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Enhancing the Durability of Asphalt Pavements

Papers from a Workshop

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Washington, D.C.

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Sponsored by

General Issues in Asphalt Technology Committee
Characteristics of Asphalt–Aggregate Combinations to Meet Surface Requirements Committee
Characteristics of Asphalt Paving Mixtures to Meet Structural Requirements Committee
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Preface

In the past several years, there have been a number of significant advances in asphalt materials and construction techniques for increasing the durability of pavements. This e-circular contains papers based on a Sunday workshop at the 92nd Annual Meeting of the Transportation Research Board in Washington, D.C., on enhancing the durability of asphalt pavements. The workshop focused on new design and construction approaches for extending pavement life while also improving their sustainability. The workshop also addressed the use of new materials, methods of mix analysis and cracking models, coupled with specific construction techniques that can significantly increase the life of the asphalt pavement. Furthermore, it covers actual application of these materials and approaches and some associated long-term benefits. The most significant outcome of this workshop was to show that these materials and approaches are readily available in the marketplace today and only require some relatively minor changes to specifications.

—Frank Fee
Frank Fee, LLC

PUBLISHER'S NOTE

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Impact of Mix Design on Asphalt Pavement Durability

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Asphalt mixture durability is defined as the ability of compacted asphalt concrete to maintain its structural integrity throughout its expected service life when exposed to the damaging effects of the environment and traffic loading (*1*). Asphalt mixture durability is one of several factors affecting pavement durability, which is defined as the ability of a pavement to retain a satisfactory level of performance over its expected service life without major maintenance (*1*). To be durable, a flexible pavement must be

1. Structurally adequate. The pavement layers must be sufficiently thick to carry the intended traffic loading and protect the supporting subgrade soil.
2. Properly drained. This includes adequate cross slope to drain water from the surface of the pavement as well as adequate slide slopes, ditches, or inlets to move water away from the pavement and minimize water infiltration into the pavement structure.
3. Properly constructed. Good construction practices must be used for all pavement layers. This includes proper grading and compaction of the subgrade, proper thickness and compaction control for all layers, and proper bond between asphalt layers. For surface layers, minimizing segregation and proper joint construction are important aspects of quality construction needed for a durable flexible pavement.
4. Built with durable materials. The materials used in the pavement must be able to withstand the effects of aging, traffic, and the environment. Base and subbase materials should not be susceptible to moisture or frost. Asphalt concrete surfaces must resist the effects of aging and moisture, as well as the forces applied by traffic and environmental loading.

The fourth item above is the primary subject of this review. Raveling and surface-initiated cracking are the primary distresses associated with asphalt mixture durability issues. Traditionally, durability has been addressed in asphalt mixture design and construction through a combination of the following:

1. Asphalt binder specifications that limit changes in binder properties under simulated aging. Examples include retained penetration, minimum ductility, and maximum viscosity after thin film oven conditioning in the penetration and viscosity grading systems; and the maximum intermediate stiffness after rolling thin film oven test (RTFOT) and pressure aging vessel (PAV) conditioning in the performance grading system.
2. Aggregate specifications that limit the amount of clay and other deleterious materials and guard against breakdown of aggregates during production and under traffic and environmental effects during the service life of the pavement.
3. Limits on volumetric properties to provide a sufficient volume of asphalt binder in the mixture to properly coat the aggregates and to minimize aging during production and the service life of the mixture.
4. Testing and requirements to ensure that the mixture is not sensitive to moisture.

5. In-place compaction requirements to minimize permeability which minimizes water infiltration and slows the rate of age hardening in the mixture.

Although these requirements have been largely successful, highway agencies question whether the durability of asphalt concrete surface mixtures can be improved through changes to mixture composition. Of particular interest are mixtures with moderate to high percentages of recycled asphalt material, either reclaimed asphalt pavement (RAP) or recycled asphalt shingles (RAS).

FACTORS AFFECTING ASPHALT MIXTURE DURABILITY

Table 1 summarizes a number of factors that affect the durability of an asphalt mixture in a flexible pavement. Although the emphasis of this review is the effect of the mixture composition category on durability, the other categories are discussed briefly to provide more complete coverage of the topic.

Environment

Environmental conditions at the project location have a major effect on asphalt concrete durability. Temperature is a primary consideration when designing flexible pavements and asphalt concrete mixtures. Temperature affects the structural stiffness, rutting resistance, and cracking resistance of asphalt concrete mixtures. The performance grading system for asphalt binders was developed to select binders that, for properly designed and constructed asphalt concrete mixtures, will provide acceptable rutting and cracking performance over the range of temperatures at a project location (*2*). Although much research has been done in an effort to improve the performance grading system, it has not changed substantially since its introduction more than 20 years ago. The possible exception is the use of the multiple stress creep recovery (MSCR) test for high-temperature grading, which appears to provide a more accurate assessment of the rutting potential of a wide range of polymer-modified binders (*3*).

TABLE 1 Factors Affecting Asphalt Mixture Durability

General Category	Specific Factors
Environment	Temperature Moisture
Drainage	Surface Subsurface
Construction	Weather conditions Segregation Compaction Joints Layer bond
Mixture composition	Aggregate properties Binder properties Gradation Volumetric properties

Temperature also has an important effect on the aging of asphalt binders and asphalt concrete mixtures during the service life of the pavement. The detrimental effects of binder aging on the durability of asphalt concrete mixtures has been long recognized (*4*); therefore, specifications for asphalt binders include limits on the change in properties after simulated aging. The performance grading system uses tests before and after RTFOT conditioning to evaluate stiffening during plant aging, and a maximum stiffness limit after PAV conditioning to limit stiffening due to in-service aging. Since asphalt binder aging is an oxidation reaction, it is significantly affected by the pavement temperature. Aging rates increase as pavement temperatures increase. The performance grading system accounts for this effect by varying the temperature of the PAV conditioning from 90°C in cool climates to 110°C for desert climates (*2*). Short-term and long-term oven conditioning procedures have also been developed for asphalt concrete mixtures to simulate the binder aging that occurs during production and during the service life of the pavement (*5*). Short-term conditioning is routinely used during mixture design; however, because mixture performance testing is not routinely performed, the long-term conditioning procedure has only been used in research projects.

Moisture is the second environmental factor affecting asphalt concrete durability. There are three ways that moisture may damage asphalt concrete mixtures: (*a*) loss of cohesion within the asphalt binder or mastic; (*b*) loss of adhesion between the asphalt binder and the aggregate; and (*c*) aggregate degradation particularly when freezing occurs in the mixture (*6*). A number of tests have been developed to identify asphalt mixtures that may be susceptible to moisture damage and nearly every highway agency includes a moisture sensitivity test in their mixture design process. The most common moisture sensitivity tests used in practice today are the Modified Lottman Test (AASHTO T283) and the Hamburg Wheel Tracking Test (AASHTO T324).

Drainage

The importance that proper drainage plays in the durability of asphalt concrete mixtures and flexible pavements cannot be overstated. As discussed above, moisture damage may occur in asphalt concrete mixtures if moisture is permitted to enter through interconnected voids and becomes trapped in the mixture. Additionally, aggregate bases and subgrade soils in flexible pavements lose strength and stiffness with increasing moisture content. Water can enter a flexible pavement structure from all directions. It can enter from the surface if the asphalt concrete wearing surface is permeable or has cracks and joints. It can enter from the sides and from below depending on the depth of ditches and the location of the water table. Therefore proper drainage is needed to remove water from the surface and to keep water from infiltrating into the base and foundation layers of the pavement.

Construction

The way a pavement is constructed has a major effect on the durability of asphalt concrete mixtures. Construction-related issues usually result in localized defects and distresses while deficiencies associated with mixture composition are usually more widespread. Construction issues are not always in the complete control of the paving contractor. Decisions made during design and project delivery can significantly affect how the pavement is constructed. Examples include: (*a*) selection of mixtures and layer thickness that do not provide sufficient lift thickness to obtain adequate compaction; (*b*) insufficient depth of milling to remove surface initiated

cracking; (*c*) inadequate treatment of areas exhibiting fatigue failure; and (*d*) bidding schedules that result in late season paving.

The weather during construction can affect the durability of asphalt concrete mixtures. The rate of cooling of asphalt concrete, and therefore the time available for compaction, is affected by temperature, moisture, and wind. Temperature and moisture also affect the bond between lifts. Although warm-mix asphalt (WMA) permits compaction at lower temperatures, the underlying layer must be heated sufficiently by the layer being placed to ensure adequate bond between layers which is critical to the structural integrity of the pavement.

Segregation is a common construction problem that significantly affects asphalt mixture durability. Segregation is defined as localized areas of either coarse or fine aggregates in the finished mat. Areas of the pavement that are segregated coarse have lower binder content, higher air void content, and greater permeability compared to nonsegregated areas. These areas are prone to durability distresses including raveling, accelerated aging, and damage from moisture infiltration (*7*).

Many asphalt technologists will agree that the degree of compaction of an asphalt concrete mixture, measured by the volume of air voids, is probably the most important factor affecting the performance of the mixture. For dense-graded mixtures it is generally agreed that the air void content of the pavement should be no higher than 8% and should never fall below 3% during the service life of the pavement (*8*). High in-place air voids allow air and water to penetrate into the asphalt mixture resulting in more rapid aging and potential for moisture damage. Asphalt concrete mixtures with low air voids are prone to rutting and shoving. One rule of thumb based on field performance data that is often cited for dense-graded mixtures is that pavement life is reduced about 10% of each 1% increase in in-place air voids above 7% (*9*). Many researchers have studied the effects of compaction on the properties of asphalt concrete mixtures. The general consensus of these studies is increased compaction or decreased air voids had the following effects (*10*):

- Reduced oxidative aging of the binder,
- Decreased permeability,
- Increased strength,
- Increased resistance to moisture damage,
- Increased mixture stiffness,
- Increased resistance to rutting except at very low air void contents where instability may occur, and
- Increased resistance to fatigue cracking.

Often the performance of the longitudinal joints governs the service life of the asphalt concrete wearing surface (*11*). All joints in asphalt concrete are locations of potential weakness where the mixture is likely to be less compacted, more permeable, and possibly segregated. Therefore, the number of both transverse and longitudinal joints should be minimized. Transverse joints are easily minimized through proper planning and scheduling; however, it is difficult to eliminate longitudinal joints on most projects. It is possible to eliminate some longitudinal joints in new construction by paving in echelon or using wide pavers; however, most projects are paved under traffic one lane at a time and include one or more longitudinal joint. The Asphalt Institute and the FHWA recently developed a best practices guide for constructing and specifying longitudinal joints (*11*). This document and associated training

materials confirm that durable longitudinal joints can be constructed in a number of ways. Best practices are provided for

1. Specifications and associated testing,
2. Project planning,
3. Mixture selection,
4. Laydown operations,
5. Compaction operations, and
6. Use of joint adhesives or overbanding with asphalt binder.

Layer bond is an element of pavement construction that has received considerable attention in recent years (*12*, *13*). A major assumption in the design of flexible pavements is that there is full bond between asphalt concrete layers. If full bond is not achieved during construction, the pavement may fail prematurely as a result of slippage of the wearing surface or cracking because tensile strains in the as-constructed pavement are much higher than considered in design. Therefore, it is imperative that the assumption of full bond be followed through during construction. This can be done through the proper application of tack coats at each layer interface (*13*). In addition to providing bond between layers, uniform tack coat application will resist the infiltration of water between pavement layers.

Mixture Composition

Mixture composition, which is the subject of this review, also has a major effect on the durability of asphalt concrete mixtures. Much research has been performed on how the properties of asphalt concrete mixtures are affected by their composition. The sections that follow summarize the major findings of this research and discuss recent research aimed primarily at mixtures with moderate to high recycled binder contents.

Aggregate Properties

The properties of the aggregates used in asphalt concrete that are generally associated with asphalt mixture durability are as follows (*14*):

- Toughness and abrasion resistance as measured by AASHTO T96: Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine. For an asphalt mixture to be durable, the aggregates must be resistant to degradation during production and under traffic loading.
- Durability and soundness as measured by AASHTO T104: Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate. For an asphalt mixture to be durable, the aggregates must be sound to resist disintegration due to weathering, particularly freezing and thawing.
- Plastic fines as measured by the Sand Equivalent (AASHTO T176: Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test or the Plasticity Index; AASHTO T90: Determining the Plastic Limit and Plasticity Index of Soils). Clay particles are undesirable in asphalt concrete mixtures because they weaken the bond between the asphalt binder and the aggregate resulting in the potential for moisture damage.

Although relationships between these aggregate properties and asphalt mixture durability are not available (*14*), it is generally accepted that current specification limits provide suitable aggregates for durable asphalt concrete mixtures (*8*).

Binder Properties

As discussed earlier, selecting an appropriate binder for the environmental conditions at the project location is critical to ensuring that an asphalt mixture will be durable. Much of the published research related to asphalt durability addresses the aging characteristics of the asphalt binder, and the development of laboratory tests to simulate the aging that occurs during construction and during the service life of the pavement (*15*). The performance grading system, which is used by all states in the United States for neat asphalt binders (*16*), includes intermediate and low-temperature tests and criteria on binder that has been conditioned in the RTFOT to simulate construction aging and the PAV to simulate long-term in-service aging. The widespread use of the performance grading system over the past 20 years demonstrates the reasonableness of binder selection using this system.

The economic and environmental benefits associated with recycling have resulted in an increase in the use of recycled materials in asphalt mixtures. Most mixtures produced today contain either RAP or RAS. Research completed in NCHRP Project 9-12 recommended using linear blending charts to select an appropriate grade of virgin binder for mixtures with recycled binder such that the blend of the recycled and virgin binder meets performance grading criteria at the project location (*17*). Based on the properties of binders from a limited number of RAP sources tested in NCHRP Project 9-12, the following guidance was provided for selecting virgin binders based on the expected RAP content of the mixture:

- If the RAP content is less than 15%, no change in binder grade;
- If the RAP content is between 15% and 25%, select a virgin binder that is one grade softer; and
- If the RAP content is greater than 25%, use a blending chart analysis to select an appropriate grade of virgin binder.

These recommendations assumed that the binder content of the RAP was approximately equal to that in the mixture, so the recycled binder ratio, defined as the proportion of recycled binder in the mixture (*18*), was approximately equal to the RAP content. These recommendations were incorporated into AASHTO M323: Superpave Volumetric Mix Design. Several state highway agencies allow greater than 15% recycled binder without changing the virgin binder grade and treat recycled binder from RAP and RAS the same even though RAS binders are much stiffer than RAP binders and change the grade of the blend of virgin and recycled binders approximately twice as quickly as RAP binders (*19*). Recycled binder ratios as high as 0.25 to 0.35 are permitted before changing the grade of the virgin binder (*19*).

Several researchers have used various engineering property and performance tests to evaluate the effect of recycled binders on asphalt mixtures (*18*). The properties that have been measured include volumetric properties, dynamic modulus, indirect tensile strength, rutting resistance, fatigue cracking resistance, reflective cracking resistance, fracture energy, low-temperature compliance and strength, and moisture sensitivity. There is general agreement among the various studies that stiffness and rutting resistance increase with increasing recycled

binder ratio. For this paper the findings associated with resistance to moisture damage and cracking are of the greatest interest. Table 2 summarizes the findings of several studies that included an evaluation of the effect of recycled binder on moisture sensitivity as measured by the Modified Lottman test and the Hamburg Wheel Track test. Based on these findings, it appears that recycled binders do not have an adverse effect on the moisture sensitivity of most asphalt mixtures. Table 3 summarizes the findings of several studies that included an evaluation of the effect of recycled binder on load associated cracking resistance using a variety of tests. Based on these findings, recycled binder appears to have an adverse effect on the load associated cracking resistance of most asphalt mixtures. Table 4 summarizes the findings of several studies that included an evaluation of the effect of recycled binder on thermal cracking resistance using a variety of tests. The resistance to thermal cracking generally decreased for mixtures with greater than about 25% RAP. The findings summarized in Tables 2 through 4 support limiting recycled binder ratios when no modifications to the mixture are made to improve cracking resistance.

