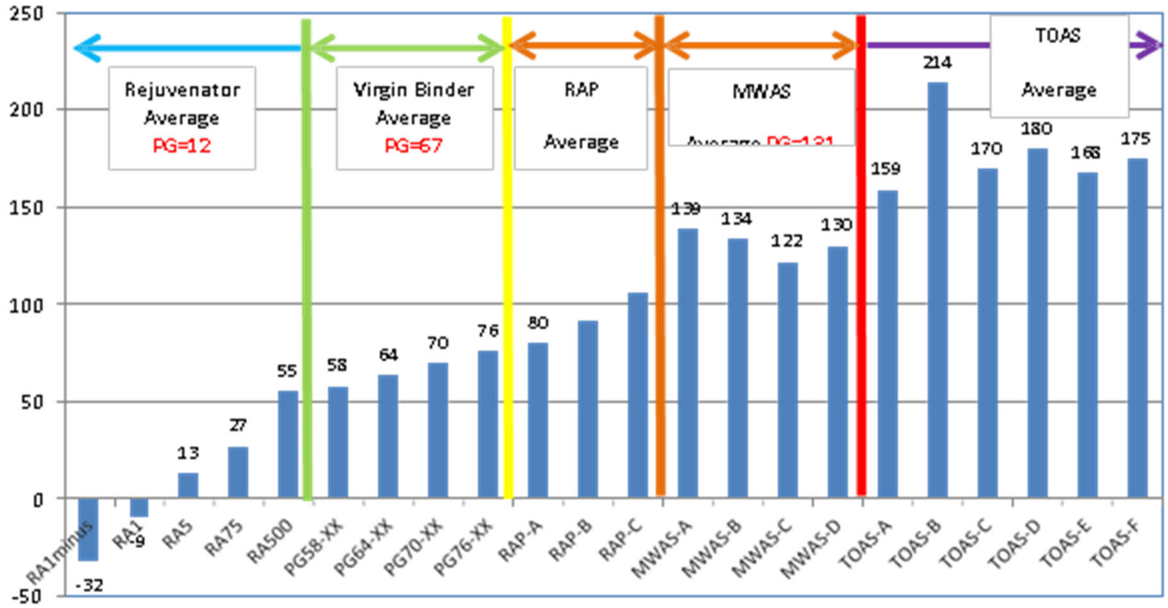


FIGURE 1 PG high-temperature grades: virgin, RAP, RAS binders, and recycling agents.

binders. From chemical point of view, the structure of asphalt binder is often treated as a complex colloid in which insoluble asphaltenes are dispersed in the soluble maltenes. With sufficient maltene components, the asphaltene micelles under applied stress can move smoothly within asphalt. As asphalt ages, the maltenes are transformed to asphaltenes through oxidation, which results in unbalanced ratio of maltenes to asphaltenes, and consequently a stiff binder. The same principle can be used to explain stiffness of RAS binder which is often produced from a flux through air-blowing process.

To compensate the super-stiff RAP–RAS binders, recycling agents are one of options for pavement engineers to use. The purpose of recycling agent is to (1) restore the aged asphalt characteristics to a consistency level appropriate for construction purposes and for the end use of the mixture; (2) restore the aged asphalt to its optimal chemical characteristics for durability, and (3) provide sufficient additional binder to coat new aggregate and to satisfy mix design requirements (*6*). According to Carpenter and Wolosick (*7*), the working mechanism (or diffusion process) of a recycling agent consists of the following four steps:

1. The recycling agent forms a low-viscosity layer that surrounds the asphalt-coated aggregate which is highly aged binder layer.
2. The recycling agent begins to penetrate into the aged binder layer, decreasing the amount of raw recycling agent that coats the particles and softening the aged binder.
3. No raw recycling agent remains, and the penetration continues, decreasing the viscosity of the inner layer and gradually increasing the viscosity of the outer layer.
4. After a certain time, equilibrium is approached over the majority of the recycled binder film.

Over the years many materials have been suggested as recycling agents, such as aromatic oils, paraffinic oils, napthenic oils, waste engine oils, waxes plus fatty acids, and tall oils. To better classify available materials as different groups, the Pacific Coast User–Producer Group evaluated a variety of recycling agents (or rejuvenators) in late 1970s and early 1980s (*8*), and its research results led to ASTM D4552: Standard Practice for Classifying Hot-Mix Recycling Agent. The recycling agents are classified into six grades (or groups) mainly through viscosity measured at 60°C (140°F), as shown in Table 1.

To compare the recycling agents with virgin and RAP–RAS binders, five recycling agents were selected and characterized, they being Hydrogreen, Evoflex, EAR, AC0.6, and AC5. The characteristics of these five recycling agents were determined in terms of viscosity and PG high grade. Table 2 shows viscosity test results of the five recycling agents. Compared to the recycling agent grade specification (Table 1), the five recycling agents cover all five recycling agent grades and one of them—Evoflex—is even beyond RA1 named RA1-minus. The dynamic shear rheometer test result for PG high grade of each recycling agent is presented in Figure 1. It is obvious that lower recycling agent grade (RA1 or even RA1 minus) should be selected for balancing stiff RAS and RAP binders so that a workable asphalt binder blend could be reached through blending. The stiffer the RAP–RAS binder, the lower recycling agent grade required.

CHARACTERISTICS OF RAP–RAS–RECYCLING AGENT–VIRGIN BINDER BLEND

The goal of determining recycling agent content is to ensure that the blend of RAP–RAS–recycling agent–virgin binder meets the specified PG high- and low-temperature grade requirements, and has similar (or even better) aging resistance than the originally specified virgin binder. Therefore, the rheological PG grade (blending chart) and aging characteristics of recycling agent–RAP–RAS–virgin binder blend are discussed below, respectively.

REGIONAL LINEAR-BLENDING CHART FOR RAP–RAS–RECYCLING AGENT–VIRGIN BINDER BLENDS

Blending characteristics between virgin and RAP binders have been well studied, and the linear blending chart is valid for determining either RAP or virgin binder content. Recently, Zhou et al. (*3*) studied blending characteristics of RAS–virgin binder blends and RAP–RAS–virgin binder blends. The overall blending chart is not linear rather it is a nonlinear line when significant difference in stiffness exists between the blending materials. Similar nonlinear blending observations for blends of two materials with large difference in stiffness or viscosity have been previously reported by Irving (*9*), Chaffin et al. (*10*), and Soleymani et al. (*11*). Specifically, the study performed by Chaffin et al. (*10*) was about the blend between aged binders and recycling agents. Based on the data presented in Figure 1 and the previous work described above, recycling agent–RAP–RAS–virgin binder blends may follow a nonlinear blending curve. Regardless of linear or nonlinear blending, a regional linear-blending chart can likely be used for selecting recycling agent amount, as demonstrated in Figure 2. If this concept is validated through laboratory testing, selecting the optimum recycling agent amount becomes much easier. In this study, three recycling agents (Evoflex, Hydrogreen, and ERA), two RAS binders (TOAS and MWAS), two PG 64-22 virgin binders, and one RAP binder were selected for validating the regional linear-blending concept. Detailed information is given below.

TABLE 1 Physical Properties of Hot-Mix Recycling Agents (ASTM D4552-10)

Test	ASTM Test Method	RA1	RA5	RA25	RA75	RA250	RA500
Viscosity, 60°C, mm^2/s (cSt)	D2170 or D2171	50-175	176-900	901-4500	4501-12500	12501-37500	37501-60000
Flash point, COC, °C (°F)	D92	219 (425)	219 (425)	219 (425)	219 (425)	219 (425)	219 (425)
Saturates, wt, %	D2007	Max. 30	Max. 30	Max. 30	Max. 30	Max. 30	Max. 30
Tests on residue from RTFO or TFO oven 163°C (325°F): 1) viscosity ratio*;	D2872 or D1754	Max. 3	Max. 3	Max. 3	Max. 3	Max. 3	Max. 3
2) wt change, %		Max. 4	Max. 4	Max. 3	Max. 3	Max. 3	Max. 3
Specific gravity	D70 or D1298	Report	Report	Report	Report	Report	Report

TABLE 2 Laboratory Test Results of the Five Recycling Agents

Parameter	Aging Condition	Recycling Agents				
		Evoflex	Hydrogreen	ERA	AC0.6	AC5
Viscosity, 60°C, mm^2/s (or cSt)	Original	47.6	58.7	227	7300	53700
	RTFO aged	125.0	106.0	401	13300	142300
Viscosity ratio	RTFO/original	2.6	1.8	1.8	1.8	2.6
RA grade (or group)		RA1 minus	RA1	RA5	RA75	RA500

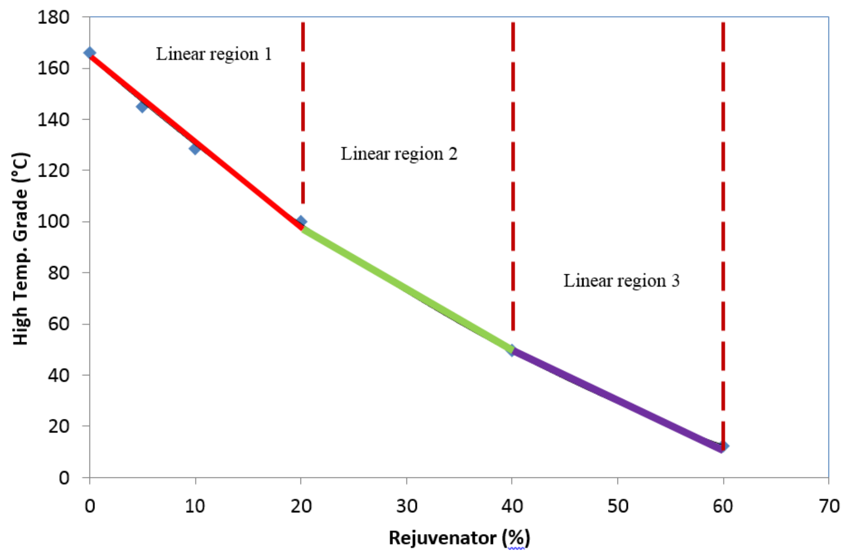


FIGURE 2 Regional linear blending concept.

Binary Blends: PG 64-22–Hydrogreen and TOAS–Hydrogreen

As the first step of the validation process, the researchers chose one recycling agent—Hydrogreen, one virgin binder—PG 64-22 (the most frequently used binder grade for RAP–RAS mixes in Texas), and one TOAS binder (having average stiffness of TOAS binders). Six and seven Hydrogreen blending ratios were selected for the validation, respectively. For simplicity, only the high-temperature PG grade of each blend was evaluated. Figure 3 presents the high-temperature PG grade measured for blends of PG 64-22–Hydrogreen and TOAS binder–Hydrogreen. The regional linear-blending concept is valid for any region for both cases as long as it is within the 20% range. For practical applications, a region of 20% recycling agent is large enough for asphalt mixes. Normally, less than 10% recycling agent is needed to make the RAP–RAS mixes meet the binder specification (i.e., PG 70-22). Therefore, this concept is validated for binary blends. Next validation focuses on multiple blends and both high and low PG grades.

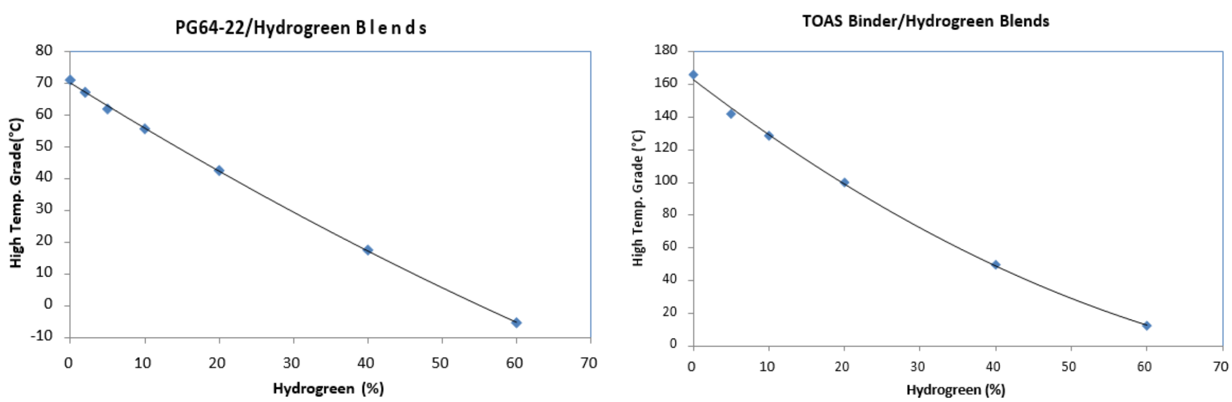


FIGURE 3 Validation of regional linear-blending concept through binary blends.

Multiple Blends: RAP–RAS–PG 64-22–Recycling Agents

Materials from the field demonstration project on SH31 (to be discussed later) were used for validating the applicability of the regional linear-blending concept. The mix design used 10% RAP and 5% MWAS and a Lion PG 64-22 virgin binder. Three recycling agents—Evoflex, Hydrogreen, and ERA—were blended with RAP–MWAS–PG 64-22 binders. For each blend, 10.8% RAP binder and 18.4% MWAS binder are fixed but varying amount of PG 64-22 and each recycling agent. Four blending ratios of each recycling agent to the total binder (by weight) were used in this study; namely 0%, 2%, 5%, and 10%. A total of 10 blends, as shown in Table 3, were graded through Superpave PG system. Figure 4 shows PG high and low grades for each blend, and that the linear blending concept is valid for all 10 blends. Meanwhile, there is no need for the recycling agent content to go beyond 10%, because 10% recycling agent is high enough to make the final blend meet the specification requirements for both high and low PG grades (say PG 70-22) for Texas conditions on SH31.

As shown in Figure 4, all three recycling agents can effectively rejuvenate aged binders. However, the long-term effectiveness of recycling agents has been questioned. Therefore, this issue must be addressed when selecting recycling agent type and amount. In the last several years the combination of Glover-Rowe (G-R) damage parameter:

$$\frac{G^*(cos\delta)^2}{sin\delta} = 180kPa : Damage\ Onset$$

$$\frac{G^*(cos\delta)^2}{sin\delta} = 450kPa\ Significant\ Cracking$$

and black space diagram has been widely used for evaluate asphalt binder cracking resistance (*12*, *13*). Anderson et al. (*14*) proposed to use 20-, 40-, and 80-h pressure aging vessel (PAV) aging conditions for analyzing long-term aging characteristics of asphalt binders and potential for block cracking. Although it is not known what the 20-, 40-, and 80-h PAV aging truly represent in terms of aging in the field, different aging characteristics of asphalt binders can be identified in the black space diagram. Muncy and Steger (*15*) recently extended the G-R parameter to quantify the relative aging characteristics between a virgin binder and a rejuvenated binder. The basic idea is described below:

- Step 1: Measure G* and phase angle (at 15°C and 0.005 Rad/s) of the virgin, originally specified asphalt binder (say PG 70-22 in this study) and the rejuvenated binder (say

TABLE 3 Multiple Blends for RAP–RAS–PG 64-22–Recycling Agents

RAP Binder–RAS Binder–Virgin Binder	Hydrogreen	Evoflex	ERA
10.8%/18.4%/70.8%	0%	0%	0%
10.8%/18.4%/68.8%	2%	2%	2%
10.8%/18.4%/65.8%	5%	5%	5%
10.8%/18.4%/60.8%	10%	10%	10%