TABLE 2 Effect of Recycled Binder on Mixture Resistance to Moisture Damage

Study	Mix Type	RAP Content	Test Method	
			Tensile Strength Ratio	Hamburg
Stroup-Gardner and Wagner (*20*)	Lab	0, 15–40	Improves	
Mogawer et. al (*21*)	Plant	0–40		No difference
Zhao et. al (*22*)	Plant WMA	0, 30, 40, 50	Improves	Improves
	Plant HMA	0, 30	Improves	Improves
Hajj et. al (*23*)	Plant and lab	0, 15, 30	No difference	
West et. al (*18*)	Lab	0, 25, 40, 55	Mix dependent	

NOTE: RAP = recycled asphalt pavement; WMA = warm-mix asphalt; HMA = hot-mix asphalt.

TABLE 3 Effect of Recycled Binder on Mixture Resistance to Load-Associated Cracking

Study	Mix Type	RAP Content	Test				
			Flexural Fatigue	Energy Ratio	Overlay Tester	Cyclic Direct Tension	Indirect Tension Fracture Energy
McDaniel et al. (*17*)	Lab	0, 10, 20, 40	Decreases				
Shu et al. (*24*)	Lab	0, 10, 20, 30	Decreases	Decreases			
Hajj et al. (*25*)	Lab	0, 15, 30	Decreases				
Mogawer et al. (*21*)	Plant	0–40			Decreases		
Zhao et al. (*22*)	Lab WMA	0, 30, 40, 50	Increases	Increases			
	Lab HMA	0, 30	Decreases	Increases			
West et al. (*18*)	Lab	0, 25, 40, 55					Decreases
Lee and Gibson (*26*)	Lab	0, 20, 40				Decreases	

TABLE 4 Effect of Recycled Binder on Mixture Resistance to Low-Temperature Cracking

Study	Mix Type	RAP content	Low-Temperature Cracking			
			Disc-Shaped Compact Tension	Semicircular Bend	Indirect Tension	Thermal Stress Restrained Specimen Test
Li et al. (*27*)	Lab	0, 20,40		Lower fracture energy for 40%		
McDaniel et al. (*28*)	Plant	0, 15, 25, and 40			Lower cracking temperature for 40%	
Hajj et al. (*23*)	Plant and lab	0, 15, 50				Higher fracture temperature for 50%
Behnia et al. (*29*)	Lab	0, 30	Lower fracture energy for 30%			
West et al. (*18*)	Lab	0, 25, 40, 55		Mixture and temperature dependent		

The use of polymer-modified binders has grown significantly since the implementation of the performance grading system for asphalt binders. The specifications for nearly all state highway agencies in the United States include one or more binder grades that require polymer modification (*16*). Polymer-modified binders were first specified to improve rutting resistance on heavily trafficked pavements. A study comparing the performance of overlays constructed with polymer-modified binder with comparable overlays constructed with neat binder concluded that the use of polymer-modified binders reduced all forms of distress, increasing the life of flexible pavements by 2 to 10 years (*30*). Some states make extensive use of polymer-modified binders. For example, the Nevada Department of Transportation (DOT) specifies polymer-modified binder for all surface course mixtures (*25*). In a laboratory study of the fatigue resistance of Nevada mixtures, the Western Regional Superpave Center found the fatigue resistance of mixtures made with both neat and polymer-modified binder decreases with increasing RAP content; however, the fatigue resistance of polymer-modified binder mixtures with up to 30% RAP was significantly greater than that of virgin neat binder mixtures (*25*). This finding indicates that it may be possible to use polymer modification to counteract the detrimental effect of recycled binder on the cracking resistance of asphalt mixtures.

Gradation

The nominal maximum aggregate size (NMAS) and gradation of an asphalt mixture affect its durability in three ways. First, smaller NMAS mixtures are designed and constructed with a higher effective volume of binder (VBE). As will be discussed in greater detail in the next

section, mixtures with higher VBE have greater resistance to cracking which is often used as a measured of mixture durability. Second, for the same density, smaller NMAS mixtures, and finer mixtures have lower permeability (*31–36*). Moisture infiltration and binder age hardening are less in mixtures with lower permeability. Finally, evidence is beginning to appear in the literature where mixtures with smaller NMAS have improved fatigue resistance compared to mixtures with larger nominal maximum size mixtures (*18*, *37*). The improved fracture resistance for small nominal maximum size mixtures is likely due to the higher VBE and smaller flaw size (air voids) in these mixtures.

Volumetric Properties

The NCHRP recently completed three research studies evaluating the effect of mixture volumetric properties on the performance of asphalt mixtures (*34*, *38*). These studies concluded that when quality aggregates and an appropriate binder are used, the in-place air void content and the effective VBE in the asphalt concrete mixture are the two volumetric properties that most affect both the durability and fatigue cracking resistance of asphalt concrete mixtures. In-place air voids are primarily controlled by construction, and the durability and fatigue life of asphalt concrete mixtures increases with decreasing in-place air void content. As discussed earlier, asphalt concrete mixtures with lower in-place air void contents are less permeable to both air and water, reducing binder age hardening and the potential for moisture damage. Asphalt concrete mixtures with lower in-place air voids also have greater strength and are more resistant to fatigue damage.

VBE is the primary mixture design factor affecting both durability and fatigue cracking resistance. Durability and fatigue resistance improve with increasing VBE. VBE is equal to the voids in the mineral aggregate (VMA) minus the air void content. The minimum VBE in current mix design procedures is controlled by the minimum VMA and the design air voids. Table 5 summarizes the design minimum VBE for different mixtures from AASHTO M323 and M325. For dense-graded mixtures, the minimum design VBE increases with decreasing NMAS; therefore, smaller NMAS mixture should have improved durability and resistance to load associated cracking. Stone matrix asphalt (SMA) mixtures, which are considered to be extremely durable and crack resistant, have the highest minimum design VBE.

TABLE 5 Summary of AASHTO M323 and AASHTO M325 Design Minimum VBE

Mixture Nominal Maximum Aggregate Size, mm	Minimum Design VMA, vol %	Design Air Voids, vol %	Minimum Design VBE, vol %
37.5	11	4	7
25.0	12	4	8
19.0	13	4	9
12.5	14	4	10
9.5	15	4	11
4.75	16	4	12
All SMA	17	4	13

NOTE: VMA = voids in mineral aggregate; VBE = effective volume of binder.

MIX DESIGN CONSIDERATIONS TO IMPROVE DURABILITY

Researchers have recommended and several state highway agencies have tried various approaches to improve the durability of asphalt mixtures and flexible pavements. Some of these were in response to local conditions and some were the result of perceived deficiencies in mixtures designed in accordance with AASHTO M323 (*39*). The sections that follow discuss several of these approaches.

Polymer Modification

It is common practice to specify polymer-modified binders in mixtures subjected to high traffic volumes primarily to improve rutting resistance. Some state highway agencies, however, specify polymer-modified binders for all surface course mixtures to improve durability. The Nevada DOT was one of the first states to adopt this philosophy. The extreme range in daily and seasonal temperatures in Nevada was the justification to use polymer-modified binder (*25*). The Louisiana Department of Transportation and Development is another agency that specifies polymer-modified binders in all surface course mixtures.

Increasing Effective Binder Content

Several methods have been recommended and used to increase the effective binder content of asphalt mixtures. The mix design manual developed in NCHRP Project 9-33 recommends that agencies should consider increasing the design VMA by 1.0% "to obtain mixtures with increased asphalt binder content, which can improve field compaction, fatigue resistance, and general durability" (*38*). Increasing the design VMA by 1.0% while keeping the design air void content at 4.0% will increase the VBE of the mixture by 1.0%. The NCHRP 9-33 mix design manual cautions that the increased design VMA may have an adverse effect on rutting resistance and the mixtures should be tested to ensure that they maintain adequate rutting resistance. For the same NMAS, the design VMA for airfield mixtures designed in accordance with the P-401 specification is 1.0% higher than the design VMA in AASHTO M323 (*40*). Durability is an extremely important consideration for airfield mixture design to minimize the potential for foreign object damage due to surface raveling. A survey of the life of airfield pavements conducted by the FAA showed the average pavement condition index for airfield runway and taxiways remained in the good range through 20 years of service (*41*). Several state highway agencies have increased design VMA to increase the effective binder content of mixtures (*39*).

Another way to increase the design VBE is to decrease the design air void content. Several state highway agencies have decreased the design air void content from 4.0% to 3.5% (*39*). This increases the design VBE by 0.5%.

A number of state highway agencies have decreased the design gyration levels in an attempt to increase effective binder contents (*39*). For the same aggregates and gradation, the optimum binder content will increase with decreasing design gyration level. However, decreasing the design gyrations may not always produce mixtures with higher VBE. If a producer is able to change gradation or the source of some of the aggregates in the mixture, it may be possible to remain near the minimum design VBE at the lower gyration level.

Another approach that has been used to increase VBE is to use smaller NMAS mixtures or to use SMA mixtures. Some states that initially used 12.5-mm NMAS mixtures for surface

courses early during the implementation of AASHTO M323 have changed to 9.5-mm NMAS mixtures (*39*). This increases the design VBE of the surface course by 1.0%. Another option that was adopted by the Maryland State Highway Administration (SHA) is to use SMA mixtures whenever high durability is required. This approach combines the beneficial effects of high VBE and polymer modification. The Maryland SHA believes that the benefits obtained from the additional pavement life exceed the higher initial cost of the SMA mixtures (*42*).

Use of Softer Binders in Recycled Mixtures

A number of state highway agencies specify the use of softer binder in mixtures when the recycled binder ratio exceeds 0.25. Although pavement performance data verifying the effectiveness of this approach is not available, it has been evaluated using various performance tests in several research studies (*21*, *29*, *43*, *44*). Using the Texas Overlay Tester, Mogawer et al. (*21*) found the use of a softer binder was not effective in improving the resistance of mixtures with recycled binder to reflection cracking. Behnia et al., on the other hand, found that the use of a softer binder was effective at increasing low-temperature fracture energy of mixtures (*29*).

Two studies compared the effectiveness of using a softer binder to increase the binder content for improving the cracking resistance of mixtures with recycled binder. Willis et al. evaluated laboratory produced mixtures with 10%, 25%, and 50% RAP (*43*). The recycled mixtures were produced at the design binder content with performance grade (PG) 67-22 and PG 58-28 binders. Recycled mixtures with PG 67-22 binder were also produced with 0.25% and 0.50% additional binder. The energy ratio and Texas overlay tests were used to evaluate the cracking resistance of the mixtures. Both using a softer binder and increasing the binder content improved the cracking resistance as measured by the energy ratio analysis for the 10% and 50% RAP mixtures. Both methods also increased the cycles to failure in the Texas overlay test, but because of the high variability of this test, the improvements were not statistically significant. Bennert et al. (*44*) summarized the results of various cracking tests that were conducted on plant mixtures to evaluate the effectiveness of using a soft binder in RAP mixtures and increasing the binder content of RAP mixtures by limiting the RAP binder contribution. When using a softer binder in RAP mixtures, the conclusions were test dependent. The softer binder improved the low-temperature cracking resistance of mixtures with 20%, 30%, and 40% RAP when measured with the Thermal Stress Restrained Specimen Test, but not when measured with the low-temperature indirect tensile creep and strength testing. The softer binder also improved the fatigue cracking resistance of the same mixtures as measured by the flexural fatigue test, but not the reflective cracking resistance as measured by the Texas overlay test. For a 20% RAP mixture, increasing the binder content by limiting the RAP binder contribution to 75% and 50% of the RAP binder content improved both the fatigue cracking resistance as measured by flexural fatigue test and the reflective cracking resistance as measured by the Texas overlay test. The increase in binder content was 0.3% when 75% RAP binder contribution was assumed and 0.5 percent with 50% RAP binder contribution was assumed. These increases are approximately equal to increases in VBE of 0.50% to 1.0%.

Warm-Mix Asphalt

One of the benefits that are often cited for WMA is improved asphalt mixture durability as a result of reduced aging of the binder during plant production (*45*). Research completed to date

has not documented that WMA mixtures have improved durability; however, research documenting the long-term performance of WMA is currently underway (*46*). In the previously described laboratory study of the cracking resistance of RAP mixtures, Willis et al. (*43*) found that producing RAP mixtures as WMA improved the cracking resistance as measured by the energy ratio test. Producing RAP mixtures as WMA also improved the cycles to failure in the Texas overlay test, but because of the high variability of this test, the improvements were not statistically significant. Based on the testing, producing RAP mixtures as WMA was considered a viable alternative to using a softer binder or increasing the effective binder content for improving the cracking resistance of RAP mixtures.

Balanced Mixture Design

Recently three state highway agencies have reported on research to develop and implement the concept of balanced mixture design (*44, 47–49*). This approach uses performance tests for rutting resistance and load-associated cracking resistance to select volumetric and binder properties that will provide adequate resistance to both rutting and load associated cracking. The balanced mix design concept was initially developed by researchers at the Texas Transportation Institute using the Hamburg wheel track test to evaluate rutting resistance and the Texas overlay test to evaluate cracking resistance (*47*). The Louisiana Transportation Research Center developed a similar approach using the Hamburg wheel track test to evaluate rutting resistance and the semicircular bend test to evaluate cracking resistance (*48*). The New Jersey DOT has implemented performance testing in the design and production of some asphalt concrete mixtures in an effort to improve the cracking resistance of asphalt mixtures (*44*, *49*). New Jersey's approach uses the APA to evaluate rutting resistance and the Texas overlay test to evaluate cracking resistance. The tests are used during mixture design and for mixture acceptance (*44*, *49*). In a recent evaluation of field sections using the balanced mix design approach, the Texas Transportation Institute researchers concluded that it is necessary to vary the criteria in the Texas balanced mix design approach depending on the climate at the project location (*50*).

Matching Design and Field Compaction

In a project for the Indiana DOT, researchers at Purdue University are investigating if asphalt mixture durability can be improved by making mixtures more compactable without sacrificing rutting resistance (*51*). The philosophy behind this research is that mixtures should be designed in the laboratory using an air void content that is achievable during construction. The recommended target for both laboratory design and field compaction is 5% air voids. Currently mixtures designed in accordance with AASHTO M323 are designed for 4% air voids. Typical in-place compaction specifications result in these mixtures being constructed at an average of 7% in-place air voids. As discussed earlier, reducing in-place air voids reduces aging and the potential for moisture damage. The design procedure that was developed uses a design gyration level of 50, a design air void content of 5.0%, and the same design VBE that is currently included in AASHTO M323. Dynamic modulus and flow number testing on mixtures produced using this design procedure and compacted to 5.0% air voids showed improved stiffness and rutting resistance compared to mixtures designed in accordance with AASHTO M323 using a design gyration level of 100 and compacted to 7.0% air voids. In a field project, a mixture designed using the procedure was successfully placed and compacted to the target 5.0% air

voids. The field performance of the project is being monitored. The laboratory study was expanded to include an evaluation of the fatigue resistance of mixtures designed using the procedure and those designed in accordance AASHTO M323.

SUMMARY AND CONCLUSIONS

The composition of an asphalt mixture is an important factor affecting the durability of asphalt concrete mixtures and one of several factors affecting the durability of flexible pavements. The durability of asphalt concrete mixtures is affected by the properties of the aggregates and binder used in the mixture as well as the gradation and volumetric properties of the mixture; all of which are addressed during mixture design. A number of changes to mixture design have been recommended in an effort to improve asphalt mixture durability. The most common include:

1. **Increase the effective binder content.** It is generally agreed that increasing the effective binder content of an asphalt concrete mixture improves the resistance to aging, moisture damage, and load-associated cracking. SMA mixtures, which are considered extremely durable, are designed to have effective binder contents that are 2 to 3 volume percent higher than dense-graded surface mixtures. Recent research suggests that the cracking resistance of mixtures with recycled binders can be improved by increasing the effective binder content of these mixtures. The effective binder content of dense graded mixtures can be increased by increasing the design VMA, decreasing the design air voids, or using a smaller NMAS mixture.
2. **Use a smaller nominal maximum size mixture or fine gradation mixture.** In addition to providing an increase in the effective binder content, smaller NMAS mixtures are less permeable at the same in-place air void content. Finer gradation mixtures are also less permeable at the same in-place air void content. Reduced permeability reduces binder age hardening and the potential for moisture damage.
3. **Use polymer-modified binders.** Polymer modification has been shown to reduce all forms of pavement distress, increasing the life of flexible pavements by 2 to 10 years.
4. **Use a softer grade of binder or warm mix for mixtures with recycled binders.** The detrimental effect of aged recycled binders on the resistance to low temperature and load-associated cracking can be offset by using a softer grade of binder or producing the mixture as warm mix. Blending charts are a tool that can be used to select an appropriate virgin binder for mixture with recycled binders.

In addition to the approaches listed above, there is growing interest in the concept of balanced mixture design. Balanced mixture design uses performance tests for rutting resistance and load associated cracking resistance to select volumetric and binder properties that will provide adequate resistance to both rutting and load-associated cracking.

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Use of Models to Enhance the Durability of Asphalt Pavements

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The use of models for asphalt concrete and asphalt concrete pavement evaluation has increased in recent years. The increase may be attributed to a paradigm shift in pavement analysis and design from empirical to mechanistic and mechanistic-empirical methods. At the same time that models have taken on a greater emphasis in pavement design analysis, changes in agency operations have focused their attention on improving the durability of preserved and rehabilitated pavement structures. Models can provide both abstract and tangible benefits for this task as well. In the abstract, the process of creating new analysis methods can introduce new ideas and viewpoints for examining asphalt concrete behaviors. These ideas may refine currently held theories or lead to entirely new approaches, which can then be used to improve materials and pavement performance. In a more tangible sense, models can be used alone or as part of an analysis system to explore physical interactions that cannot be readily examined through experiment and engineering experience alone.

Durability of asphalt pavements includes many aspects related to the mechanical responses of materials, the interactions between structure and materials, and the influence of singular and interactive non-load related mechanisms like oxidative aging and moisture damage. In this paper, the use of models to enhance and improve the durability of asphalt pavements is examined. As background, some of the more widespread models that are used in asphalt concrete technology are classified with respect to their defining characteristics and uses. The differentiation between a singular model and a pavement analysis system is an important part of this discussion. Subsequently, some of the better known pavement analysis systems are briefly described and some key findings from the development and use of these tools are presented. Descriptions of how models and pavement analysis systems are useful in various aspects of pavement technology and thus how these tools contribute to enhancing asphalt pavement durability are also given. Following this discussion, some comments are given regarding the two issues raised during the workshop that has led to this document: (1) the use of models to guide mixture composition decisions and (2) the potential for model *fatigue* in the asphalt concrete community. Finally, the last section provides a brief summary and overview of the key points.

STRUCTURE AND CLASSIFICATION OF MODELS USED IN ASPHALT CONCRETE TECHNOLOGY

A review of the asphalt concrete modeling literature reveals a wide range of approaches, terminologies, and applications within four general categories: (1) mechanistic material models, (2) structural analysis models, (3) non-load associated models, and (4) micromechanical models. Each of these categories may themselves consist of multiple subcategories, which can make it very difficult to understand and contextualize the findings from any single study within a broader set of work. Since this issue is relevant to the discussion of models for enhancing durability, a brief description of the model categories and their usage is worthwhile. At the most essential level, a model is an idealization of some physical reality. A globe, for example, is a model of the

earth, which captures some, but not all of the major physical characteristics of the planet. In asphalt concrete, models are formulated around key distresses like rutting, fatigue cracking, and thermal cracking. Table 1 summarizes some of the more commonly described models, separated by category, along with their distinguishing characteristic. Table 1 is not an exhaustive list, and is meant only to provide readers with a basic summary of the different aspects of modeling in the asphalt concrete literature as well as give references from which more information can be found.

TABLE 1 Summary of Models and Modeling Approaches Used in Asphalt Concrete Studies

Model Category	Model Type	Name	Related Mechanism or Distress	References
Mechanistic material models	Empirical	Fatigue law	Fatigue	*1, 2*
		Power-law hardening	Rutting	*3, 4*
		Strain ratio model		*5, 6*
		Minimum strain rate		*7*
	Theory derived	TC-Model	Fracture	*7*
		Cohesive zone model		*9–11*
		Dissipated creep strain energy	Fatigue	*12*
		Viscoelastic continuum damage		*12, 13*
		Viscodamage model	Fatigue and rutting	*14, 15*
		Drucker-Prager	Rutting	*16*
		HISS-Perzyna		*17, 18*
		Rate-dependent hardening		*19, 20*
		Hardening-relaxation		*21*
		Strain hardening		*15, 22*
Structural analysis models	Theory derived	Layered elastic analysis	Response model	*23*
		Layered viscoelastic analysis		*24, 25*
		Finite element method		*26, 27*
		Boundary element method		*28*
		Fourier Finite Element		*29*
Nonload associated	Empirical	Global aging system	Aging	*30*
		LTPP bind equation	Temperature in pavement	*31*
	Theory derived	Diffusion aging	Aging	*32, 33*
		Micro-damage healing	Healing	*34–36*
		Moisture damage model	Moisture damage	*38, 39*
		Enhanced integrated climatic model	Temperature and moisture profiles	*37*
Micro-mechanical models	Analytical	Generalized self-consistent model	Modulus	*40, 41*
		Self-consistent model	Modulus	*42*
		Differential model	Modulus	*43*
	Numerical	Lattice model	Modulus, fatigue	*44*
		Discrete element based method	Modulus, fatigue, rutting	*45*
		Two-way coupled approach	Modulus, fatigue	*46*

NOTE: LTTP = long-term pavement performance.

Mechanical models are regularly used to describe the distress phenomena in a way that is useful for engineers. These models attempt to describe physical phenomenon using a series of mathematical functions and can be either theory derived or empirically based. An example of an empirical mechanical model that is commonly encountered in asphalt technology is the fatigue law function, Equation 1, which relates the strain amplitude, ε_t, and modulus, E, to the number of cycles to failure, N_f, by using three empirically calibrated coefficients $K_{1–3}$.

$$N_f = K_1 \left(\frac{1}{\varepsilon_t}\right)^{K_2} (E)^{K_3} \tag{1}$$

By contrast to the empirical model relating cycles to failure with strain and modulus, theory derived models are typically constitutive, that is they mathematically relate stress, σ, and strain, ε. Some may also relate parameters like crack depth or permanent strain accumulation with cycles or time. These theory derived models attempt to capture some deeper fundamental characteristic that underlie the observed material behaviors. The simplest example of a constitutive model is the 1-D Hooke's law, Equation 2, where the elastic modulus, E, is the parameter that conveys the *deeper fundamental understanding* of the material. Theory derived models that are relevant to asphalt pavement durability are considerably more complex than Equation 2. In the modern literature, when the term "mechanical model" is used it generally refers to these theory derived-type models and many examples exist (see Table 1 for a summary).

$$\sigma = E \times \varepsilon \tag{2}$$

The second category of models is used to describe the pavement structural influences. These models are also mechanistic, but are considered separate because the primary application is different from that of the first category (structural versus material only). Although the notion of structure can have scale-dependent characteristics, for example, an asphalt concrete sample can be considered in a structural context where the aggregate forms a skeleton with load paths to transfer the forces between aggregate particles (for the purposes of simplicity, structure here refers only to the pavement structure itself). The simplest version of a structural model is the layered elastic method. More advanced approaches use layered viscoelastic analysis as well as finite element and boundary element methods in both frequency and time–domain. Increases in complexity are generally believed to lead to increases in accuracy, but may also require substantially more computational run time.

The third model category focuses on nondistress specific phenomena, such as moisture damage, oxidative aging, healing, climate, traffic, etc. The theories and methods used in this category can vary widely and so it is difficult to identify consistency in the formulations. While these models can be useful on their own, their most significant contribution is made when they are included as part of an integrated pavement analysis system that incorporates both material and structural models. It is this system of models that has the greatest potential for enhancing the durability of asphalt pavements because it involves the structural and material level interaction and the flexibility to account for factors like oxidation, moisture damage, healing, etc. The integration framework is demonstrated schematically in Figure 1. Some examples of emerging integrated pavement analysis systems are discussed later in this paper.

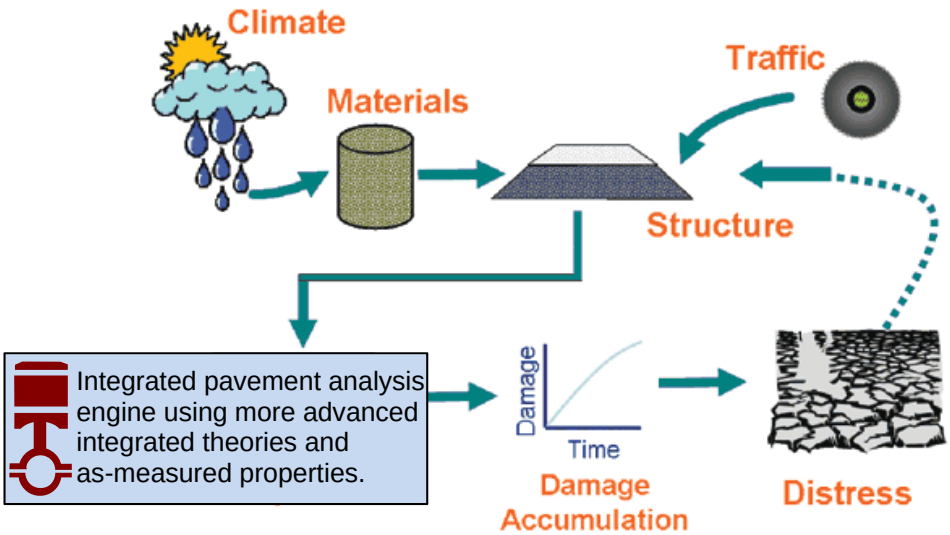

FIGURE 1 Analysis framework for integrated pavement analysis system (*47*).

The final category of includes the micromechanical formulations. These models are used to investigate the multiscale characteristics of asphalt concrete mixtures by explicitly considering the heterogeneity of the material. These methods share commonalities with the mechanistic model category since they too focus on the material mechanical characteristics, but are distinct in the sense that they focus on the mechanics at different length scales. This category has received substantial interest from the modeling community in the past few years. Traditionally, efforts were extended for analytical modeling and prediction of mixture modulus from only asphalt binder and aggregate properties. More recent efforts have extended this work using numerical methods to examine cracking and permanent deformation behaviors. This category does have an important role in enhancing the durability of asphalt concrete pavements as will be discussed in a later part of this paper. However, attention is first given to the aforementioned integrated pavement analysis systems.

INTEGRATED PAVEMENT ANALYSIS SYSTEMS

Table 2 summarizes six relevant integrated pavement analysis systems. Together these tools cover a range of distresses; fatigue cracking, rutting, thermal fracture (generally associated with single low temperature events), and thermal fatigue (associated with accumulated distress from daily temperature fluctuations). The system that has received the most interest over the past 10 years is probably the Pavement ME (mechanistic–empirical) platform, which evolved from multiple NCHRP projects notably including the Mechanistic–Empirical Pavement Design Guide (MEPDG) from NCHRP 1-37A. The next system, ILLI-TC, focuses on thermal fracture, while the third and fourth [Hot-Mix Asphalt (HMA)–Fracture Mechanics System and VECD-FEP++ method] deal with fatigue cracking and thermal fatigue only. The remaining systems include models to evaluate both fatigue and rutting distresses. The purpose in this document is to identify the major tools and their key components, and then provide references to information for interested readers to learn more.

Pavement Mechanistic–Empirical Design: MEPDG and DARWin ME

Pavement ME (and its predecessors DARWin ME and the NCHRP 1-37A MEPDG) is probably the most well-known pavement analysis system. Many states are currently in the process of calibrating this tool for new pavement design and so there is likely some interest in its use to examine durability. Technically, there are many differences between this tool and the others discussed here, which can be identified from the descriptions given in Table 2. While this tool shares a common and basic framework with the other tools, one major functional difference is that pavement response analysis (the calculation of stresses and strains resulting from vehicular loading) is carried out separate from the damage–distress analysis. In the other pavement analysis tools the constitutive material theories can be naturally integrated into the structural analysis platform to more accurately capture the structural influences of damage. Such influences are only implicitly accounted for in the Pavement ME through calibration. Additionally and owing to its use as a design tool, the Pavement ME system also has a more detailed soil–structure interaction algorithm than the other tools.

ILLI-TC

ILLI-TC is a single-event thermal fracture propagation system developed through efforts at the Universities of Minnesota, Illinois, Wisconsin, and Iowa State. The basic structure and key components of this method are summarized in Figure 2. The tool utilizes viscoelastic and fracture theories integrated with finite element structural analysis. The primary damage theory involves cohesive zone elements, which permit the propagation of cracks with an appropriately defined and characterized crack tip traction–displacement relationship. The effects of aging, healing, and moisture damage are accounted for in the selection of model calibration coefficients. Output is provided in the form of the extent of pavement damage and cracking with the aid of probabilistic transfer function first derived in the SHRP research program (*7*).

Hot-Mix Asphalt–Fracture Mechanics System

The HMA–FM model was developed at the University of Florida under NCHRP 1-42A. Its intended purpose is to examine the mechanics of top-down fatigue cracking, but it may also consider both top-down and bottom-up behaviors. An approximate viscoelastic fracture theory, dissipated creep strain energy, serves as the primary material damage model in this method. This material model is coupled with both a viscoelastic pavement response model and a boundary element structural model to create a tool that bridges the gap between crack initiation and propagation. The basic flowchart for this integrated system is given in Figure 3. Although it is not apparent from this figure, empirical models are included to account for oxidation and healing of the material. The final output for this model is the crack initiation time and the amount of fatigue cracking, which is calculated using a probabilistic method similar to that used in ILLI-TC.

TABLE 2 Summary of Integrated Pavement Analysis Tools

Name	Distress	Structural Analysis Engine	Major Mechanical Theories	Nonload-Associated Models	Sources
Pavement ME	TF, FC, R	LEA	E, PhF, PhR	GAS EICM Ph. soil modulus relationships Soil moisture and temperature effects	*48*
ILLI-TC	TF	2-D FEM	VE-FM (CZM)	ICM-created library of climate files	9
HMA-FM	FC, TFC	LEA and 2-D DDBEM	E and VE-FM	GAS-based aging model EICM Phenomenological healing model	*12*
VECD-FEP++	FC, TFC	2-D FEM	VE and VECD	GAS- and Ph.-based aging model EICM Ph./continuum healing model Damage factor for viscoplastic influences	*12*
PANDA	FC, R	2-D and 3-D FEM	VE, VP, VECD	Continuum-based healing model Continuum-based moisture damage model Kinetics-based aging model	*46*
HMA-PRS	FC, TFC, R	2-D FEM or LVEA	VE, VP, VECD	GAS- and Ph.-based aging model EICM Continuum-based healing model	*49*

NOTE: TF = thermal fracture at low temperatures; FC = fatigue cracking; TFC = thermal fatigue cracking at low and intermediate temperatures; R = rutting; FEM = finite element method; LEA = layered elastic analysis; DDBEM = Displacement discontinuity boundary element method; LVEA = layered viscoelastic analysis; VE = viscoelasticity; FM = fracture mechanics; CZM = cohesive zone model; E = elasticity; VP = viscoplasticity; VECD = viscoelastic continuum damage; PhF = phenomenological fatigue relationship; PhR = phenomenological rutting relationship; ICM = integrated climatic model; EICM = enhanced integrated climatic model; GAS = global aging system; Ph. = phenomenological.

VECD-FEP++ Method

VECD-FEP++ was developed at North Carolina State University under the NCHRP 1-42A effort. The name reflects two key components of the analysis system: VECD (Viscoelastic Continuum Damage model) and FEP++, which indicates that a finite element program is used to generate the structural model. The key difference between the two NCHRP 1-42A methods is the material model theory adopted, fracture mechanics versus continuum damage mechanics. Both tools focus on the same basic phenomenon (differentiating between bottom-up and top-down cracking), which means that while HMA–FM places the emphasis in pavement cracking on macrocrack propagation, the VECD-FEP++ places more emphasis on microcrack formation, coalescence, and propagation. The two phenomena are related, which is why both approaches may be capable of meeting their stated objective after appropriate calibration. The framework for the VECD-FEP++ model is summarized in Figure 4, which demonstrates that multiple individual theories and submodels (at both the structural and material levels) are needed to create an integrated pavement analysis system. It should be noted that like the HMA–FM method the VECD-FEP++ model considers nonload effects using a combination of mathematical and empirical techniques.