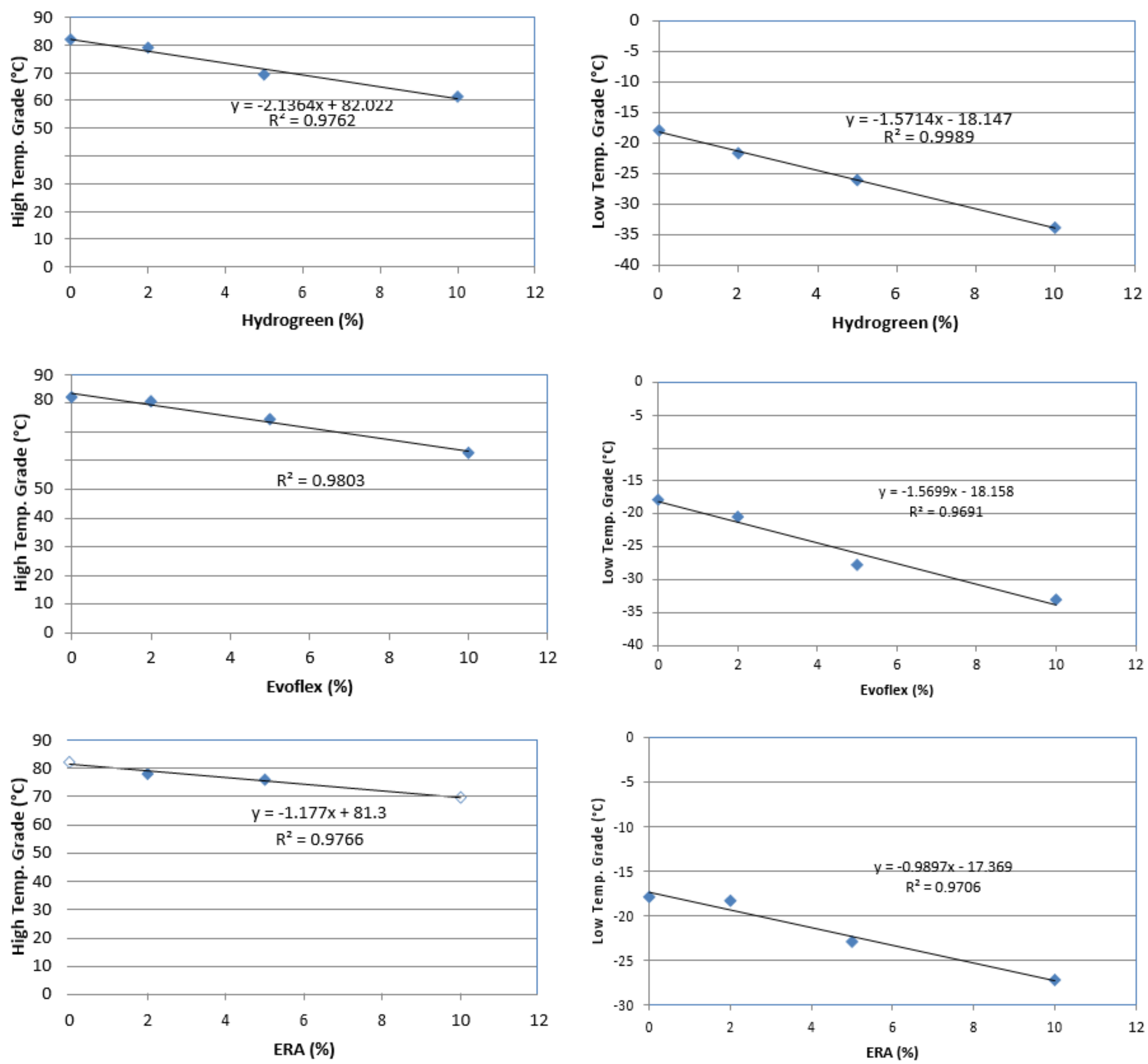


FIGURE 4 Validation of regional linear-blending concept for multiple blends.

with 2% Hydrogreen) before any aging;

- Step 2: Age the virgin binder and the rejuvenated binder through 20-, 40-, and 80-h PAV testing;
- Step 3: Measure the G* and phase angle (at 15°C and 0.005 Rad/s) of each PAV-aged virgin binder and each rejuvenated binder.
- Step 4: Plot the measured G* and phase angles of each binder in a black-space diagram;
- Step 5: For each binder, calculate the PAV aging hours to reach the G-R damage onset curve and the significant cracking curve, respectively;
- Step 6: Calculate the ratios of the PAV aging hours of the rejuvenated binder to the virgin binder;

- Step 7: If the calculated ratios are larger than (or close to) 1, then the rejuvenated binder has better (or similar) aging resistance than the specified virgin binder.

The author demonstrates this new concept by comparing the virgin PG 70-22 binder with 2% and 5% Hydrogreen rejuvenated RAP–RAS–PG 64-22 binder (Table 3). Figure 5 shows the G-R damage curves and the measured G* and phase angles of PG 70-22 and each blend at different aging conditions. Table 4 lists the calculated hours for each binder to reach the G-R damage onset curve and the significant cracking curve, respectively. 5% Hydrogreen rejuvenated binder has similar (if not better) aging resistance than the virgin PG 70-22 binder.

Although the 2% Hydrogreen rejuvenated binder meets Superpave PG high and low grades requirement, caution should be exercised due to its poor aging characteristics.

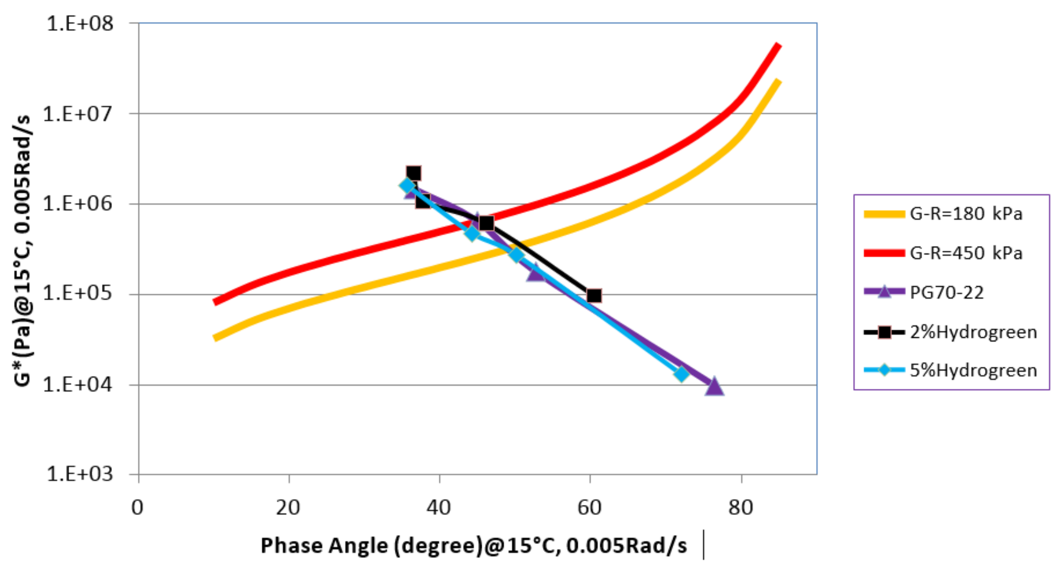


FIGURE 5 Aging characteristics of virgin PG 70-22 and Hydrogreen rejuvenated binder.

TABLE 4 PAV Aging Hours to Reach the G-R Damage Curves

Binder	G-R Damage Onset Curve			
	Required PAV Hours	Rejuvenated/ Virgin	Required PAV Hours	Rejuvenated/ Virgin
Virgin PG 70-22	26	N/A	40	N/A
2% Hydrogreen rejuvenated binder	14	0.54	200	.50
5% Hydrogreen rejuvenated binder	23	0.88	441	.10

In summary, this section evaluated the blend characteristics of recycling agent–RAP–RAS–virgin binders. The results presented above indicate that the regional linear-blending concept is accurate enough for practical application to determine the required recycling agent content for meeting specification requirements. Additionally, aging characteristics represented by the G-R parameter should be evaluated to ensure long-term performance of rejuvenated binders. Ideally, the rejuvenated binder should have similar or even better aging resistance than the original virgin binder. These findings constitute part of the proposed mix design method for asphalt mixes containing RAP–RAS–recycling agents, which is described in the next section.

BALANCED RAP–RAS–RECYCLING AGENT MIX DESIGN FOR PROJECT-SPECIFIC CONDITIONS

Properties of the rejuvenated asphalt binder are critical for obtaining a successful mix design. However, it is the properties of the entire asphalt mixture that ultimately determines field performance. Consequently it is proposed that the design method for asphalt mixes containing RAP–RAS–recycling agent includes at least three components:

1. Selection of recycling agent type,
2. Determination of the range of recycling agent amounts required to meet both the binder specification and aging characteristics, and
3. Finalize the recycling agent amount through asphalt mix property evaluation to meet project specific service conditions in terms of mixture rutting and cracking requirements.

Based on the data presented above and the previous work (*2*, *16–20*), the three-step mix design process is described below:

Step 1: Selection of Recycling Agent Type

The amount and hardness of the asphalt in aged asphalt mix are considered when selecting recycling agent (RA) grade. The general rule for selecting RA type is to use lower RA grade for stiff RAP and RAS binders. Meanwhile, the compatibility between the selected RA and RAP–RAS–virgin binder should be considered, although it is beyond the scope of this paper.

Step 2: Determination of Recycling Agent Range Through Binder Tests

As discussed in Section 3, it includes the following three steps to determine the range of recycling agents:

- **Step 1.** Determine the range of recycling agents based on the specific project PG binder requirement.

As demonstrated in Section 3.1, the regional linear-blending rule for both Superpave PG high and low grades is valid for recycling agent–RAP–RAS–virgin binder blending, which significantly reduces the blending work. Only two blending ratios of the recycling agent to RAP–RAS–virgin binder are needed. The high PG grade of blended binders controls the maximum recycling agent amount, and the low PG grade of the blended binders determines the minimum recycling agent amount (*21*). In case of recycling agent Hydrogreen (Figure 4), it will range from 2% to 5%.

- **Step 2.** Verify aging characteristics of rejuvenated binder through comparing to virgin binder.

The proposed detailed process was described in Section 3.2. At the end of the process, a range of recycling agent amounts will be determined so that the rejuvenated binder has similar or even better aging resistance to the original virgin binder. In case of recycling agent Hydrogreen (Figure 5), the range will be from 2% to 5%.

- **Step 3.** Select the final range of recycling agent amount for mix property evaluation based on Steps 1 and 2.

In case of recycling agent Hydrogreen, the final range for mixture testing will be larger than 2% but less than 5%.

SELECTION OF RECYCLING AGENT AMOUNT FOR PROJECT-SPECIFIC SERVICE CONDITIONS THROUGH BALANCED MIX DESIGN

In last 10 years TTI researchers have focused on developing the BMD method (*2*, *16–20*). A successful mix needs to have a balanced rutting and cracking resistance, and the rutting and specifically cracking requirements depend on site specific conditions. These include traffic, climate, pavement structure and layer thickness, and existing pavement conditions particularly for asphalt overlays. Currently, asphalt mix design in Texas is based on volumetric properties of asphalt mixes plus checking potential rutting and moisture damage the Hamburg wheel tracking test. Texas already established the project specific rutting–moisture damage requirements for mixes through connecting the criteria with binder PG grades, because the selection of binder PG grade is related to climate and traffic. For example, the rut depth of a mix with PG 76-22 binder should be less than 0.5 in. (12.5 mm) after 20,000 passes. However, there is no cracking requirement on dense-graded and/or Superpave mixes in the current TxDOT specification, although Texas OT has been well widely used for characterizing cracking resistance of asphalt mixes in last 15 years. As demonstrated by Zhou et al. (*20*), it is difficult (if not impossible) to establish a single cracking requirement for all scenarios, because cracking performance of asphalt mixes depends on traffic, climate, pavement structure, and existing pavement conditions for asphalt overlays. Therefore, a balanced RAP–RAS mix design system for project-specific conditions, rather than a single cracking requirement, should be developed, and then implemented to ensure the mixes designed with acceptable field performance. It is envisioned that it is a two-step process: in Step 1 the site conditions will be evaluated and the performance model will be run to predict pavement performance for a range of different materials properties (different OT cycles), and the designer then selects the OT requirement to meet the design performance goal (for example less than 50% reflective cracking after 5 years). In Step 2 a lab mix design is run to design a mix with the required OT cycles. If this does not work, the mix will be redesigned, this time changing virgin binder type, recycling agents, and others.

The BMD system for project specific conditions proposed previously can also be used for designing mixes containing recycling agent. The only difference here is to determine the optimum recycling agent content rather than virgin binder content. The revised design flowchart for recycling agent–RAP–RAS–virgin binder mix design is shown in Figure 6. Note that TGC and SGC stand for Texas Gyratory Compactor and Superpave Gyratory Compactor, respectively.

This system integrates both mix design and pavement structure design which has been pursued for long time. The proposed system is an expanded BMD procedure in which cracking performance is evaluated through a simplified asphalt overlay performance analysis system, S-TxACOL, with OT cycles as an input. Note that the same reflective cracking model as that in TxACOL (*18*) is used in the S-TxACOL.

To demonstrate and verify the proposed mix design method, five field test sections were constructed on SH31 of Tyler District, Texas. Detailed information is described in the next section.

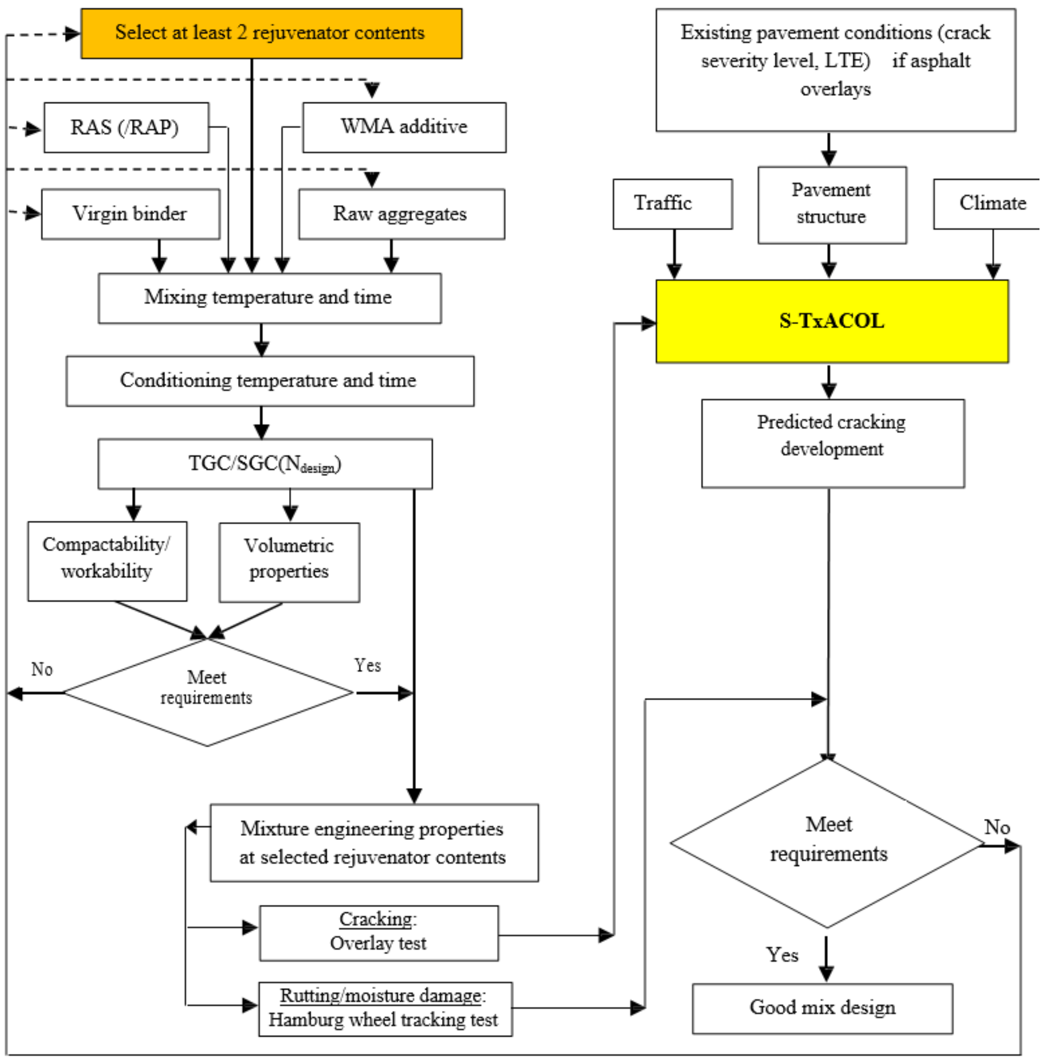


FIGURE 6 Balanced RAP–RAS–recycling-agent mix design for project-specific service conditions.

DEMONSTRATION OF THE BALANCED RAP–RAS–RECYCLING AGENT MIX DESIGN FOR PROJECT-SPECIFIC CONDITIONS

More than 15 field test sections with recycling agents have been designed following the balanced RAP–RAS–Recycling agent mix design for project specific conditions shown in Figure 6. The first five field test sections constructed on SH31, Texas in June 2014 are described for the purpose of the demonstration. SH31 is a two-way divided highway with annual average daily traffic of 9,800 and 10% truck traffic, the estimated 20-year traffic load in 18 kips is 3.5 million equivalent single-axle load (ESAL). These test sections are located between the east side of the city of Murchison and the west side of the city of Brownsboro, Texas (Figure 7). For each test section 350 tons of asphalt mix was produced. On June 3, 2014, Sections 1, 2, and 3 were constructed. Sections 4 and 5 were paved on June 4, 2014. Section 4 is followed by Section 5.

All five sections have the same pavement overlay structure: this includes, 1 in. of CAM and 2-in. dense-graded Type-C surface mix. The Type-C mix was modified as described below:

- Section 1: A virgin mix with PG 70-22 binder (no RAP–RAS–recycling agent).
- Section 2: The control mix with PG 64-22 virgin binder, 10% RAP, and 5% MWAS.
- Section 3: The same control mix with recycling agent—Hydrogreen.
- Section 4: The same control mix with recycling agent—Evoflex (3.7% weight of total asphalt binder) and Evotherm (0.3% of weight of total of asphalt binder).
- Section 5: The same control mix with recycling agent—ERA (1.3% of weight of total asphalt binder content).

The contractor designed both the virgin PG 70-22 mix and the control section with RAP and MWAS following TxDOT standard mix design procedure (Tex-204-F). The optimum asphalt binder contents for the virgin PG 70-22 mix and the control mix are 4.6% and 4.9%, respectively.

For the control mix with RAP and MWAS, the ABR ratio is 29.2%, 10.8% from 10% RAP and 18.4% from 5% MWAS.

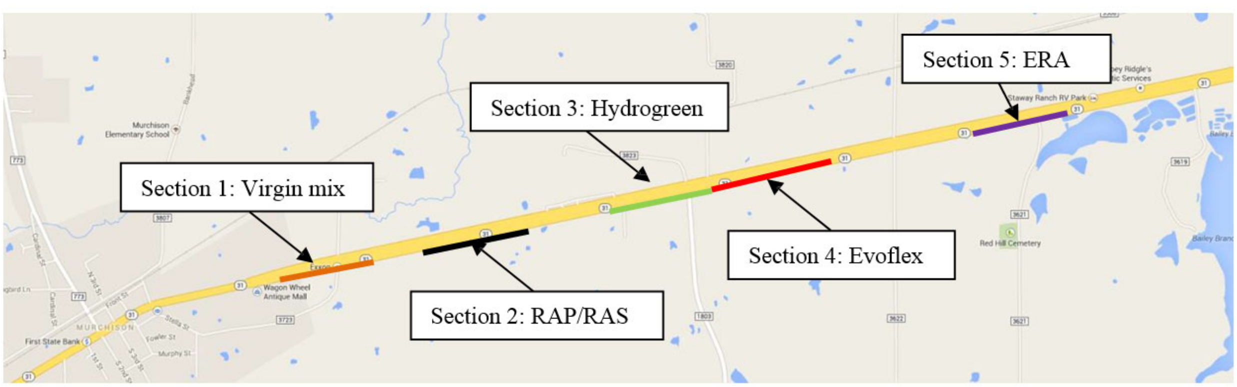


FIGURE 7 Layout of five test sections on SH31.

Both TTI and the contractor worked together to complete the mix designs for the three recycling agent test sections. The three-step of mix design process described in Section 4 was followed, and detailed information is provided below:

- **Step 1: Select recycling agent types.**

First of all, TTI researchers characterized the extracted RAP and MWAS binders. The measured high temperature PG grades for the RAP and MWAS binders are 109 and 134ºC, respectively. Both binders are stiff. The low temperature PG grades for both binders could not be measured with the bending beam rheometer. Based on the results presented in Section 2, three out of five recycling agents were selected for the test sections, and they are Hydrogreeen, Evoflex, and ERA.

- **Step 2: Determine recycling agent range through binder tests.**

The goal was to have the rejuvenated binder to meet PG 70-22 binder specification. Based on the work described previously, 2% to 5% of recycling agent can make the binders meet the PG 70-22 requirements in terms of PG high and low grades. The aging characteristics comparison between the PG 70-22 virgin binder and the recycling agent binders are shown in Figure 5 and Figure 8. Based on the measured data, the recommended range for mixture evaluation is 2% to 5% for both Hydrogreen and Evoflex within which they can meet the PG binder specification requirements and have acceptable aging characteristics. However, this is not the case for ERA. More ERA is needed to have similar aging resistance to the virgin PG 70-22. It was decided for the ERA section to add extra ERA to the original design binder content of 4.9% rather than the binder replacement concept.

- **Step 3: Select recycling agent content optimized for project-specific service conditions through mixture tests.**

Based on the results of Step 2, two trial recycling agent contents for each recycling agent were chosen for mixture testing, as listed in Table 5. Following the proposed mix design flowchart shown in Figure 6, this study focused on the engineering properties of asphalt mixes with recycling agent only, although both compactability and volumetric properties are important as well. The Hamburg wheel-tracking test (HWTT) was performed to evaluate rutting resistance of all five mixes, following Tex-242-F, Test Procedure for HWTT. The cracking resistances of the same mixes were evaluated using Texas Overlay test according to Tex-248-F Test Procedure for OT. Both the Hamburg and OT test results are shown in Figure 9. Note that the rut depth shown in Figure 9 is measured at 10,000 passes. It can be observed from Figure 9

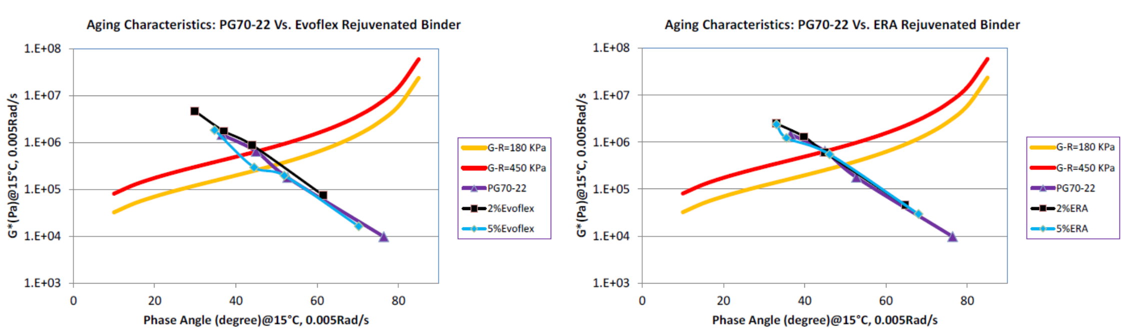


FIGURE 8 Aging characteristics of virgin PG 70-22, Evoflex- and ERA-rejuvenated binders.

TABLE 5 Selected Recycling Agent Contents for Mixture Performance Evaluation

Recycling Agent Content	Hydrogreen (weight of total asphalt binder)	Evoflex (weight of total asphalt binder)	ERA (extra addition, weight of total asphalt binder)
Low end	2.6%	3.7%	1.0%
High end	5.0%	5.0%	2.0%

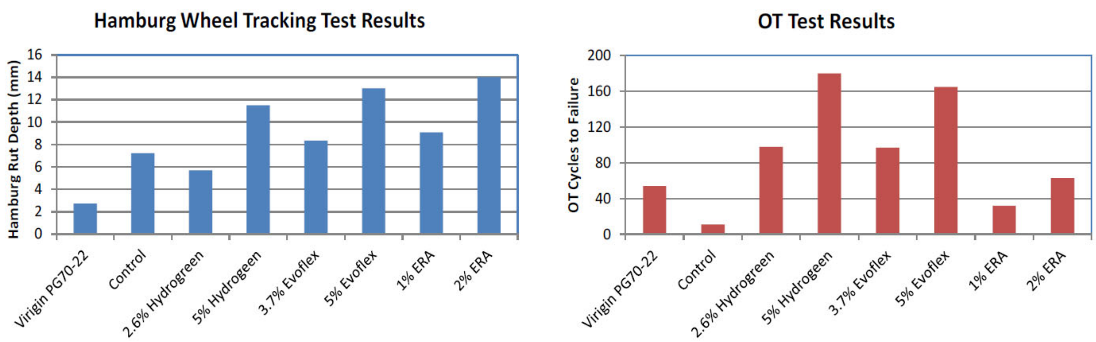


FIGURE 9 Hamburg and OT test results of SH1 mixes.

that the use of recycling agents can significantly improve the cracking resistance of RAP–RAS mixes, and the measured Hamburg rut depths are also acceptable. Another observation in Figure 9 is that the rejuvenated RAP–RAS mix can have better cracking resistance than the virgin mix.

Regarding cracking requirement, TTI researchers used the S-TxACOL overlay design program to calculate the required OT cycle for the 2-in. surface mix. The existing pavement structure after the milling is 2-in. asphalt over an old joint concrete pavement. Prior to milling there is lots of severe transverse and longitudinal cracking. The loading transfer efficiency at cracks was assumed to be 70%. Based on the traffic, weather, pavement structure and falling weight deflectometer backcalculated layer modulus, the calculated minimum OT cycles for the surface mix to meet the design life is predicted to be 50 cycles. All mixes, except the control mix and the mix with 1% ERA, meet this cracking requirement. Based on the laboratory test results, the final recycling agent contents recommended for Hydrogreen, Evoflex, and ERA are 2.6%, 3.7%, and 1.3%, respectively. All five test sections with the five mixes designed under this study have been successfully constructed on SH31. TTI researchers are closely monitoring the performance of these five sections, and will report the observed field performance at later time.

In summary, this section demonstrates the proposed mix design method through actual field project, and the performance of these five sections on SH31 being observed will be used to verify both rutting and cracking requirements of asphalt mixes.

SUMMARY AND CONCLUSIONS

This paper evaluated five recycling agents and blending characteristics of recycling agent–RAP–RAS–virgin binders in terms of their rheological and aging properties. A BMD method for project specific conditions was proposed for asphalt mixes containing RAP–RAS–recycling agents. Additionally, five field test sections were constructed with the purpose of demonstrating and validation of the proposed mix design method. The following conclusions and recommendations are made based on the research presented in this paper.

- The regional linear blending concept provides a means of selecting the amount of recycling agent required to meet the specified binder specification. The three recycling agents investigated in this study are effective to rejuvenate the aged binders. A mount of 10% or less recycling agent is needed to meet the binder grade requirement.
- Aging characteristics of virgin and rejuvenated binders were assessed using the G-R damage parameter curves. A relative aging time approach related to G-R damage onset and significant cracking curves was explored in this study, which is important to ensure that the rejuvenated binders have similar (or even better) long-term performance to virgin binders.
- A balanced RAP–RAS–recycling agent mix design method was proposed for project-specific service conditions. The proposed mix design includes
  1. Selection of recycling agent type,
  2. Determination of the range of recycling agent through binder rheological and aging characterization, and
  3. Final selection of recycling agent content through a BMD and a performance evaluation system.

The HWTT and associated criteria are used to control rutting–moisture damage and the OT and the required OT cycles determined from S-TxACOL cracking prediction with consideration of climate, traffic, pavement structure, and existing pavement conditions are employed for controlling cracking. Additionally, five field test sections were constructed on SH31 near Tyler Texas to demonstrate and verify the proposed method. Initial performance has been excellent and multiyear monitoring is underway.

DISCLAIMER

The contents and opinions of this paper reflect the views of the authors, who are solely responsible for the facts and the accuracy of the data presented herein. The contents of this paper do not necessarily reflect the official views or the policies of any agencies

REFERENCES

1. Hansen, K. R., and A. Copeland. *Annual Asphalt Pavement Industry Survey on Recycled Materials and Warm-Mix Asphalt Usage: 2009–2013*. National Asphalt Pavement Association, 2014.
2. Zhou, F., S. Hu, G. Das, et al. Successful High RAP Mixes Designed with Balanced Rutting and Cracking Requirements, *Journal of Association of Asphalt Paving Technologists*, Vol. 81, Austin, Texas, 2012, pp. 477–505.

3. Zhou, F., H. Li, S. Hu, J. W. Button, and J. A. Epps. *Characterization and Best Use of Recycled Asphalt Shingles in Hot Mix Asphalt*. FHWA/TX-12/0-6614-2. Texas A&M Transportation Institute, College Station, Texas, 2013.
4. Zhou, F., H. Li, R. Lee, T. Scullion, and G. Claros. RAS Binder Characterization and Blending with Virgin Binders. *Transportation Research Record*: *Journal of Transportation Research Record, No. 2370*, 2013, pp. 33–43.
5. Zhou, F., P. Chen, and S. Huang. Characteristics of Virgin and Recycled Asphalt Shingle Binder Blends. *Transportation Research Record: Journal of the Transportation Research Board*, *No. 2444*, 2014, pp. 78–87.
6. Epps, J. A., D. N. Little, R. J. Holmgreen, and R. L. Terrel. *NCHRP Report 224: Guidelines for Recycling Pavement Materials*, September 1980.
7. Carpenter, S., and J. Wolosick. Modifier Influence in the Characterization of Hot-Mix Recycled Material. *Transportation Research Record 777*, 1980, pp. 15–22.
8. Kari, W. J., N. E. Andersen, D. D. Davidson, H. L. Davis, R. N. Doty, S. J. Escobar, D. L. Kline, and T. K. Stone. Prototype Specifications for Recycling Agents Used in Hot-Mix Recycling. *Proc., Association of Asphalt Paving Technologists*, Vol. 49, 1980, pp. 177-192.
9. Irving, J. Viscosities of Binary Liquid Mixtures: The Effectiveness of Mixture Equations, Report 631, National Engineering Laboratory, Department of Industry of U.K., 1977.
10. Chaffin, J. M., R. R. Davison, C. J. Glover, and J. A. Bullin. Viscosity Mixing Rules for Asphalt Recycling. *Transportation Research Record 1507*, 1995.
11. Soleymani, H. R., H. U. Bahia, and A. T. Bergan. Time-Temperature Dependency of Blended and Rejuvenated Asphalt Binders. *Journal of the Association of Asphalt Paving Technologists*, Vol. 68, 1999, pp. 129–152.
12. Rowe, G. M. Prepared Discussion for the AAPT paper by Anderson et al.: Evaluation of the Relationship between Asphalt Binder Properties and Non-Load Related Cracking. *Journal of the Association of Asphalt Paving Technologists*, Vol. 80, 2011, pp. 649–662.
13. King, G., M. Anderson, D. Hanson, and P. Blankenship. Using Black Space Diagrams to Predict Age-Induced Cracking. *Proceeding of the 7th RILEM International Conference on Cracking in Pavements*, Vol. 4, 2012, pp. 453–463.
14. Anderson, R. M., G. N. King, D. I. Hanson, and P. B. Blankenship. Evaluation of the Relationship Between Asphalt Binder Properties and Non-Load Related Cracking. *Journal of the Association of Asphalt Paving Technologists*, Vol. 80, 2011, pp. 615–649.
15. Muncy, D, and R. Steger. Personal communication, April 2014.
16. Zhou, F., S. Hu, T. Scullion, M. Mikhail, and L. F. Walubita. A Balanced HMA Mix Design Procedure for Overlays. *Journal of the Association of Asphalt Paving Technologists*, Vol. 76, 2007, pp. 823–850.
17. Zhou, F., S. Hu, X. Hu, and T. Scullion. *Mechanistic-Empirical Asphalt Overlay Thickness Design and Analysis System.* Research Report FHWA/TX-09/0-5123-3. Texas Transportation Institute, College Station, October 2009.
18. Zhou, F., S. Hu, and T. Scullion. Advanced Asphalt Overlay Thickness Design and Analysis System. *Journal of Association of Asphalt Paving Technologists*, Vol. 79, Sacramento, Calif., 2010.
19. Zhou, F., H. Li, S. Hu, R. Lee, T. Scullion, G. Claros, J. Epps, and J. Button. Evaluation of Use of Recycled Asphalt Shingles in HMA. *Journal of Association of Asphalt Paving Technologists*, Vol. 82, 2013.
20. Zhou, F., S. Hu, T. Scullion, and R. Lee. Balanced RAP/RAS Mix Design for Project-Specific Conditions. *Journal of Association of Asphalt Paving Technologists*, Vol. 83, 2014.
21. Shen, J., and Y. Ohne. Determining Rejuvenator Content for Recycling Reclaimed Asphalt Pavement by SHRP Binder Specification. *International Journal of Pavement Engineering*, Vol. 3, 2002, pp. 261–268.