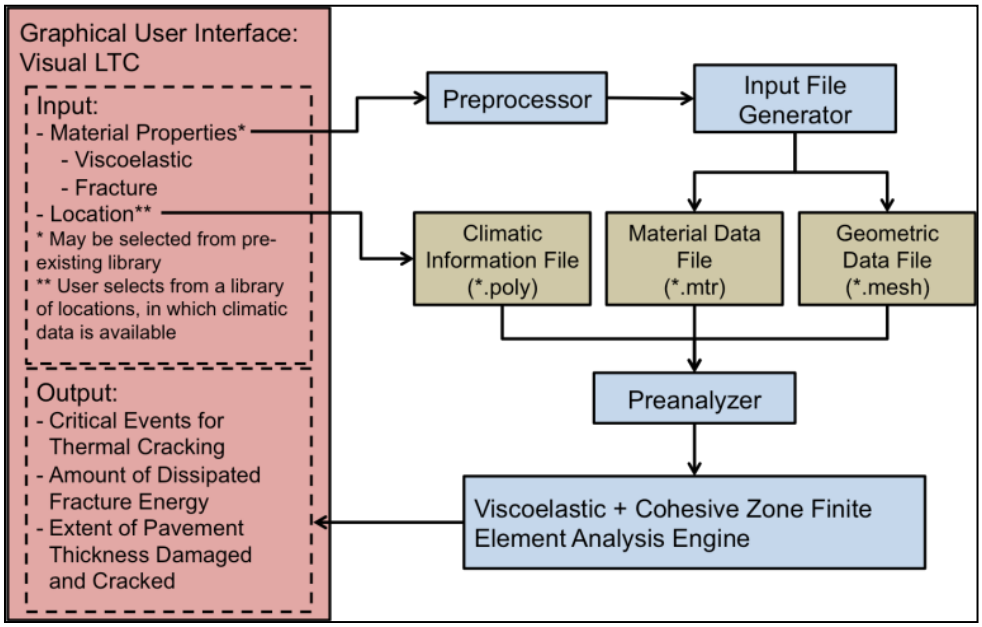

FIGURE 2 Analysis flowchart for ILLI-TC model (*9*).

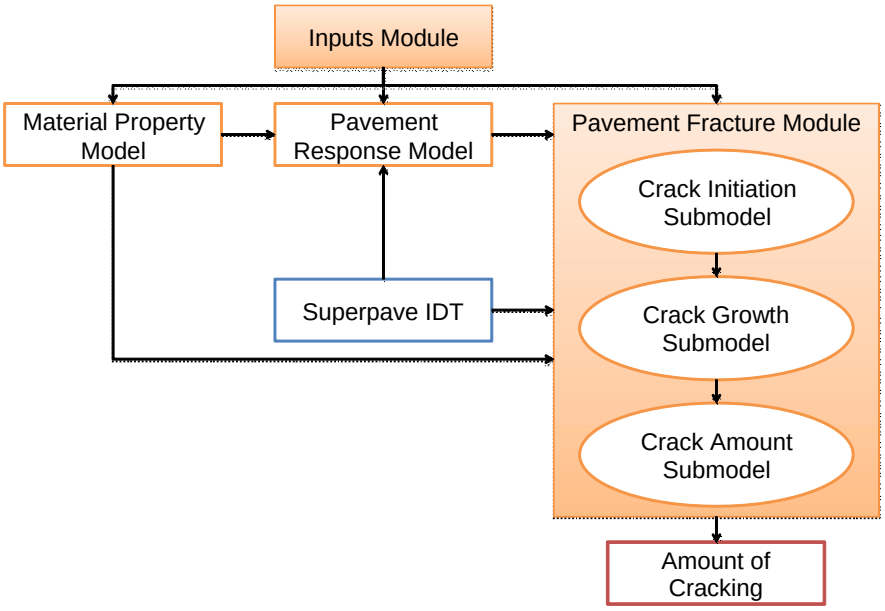

FIGURE 3 Analysis framework for HMA–FM model (*12*).

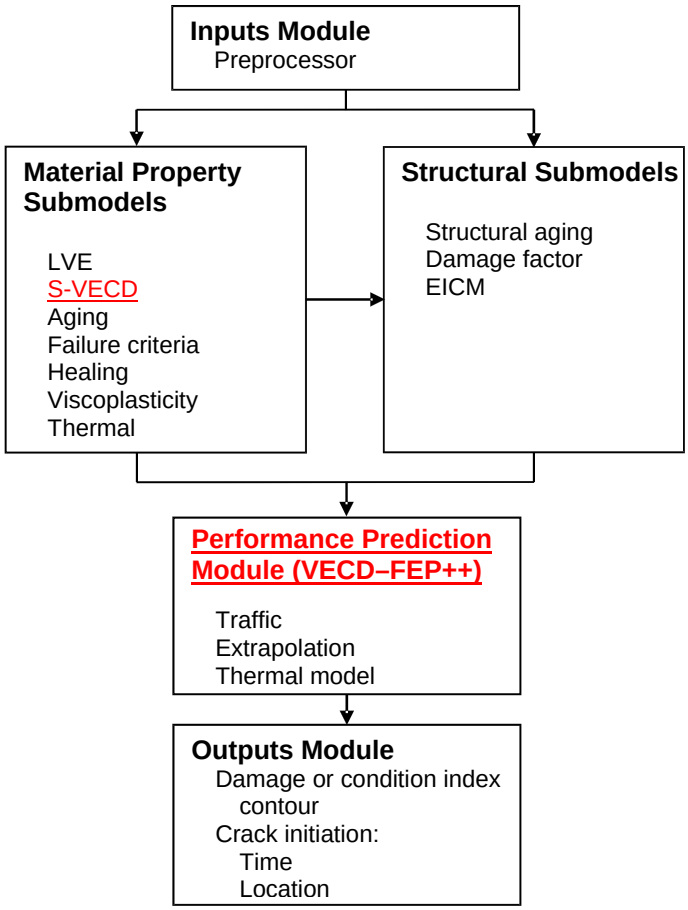

FIGURE 4 Analysis framework for VECD–FEP++ method (*12*).

Pavement Analysis Using Nonlinear Damage Approach

The Pavement Analysis Using Nonlinear Damage Approach (PANDA) model is a set of analysis tools built around a user-defined material subroutine within the ABAQUS finite element analysis software. This set of tools has been developed through efforts in the Asphalt Research Consortium (ARC) and considers the primary distresses of rutting and fatigue. Linear and nonlinear viscoelasticity, viscoplasticity, and mechanical damage are the predominant theories adopted. PANDA also considers the impacts of aging, moisture, and healing. Users determine a priori the level of detail and the particular mechanisms to consider in the model predictions, and then carry out the necessary experiments to characterize those mechanisms. Analysis outcomes are summarized in the form of contour plots of damage. The tool is still in the development phase with research focusing on the creation of user friendly interfaces and automated characterization subroutines. The analysis framework is shown in Figure 5.

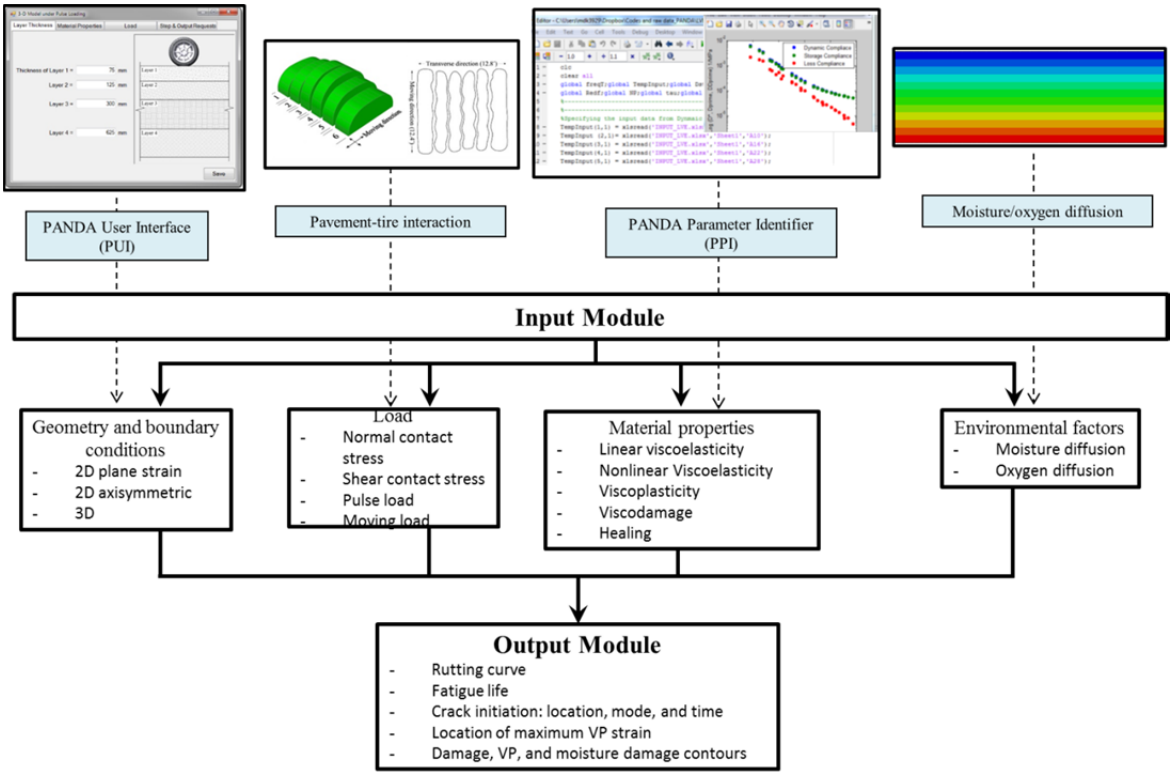

FIGURE 5 Analysis framework for PANDA model (*46*).

Hot-Mix Asphalt Performance-Related Specifications Tool

The Hot-Mix Asphalt Performance-Related Specifications (HMA–PRS) analysis tool is a series of hierarchical integrated model systems that focus on rutting and fatigue distresses. As summarized in Table 3, the tool is organized into three levels depending on the desired level of accuracy. Level 1 involves the most extensive testing and analysis effort, but provides the most detailed analysis while level 3 makes extensive use of predictive equations to relate materials and construction variables (asphalt content, density, gradation, etc.) with mechanical properties and provides the least detailed analysis. Like PANDA, this method is currently under development with ongoing work focusing on the calibration and refinement of the individual models. This tool also uses similar theories; linear viscoelasticity, viscoplasticity, and damage although the exact formulations differ substantially from those included in PANDA. More advanced models for healing and oxidative aging are under development and should be part of the final analysis methodology. Output is provided in the form of contour plots of damage, which must be interpreted by the users; however, ongoing calibration efforts should provide a suitable transfer function to relate these outcomes to engineering performance measures (cracked area, rut depth, etc.).

TABLE 3 Summary of HMA-PRS Tool Analysis Levels

	Level 1	Level 2	Level 3
Stiffness	Unconfined and confined |*E**|	|*E**| from Asphalt Mixture Performance Tester	Impact resonance |*E**| + pred. equation
Cracking	Uniaxial VECD	Simplified VECD	Pred. equations
Rutting	Multiaxial VEPCD	Repeated load test at a representative stress state	Pred. equations
Structural model	Fourier–finite element model[a]	Layered viscoelastic model	Layered viscoelastic model
Testing time	5 days	3 days	Less than 1 day
Analysis time	2 days[a]	1 day	Less than 1 day
Total time	7 days	4 days	1 day

[a] Current approach may change upon further development.

LESSONS LEARNED FROM MODELING EFFORTS

The primary factors affecting pavement performance have been known for many years. These factors are identified in the simplest pavement structural and material models (*3–5*). For example, using layered elastic analysis and power-law permanent strain models it is understood that increases in material modulus will decrease the vertical compressive strain throughout pavement layers and decrease the permanent strain accumulation. These models also demonstrate that the effect is not consistent with depth nor will it scale linearly with the magnitude of load. Identification of these first order impacts has provided pavement engineers with useful insight that has aided in the design and analysis of pavement performance for the last half century. The limitation in these simple tools is that they are incapable of showing, in a fundamental and general way, why particular behaviors occur more or less under certain situations. This limitation may exist because the methods are not formulated in a way that permits such investigation or the fundamental measures adopted are not consistent with known material behaviors. The fatigue law function and power–law permanent deformation model are examples of the former and the use of layered elastic models is an example of the latter.

By contrast to the empirical-based mechanical models, constitutive- and theory-derived models have the inherent capability to identify and quantify the underlying material mechanisms. Research with these models has revealed some useful higher level insights into the material behaviors. The first example is with respect to the use of continuum damage models for fatigue cracking prediction, which have shown the importance of decoupling fatigue into both resistance to deformation and resistance to damage. This decoupling was implied in the empirical–mechanical models (e.g., Equation 1) but continuum models have shown that the nature of decoupling is more involved that what these early models could capture. Second, fracture mechanics studies of thermal cracking have shown that both strength and fracture energy are important parameters in examining crack initiation and propagation. Such a finding affects how tests are performed and reduces the potentially infinite number of test factors to only those that identify key parameters. The most interesting findings are currently occurring with respect to the

rutting phenomenon. Constitutive modeling efforts have identified critical flaws in classic viscoplastic models with respect to the influence of load and rest period times. This finding has implications on whether existing protocols that test at a single pulse–rest–temperature are necessarily ideal indicators for the permanent deformation potential in asphalt concrete.

Many findings are also emerging with respect to material–structure–climate interaction. When individual models are integrated into a pavement analysis system the outcomes provide insight into the effect of external factors (climate, traffic, structure, rehabilitation strategy, etc.) on the manifestation of distresses like rutting and cracking. Work to date has been successful in this respect and a review of the literature reveals some specific relevant findings like quantification of impacts from design decisions (higher or lower quality base materials for example) that would not be possible without a complete pavement analysis systems (*12, 49, 50*).

ROLE OF MODELS IN ENHANCING DURABILITY

Predicting if a pavement will have durability problems a priori or identifying why an existing pavement structure exhibits durability issues is a very challenging task. In this respect the role of models to enhance pavement durability is two-fold: provide the insight to understand the mechanisms that lead to more or less durable materials so that tools can be created to screen materials and provide analysis frameworks to evaluate structural influences that can degrade or enhance the durability of asphalt pavements. To understand how models can contribute to these tasks one must consider how model outcomes match with the definition of durability.

Merriam-Webster defines durability as the "ability to exist for a long time without significant deterioration" (*51*). Engineers can readily use this definition to identify pavements that are not durable. Such pavements require more than typical maintenance and/or replacement and thereby consume greater resources and generally detract from the development of a sustainable transportation infrastructure. However, a model, being an analytical or numerical object, is not capable of describing durability in such subjective terms. Models quantify performance by effectively quantifying the deviation in response from an established standard (generally taken as the condition prior to any loading has been applied) under repeated loading. The reasons for the observed deviation is generally referred to as damage, which can include a number of different physical phenomena including permanent strain accumulation or micro- and macrocrack initiation, coalescence, and propagation. Whatever the actual cause for damage, this framework poses some conflict with respect to the way that durability is understood in engineering practice.

Deterioration is a subjective assessment of whether a material has resisted loss in performance whereas damage is related to the quantified output of a model. Damage and deterioration are related. For example, a material can be damage resistant and withstand loading without showing a loss of mechanical properties or it can be damage tolerant and have mechanical properties that are not very sensitive to the damage that occurs. In both cases we would expect that the material deterioration would be small. The lack of a mathematical link between damage and deterioration means that there still remains a qualified subjectivity in the use of mechanistic models for durability assessment. This fact must be clearly understood since it suggests that with the current level of technical achievement, engineering judgment must be exercised when applying models for durability assessment. The same condition does not necessarily exist in the application of mechanistic models for objective tasks like falling weight

deflectometer calculation, fatigue crack length prediction, thermal crack spacing, rut depth, etc. Each of these factors may contribute to determining whether a pavement is durable, but the exact contribution of each is open for a subjective interpretation.

Figure 6 summarily demonstrates the processes related to creating pavement structures. Each of these processes is linked, for example the best design of a mixture will depend on the pavement structure into which that material will be placed. Likewise the best pavement structure will depend on what materials are available, the geographic location of the pavement, and the traffic levels. Models are useful to bridge the gap between each of these tasks. The simplest example of this process is in the case of pavement design to construction where a structural analysis system can be used for quality control as demonstrated by NCHRP 1-22 (*52*). However, as mentioned earlier, models are inherently useful to provide insight and understanding of mechanisms that govern material performance. In gaining such insight engineers can understand how to develop mixtures that are optimized for the specific structure of interest and mixtures that have an acceptable level of constructability. By first understanding how certain key mechanisms manifest under a prescribed set of conditions it may be possible to identify the engineering parameters that reduce damage and thereby create materials and structures that have greater durability.