Evaluating Balanced Mixture Design for New Jersey to Enhance Asphalt Mixture Durability

THOMAS BENNERT
Rutgers University

The original intent during the development of the Superpave asphalt mixture design system was to have a volumetric design phase complimented by mixture performance and modeling to ensure the final asphalt mixture design would perform under the anticipated traffic and pavement conditions. Unfortunately, due to complexities in the testing and modeling phase, the Superpave asphalt mixture design system was left with only the volumetric design phase. And although a volumetric design system may work well in a simplistic environment, asphalt mixture design and production surely is not. The heavy use of polymer-modified binders, recycled asphalt binder, warm-mix additives, etc., have "muddied" the waters regarding whether or not the volumetrics can actually provide assurances of asphalt mixture performance.

This paper summarizes an effort that evaluated a different methodology for designing asphalt mixtures, called BMD. In this design method, the asphalt content is not determined through volumetric analysis. Optimum asphalt content, and recommended tolerances, are established by the rutting and fatigue cracking performance of the asphalt mixture, thereby "balancing" asphalt mixture performance. Volumetrics are not ignored, as they provide good guidance that has been historically verified. However, unlike the current Superpave asphalt mixture design, the volumetrics are used as a guide and not the final determining criteria.

BALANCED MIXTURE DESIGN

The concept of balancing rutting (stability) and fatigue (durability) has been around for a while and can date back to some of the original Marshall and Hveem mixture design work, as depicted in the figure below (Figure 1). When utilizing a BMD method for asphalt mixtures, the optimum asphalt content is not a function of the compacted air voids at a predetermined compaction level, but a function of optimizing the asphalt content to achieve the best rutting and fatigue performance. Obviously, from a construction standpoint, there needs to be some consideration towards the workability and in-place density levels of the final pavement. However, when utilizing the balanced design concept with established laboratory rutting and fatigue performance criteria, the final asphalt mixture should construct and perform as expected.

The original intention of the Superpave Mix Design procedure was to incorporate performance testing to verify the rutting, fatigue cracking and thermal cracking performance of the asphalt mixtures. However, due to the complexity and cost of the test procedures ultimately recommended, the performance testing was deemed to be impractical and was never implemented on a national level. However, it was soon realized that the volumetric properties alone cannot be relied on to determine if there will be issues with performance.

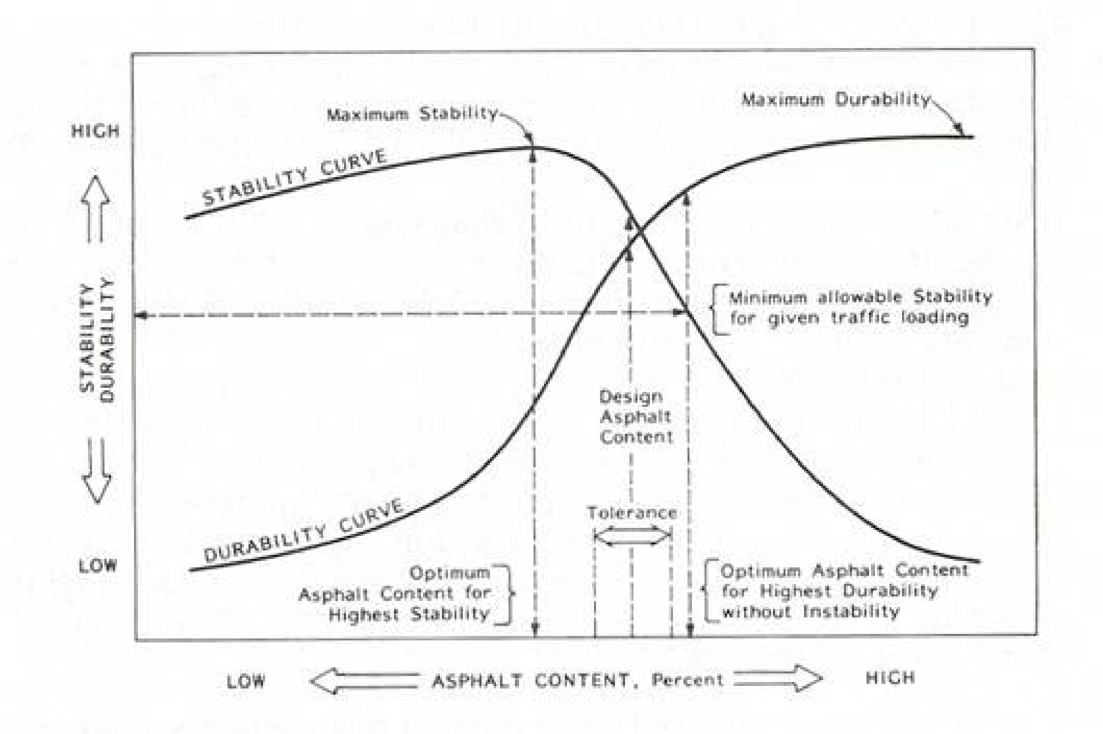


FIGURE 1 Importance of balancing stability and fatigue durability (*1*).

In 2006, TTI re-introduced the concept of BMD in their report, *Integrated Asphalt (Overlay) Mixture Design, Balancing Rutting and Cracking Requirements* (*2*). The researchers utilized the wet HWTT device to index rutting resistance, while indexing the fatigue cracking performance of asphalt mixtures with the OT. Over the past few years, TxDOT and TTI had generated a significant database of laboratory test performance that had been correlated to observed field performance with these laboratory tests and believed they could be utilized to verify asphalt mixtures during design. Their general methodology is shown below and in Figure 2.

1. Select materials (aggregate and asphalt binder).
2. Develop aggregate gradation, mix with asphalt binder at different binder contents, and compact to gyration level (based on traffic).
3. Determine volumetric properties at each asphalt content.
4. Compact HWTT and OT specimens at each asphalt content to a known air void range (typically 6% to 7% air voids to represent typical initial in-place air voids).
5. Utilize performance criteria to verify whether mixture met the rutting and fatigue requirements.
6. Adjust final asphalt content to meet the balanced performance.

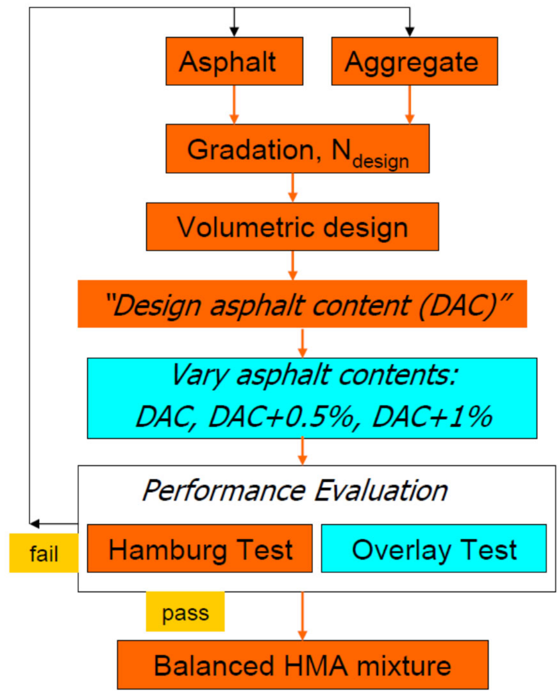


FIGURE 2 TxDOT BMD concept (*3*).

An example of what the typical BMD output looks like is shown in Figure 3. The yellow area marks the range in asphalt contents that optimizes the rutting and fatigue cracking properties of the mixture evaluated. In this case, a range in asphalt content of 5.3% to 5.8% optimizes the mixtures performance. It should be noted that this is based on the set criteria TxDOT has established using the wet HWTT device and the OT.

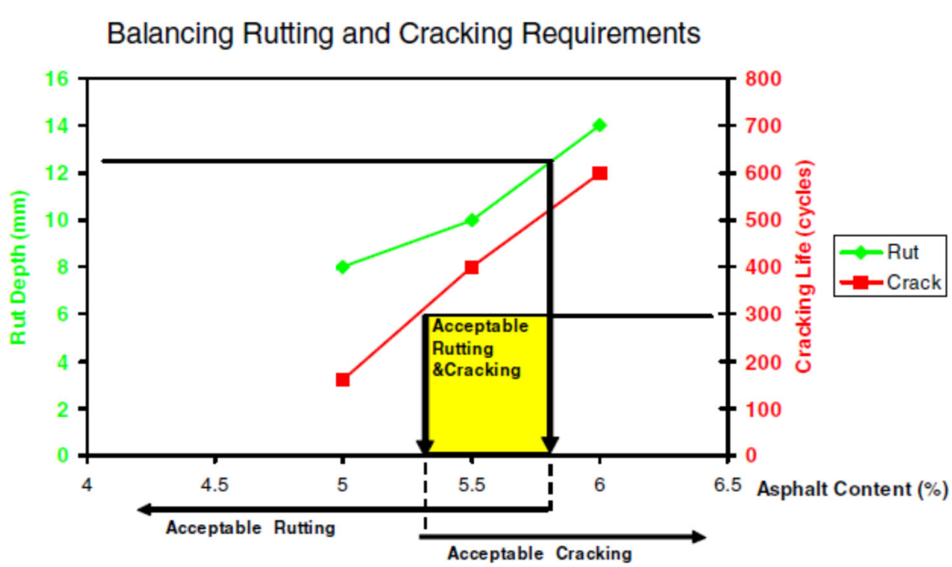


FIGURE 3 BMD results (*2*).

NEW JERSEY'S BALANCED MIXTURE DESIGN APPROACH

In the TxDOT BMD procedure, TxDOT prefers to utilize the wet HWTT device to assess rutting potential. However, New Jersey DOT (NJDOT) has had a long history of using the APA as a test to evaluate rutting potential, and therefore, it is utilized in NJDOT's BMD. The OT was also selected for the NJDOT Balanced Design due to its ability to trend with field performance, especially when RAP is used.

The selection of the performance criteria for the NJDOT BMD is based on NJDOT's high RAP asphalt mixture specification. For the fatigue resistance, a minimum of 175 cycles is required in the OT, regardless of the asphalt binder performance grade. Meanwhile, the APA rutting is dependent on the traffic level the asphalt mixture is intended to be placed on. For lower volume road (<10 million ESALs) where a PG 64-22 asphalt binder would be specified in New Jersey, the maximum APA rutting allowed is 7.0 mm. For moderate to higher volume roads (>10 million ESALs) where a PG 76-22 asphalt binder would be specified in New Jersey, the maximum APA rutting allowed is 4.0 mm.

For the New Jersey BMD approach, the flowchart shown in Figure 2 was followed, except that the APA test was substituted for the Hamburg test. Also, the mixture designs utilized were based on current NJDOT approved mix designs. This was done to compare how the current mixtures compared to the BMD approach.

Materials–Mixture Design

NJDOT approved job-mix formulas were procured for 8 different asphalt mixtures commonly used in New Jersey. The mixtures varied in NMAS (i.e., 9.5 and 12.5 mm), as well as asphalt binder grade (i.e., PG 64-22 and PG 76-22). All asphalt mixtures were designed using an N_{design} of 75 gyrations (as noted by the "M"). These include

- Trap Rock Industries (Kingston):
  - 9.5M64 and 9.5M76 and
  - 12.5M64 and 12.5M76.
- Tilcon Mt. Hope:
  - 9.5M64 and 9.5M76 and
  - 12.5M64 and 12.5M76.

Tables 1 and 2 show the aggregate gradations and optimum asphalt content for the NJDOT-approved mixtures.

As noted in the TxDOT BMD flowchart (Figure 2), each of the mixtures evaluated in this study were evaluated under volumetric criteria and performance testing. First, each of the mixtures were compacted to a design gyration level of 75 gyrations and the resultant compacted air voids were calculated at asphalt contents of 4.5%, 5%, 5.5%, and 6.0% asphalt. At the identical asphalt contents, the APA and OT performance specimens were also produced. However, all performance samples were compacted to within an air void range of 5.5% to 6.5% air voids, which represented typical in-situ pavement densities.

TABLE 1 NJDOT-Approved 9.5-mm NMAS

Property	% Passing	
Sieve Size	Tilcon - Mt Hope	Trap Rock
1/2" (12.5 mm)	100	100
3/8" (9.5 mm)	94.4	96.0
No. 4 (4.75 mm)	60.3	64.8
No. 8 (2.36 mm)	36.2	48.6
No. 16 (1.18 mm)	26.9	35.3
No. 30 (0.600 mm)	19.6	24.7
No. 50 (0.425 mm)	12.6	16.5
No. 100 (0.15 mm)	6.9	9.5
No. 200 (0.075 mm)	4.1	5.6
Asphalt Content (%)	5.0	5.4
Design VMA (%)	15.0	17.1
Effective AC by Vol (%)	11.0	13.1

TABLE 2 NJDOT-Approved 12.5-mm NMAS

Property	% Passing	
Sieve Size	Tilcon - Mt Hope	Trap Rock
3/4" (19 mm)	100	100.0
1/2" (12.5 mm)	99.2	94.0
3/8" (9.5 mm)	91.4	86.2
No. 4 (4.75 mm)	57.2	51.5
No. 8 (2.36 mm)	33.9	37.5
No. 16 (1.18 mm)	25.1	24.8
No. 30 (0.600 mm)	18.3	17.9
No. 50 (0.425 mm)	11.8	11.2
No. 100 (0.15 mm)	6.5	7.2
No. 200 (0.075 mm)	3.9	4.8
Asphalt Content (%)	5.1	4.6
Design VMA (%)	14.6	15.3
Effective AC by Vol (%)	10.6	11.3

Tilcon, Mt. Hope Mixtures

9.5-mm NMAS Mixtures

The resultant mixture performance for the 9.5 mm NMAS mixtures are shown in Figure 4 for the 9.5M64 and Figure 5 for the 9.5M76 mixtures, respectively. For the 9.5M64 mixture, the balanced design shows that the optimal range in asphalt content to achieve both good rutting and fatigue cracking properties is 5.2% to 5.9% asphalt content. Meanwhile, the balance design results for the 9.5M76 asphalt mixture indicates that an asphalt content range of approximately

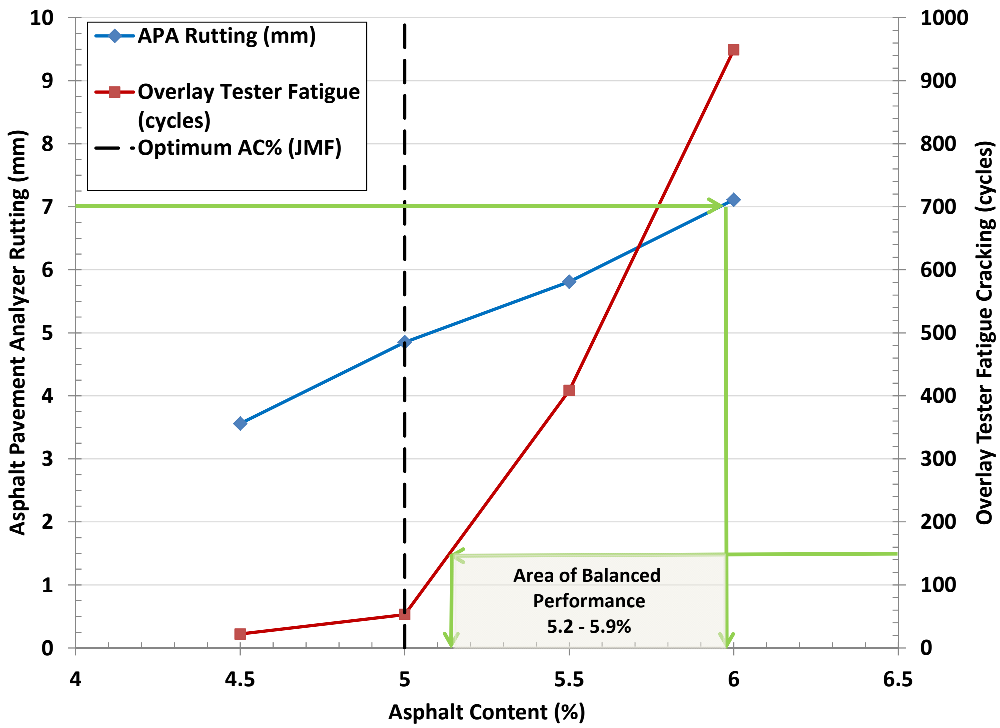


FIGURE 4 Tilcon, Mt. Hope 9.5M64 balanced performance vs. asphalt content.