MOVING FORWARD

Use of Models to Refine Mixture Design Variables

The basic goal for any model is to provide users with insight that helps them to answer questions that they cannot answer through experienced judgment alone (either because the process is too

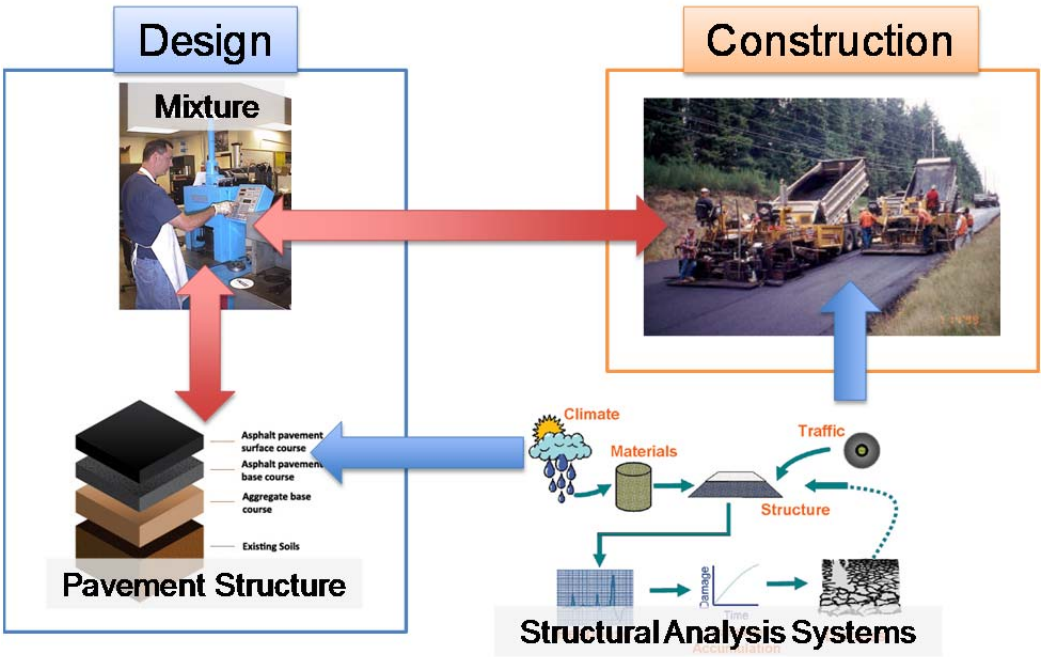

FIGURE 6 Role of models in enhancing pavement durability.

complicated or because to do so would require a substantial investment of time and resources). For the case of maximizing pavement durability, engineers have two questions that need to be answered.

- How does the given asphalt concrete mixture behave under particular structural, environmental, and traffic conditions?
- What mixture adjustments should be made so that the pavement durability is maximized?

Currently, the tools described above can only provide insight to answer the first question because each utilizes the same basic process: users evaluate their material by performing an experiment or group of experiments; the outcomes of this experiment or experiments are analyzed within the context of the applicable theory or set of theories to obtain model parameters; model parameters are input into the chosen integrated analysis tool; the analysis tool is executed and outcomes in the form of the evolution of a performance metric (rut depth, fatigue cracking, thermal cracking) over time are provided to the user. To maximize the performance of the available source materials, engineers must formulate another mixture design and carry out the same steps again. Such a method is necessary because the models that constitute the integrated analysis tools are all macroscale based, but changes to the mixture design variables occur at scales smaller than the macroscale. Thus, while the parameters in current models are affected by compositional changes, they cannot analytically predict the magnitude of these changes.

Such a limitation has been understood for many years and two strategies exist for overcoming this constraint" phenomenological calibration and multiscale micromechanical modeling. The first approach is very familiar to engineers and involves systematic evaluation of a broad cross section of materials. Each material is first characterized within the context of a chosen theory or model and then the results are analyzed using statistical or numerical methods to develop relationships. The most well-known example of this kind of approach is the Witczak model for dynamic modulus.

The second method relies on micromechanical models, which are formulated expressly to consider the interaction of individual constituents in a heterogeneous body, and thus by definition have the capability of accounting for constituent properties. This approach is more powerful than the phenomenological calibration method, but also adds considerable mathematical complication. It should be noted that work is ongoing with these approaches, and that the PANDA model, when completed, will include a type of micromechanical model capable of examining the impact of adjustments to the coarse aggregate structure (*46*). However, as of this writing and other than the Witczak dynamic modulus predictive equation that is in the Pavement ME tool, neither of these two approaches has been fully calibrated, verified, and incorporated into the aforementioned pavement analysis tools. So, while it is recognized that current models have limitations in their ability to guide mixture design decisions, efforts are underway to address this shortcoming so that the maximum possible benefit from these tools is realized.

Contending with Model Fatigue

It is understandable for engineers to be uncertain about how models can be useful in engineering practice, and thereby become fatigued by the expanding literature on the subject. As Table 1

demonstrates, modeling in the field of asphalt technology constitutes a broad topical area that can include different analytical classes (empirical, constitutive, chemical-reaction, etc.), length scales (asphalt cement, asphalt concrete, pavements, and intermediate scales), fields of mechanics (continuum mechanics, fracture mechanics, or micro-mechanics), and mechanical theory (viscoelasticity, viscoplasticity, damage, etc.). The definition of what constitutes a model is also ambiguous. For example, one might refer to the Pavement ME Design as a *model* when in light of the discussion given earlier it may be more accurately described as an integrated pavement analysis system consisting of an assemblage of different models. These two factors likely contribute greatly to the uncertainty of practitioners whose primary interest is in creating materials and structures with the maximum possible life.

However, what should be kept in mind is that the performance of asphalt concrete pavements is a function of the interactivity between materials and structures, thus without a completely integrated pavement analysis system it is difficult to show benefits of a singular model. A model describes only a single physical process in the system and researchers have a vast number of methods available and opinions about how to describe these processes. Each of these approaches is accompanied by a series of implied and explicit physical constraints, which need to be debated and discussed by all members of the community in order to identify the approach that best captures the physical or chemical phenomenon. As the paragraphs above show, consensus on the essential elements of a pavement analysis system (climate, structure, materials, and traffic) exists, but the details of how to best describe the individual elements and interaction of these pieces is still not clear. A substantial amount of modeling literature is currently being produced for the basic reason that a consensus on the analysis system framework does exist. Going forward, the existence of tools like those described in this paper establishes a basic structure from which future modeling can build upon. These future efforts may either borrow the conceptual framework to develop new tools or may involve replacing sub-models in the already existing tools. In either case, the development and discussion of new models will likely continue to be an integral part of pavement analysis and design.

SUMMARY

Designing flexible pavements is one of the most challenging tasks that civil engineers undertake. The difficulty can be attributed to the complexity of the basic engineered material in these pavements (asphalt concrete), uncertainty in loading conditions, constant exposure to the climatic elements, variability in constructed product, variability in soil conditions, and others. Owing to this complexity engineers have a strong motivation to utilize tools that can systematically and accurately account for the many factors that are present. In this respect models and pavement analysis systems play an important role in the development of durable pavements. These models help to identify the mechanisms and factors that lead to damage, which eventually causes distresses that accumulate and cause a loss in durability. The insight gained through the development and calibration of mathematical models can inform mixture design and pavement design decisions. It can also provide guidance on construction control, required specifications, and quality control and quality acceptance tests that capture fundamental material properties and thus require less overall experimental effort. The current state of the art has provided robust material level models that are identifying the materials that will perform better or worse in a given pavement situation. Future research and study will continue to identify the

links between asphalt mixture constituents, asphalt concrete damage, and pavement distresses. These more advanced models and integrated pavement analysis tools can be useful for developing a more durable pavement infrastructure.

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Optimization of Tack Coat for Hot-Mix Asphalt Placement

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Selection of an optimum tack coat material and application rate is critical in the development of proper bond strength between pavement layers. The main objectives of NCHRP Project 9-40 were to determine optimum application methods, equipment type and calibration procedures, application rates, and asphalt binder materials for the various uses of tack coats and recommend revisions to relevant AASHTO methods and practices related to tack coats. During the course of this project, the Louisiana Tack Coat Quality Tester (LTCQT) was developed to evaluate quality of the bond strength of tack coat in the field. Further, the Louisiana Interlayer Shear Strength Tester (LISST) was developed for the characterization of interface shear strength of cylindrical specimens in the laboratory. The LISST device was designed such that it will fit into any universal testing machine. As part of the experimental program, the research team constructed full-scale test overlays including different tack coat application rates between a new hot-mix asphalt (HMA) overlay installed over several types of pavements surfaces, including old HMA, new HMA, milled HMA, and grooved portland cement concrete (PCC). Five types of tack coat materials were each applied at three application rates. Optimum tack coat application rates were developed for different pavement surface types.

INTRODUCTION

Tack coat is a light application of asphalt, usually asphalt emulsion diluted with water, onto an existing relatively nonabsorptive pavement surface (*1*). It is used to ensure adequate bond between the pavement being placed and the existing surface. A tack coat provides necessary bonding between pavement layers to ensure that they behave as a single system to withstand traffic and environmental stresses. Tack coat is normally applied to an existing pavement surface before a new layer of asphalt concrete is placed. It may also be applied to the surface of a new HMA pavement layer before the next layer is placed, such as between a HMA leveling course and a HMA surface course.

Selection of an optimum tack coat material and application rate is critical in the development of proper bond strength between pavement layers. Pavement surfaces with different conditions (e.g., new, old, milled, grooved, cracked) require different tack coat application rates to achieve proper interface bond strength. In most paving operations, tack coat covers less than 90% of the existing surface. On the other hand, excessive tack coat may promote shear slippage at the interface. Most importantly, it is the residual amount of asphalt, not the quantity of diluted asphalt emulsion that should be specified in tack coat applications. Few guidelines are available for the selection of tack coat material type, application rate, placement, and evaluation. In general, selection of tack coats has been mainly based on experience, convenience,or empirical judgment. In addition, quality control and quality assurance testing of the tack coat construction

process is rarely conducted, resulting in the possibility of unacceptable performance, even premature pavement failure.

NCHRP Project 9-40: Optimization of Tack Coat for HMA Placement assessed the effects of many plausible factors on interface bond characteristics (*2*). The outcomes of this project included the development optimum application methods, equipment type and calibration procedures, tack coat application rates, asphalt binder materials for the various uses of tack coats, and revisions to AASHTO methods and practices related to tack coats. This paper presents the results of NCHRP Project 9-40: Optimization of Tack Coat for HMA Placement.

DEVELOPED TEST DEVICES

Direct Shear Device: Louisiana Interface Shear Strength Tester

A direct shear device was developed for the characterization of interface shear strength of cylindrical specimens (Figure 1). The device is referred to as LISST (Louisiana Interface Shear Strength Tester). It consists of two main parts, a shearing frame, and a reaction frame. Only the shearing frame is allowed to move while the reaction frame is stationary. A cylindrical specimen is placed inside the shearing and reaction frames and is locked in place with collars. Loading is then applied to the shearing frame. As the vertical load is gradually increased, shear failure occurs at the interface.

The LISST device was designed such that it will fit into any universal testing machine. It has a nearly frictionless linear bearing to maintain vertical travel and can accommodate sensors that measure vertical and horizontal displacements. The device also provides a specimen locking adjustment, applies a constant normal load up to 100 psi, and accommodates a specimen with 4- or 6-in diameters. The gap between the shearing and the reaction frame is 0.5 in. It is noted that a number of experiments were conducted in order to evaluate the ruggedness and reliability of the LISST. Experiments were also conducted comparing the results from this device to those of the Superpave Shear Tester. Details of these experiments are described elsewhere (*3*).

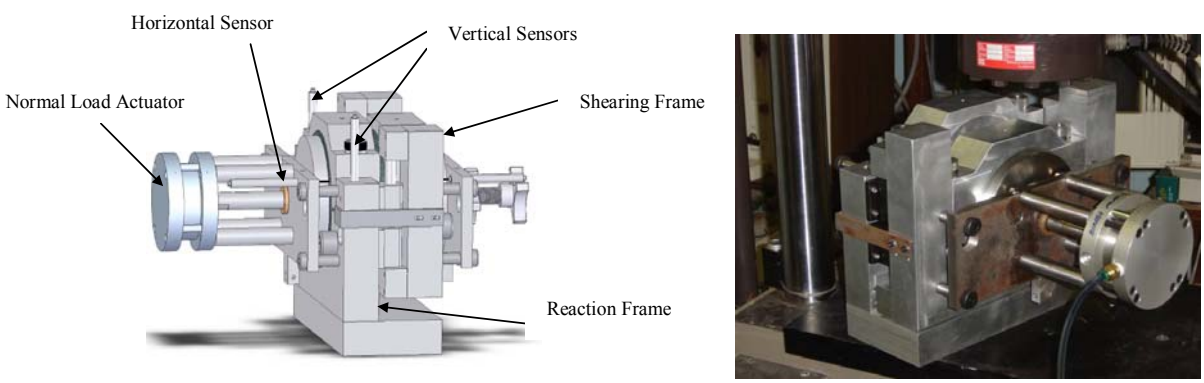


FIGURE 1 LISST.

Testing Procedure

Test specimens 4-in. in diameter are positioned inside the LISST device such that the tack coat interface is placed directly in the middle of the gap between the shearing and the reaction frames. The interface shear strength is estimated through measuring the shear strength of the test specimen at the interface. A shearing load is applied at a constant rate of 0.1 in./min on the specimen until failure. A normal load actuator is used to apply a confining pressure of 20 psi for those samples tested under confinement condition. Figure 2 shows a typical result of shear stress versus displacement curve. The interface shear strength (ISS) is computed as follows:

$$\text{ISS} = P_{\text{ULT}}/A = \frac{4\,P_{\text{ULT}}}{\pi D^2} \tag{1}$$

where

ISS = interface shear strength (psi);
P_{ULT} = ultimate load applied to specimen (lb);
A = cross-sectional area of test specimen (in.^{2}); and
D = specimen diameter (in.).

ISS versus displacement curve is plotted for each specimen. Shear strength is determined from the peak load and used in the analysis.

Pull-Off Field Test Device: Louisiana Tack Coat Quality Tester

The Louisiana Transportation Research Center (LTRC) and InstroTek, Inc., manufacturer of the ATacker, partnered to develop the LTCQT during the NCHRP Project 9-40 (Figure 3). The LTCQT is used to evaluate the quality of the bond strength of tack coat in the field. It is a modification of an existing device named ATacker (*4*). The final generation of the device is equipped with a closed-loop servo motor actuator for precision control of the rate of displacement during testing. It is capable of measuring loads of up to 100 lb with an accuracy of ±1%. The displacement of the actuator is measured using a position transducer that has a total travel of 4-in.

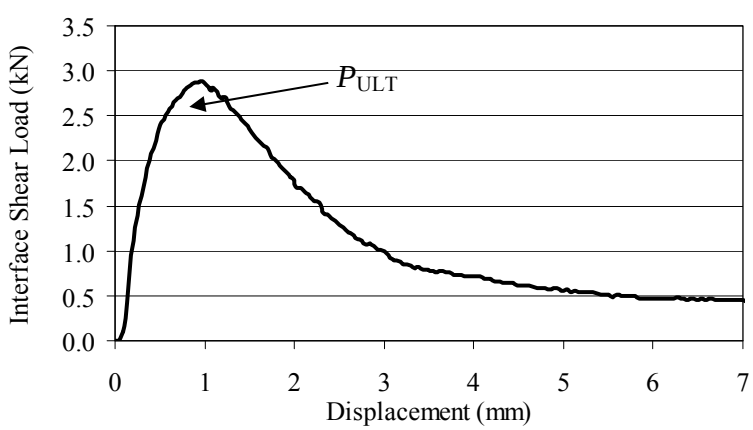


FIGURE 2 Typical load-displacement curve.

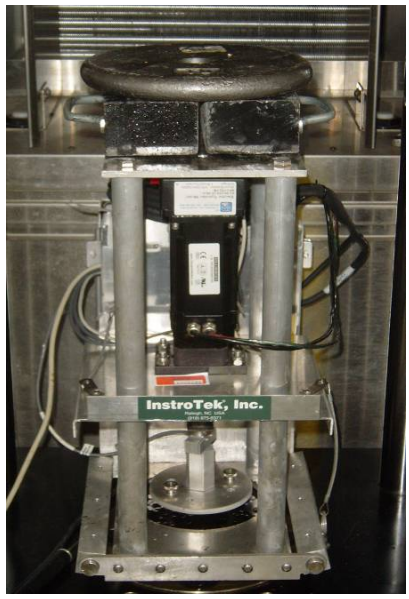


FIGURE 3 Description of the LTCQT.