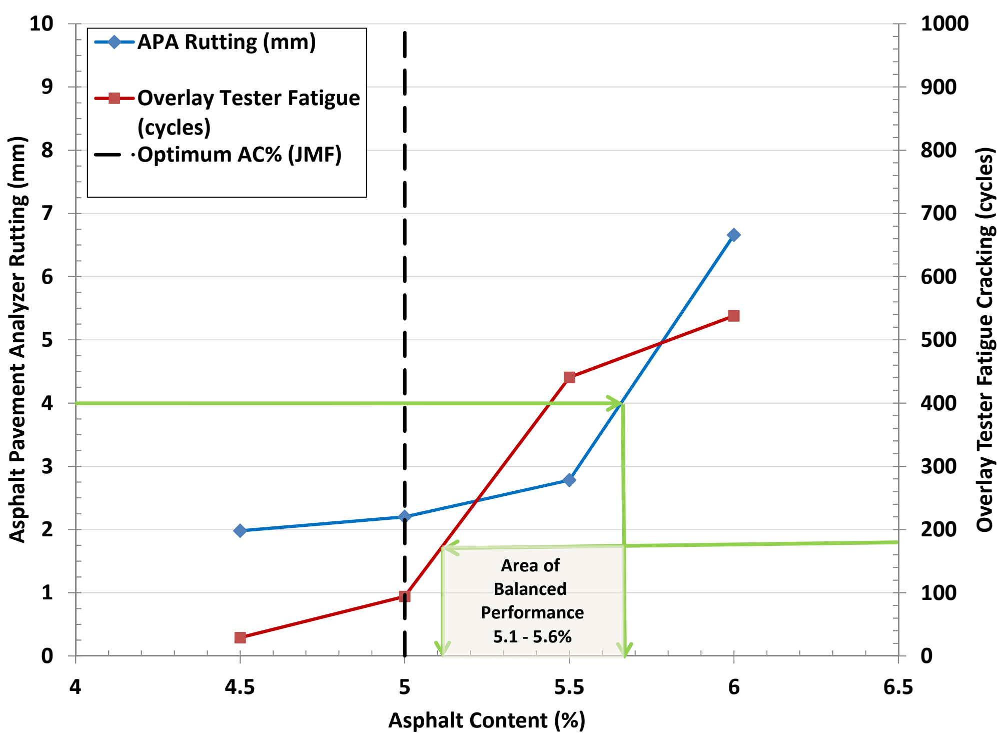


FIGURE 5 Tilcon, Mt. Hope 9.5M76 balanced performance vs. asphalt content.

5.1% to 5.6% would result in an asphalt mixture with good rutting and fatigue resistance. Both of the balanced design results indicate that the volumetric-based design results in under-asphalting the asphalt mixture.

12.5-mm NMAS Mixtures

The Tilcon–Mt. Hope 12.5M64 and 12.5M76 asphalt mixtures are shown in Figures 6 and 7. Similar to the 9.5 mm Tilcon–Mt. Hope mixtures, the balanced design performance results indicated that an optimal asphalt content is higher than what the current volumetric analysis determined. For the 12.5M64 mixtures, the balanced design asphalt content falls between 5.2% and 5.8%, while for the 12.5M76 asphalt mixture, the balanced design calls for an asphalt content between 5.5% and 6.0% asphalt content.

In summary, an increase in asphalt content is required for the Tilcon–Mt. Hope asphalt mixtures when comparing the current volumetic-based asphalt mixture design to the BMD (Table 3). A quick comparison of the volumetrics at optimum asphalt content, as determined from the middle of the balanced design range show that, on average, design air voids would actually need to be reduced to almost 3.0% air voids. However, the average is not well-defined and shows that it varies with mixture type and its respective components, and not a universally defined, as we currently assume it to be under volumetric design.

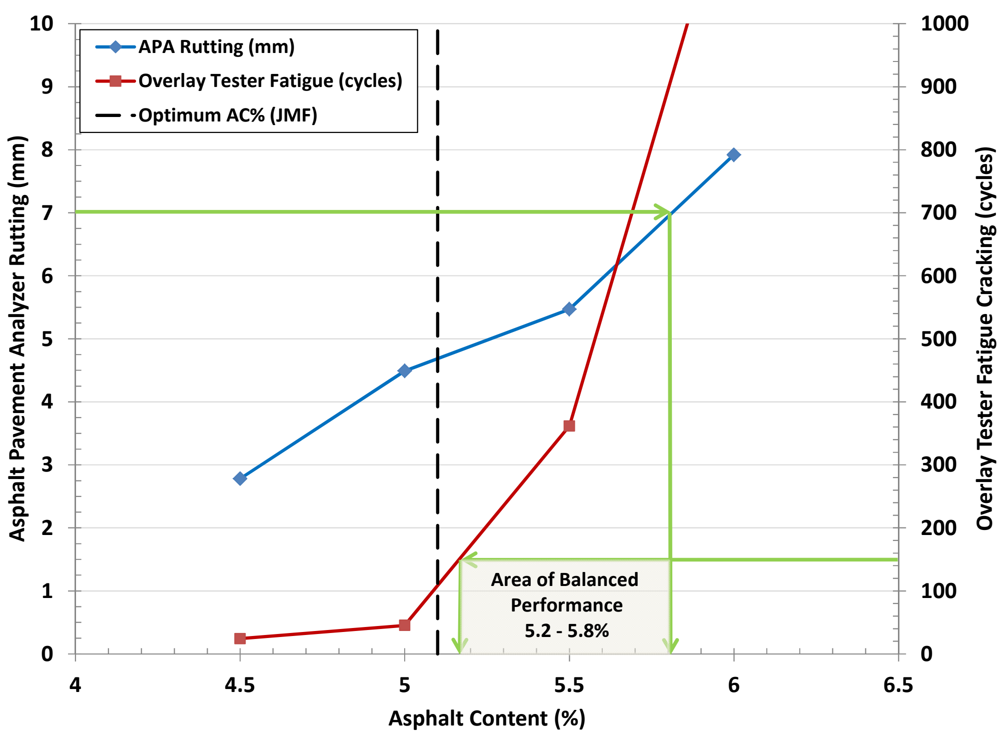


FIGURE 6 Tilcon–Mt. Hope 12.5M64 balanced performance vs. asphalt content.

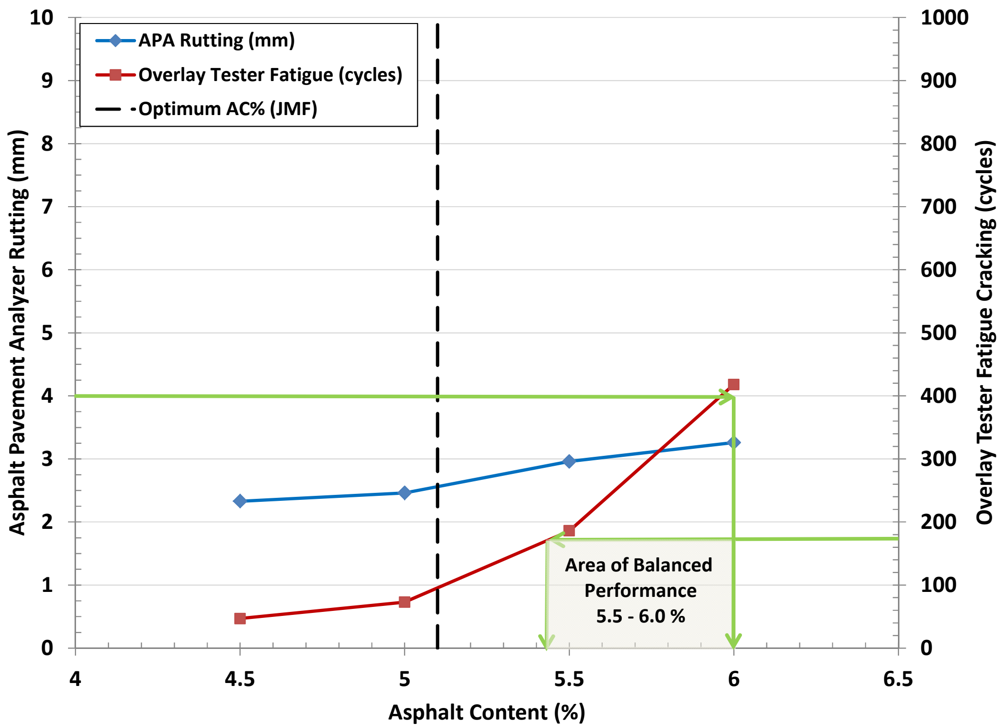


FIGURE 7 Tilcon–Mt. Hope 12.5M76 balanced performance vs asphalt content.

TABLE 3 Summary of Determined Asphalt Contents for Tilcon–Mt. Hope Asphalt Mixtures

Mix Type (Mt Hope)	Volumetric Optimum AC% (N_{des} = 75	Balanced Mix Design	
		Optimum AC (%)	Air Voids @ AC% (N_{des} = 75 gyrations)
#1, 9.5M64	5.0	5.2 - 5.9 (5.6%)	2.8
#1, 9.5M76	5.0	5.1 - 5.6 (5.4%)	3.9
#1, 12.5M64	5.1	5.2 - 5.8 (5.5%)	3.0
#1, 12.5M76	5.1	5.5 - 6.0 (5.8%)	3.5

Trap Rock Industries Mixtures

Similar to the Tilcon–Mt. Hope mixtures, four different asphalt mixtures were produced using aggregates and RAP materials from Trap Rock Industries (TRI). For the volumetric analysis, three specimens were mixed for each asphalt content and compacted to a design gyration level of 75 gyrations. For each asphalt content, the average compacted air voids were determined. The balanced design specimens were produced in a similar manner and were evaluated for their respective rutting resistance and fatigue resistance using the APA and OT.

9.5-mm NMAS Mixtures

The results for the 9.5-mm NMAS TRI mixtures are shown below. Figures 8 and 9 present the test results for the 9.5M64 and 9.5M76 asphalt mixtures. The balanced design results for the 9.5M64 asphalt mixture indicates that a range of asphalt content of 5.2% to 5.9% would result in a good performing asphalt mixture that is balanced for both rutting and fatigue cracking resistance. Meanwhile, the balanced design results for the 9.5M76 TRI indicates that an optimal range of asphalt content to achieve a rutting and fatigue resistance mixture should be approximately 5.8% to 6.0%. This is approximately 0.5% higher than what the currently approved asphalt mixture contains.

12.5-mm NMAS Mixtures

Both the NJDOT-approved PG 64-22 and PG 76-22 12.5M asphalt mixtures from TRI was evaluated for their volumetric and balanced blend performance properties. Figures 10 and 11 show the results for the respective results. The balanced design performance indicated for the 12.5M64 asphalt mixture indicates that a range between 5.1% to 6.1% asphalt content would provide the balanced design. This was the widest range of potential asphalt contents found in the balanced blend analysis work. Meanwhile, the TRI 12.5M76 asphalt mixture is shown in Figure 11. The balanced design performance shows an optimum asphalt content in the range of 5.6% to 6.1% asphalt binder. This is approximately 1% more asphalt binder required than the volumetric design method indicated.

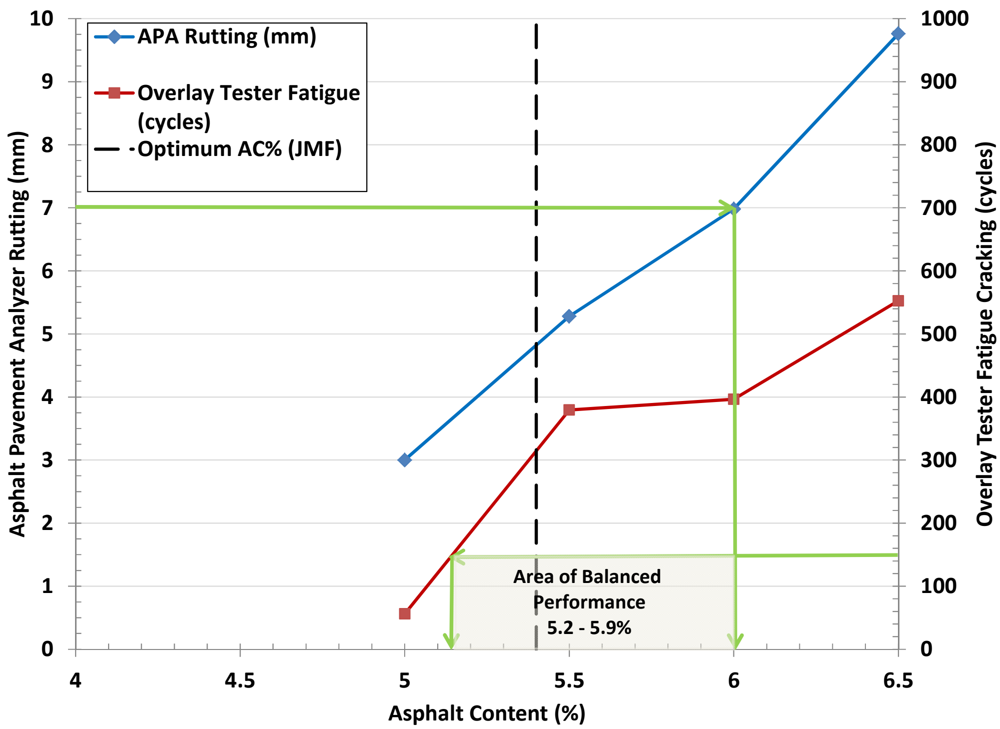


FIGURE 8 TRI 9.5M64 balanced performance vs. asphalt content.

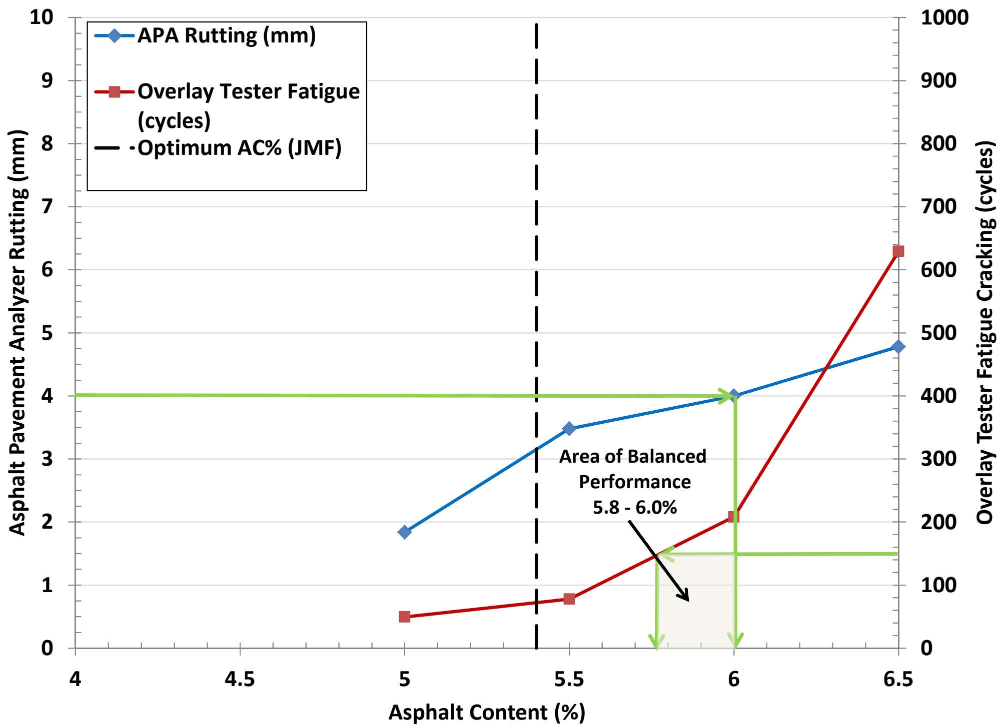


FIGURE 9 TRI 9.5M76 balanced performance vs. asphalt content.