Software was developed to display the time, normal load, and displacement of the actuator continuously during testing while graphically illustrating the relationship of the normal load and time. It also allows the user to input the required compressive load, the time to hold the compressive load, and the displacement rate required. The actual holding time of the compressive load is displayed during testing as well as the actual displacement rate. In addition, the software allows the user to move the actuator manually.

Test Procedure

Prior to testing, the tacked areas should be conditioned to the correct testing temperature using a heat blower or fan. It is recommended that the testing temperature should be the softening point of the tack coat material. After conditioning, the LTCQT is placed directly above the tacked surface to be tested. The correct weight is then placed on top of the LTCQT device and the compressive load, time to hold the compressive load, and the displacement rate are entered into the computer by the user. The compressive load should not exceed the weight placed on top of the LTCQT device. Immediately following the initiation of the test, the load shall be offset such that the software displays a load of 0 lb prior to the contact between the loading plate and the tacked surface. It is also recommended that the plate be positioned as close as possible to the tacked surface prior to testing so as to minimize the change in temperature. The initial position of the loading plate should be determined to allow sufficient time for the observation of the initial load and application of the offset. The compressive load is then mechanically applied to the tacked surface for the specified amount of time. Once the allotted time has ended, the loading plate is automatically moved away from the tacked surface at the prescribed displacement rate. The software records the normal load, vertical displacement, and time throughout the test.

The ultimate tensile load, P_{ult}, of the tack coat material is obtained from the test measurements and the tensile strength is calculated as follows:

$$\mathrm{TS} = \frac{P_{\mathrm{ULT}}}{\Delta} = \frac{4\,P_{\mathrm{ULT}}}{\pi D^2} \tag{2}$$

where

TS= tensile strength (psi);
P_{ULT} = ultimate tensile load (lb); and
D = diameter of the loading plate (in.).

EXPERIMENTAL PROGRAM

The majority of the research activities conducted in this project was based on tack coat experiments conducted in a field environment. Field experiments were complemented with a number of laboratory experiments to assess the influence of variables, such as laboratory compaction, rheological properties of tack coat materials, and test temperature. The experimental program was divided into experimental test matrices, which answered specific objectives of the experimental program. Since all experiments made use of full-scale test lanes, a description of the construction process and the test variables in the field experiment is presented in the following section.

Tack Coat and Overlay Construction at the Test Site

Table 1 presents the test matrix simulated in the LTRC Pavement Research Facility (PRF) field experiment, which used conventional paving equipment and a computerized tack coat distributor truck. Four types of pavement surfaces and five tack coat materials were evaluated. However, only one emulsion (SS-1h) was used on the new HMA surface and two emulsion grades (SS-1h and SS-1) were used on the milled surface. Four residual application rates were selected including zero (no-tack). Effects of wet and dusty conditions during construction operations were simulated for the different surface types as part of the experimental program. To evaluate variation in the results, triplicate samples were tested for each condition; 474 samples were tested as part of the test matrix. Laboratory specimens (cores) were obtained from the pavement test sections. Details about the conditioning and testing procedures have been presented elsewhere (*2*).

TABLE 1 Test Factorial for Field-Prepared Samples

Variables	Content	Levels
Pavement surface type	Old HMA, new HMA, PCC, milled HMA	4
Tack coat material	SS-1h, SS-1, CRS-1, trackless, PG 64-22	5
Residual application rate	0 (no tack), 0.031, 0.062, 0.155 gal/yd^2	4
Wet (rain) condition	Wet, dry	2
Dusty condition	Dusty, clean	2
Test temperature	25°C	1
Confinement pressure (psi)	0, 20	2
Tack coat coverage	50%, 100%	2
Number of replicates	3	3
Total number of samples		474

Three test lanes were prepared at the LTRC PRF to accommodate the test factorial presented in Table 1. The total length of each lane was 185 ft in length and 13 ft in width. It is noted that each lane contained test and distributor truck access sections. Each test section had a length of 15 ft and a width of 6.5 ft. The length of the access section, however, was selected to ensure that the distributor truck could achieve the required speed in order to apply the correct tack coat application rate. Test specimens for a specific factorial level were cored from each corresponding test section. Surface roughness values for each lane were measured using a laser-type surface texture evaluation device (DYNATEST 5051 Mark III road surface profiler) according to ASTM E 1845: Standard Practice for Calculating Pavement Macrotexture Mean Profile Depth. The measured roughness value for Lane 1, Lane 2, and Lane 3, were 1.07, 1.09, and 1.09 mm, respectively.

Simulation of Dusty and Wet Conditions

The effects of construction conditions such as dustiness of existing pavement surface and wetness (rainfall) of tacked surface were investigated. In order to simulate dusty conditions, a silty–clay type soil classified as A4 based on the AASHTO soil classification, was uniformly applied at a rate of 0.07 lb/ft^2 on the existing HMA surface prior to tack coat application (Figure 4*a*). The wet condition was simulated by uniformly spraying water at a rate of 0.06 gal/yd^2 on tacked surfaces and prior to placement of the HMA mixture (Figure 4*c*). Wet condition was only considered for the SS-1h tack coat, due to the limited number of test lanes at the PRF facility.

Tack Coat Distributor Truck

An Etnyre computerized tack coat distributor truck, Model 2000, was used in the application of tack coat materials (Figure 4*b*). The truck had a heated tank for holding tack coat materials at the desired application temperature. While the trackless tack coat was applied at a temperature of 82°C, the SS-1h and CRS-1 tack coat materials were applied at a temperature of 688C. Tack coats were applied in the undiluted state. Mounted on the back of the truck, a spray bar fitted with nozzles distributed tack coat material at the specified application rate. The total width of the spray bar was extended to 14.1 ft in order to provide full coverage of a single lane. The computerized system inside the truck can be programmed to multiple application rates. Application rate was adjusted by altering the truck speed and nozzle type and size.

Measurements of Applied Tack Coat Rate

Several tack coat application rate verification experiments were conducted in order to calibrate the distributor truck prior to the actual spraying of the tack coat materials at the PRF site. These experiments were performed according to test method A of ASTM D2995 (*5*). Two nozzle types were evaluated during these experiments, a 0.16-cm coin slot and a S36-4 V slot. The S36-4 V slot was selected for subsequent tack coat application since it provided the most consistent results for the residual application rates. In addition, the residual application rates were measured during the construction of the test lanes. Seven geotextile pads (30.5 by 30.5 cm) were placed across the width of each test section prior to tack coat application. Each pad was collected subsequent to application of the tack coats and weight measurements were performed at 30-min intervals until no increase in weight was observed between two consecutive measurements. The original and the final weights of the pads were used to compute the residual application rates.

Table 2 presents a comparison of the target and measured application rates. The consistency of the measured application rates, as determined by the coefficient of variation (COV), is shown in Table 2. In general, the measured application rates were fairly consistent as evident by the low COV values. The measured application rates were slightly different than the target ones; however, the measured rates met the objectives of the test matrix to simulate low, medium, and high levels.

Overlay Construction

A 12.5-mm HMA mixture was placed on top of the tacked surfaces at a thickness of approximately 3-in. It is noted that a material transfer device was used to transfer the mixture from the haul trucks to the hopper of the paver (Figure 4*d*) in order to eliminate problems associated with construction traffic on tacked surfaces. Subsequent to completion of the HMA overlay placement, each lane was then marked based on previously documented reference points identifying the various test sections within each lane (Figure 4*a*).

RESULTS AND ANALYSIS

The mean ISS along with their standard deviations and coefficients of variation were obtained for each condition considered in the test factorial. Triplicate samples were tested for each test condition defined by tack coat type, application rate, confining pressure, and dusty and wet conditions. The COV in the test results were less than 15% for all simulated conditions. As presented in this section, test results were analyzed to investigate the effects of the variables considered in the test factorial on ISS.

TABLE 2 Tack Coat Distribution Results at the PRF Site

Tack Coat	Target Residual Application Rate (gal/yd^2)	Measured Residual Application Rate		
		Average (gal/yd^2)	Standard Deviation (gal/yd^2)	COV (%)
SS-1h	0.031	0.044	0.004	8.8
	0.062	0.073	0.007	9.1
	0.155	0.139	0.022	16.6
Trackless	0.031	0.040	0.002	6.1
	0.062	0.068	0.004	7.7
	0.155	0.177	0.011	5.9
CRS-1	0.031	0.035	0.004	15.3
	0.062	0.062	0.004	5.7
	0.155	0.152	0.007	4.5

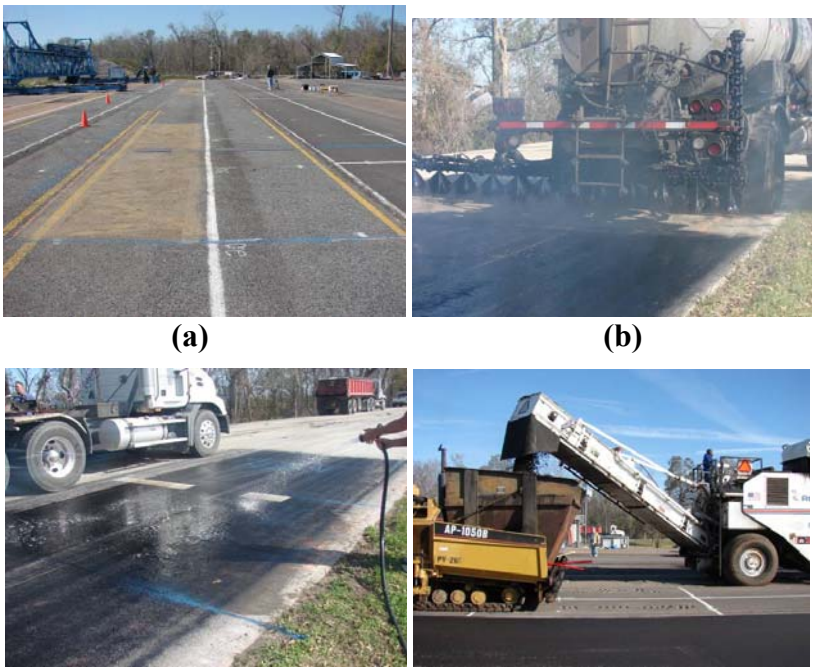
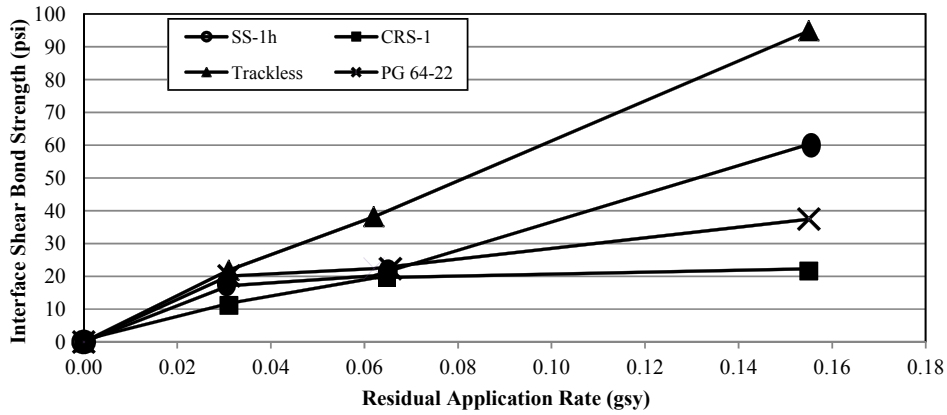


FIGURE 4 Procedures for tack coat application and overlay construction: (*a*) dirt application (left side); (*b*) tack coat application; (*c*) water spraying using a hose; and (*d*) overlaying on tacked surface.

Effect of Tack Coat Type and Residual Application Rate

Figure 5 (*a* through *d*) presents the variation of ISS with emulsified tack coat types and residual application rates for the different surface types (existing HMA surface, PCC surface, milled HMA surface, and new HMA). These results were obtained for clean and dry samples with no confinement pressure at 25°C. As shown in these figures, all tack coat materials showed that the interface shear strength increased as the residual application rate increased within the evaluated application rate range (from 0.031 to 0.155 gal/yd^2). It was not possible to identify an optimum residual application rate. This may indicate that, under actual field conditions, optimum application rates may be greater than what is commonly predicted from laboratory-based experiments. However, while higher application rates may increase ISS, excessive tack coat may migrate into the HMA mat during compaction and service causing a decrease in the air void content of the mix and may even cause the appearance of fat spots on the HMA surface. One study reported that excess tack might be picked up by hauling trucks and paving equipment causing safety concerns when tracked onto pavement markings in traffic intersections close to the construction area (*6*).

For existing HMA and PCC surface types, the trackless tack coat exhibited the highest shear strength and CRS-1 and SS-1 exhibited the lowest. Trackless tack coat consists of a polymer-modified emulsion with hard base asphalt cement. These results relate directly to the viscosity of the residual binders at the test temperature (*7*). The influence of tack coat type appears to increase with the increase in the residual application rate. Except for the milled HMA surface, the no-tacked cores failed during extraction due to the poor bonding at the interface. This reiterates the importance of using a tack coat material at the interface to avoid poor



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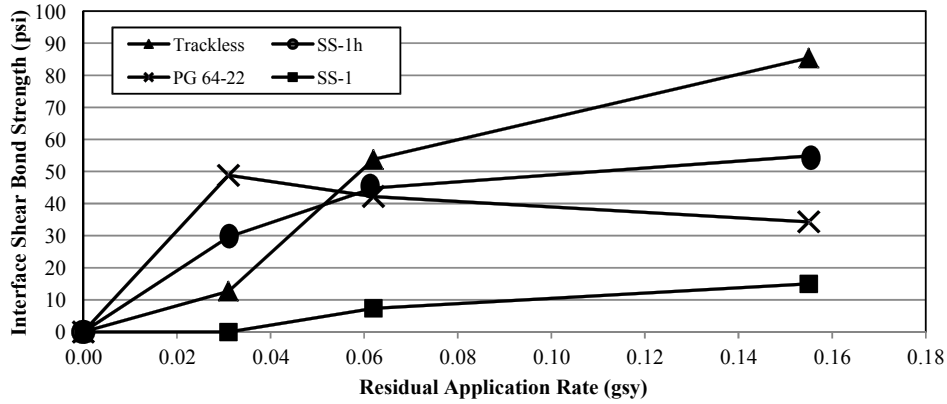

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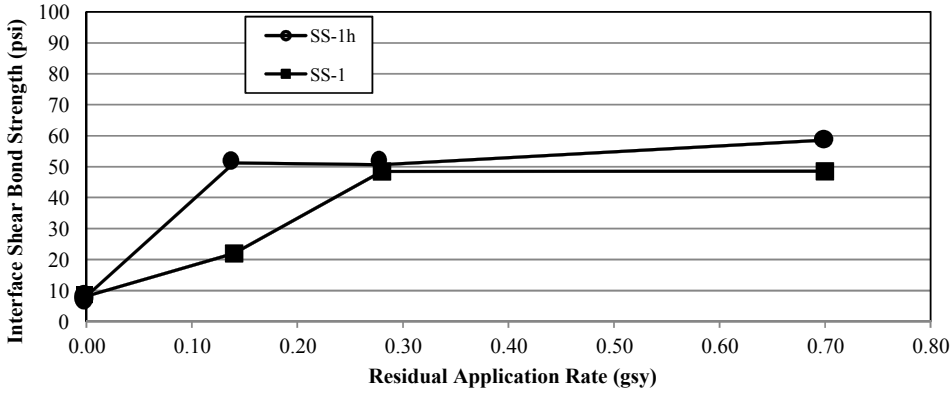

(c)

FIGURE 5 Effects of residual application rates and tack coat types on ISS for (*a*) existing HMA surface; (*b*) PCC surface; and (*c*) milled HMA surface.