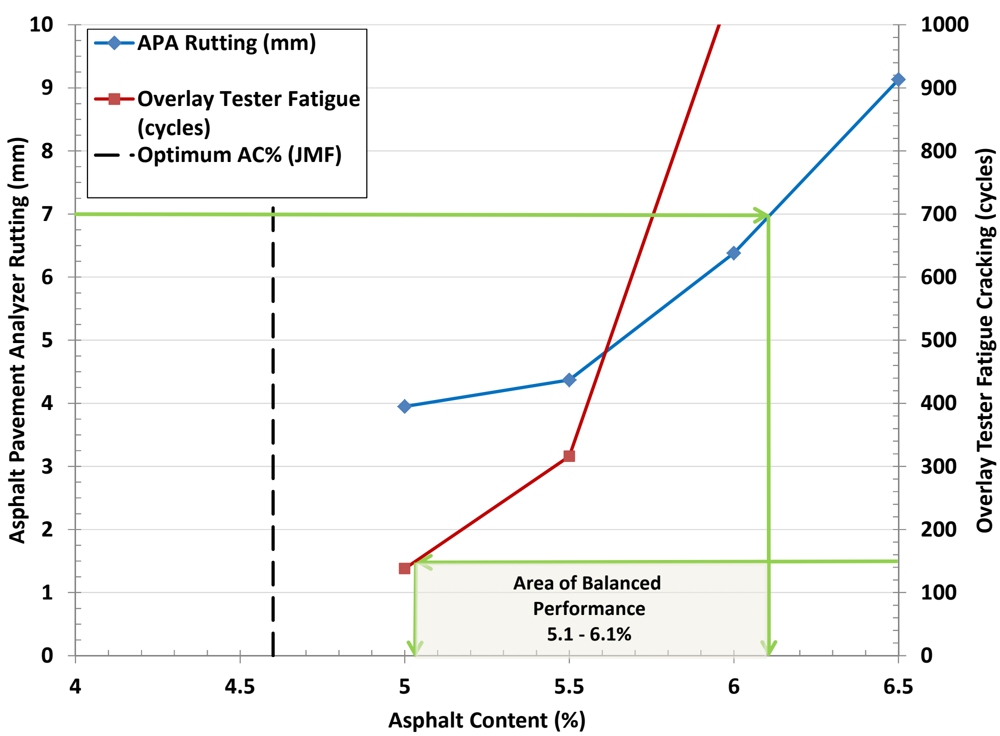


FIGURE 10 TRI 12.5M64 balanced performance vs. asphalt content.

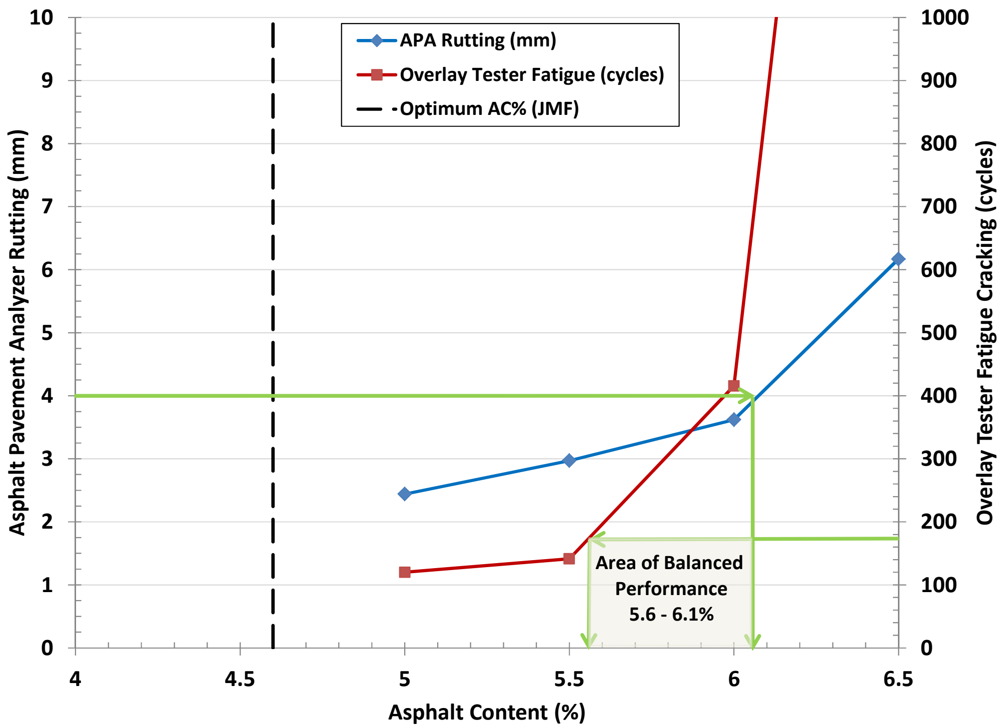


FIGURE 11 TRI 12.5M76 balanced performance vs. asphalt content.

A summary of the TRI asphalt mixtures are shown in Table 4. Similar to the Mt. Hope asphalt mixtures, the balanced design procedure indicates a higher asphalt content is required than what the volumetric method currently provides. Again using the center of the range as “optimum” asphalt content, an equivalent target air void level would again be close to 3.0%, yet again, there is variation indicating not all asphalt mixtures have the same volumetric requirements.

Using Balanced Mixture Design to Improve Volumetric Specifications

Durability in asphalt mixtures is a broad characteristic, but it generally covers the asphalt mixtures’ ability to resist cracking, raveling, and brittle-type failures. For years, pavement

TABLE 4 Summary of Determined Asphalt Contents for TRI Asphalt Mixtures

Mix Type (Trap Rock Industries)	Volumetric Optimum AC% (N_{des} = 75	Balanced Mix Design	
		Optimum AC (%)	Air Voids @ AC% (N_{des} = 75 gyrations)
#2, 9.5M64	5.4	5.2 - 5.9 (5.6%)	2.9
#2, 9.5M76	5.4	5.8 - 6.0 (5.9%)	3
#2, 12.5M64	4.6	5.1 - 6.1 (5.6%)	2.8
#2, 12.5M76	4.6	5.6 - 6.1 (5.9%)	3.4

engineers and asphalt material technicians have utilized the volumetric property VMA as a general "durability index" parameter. In general, the higher the VMA at design, the greater the amount of effective asphalt (by volume), the better the durability of the asphalt mixture. Finer aggregate gradations, with increased surface area, require higher levels of VMA to ensure adequate asphalt film thickness around the aggregates occur. The current NJDOT volumetric requirements for asphalt mixture design are shown in Table 5. Unfortunately, since the VMA is comprised of both effective asphalt content and air voids, established criteria for VMA only have meaning during mixture design and asphalt plant QC testing. Therefore, instead of utilizing VMA as an indicator of durability, it was proposed in this study to look at the effective binder content by volume (EBCV). Since the EBCV does not change like VMA due to varying air voids, the EBCV is a much more stable parameter and can be easily evaluated and compared to during a BMD approach.

The balanced design performance test results for all eight asphalt mixtures evaluated are shown in Figure 12. The test results do show some scatter, which would be expected since the data is comprised of different PG grades, different NMAS, and different aggregate sources. However, the trend shows that as the EBCV increases:

- The rutting potential increases as measured in the APA; and
- The fatigue cracking potential decreases as measured in the OT.

Both the 9.5-mm NMAS and 12.5-mm NMAS mixtures were separated out, along with the PG grade of the asphalt binder in Figures 13 and 14, to illustrate where the performance of the asphalt mixtures fit into the current design VMA specifications. Since VMA is the EBCV plus the air voids, simply subtracting 4% air voids from the VMA criteria results in the EBCV. For 9.5-mm asphalt mixtures evaluated, a design VMA of 15% (resulting in an EBCVof 11%), an average OT value of approximately 225 cycles results. Figure 13 also shows that the 9.5-mm mixtures evaluated show good rutting resistance when compared to the proposed APA criteria. Similar observations were made when looking at the results of the 12.5-mm NMAS mixtures evaluated (Figure 14). Both the PG 64-22 and PG 76-22 mixtures were found to meet the rutting requirement while the OT was approximately 300 cycles.

TABLE 5 NJDOT Asphalt Mixture Design Volumetric Requirements

Table 902.02.03-3 HMA Requirements for Design

Compaction Levels	Required Density(% of Theoretical Max. Specific Gravity)		Voids in Mineral Aggregate (VMA),% (minimum) Nominal Max. Aggregate Size, mm						Voids Filled With Asphalt (VFA)[1] %	Dust-to-Binder Ratio
	$@N_{des}$[2]	$@N_{max}$	37.5	25.0	19.0	12.5	9.5	4.75		
L	96.0	≤ 98.0	11.0	12.0	13.0	14.0	15.0	16.0	70 - 80	0.6 - 1.2
M	96.0	≤ 98.0	11.0	12.0	13.0	14.0	15.0	16.0	65 - 78	0.6 - 1.2
H	96.0	≤ 98.0	11.0	12.0	13.0	14.0	15.0	16.0	65 - 75	0.6 - 1.2

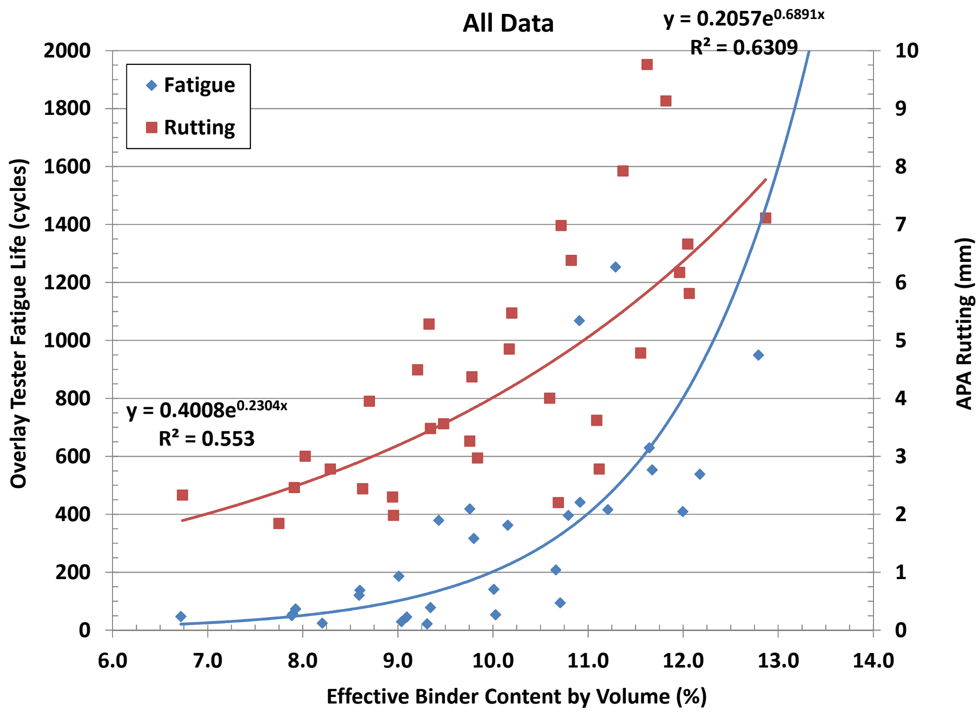


FIGURE 12 EBCV (%) vs. balanced design performance, all test data.

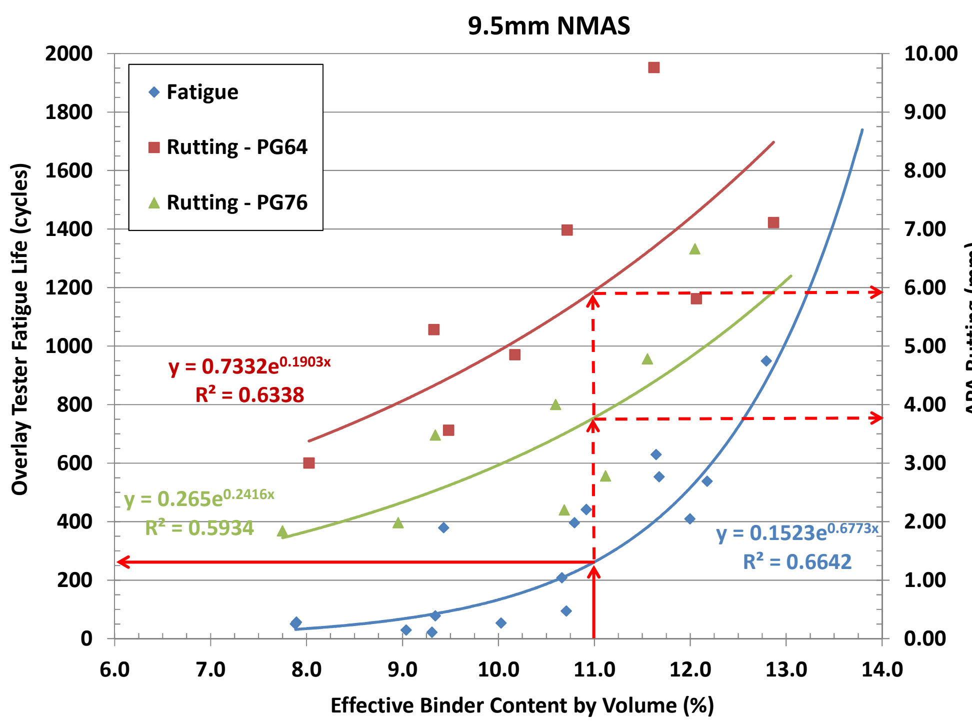


FIGURE 13 EBCV (%) vs. balanced design performance—
9.5-mm NMAS with current NJDOT design VMA criteria.

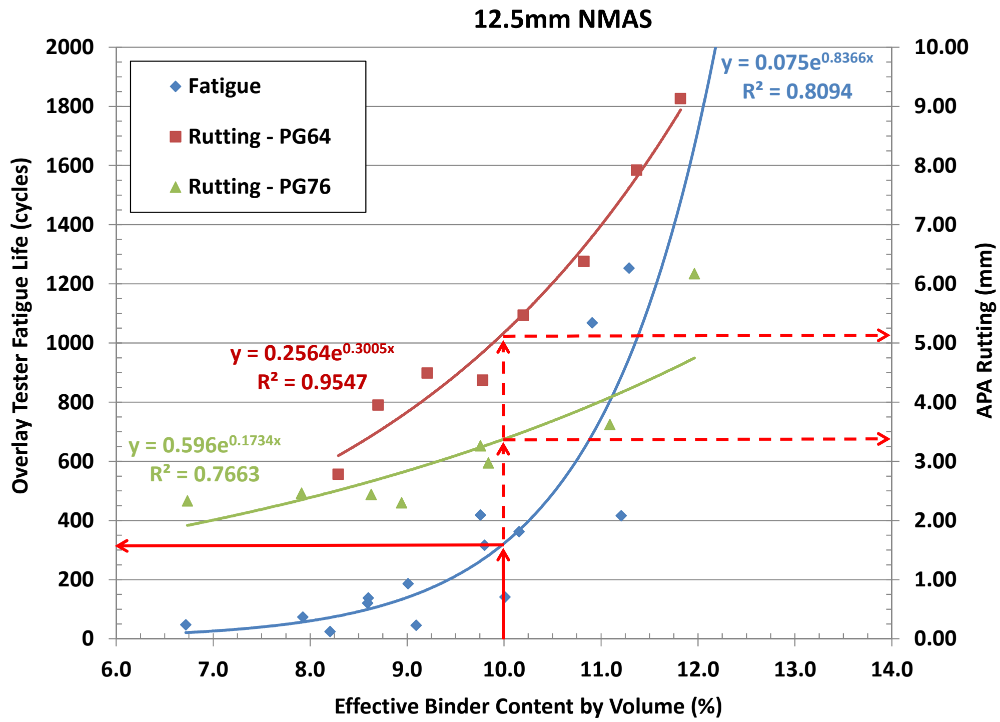


FIGURE 14 EBCV (%) vs. balanced design performance: 12.5-mm NMAS with current NJDOT design VMA criteria.

Both the 9.5- and 12.5-mm asphalt mixtures were found to meet the minimum OT requirements developed in this study while still meeting the rutting. However, with the current asphalt binder production tolerances, the effective asphalt content by volume would most likely decrease as asphalt suppliers are commonly producing asphalt mixtures towards the lower end of the allowable production tolerance. To help ensure enough asphalt binder is in the mixture to achieve higher effective asphalt content by volume values, it is proposed to look at increasing the design VMA by 1%. This would ultimately increase the effective asphalt content by volume by 1% as well. Figures 15 and 16 show the same data set generated during this study compared to the proposed 1% increase in the design VMA (resulting in a 1% increase in the EBCV). The proposed increase in EBCV shows:

- An average improvement in the overlay fatigue resistance of 58% when comparing the current design VMA spec to the proposed design VMA spec.
- An average increase in the APA rutting of 19% when comparing the current design VMA spec to the proposed design VMA spec. However, even though there was an increase in the APA rutting, only the 9.5-mm NMAS with PG 76-22 asphalt binder exceeded the maximum recommended APA rutting (i.e., 4.0 mm for a PG 76-22 asphalt binder).