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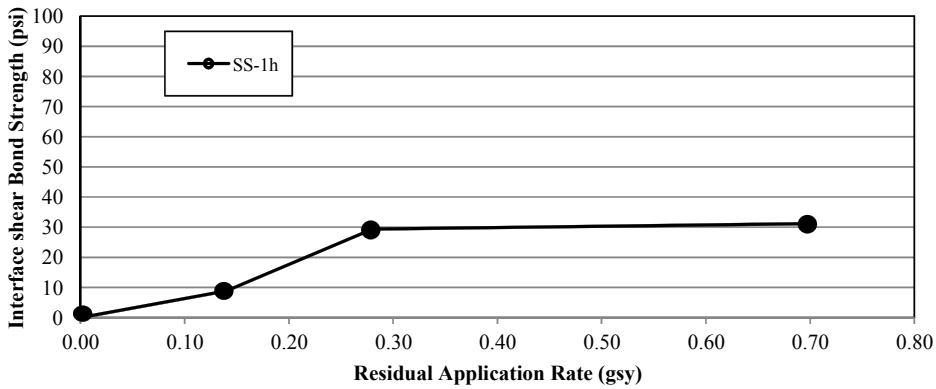


(d)

FIGURE 5 (*continued*) **Effects of residual application rates and tack coat types on ISS for (*d*) new HMA surface.**

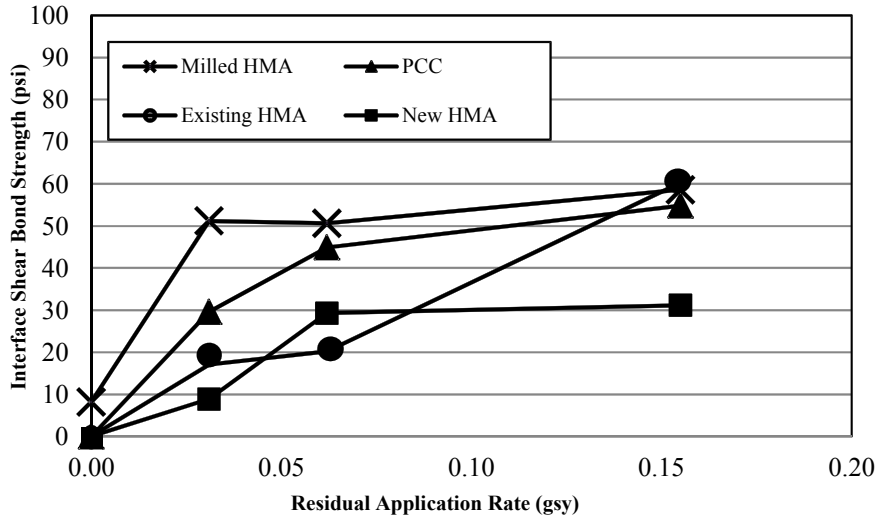
bonding between the layers. To balance the aforementioned factors, one should select a tack coat application rate that would ensure that the ISS is greater than the calculated shear stress at the interface due to traffic and thermal loading.

Effect of Surface Type

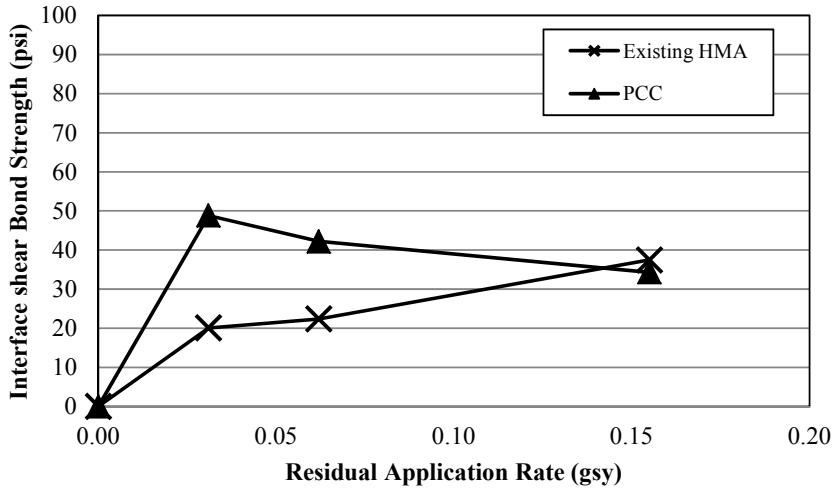
SS-1h-emulsified tack coat was evaluated for all four surface types. On the other hand, the trackless tack coat and PG 64-22 asphalt binder were evaluated for two surface types: existing HMA and PCC surfaces. PCC samples were tested parallel to the direction of the grooves. This test arrangement should generate the lowest ISS, and therefore, is more conservative. Figure 6 (*a* through *c*) presents the variation of the ISS with surface types and residual application rates. As shown in these figures and due to its high roughness, the milled HMA surface provided the highest ISS, followed by the PCC surface. In most cases, the existing HMA surface provided greater interface strength than the new HMA surface. It is noted that differences are more pronounced at low and intermediate application rates and less pronounced at high application rates. It is possible that the effects of microstructure features that contribute to the surface roughness or texture are less pronounced when they are filled with tack coat materials.

Effect of Dusty Conditions of HMA Surface

Figure 7 presents the effects of dust on the interface shear strength values at no confinement and confinement (20 psi) test conditions. As shown in this figure, the majority of the cases showed differences between clean and dusty conditions. In general, dusty conditions exhibited higher interface strength than clean conditions, especially when tested with a confinement condition. One possible explanation for these results is that a high-viscosity, gritty mastic was formed when tack coat combined with dust and, thus, provided a greater resistance to shear movement.



(a)



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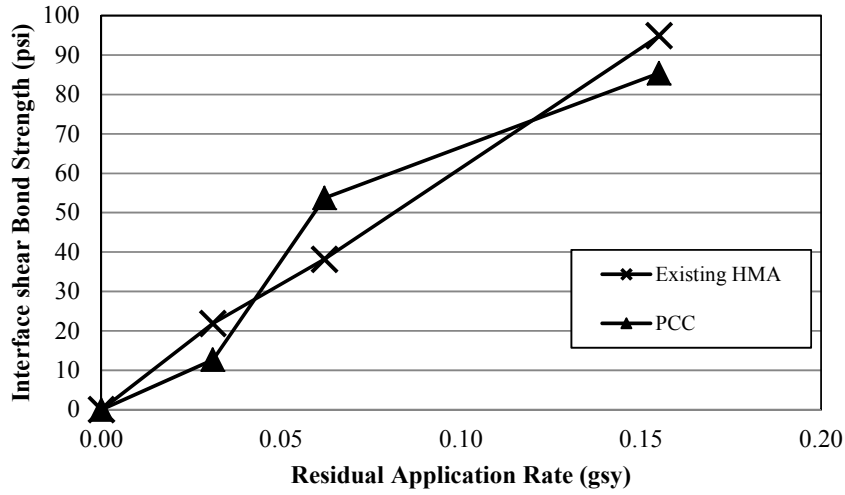
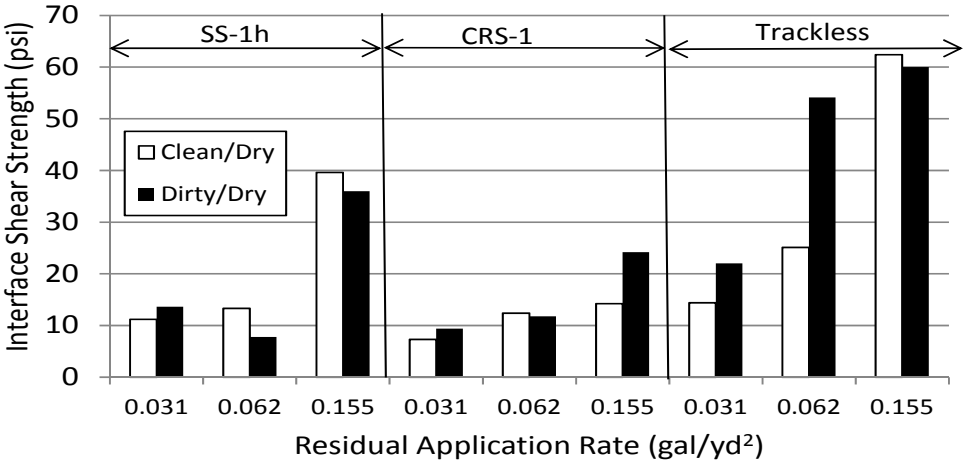


(c)

FIGURE 6 Effects of surface types on ISS for (*a*) SS-1h tack coat; (*b*) PG 64-22; and (*c*) trackless tack coat.



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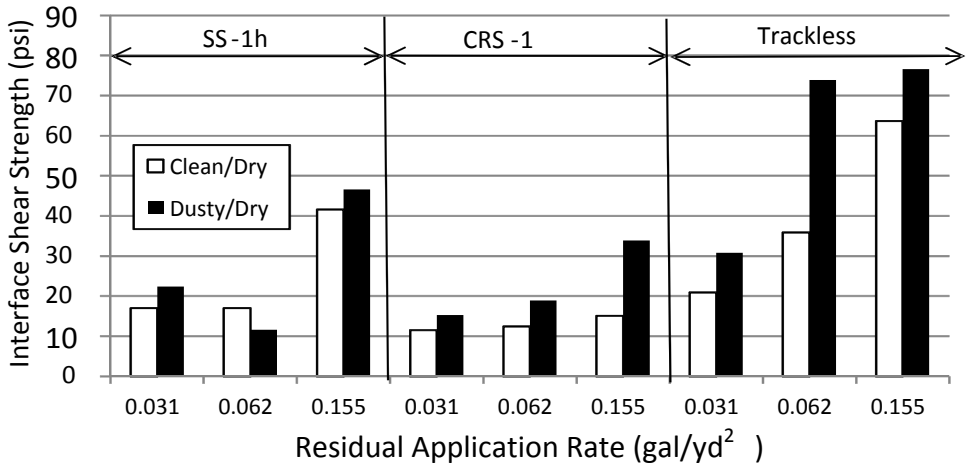
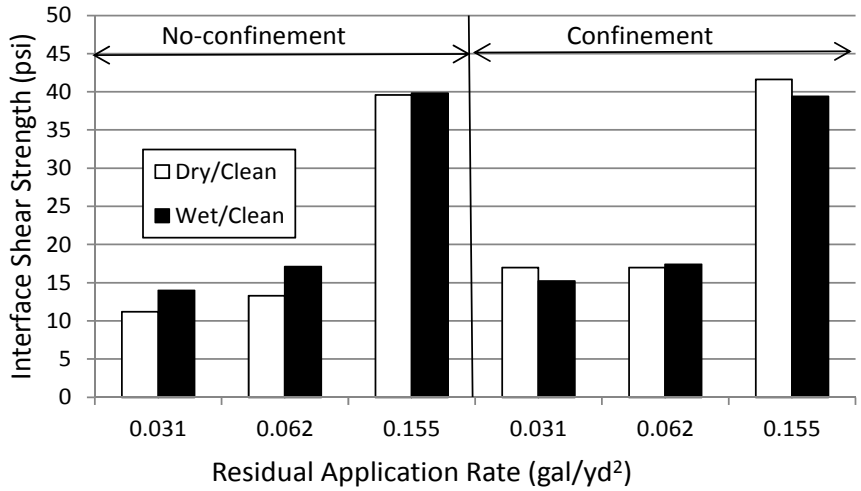


(b)

FIGURE 7 Dust effect on ISS with (*a*) no confinement and (*b*) confinement.

Effect of Wet or Rainfall Conditions of Tacked Surface

Figure 8 presents the effects of water (light rainfall) of a tacked surface at no confinement and with confinement (20 psi) on ISS. It is noted that the majority of the cases showed no significant differences between dry and wet conditions. This indicates that, even in the presence of light rains, the placement temperature of an overlay on an HMA surface will cause the water to evaporate or to infiltrate to the underlying layers with no practical consequence on the interface bond strength.



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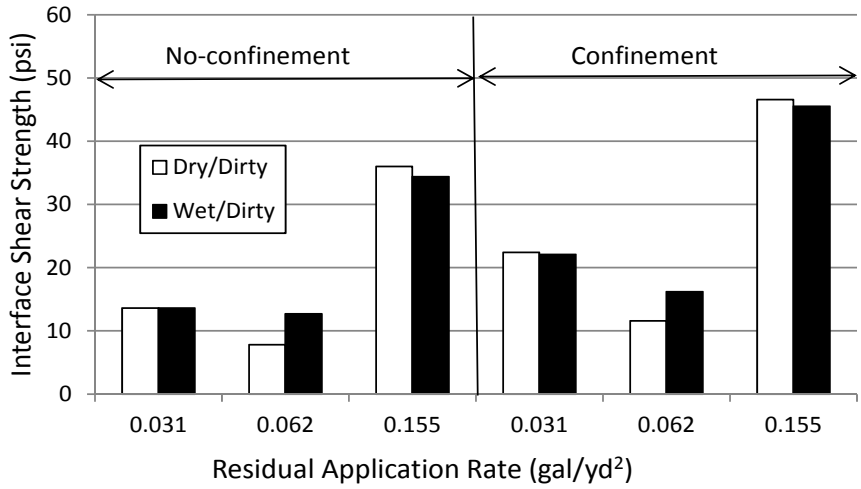

(b)

FIGURE 8 Effect of wetness on ISS for SS-1h tack coat with (*a*) clean and (*b*) dusty conditions.

Effects of Preparation Methods

To assess the influence of sample preparation methods, Figure 9 compares the ISS of laboratory-fabricated samples to that of field-extracted cores for tack coat SS-1h in the case of the new HMA surface. As shown in this figure, laboratory-prepared samples grossly overestimated the ISS by a factor ranging from two to 10 when compared to field-extracted cores. In addition, while a decreasing trend was observed in the laboratory, an increasing trend in the measured ISS was observed in the field. A number of factors may cause this discrepancy including the difference in mixing and compaction methods and application method for the tack coat materials. Difference in compaction methods may result in differences in air voids' values and distributions in the sample, mix resistance to shear loading, and mix density.

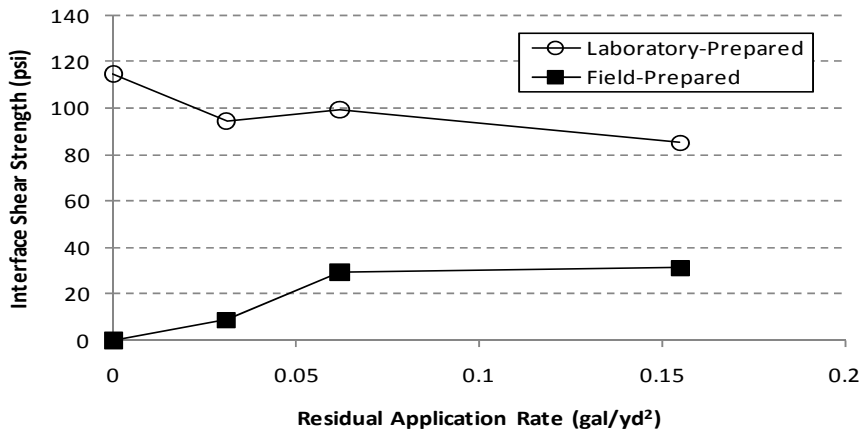

FIGURE 9 Effects of sample preparation methods on the ISS.

SUMMARY AND CONCLUSIONS

The main objectives of this project were to determine optimum application methods, equipment type and calibration procedures, residual application rates, and asphalt binder materials for the various uses of tack coats and to recommend revisions to relevant AASHTO test methods and practices related to tack coats. During the course of this project, the research team developed the LTCQT to evaluate the quality of tack coat spray application in the field. Further, the LISST was developed for characterization of interface shear strength of cylindrical specimens in the laboratory.

Research in this project also quantified the effects of tack coat material type, tack coat application rate, and pavement surface type (i.e., HMA versus PCC) including milled versus unmilled surfaces on the ISS based on full-scale field tests. The variation of ISS between field- and laboratory-prepared samples was also investigated. To achieve this objective, five types of tack coat materials were applied at three application rates on four different types of surfaces at the PRF site. Samples were cored from the constructed test lanes, and the ISS was measured using the LISST. Based on the results of this analysis, the following conclusions may be drawn:

- **Application rate.** All tack coat materials showed the highest shear strength at an application rate of 0.155 gal/yd^2. Within the tested application rate range, it was difficult to precisely determine the optimum residual application rate. While higher application rates may increase ISS, excessive tack coat may migrate into the HMA mat during compaction causing a decrease in the air void content of the mix. One should select the tack coat application rate that would ensure that the ISS is greater than the predicted shear stress at the interface due to traffic and thermal loading. Table 3 presents the recommended tack coat residual application rates for different surface types.
- **Surface type.** A direct relationship is observed between the roughness of the existing surface and the developed shear strength at the interface. Therefore, the milled HMA surface provided the greatest ISS followed by the PCC surface, the existing HMA, and the new HMA surface. The new HMA surface was smooth, unweathered, and had already a coating of asphalt.