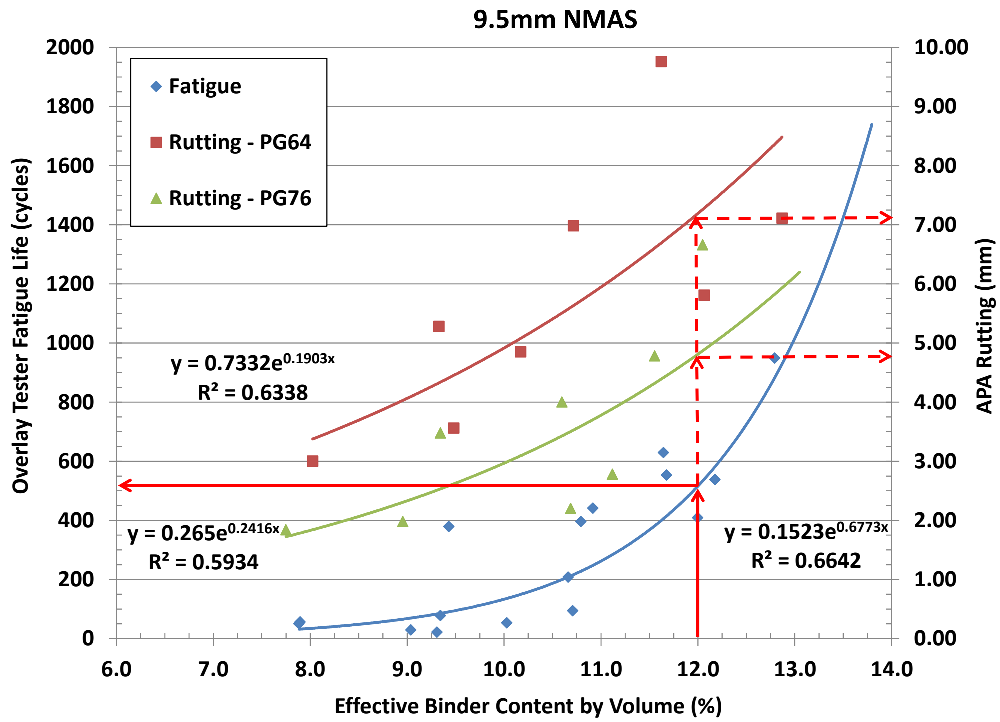


FIGURE 15 EBCV (%) vs. balanced design performance— 9.5-mm NMAS with proposed NJDOT design VMA criteria.

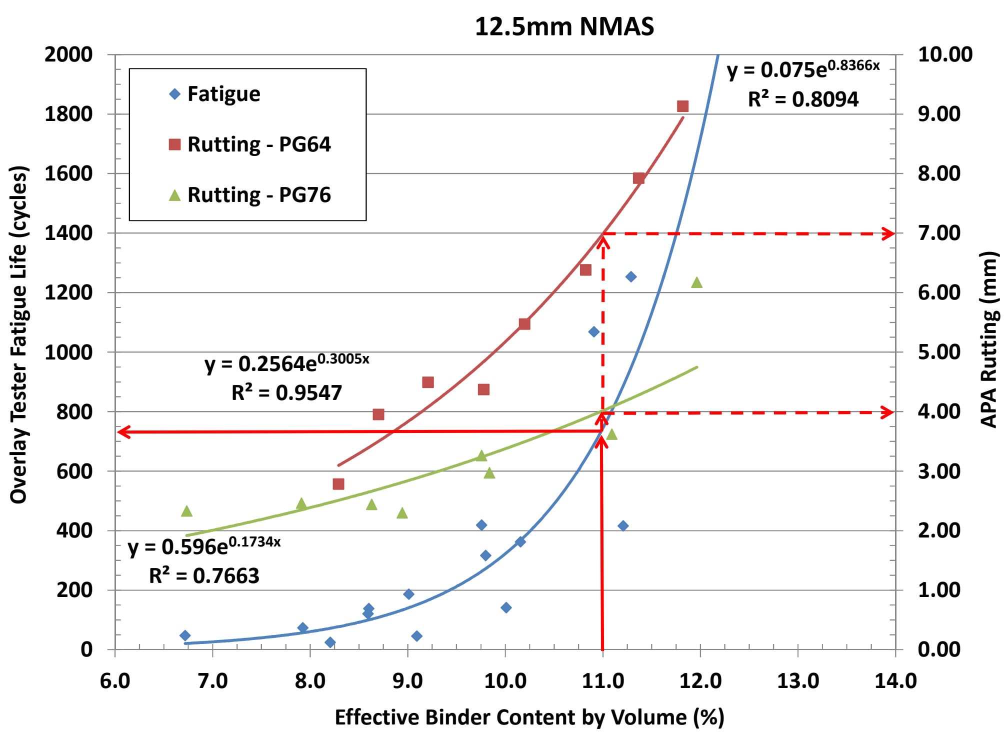


FIGURE 16 EBCV (%) vs. balanced design performance— 12.5-mm NMAS with proposed NJDOT design VMA criteria.

SUMMARY OF BALANCED MIXTURE DESIGN WORK FOR NEW JERSEY

A new mixture approach was evaluated to determine its applicability to New Jersey asphalt mixtures. The methodology, called BMD, incorporates an asphalt rutting and fatigue test to determine the appropriate asphalt content instead of the current volumetric procedure outlined in Superpave. And even though volumetric (air voids, VMA) are measured, ultimately the methodology relies on the performance (rutting and fatigue) of the asphalt mixture. The methodology is beneficial over conventional volumetric design procedures as a state agency can established threshold criteria that would provide them with a level of assurance that the asphalt mixture designed and produced will meet some level of field performance expectations.

The results of the BMD demonstrated that for almost all mixtures evaluated, an increase in asphalt content was required over the current NJDOT approved mixture design. It is apparent that asphalt mixtures produced in New Jersey are under-asphalted based on the fatigue cracking requirements. Based on the information generated in this study, the mixtures are under-asphalted on average by 0.6%. Although this was a limited dataset, the general trend is still troubling but does mirror typical field observations.

The BMD methodology was also showed that it could be utilized to evaluate current state agency volumetric specifications and determine if current values need to be edited. Similar work can be done for ABR when utilizing RAP and/or RAS, although this was not shown in this study.

FUTURE NEEDS TO IMPLEMENT BALANCED MIXTURE DESIGN

Although the general methodology of BMD may sound easy to apply, there are a number of obstacles that a state agency would still need to determine. The two major ones being determining “optimum” asphalt content and also establishing production tolerances.

When utilizing BMD, it is typical that a range of “balanced” performance occurs, as what was shown earlier. However, now that a range is determined, how does an agency actually specify “optimum” asphalt content? Is it simply the middle of the range? Does an agency take a similar approach to Hveem and use the highest asphalt content possible until rutting/stability becomes an issue?

Regarding production tolerances, would a state agency target the center of the Balanced range and use the Balanced range as tolerances? Should current agency production tolerances be incorporated using the “balanced” asphalt content? These are a few questions that agencies need to consider before moving towards BMD. However, the methodology does seem to provide an improvement over current volumetric procedures, especially when considering how to enhance durability and fatigue resistance of asphalt mixtures.

REFERENCES

1. Hveem, F. N. Gradation of Mineral Aggregate for Dense Graded Bituminous Mixtures. *Proceedings Association of Asphalt Paving Technologists*, Volume 11, 1940.
2. Zhou, F., S. Hu, and T. Scullion. *Integrated Asphalt (Overlay) Mixture Design, Balancing Rutting and Cracking Requirements*. FHWA/TX-06/0-5123-1. Texas Department of Transportation, 2006,162 pp.

3. Rand, D. *Balanced HMAC Designs*. Presented at IHEEP Conference, San Antonio, Texas, September 29, 2009.

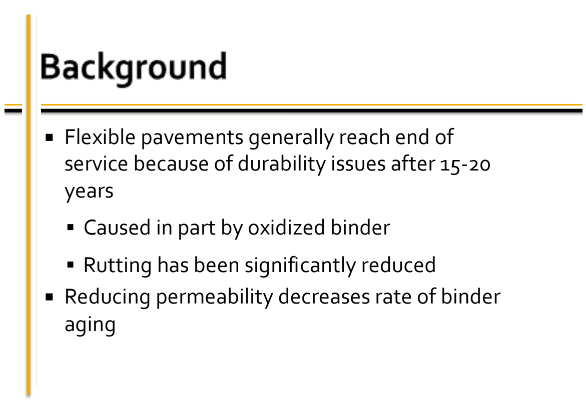
Mixture Design for Better Field Compaction

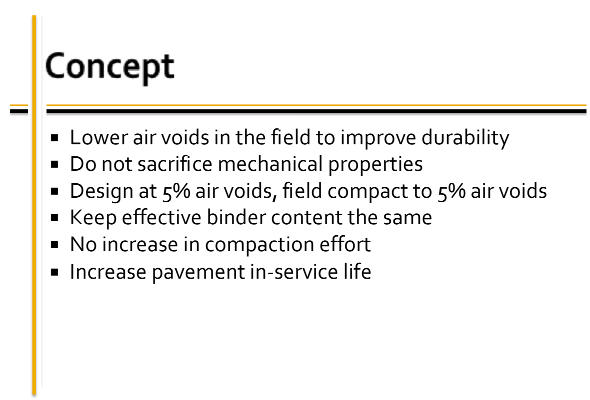
JOHN E. HADDOCK
Purdue University

In Indiana, rutting distress has been significantly reduced, and in many cases eliminated and most flexible pavements reach the end of their 15- to 20-year service lives due to durability issues, caused in part by asphalt binder oxidation. Reducing the in-place permeability of asphalt mixtures can aid in reducing the rate of in-service binder aging, thus increasing asphalt mixture durability and extending pavement service lives.

Decreasing in-place air voids in asphalt mixtures will reduce in-place asphalt mixture permeability and thereby increase asphalt pavement durability. However, when decreasing in-place air voids it is imperative that good mechanical mixture properties not be sacrificed. This research proposes to alter the standard mixture design procedure to determine optimum binder content at 5% air voids, rather than 4%, and then compact the resulting mixtures to 5% in-place air voids (95% G_{mm} density), rather than the more typical 7% to 8% in-place air voids (92% to 93% G_{mm} density) during construction.

When changing the mixture design procedure it is important the effective binder content (P_{be}) not be decreased from that P_{be} that would result from a standard 4%-designed mixture, thus precluding an increase in the design air voids by simply removing asphalt binder from the mixture, a result that would act to decrease mixture durability. Additionally, any mixture designed at 5% air voids should not require a compactive effort to reach 95% G_{mm} density over and above the effort typically needed to achieve current in-place densities for standard mixtures. A proof-of-concept laboratory study was conceived to use three standard 100-gyration Superpave-designed mixtures that were previously placed on paving projects in Indiana. Each standard mixture was to be re-designed three times, once each at gyrations levels of 30, 50, and 70. For each of the re-designed mixtures, optimum binder content would be chosen at 5% air voids, while maintaining the re-designed mixture P_{be} equivalent to the corresponding standard mixture.





Scope

- Laboratory study (proof of concept)
  - Design 3 standard mixtures
  - Re-design each mixture at 5% air voids
    - Maintain effective binder content
    - Use 70, 50, 30 gyrations
  - Test all mixtures for dynamic modulus and flow number (anticipated in-service air voids)
- Place field projects

Experimental Matrix

Traffic (MESAL)	No. Gyrations	9.5-mm	19.0-mm
Category 3 (3-10)	30	x	
	50	x	
	70	x	
Category 4 (10-30)	30	x	x
	50	x	x
	70	x	x

Once the redesigned mixtures were designed, specimens would be produced and tested in the laboratory to determine dynamic modulus and flow number, this data being compared to that of the corresponding standard mixtures. If the redesigned mixtures have dynamic modulus and flow numbers as good as, or better than the corresponding standard mixtures, then the redesigned mixture would be judged to have adequate mechanical properties. For this process, the standard mixture specimens would be compacted to 7±1% air voids, as is the current standard test method. However, the redesigned mixture specimens would be compacted to 5±1% air voids, since it is anticipated this would be the in-service air voids level.

If the 5% design concept proved viable, trial field projects could be placed to determine if the mixtures could be compacted in the field without additional compactive effort.

The experimental matrix shows the three mixtures chosen for the laboratory study: one Category 3 9.5-mm mixture and two Category 4 mixtures, one 9.5 mm and one 19.0 mm. The "category" number is an Indiana DOT (INDOT) convention denoting traffic levels. Both Categories 3 and 4 require 100-gyration mixture designs, but Category 4 requires specific types of aggregates, typically a blend of dolomite and blast furnace slag.

Materials for the redesigned mixtures were the same as those used in the three standard mixtures, limestone, dolomite, and blast furnace slag coarse aggregates, and limestone, dolomite, and natural sand fine aggregates. A PG 64-22 asphalt binder was used for each mixture. No recycled materials were used in the laboratory experiment, although both RAP and RAS are widely used in Indiana. Recycled materials were excluded to avoid confounding the experiment with too many variables.

Data for the Category 4, 19.0-mm mixture show the P_{be} of the three redesigned mixtures are essentially equivalent to the standard, 100-gyration mixture. Of course the VMA of the redesigned mixtures are 1% higher than that of the standard mixture because the air voids are 1% greater.

The Category 4, 19.0-mm mixture gradations are slightly different, mainly in the larger aggregate sizes. Changing the aggregate gradations of the mixtures facilitates raising the design air voids by 1% while maintaining constant P_{be}, and varying the number of design gyrations.

The dynamic modulus master curves for the Category 4, 19.0-mm mixtures indicate that all three redesigned mixtures (30, 50, and 70 gyrations) have stiffness values as high, or higher than does the standard (100 gyration) mixture, indicating their mechanical performance should be as good as the latter.

Materials

- Coarse aggregates
  - Limestone, dolomite, blast furnace slag
- Fine aggregates
  - Limestone, dolomite, natural sand
- PG 64-22
- No recycled materials

Category 4, 19.0-mm Designs

	N100	N70	N50	N30
P_b, %	4.7	4.7	5.1	5.1
P_{be}, %	4.1	4.1	4.1	4.3
V_a, %	4.0	4.9	4.9	4.9
VMA, %	13.6	14.5	14.4	14.9
VFA, %	70.6	66.3	66.0	67.2

Standard and re-designed mixture specimens have different air void contents, as noted earlier. The 100-gryation mixture specimens tested had air voids of 7±1%, while redesigned mixture specimens had air voids of 5±1%.

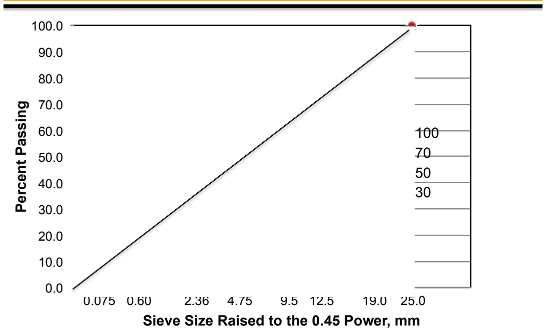
The flow number data for the Category 4, 19.0-mm mixture indicates the redesigned mixtures all had flow numbers equivalent to, or higher than the standard 100-gyration mixture. This result suggests the redesigned mixtures should have rutting resistance equivalent to, or better than the standard mixture.

Data for the Category 3, 9.5-mm mixture show the P_{be} of the three redesigned mixtures are essentially equivalent to the standard, 100-gyration mixture. Again, the VMA of the redesigned mixtures are approximately 1% higher than that of the standard mixture because the air voids are 1% greater.

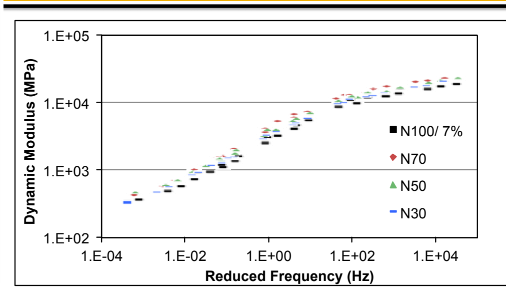
The Category 3, 9.5-mm mixture gradations are slightly different, mainly in the intermediate aggregate sizes.

The dynamic modulus master curves for the Category 3, 9.5-mm mixtures indicate that all three redesigned mixtures (30, 50, and 70 gyrations) have stiffness values as high, or higher than does the standard (100 gyration) mixture, indicating their mechanical performance should be as good as the latter.

Category 4, 19.0-mm



Category 4, 19.0-mm



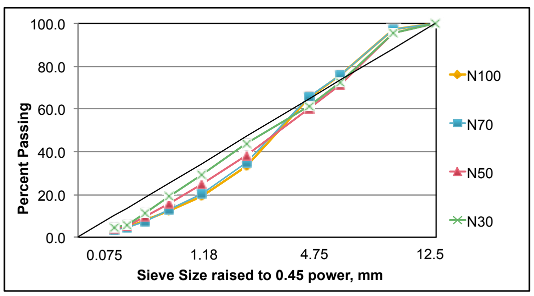
Category 4, 19.0-mm

Gyrations	Average Flow Number	Average Strain at FN (µm)
100	162	23,983
70	386	18,269
50	348	19,882
30	185	22,090

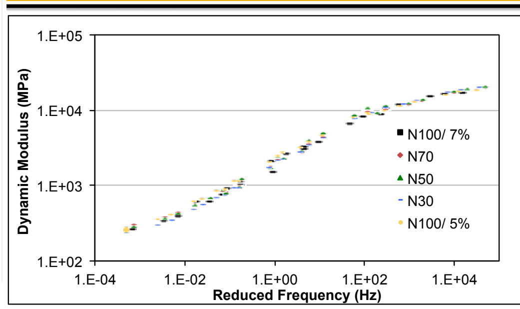
Category 3, 9.5-mm Designs

	N100	N70	N50	N30
P_b, %	5.9	5.9	6.0	6.0
P_{be}, %	4.6	4.6	4.6	4.7
V_a, %	4.1	5.1	4.9	5.3
VMA, %	15.0	16.0	15.8	16.3
VFA, %	72.9	67.9	68.9	67.6

Category 3, 9.5-mm



Category 3, 9.5-mm



As indicated previously, the standard and redesigned mixture specimens have different air void contents. The 100-gryation mixture specimens tested had air voids of 7±1%, while redesigned mixture specimens had air voids of 5±1%. However, in this case, a set of standard mixture specimens were produced at 5±1% air voids and tested for comparison purposes. The data indicate this set of specimens had dynamic modulus master curve equivalent to the other mixture specimens.

The flow number data for the Category 3, 9.5-mm mixture indicates the redesigned mixtures all had flow numbers higher than the standard 100-gyration mixture and equivalent to the standard mixture specimens compacted to 5±1% air voids. This result suggests the redesigned mixtures should have rutting resistance equivalent to, or better than the standard mixture.

After completing the first two mixture redesigns and gathering mechanical data from them, it was obvious that little difference existed between the standard 100-gyration mixture and the corresponding 70-gyration design. Removing 30 gyrations from a mixture raises the mixture air voids by 1%, leaving the gradation and binder content virtually unchanged. Therefore, to save time and materials, for the Category 4, 9.5-mm mixture redesigns were completed for only 30 and 50 gyrations.

Data for the Category 4, 9.5-mm mixture show the P_{be} of the two re-designed mixtures are essentially equivalent to the standard, 100-gyration mixture. As expected, the VMA of the

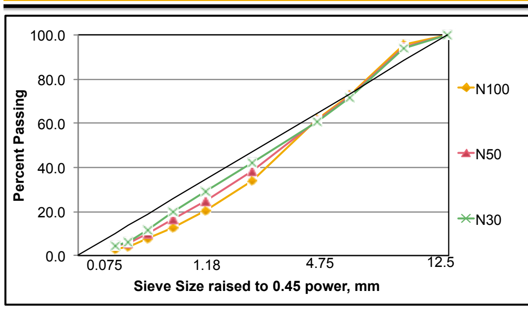
Category 3, 9.5-mm

Gyrations	Average Flow Number	Average Strain at FN (μm)
100- 7%	91	18,114
100- 5%	166	18,174
70	167	17,704
50	163	20,300
30	156	19,204

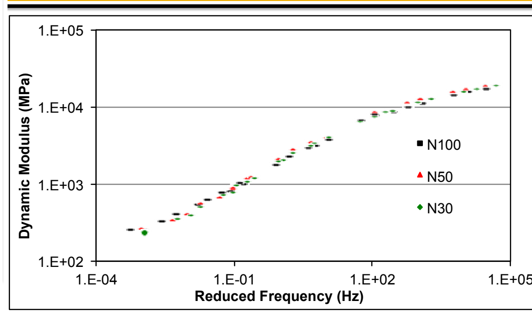
Category 4, 9.5-mm Designs

	N100	N50	N30
P_b, %	6.5	6.4	6.4
P_{be}, %	4.8	5.0	5.0
V_a, %	3.8	4.9	5.0
VMA, %	15.0	16.4	16.4
VFA, %	74.9	70.0	69.6

Category 4, 9.5-mm



Category 4, 9.5-mm



redesigned mixtures are approximately 1% higher than that of the standard mixture because the air voids are 1% greater.

As with the Category 3, 9.5-mm mixture, the Category 4, 9.5-mm mixture gradations are slightly different, mainly in the intermediate aggregate sizes.

The dynamic modulus master curves for the Category 4, 9.5-mm mixtures indicate that all both redesigned mixtures (30 and 50 gyrations) have stiffness values as high, or higher than does the standard (100 gyration) mixture, indicating their mechanical performance should be as good as the latter.

Again, standard and redesigned mixture specimens have different air void contents, as noted earlier.

The flow number data for the Category 4, 9.5-mm mixture indicates the redesigned mixtures have flow numbers higher than the standard 100-gyration mixture, again suggesting the redesigned mixtures should have rutting resistance equivalent to, or better than the standard mixture.

The proof-of-concept having been considered promising, it was decided to place a field trial. The INDOT chose SR-13 near Ft. Wayne, Indiana, as the site for the trial. A Category 4, 9.5-mm overlay was being placed on SR-13, providing an opportunity to re-design the mixture and place trial mixture. The original mixture placed on the project was designed using the standard 100-gyration Superpave design method with optimum binder contend chosen at 4% air voids; in-place air voids were anticipated to be 7% (93% G_{mm} density) during construction.

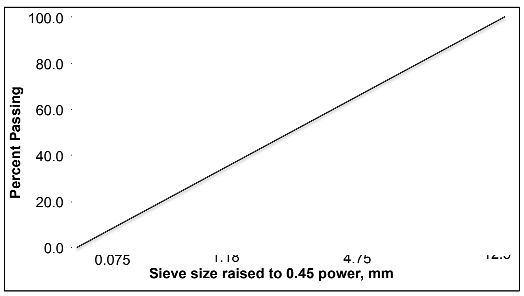
Category 4, 9.5-mm

Gyrations	Average Flow Number	Average Strain at FN (µm)
100	160	23,983
50	253	20,935
30	211	21,033

Field Trial 1

- SR-13 near Ft. Wayne, IN
- New overlay, Category 4, 9.5-mm
- Original design, N100, 4%, 7%
- Redesigned, N50, 5%, 5%
- Steel slag and limestone coarse aggregates, limestone and natural sands, RAS, PG 70-22

Mixture Gradations



In-place Densities

- N100, 18 cores, average density 91.8%
- N50, 6 cores, average density 94.7%
- Same rollers and rolling patterns

The redesigned mixture used 50 gyrations of the SGC and optimum binder content was chosen at 5% air voids. The P_{be} of the redesigned mixture was approximately equivalent to the original mixture design, with the resulting VMA being 1% higher. The in-place air voids goal was 5% (95% G_{mm} density).

The same materials were used for both the original and redesigned mixtures. Limestone and steel slag coarse aggregates were blended with limestone and natural sand fine aggregates and mixed with a PG 70-22 binder. RAS was also included in both mixtures.

The mixture gradations are slightly different, mainly in the intermediate aggregate sizes.

The INDOT considers 600 tons to be one sub-lot of asphalt surface mixture. Nine sub-lots of the original mixture and three sub-lots of the redesigned mixture were produced and placed. Two cores were taken from each sub-lot to determine the in-place densities. The average in-place densities were 91.8% (8.2% air voids) and 94.7% (5.3% air voids) for the original and re-designed mixtures respectively. The former is fairly typical for standard mixtures. The latter is close to the project goal of 5%.

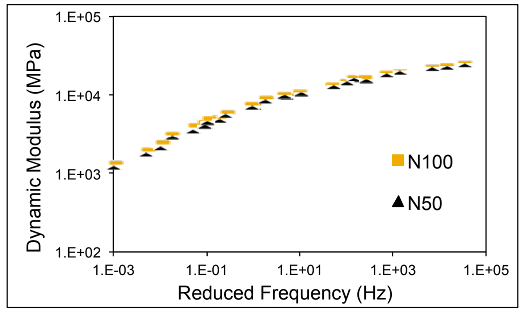
The increase in density was achieved with the same rollers and roller patterns for both mixtures.

During construction, mixture samples from both the original and redesigned mixtures were collected and returned to the laboratory. These mixtures were used to prepare laboratory-compacted specimens for dynamic modulus, flow number, and BBF testing.

Post-Construction Testing

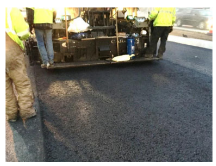
- Plant-mixed, laboratory compacted
  - Dynamic modulus, flow number, fatigue properties (beam), binder recovery and grading

Dynamic Modulus Results



Field Trial 2

- Georgetown Road, Indianapolis , IN
- Intermediate layer, Category 3, 19.0-mm
- Original design, N100, 4%, 7%
- Redesigned, N30, 5%, 5%
- Limestone coarse aggregates, dolomite sand, RAS, RAP, PG 64-22



In-place Densities

- N100, 20 cores, average density 94.0%
- N30, 20 cores, average density 95.2%
- Same rollers and rolling patterns

Sampled mixture was also used to extract, recover, and grade the asphalt binders from the two mixtures.

For the sake of brevity, only the dynamic modulus master curves data from the project is shown. The data indicate the two mixtures have similar master curves. The stiffness values of the redesigned mixture should be similar to the original mixture, indicating its mechanical performance should be as good as the latter.

Standard and redesigned mixture specimens had air void contents of 7±1% and 5±1%, respectively.

The SR-13 project is currently in its second year of life and no problems have been encountered to date. The INDOT plans on coring the original and redesigned sections again to compare the densities.

After the success of the SR-13 project, it was decided to place a second field trial, this time on Georgetown Road in Indianapolis. A Category 3, 19.0-mm intermediate layer was placed, the original mixture designed using the standard 100-gyration Superpave design method with optimum binder contend chosen at 4% air voids; in-place air voids were anticipated to be 7% (93% G_{mm} density) during construction.

The redesigned mixture used 30 gyrations of the SGC and optimum binder content was chosen at 5% air voids. The P_{be} of the redesigned mixture was approximately equivalent to the original mixture design, with the resulting VMA being 1% higher. The in-place air voids goal was 5% (95% G_{mm} density).

The same materials were used for both the original and redesigned mixtures, limestone coarse aggregate blended with dolomite fine aggregate and mixed with a PG 64-22 binder. Both RAP and RAS were included in both mixtures.

Twenty cores were taken from each of the two mixture sections to determine the in-place densities. The average in-place densities were 94.0% (6.0% air voids) and 95.2% (4.8% air voids) for the original and redesigned mixtures respectively. These density values are good, especially since the placement of both sections were done in early December. In this case, the project goal of 5% air voids was slightly exceeded.

Both sections were compacted with the same rollers and roller patterns.

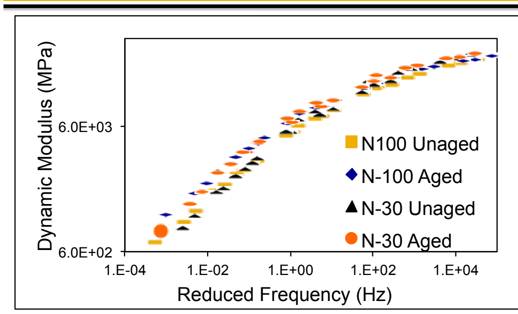
During construction, mixture samples from both the original and redesigned mixtures were collected and returned to the laboratory. These mixtures were used to prepare laboratory-compacted specimens for dynamic modulus, flow number, and SCB fatigue testing.

The 20 cores from each of the two sections were used to determine in-place densities, dynamic modulus testing, SCB fatigue testing, and HWTT. Mixture from cores was also used to extract, recover, and grade the asphalt binders from the two mixtures.

Post-Construction Testing

- Plant-mixed, laboratory compacted
  - Dynamic modulus, flow number, fatigue properties (SCB)
- Plant-mixed, field compacted (cores)
  - Density, dynamic modulus, fatigue properties (SCB), Hamburg wheel test, binder recovery and grading

Dynamic Modulus Results



Conclusions

- Mixtures can be designed at 5% air voids without lowering effective binder content
- Mixtures designed and placed at 5% air voids can have equivalent mechanical properties as traditional Superpave-designed mixtures
- Asphalt mixtures designed at 5% air voids can be field compacted to 95% density without additional compaction effort

Recommendations

- 50 design gyrations for medium to high traffic levels
- 30 design gyrations for low traffic levels
- Perform low-temperature mixture testing
- Include additional traffic levels, mixtures containing RAP, RAS, or both, additional binder grades, aggregate types, mixture sizes
- Place additional field projects, monitor field projects

For this testing, both un-aged and aged specimens were tested in each of the tests. The un-aged specimens were those made from the sample mixture or from the cores. Aged specimens were made from mixture or cores after they had been conditioned in accordance with the AASHTO R30 long-term oven-aging protocol.

For the sake of brevity, only the dynamic modulus master curves data from the project is shown. The data indicate the two mixtures have similar master curves for the two groups of un-aged and aged. The stiffness values of the redesigned mixture should be similar to the original mixture, indicating its mechanical performance should be as good as the latter.

Again, the standard and redesigned mixture specimens prepared in the laboratory had air void contents of 7±1% and 5±1%, respectively. The specimens taken from field cores were tested at the in-place density.

The Georgetown Road project is currently in its second year of life and no problems have been encountered to date. The mixture has been covered with a surface and there are no plans to core the sections again to compare the densities.

The experimental results of the project indicate that asphalt mixtures can indeed be designed by choosing optimum asphalt binder content at 5% air voids without lowering the effective binder content lower than mixtures designed using the standard Superpave mixture design method. Additionally, these 5% air void mixtures can have mechanical properties equivalent to the standard mixtures. Thus no rutting performance should be lost while mixture durability should be increased.

Finally, two field trials demonstrated the 5% air void mixtures can be placed and field compacted 95% G_{mm} density without additional compactive effort above that needed for standard mixtures.

In reviewing all the project data, 50-gyration mixtures with optimum asphalt binder content chosen at 5% air voids are recommended for mixtures to be placed on roads with medium to high traffic. Similarly, 30 gyration mixtures are suggested for use on low traffic roads.

Moving forward, additional work that gathers more information and assists in further method validation would be helpful. This work would include additional traffic levels, mixtures containing RAP, RAS, or both, additional binder grades, aggregate types, and mixture sizes. Additional field projects that are constructed and monitored would also be helpful.

Abbreviations and Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ABR	asphalt binder replacement
AC	asphalt concrete
ALF	Accelerated Loading Facility
APA	Asphalt Pavement Analyzer
BBF	bending beam fatigue
BMD	balanced mix design
CAA	coarse aggregate angularity
CAM	crack attenuating mix
COV	coefficient of variation
CRS	condition rating survey
DCT	disc-shaped compact tension
DIC	digital image correlation
DOT	departments of transportation
EBCV	effective binder content by volume
ESAL	equivalent single-axle load
F&E	flat and elongated
FAA	fine aggregate angularity
FHWA	Federal Highway Administration
FI	flexibility index
HLWT	Hamburg loaded wheel tester
HMA	hot-mix asphalt
HWT	Hamburg wheel tracker
HWTT	Hamburg wheel-tracking test
ICT	Illinois Center for Transportation
IDOT	Illinois Department of Transportation
IDT	indirect tension
INDOT	Indiana Department of Transportation
LADOTD	Louisiana Department of Transportation and Development
LTRC	Louisiana Transportation Research Center
LWT	loaded wheel test
MWAS	manufacture waste asphalt shingles
NCHRP	National Cooperative Highway Research Program
NJDOT	New Jersey Department of Transportation
NMAS	nominal maximum aggregate size
OMEGA	Optimized Mix Design Approach
OT	overlay test
PAV	pressure aging vessel
PG	performance grade
PWL	percent within limit
QC	quality control
RA	recycling agent
RAP	reclaimed asphalt pavement

RAS	recycled asphalt shingles
RTFO	rolling thin film oven
SCB	semicircular bend
SE	sand equivalency
SGC	Superpave gyratory compactor
SHRP	Strategic Highway Research Program
SIP	stripping inflection point
TFO	thin film oven
TGC	Texas Gyratory Compactor
TOAS	tear-off asphalt shingles
TRB	Transportation Research Board
TRI	Trap Rock Industries
TTI	Texas A&M Transportation Institute
TxDOT	Texas Department of Transportation
VFA	voids filled with asphalt
VMA	void in mineral aggregate

TRB
TRANSPORTATION RESEARCH BOARD
500 Fifth Street, NW
Washington, DC 20001