TABLE 3 Recommended Tack Coat Residual Application Rates

Surface Type	Residual Application Rate (gsy)
New asphalt mixture	0.035
Old asphalt mixture	0.055
Milled asphalt mixture	0.055
PCC	0.045

- **Wetness.** A small amount of water appeared to negatively affect ISS in case of the use of PG 64-22 as a tack coat material. However, the effect of surface wetness on ISS was less evident for emulsion-based tack coat materials. In this case, results indicate that a small amount of water can be flashed away by the hot HMA mat and, practically speaking, have inconsequential effects on the quality of the tack coat.
- **Preparation method.** Laboratory-prepared samples grossly overestimated the ISS when compared to pavement cores. In addition, while a decreasing trend was observed in the laboratory, an increasing trend in the measured ISS was observed in the field.

ACKNOWLEDGMENTS

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Implementation of Performance-Based Mixture Designs to Enhance the Durability of Asphalt Pavements in New Jersey

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The Superpave® mixture design system provided an excellent foundation for the proper selection of materials to conduct a volumetric-based asphalt mixture design. However, one thing lacking in the Superpave system is a test procedure to verify that the performance of the asphalt mixture produced will perform as anticipated by the designer. Another important aspect generally not linked to pavement design is the quality control during production of these asphalt mixtures. It is well understood that there are differences in the asphalt mixture properties between lab-prepared and plant-prepared asphalt materials, therefore a good performance-based system should include a quality control component to ensure that the asphalt mixture design performs as intended. With budgets for roadway infrastructure extremely tight, owners want the maximum service life from every dollar invested.

This paper covers the implementation of performance-based mixture design and quality control for "specialty" asphalt mixes currently specified by the New Jersey Department of Transportation (DOT). The New Jersey DOT utilizes different mixture types for specific pavement applications and bridge deck overlays; therefore, different test procedures and performance criteria are established for the different mix applications. For all of the specialty asphalt mixtures utilized by New Jersey DOT, performance-based testing is conducted on the mix produced during mix design, test strip construction, and project production to ensure that the mixture meets the performance requirements and that the asphalt mixture produced for the project continually meets the specified performance criteria. The adoption of the performance-based design system for these asphalt mixtures has seen a dramatic improvement in the durability and cracking resistance of asphalt mixtures in these problematic areas.

NEW JERSEY'S CONDITIONS

The predominant pavement distress in New Jersey is longitudinal cracking. In the past, typical rehabilitation projects involving mill 2 in.–pave 2 in. have lasted approximately 7 to 8 years, while a mill 2 in.–pave 4 in. have lasted 8 to 9 years. Composite pavements [asphalt overlaid on aging portland cement concrete (PCC) pavements] are lasting 6 to 8 years. The underperforming nature of these asphalt pavements is a primary function of the climate and traffic conditions of New Jersey. New Jersey's climate and traffic conditions result in a unique combination of environmental and mechanical loading that has caused a severe impact on the state's transportation system. With respect to climate, New Jersey typically witnesses the following:

- 30 to 45 days of >90°F air temperature depending on location in the state;
- 85 to 95 days of <32°F air temperature, depending on the location in the state; and
- Over 120 days of measureable precipitation throughout the year.

The general climate conditions shown above indicates that New Jersey roadways undergo a severe swing in environmental conditions with approximately one-third of the year having some type of measurable precipitation (snow or rain). New Jersey's latitude results in an extreme number of freeze–thaw cycles that cause significant damage to pavements.

In addition to the broad temperature ranges, New Jersey also exhibits extremely high levels of traffic and congestion. New Jersey DOT's funding for its aging transportation infrastructure is insufficient to maintain the network at an acceptable level of service. The TRIP Report, *Future Mobility in New Jersey* (June 2009), offered the following conclusions:

- Over the next 10 years, New Jersey faces an estimated transportation funding shortfall of $35 billion. This limits the number of projects that can be conducted to maintain and expand the current transportation system.
- Nearly half of the major roads in the state, which are maintained by municipal, county, and state governments, are rated as deficient.
- Vehicle travel on New Jersey's major highways increased by 29% from 1990 to 2006, jumping from 59 billion vehicle miles traveled in 1990 to 76 billion vehicle miles traveled in 2006. Travel in New Jersey is expected to increase by 30% by 2025, reaching approximately 99 billion vehicle miles traveled.
- The general economy of New Jersey is highly influenced by the state of its transportation system. Approximately $287 billion in goods are shipped annually from sites in New Jersey and another $267 billion in goods are shipped to locations within New Jersey, mostly by commercial trucks on the state's highways.

One of the last major issues pertaining to the state of New Jersey's roadways is the pavement structure. Approximately 50% of the roadways New Jersey DOT oversees are composite pavement (i.e., asphalt overlay on top of aging PCC pavement). Current geometric constraints including curb, guiderail, and overpass structures limit increases in pavement thickness during rehabilitation. Typical composite pavements exhibit extensive reflective cracking within 5 years after asphalt overlay construction.

With deteriorating transportation infrastructure, limited funding for maintenance and projected traffic loading increasing significantly, the New Jersey DOT needed assurance they were getting high-performance asphalt mixtures in areas with high stress and limited options. (i.e., pavement structure limitations, high deflections).

In 2006, the New Jersey DOT began utilizing established hot-mix asphalt (HMA) performance tests in conjunction with the Superpave volumetric procedure on specialty mixes used by the New Jersey DOT. These specialty mixes are utilized in specific areas and locations in the pavement structure where the New Jersey DOT has observed premature pavement deterioration when conventional HMA was previously specified. At this time, the New Jersey DOT currently specifies four different specialty mixes: (*a*) high-performance thin overlay (HPTO); (*b*) binder-rich intermediate course (BRIC); (*c*) bridge deck waterproofing surface course (BDWSC); and (*d*) bottom rich base course (BRBC). Each one of these mixtures incorporates a volumetric design procedure with some of the materials and volumetric properties changed from conventional HMA.

Each one of the mixtures also requires HMA performance testing to evaluate their general fatigue and rutting properties at the mixture design, test strip, and production levels.

NEW JERSEY DOT'S PERFORMANCE-BASED ACCEPTANCE PROCEDURE

The New Jersey DOT has established a general acceptance procedure that the asphalt plants–contractors are required to follow to be allowed to produce and place their respective specialty mixture. The general procedure is as follows:

Step 1. The asphalt plant must conduce a volumetric design using the proposed materials and mixture design specifications for that particular specialty mixture. After the asphalt plant has successfully conducted their own volumetric design, the New Jersey DOT Regional Offices verify the volumetric at their laboratory. Once the volumetric have been verified and the constituents (aggregates and asphalt binder) of the asphalt mixture have been approved, the asphalt plant–contractor is allowed to proceed to Step 2.

Step 2. The asphalt plant–contractor must submit either laboratory-prepared loose mix or the virgin materials to a laboratory approved by the New Jersey DOT Bureau of Materials. The laboratory will then prepare the required test specimens for the respective performance tests. If the test specimens meet the specified performance criteria, the asphalt plant–contractor is then allowed to move to Step 3. Otherwise, the mixture must be redesigned.

Step 3. The asphalt plant–contractor must produce the mixture through the asphalt plant and construct a test strip. The location of the test strip is preferred to be close to the actual location of construction (i.e., shoulder area), but it is at the discretion of the contractor as long as it is approved by the New Jersey DOT. Loose mix used to produce the test strip is sampled and supplied to a laboratory approved by the New Jersey DOT Bureau of Materials. The same test procedure and performance criteria from Step 2 must again be met with the plant-produced material. If the test specimens fail, the asphalt plant–contractor must again produce the mixture through the plant and construct another test strip, essentially repeating Step 3 until the mixture passes the performance criteria established. Once the test strip material passes the loose mix criteria, the asphalt plant–contractor is allowed to produce and place the material on the project.

Step 4. The contractor must sample material during production for continued performance testing to ensure the mixture properties still meet the required specifications. The frequency of sampling is dependent on the specialty mixture being produced, as well as the quantity. In most cases, the total production of specialty mixes in New Jersey is under 5% of the total tonnage produced for New Jersey DOT projects.

Three different HMA performance test methods are utilized to characterize the specialty mixes. In most cases, both rutting and fatigue cracking are evaluated using one of the following test procedures:

- Asphalt Pavement Analyzer (APA) (AASHTO T340: Determining Rutting Susceptibility of Hot-Mix Asphalt Using the Asphalt Pavement Analyzer);
- Flexural Beam Fatigue (AASHTO T321: Determining the Fatigue Life of Compacted Hot-Mix Asphalt Subjected to Repeated Flexural Bending); and
- Overlay Tester (Texas DOT TEX 248-F: Test Method for the Overlay Test).

The type of fatigue test utilized is dependent on whether the mode of cracking is dependent on the flexural properties of the pavement or the expansion–contraction of PCC slabs.

NEW JERSEY DOT'S SPECIALTY MIXES: SPECIFICATIONS AND FIELD INSTALLATION

Currently, the New Jersey DOT has four specialty mixes that require the performance-based testing and protocols previously mentioned. These four mixtures include

1. HPTO;
2. BRIC;
3. BDWSC; and
4. BRBC.

Each mixture is explained in detail in the following sections.

High-Performance Thin Overlay

By Superpave definition, the New Jersey DOT's HPTO is a fine-graded, 9.5-mm nominal maximum aggregate size (NMAS) mixture. The HPTO is used as a rut-resistant and durable thin lift mixture for maintenance–pavement preservation applications, as well as a superior leveling course when extended staging time is expected. When small quantities are needed (< 100 mix tons), the HPTO has also been used for overlays on top of bridge decks. The required aggregate blend gradation, minimum asphalt content and design–production volumetric requirements are shown in Tables 1 and 2. The HPTO requires the use of a polymer-modified Performance Grade (PG) 76-22 asphalt binder and the addition of natural sand or recycled asphalt pavement (RAP) is not allowed.

Rutting performance testing, using the APA, is required during mixture design, test strip production, and mainline production for the HPTO. For acceptance, the HPTO must achieve a maximum of 4.0 mm of rutting at 8,000 loading cycles in the APA at testing conditions of 64°C, 100-psi hose pressure, and 100-lb wheel loads.

TABLE 1 Aggregate Blend Gradation of New Jersey DOT's HPTO

Sieve Size	Percent Passing by Mass
3/8 in.	100
#4	65–85
#8	33–55
#16	20–35
#30	15–30
#50	10–20
#100	5–15
#200	5.0–8.0
Minimum percent asphalt by mass of total mix	7.0

TABLE 2 Volumetric Requirements for New Jersey DOT's HPTO

	Required Density (% of Gmm)		Voids in Mineral Aggregate	Dust-to-Binder Ratio	Draindown AASHTO T 305
	N_{des} (50 gyr)	N_{max} (100 gyr)	VMA		
Design requirements	96.5	≤99.0	≥18.0%	0.6–1.2	≤0.1%
Control requirements	95.5–97.5	≤99.0	≥18.0%	0.6–1.3	≤0.1%

HPTO Field Implementation: Interstate 287 Southbound

An example of the application and performance of New Jersey DOT's HPTO can be found on Interstate 287 (I-287) Southbound (SB) between mileposts 30.2 and 35.5. The full-depth asphalt pavement in that area carries approximately 44 million equivalent single-axle loads (ESALs). In 2008, the pavement distress survey conducted within the New Jersey DOT's Pavement Management Program identified milepost section 30.2 to 35.5 I-287 SB as having a structural distress index (SDI) of 1.7 (with 0 being worst and 5 being the best condition), triggering a rehabilitation requirement (Figure 1). The primary distress associated was top-down, longitudinal fatigue cracking. It should be noted that the distressed overlay (from a mill 2 in./pave 2 in. application) had lasted 8 years.

A field forensic program identified that a 1-in. mill could be utilized to limit the amount of RAP produced on the job while eliminating the top-down cracking and crack sealer previously used to seal the exposed cracking. After milling, a 1-in. HPTO overlay was applied to help improve the cracking resistance along this section of I-287. The HPTO was placed after a hot PG 64-22 tack coat was applied to ensure sufficient bonding to the milled surface was achieved.

Additional SDI testing and analysis was conducted in 2010 and 2012 and shown in Figure 1. The SDI results, 1.5 and 3.5 years after the HPTO was placed, show that the current HPTO application is performing exceptionally well with an SDI = 3.9 and has not changed within the past 2-year measurements.

Binder-Rich Intermediate Course

The main use of New Jersey's BRIC is for placement over existing PCC and at the bottom of an HMA overlay to aid in minimizing reflective cracking of the HMA overlay due to horizontal and vertical movements at the PCC joint–crack due to environmental and traffic loading. The BRIC is a 4.75-NMAS mixture consisting of the aggregate gradation shown in Table 3 and a minimum asphalt content of 7.0%. The grade of asphalt binder is required to be at least a PG 70-28. Additional volumetric requirements for design and during production are shown in Table 4. New Jersey's BRIC mixture was adapted from the crack attenuating mixture (CAM) developed and used by the Texas DOT.

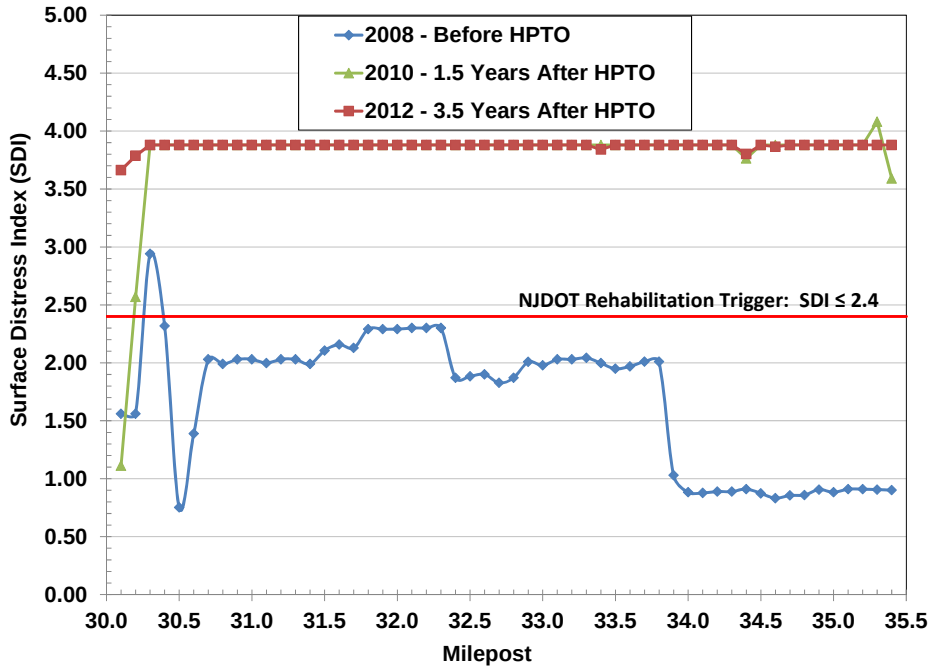

FIGURE 1 SDI for before and after HPTO application on I-287 in New Jersey.

TABLE 3 Aggregate Blend Gradation of New Jersey's BRIC

Sieve Size	Percent Passing by Mass
3/8 in.	100
#4	90–100
#8	55–90
#30	20–55
#200	4–10
Minimum percent asphalt by mass of total mix	7

TABLE 4 Volumetric Requirements for New Jersey's BRIC

	Required Density (% of Gmm)		Voids in Mineral Aggregate	Dust-to-Binder Ratio	Draindown AASHTO T 305
	N_{des} (50 gyr)	N_{max} (100 gyr)	VMA		
Design requirements	97.5	≤99.0	≥18.0%	0.6–1.2	≤0.1%
Control requirements	96.5–98.5	≤99.0	≥18.0%	0.6–1.3	≤0.1%

In the past, the New Jersey has never specified the use of a PG 70-28 asphalt binder. However, during initial research studies to evaluate its possible use, it was found that the PG 70-28 asphalt binder performed better than PG 64-22 and PG 76-22, two commonly used asphalt binder grades in New Jersey, in both the Flexural Beam Fatigue (AASHTO T321) that simulates vertical deflection at the PCC joint–crack due to traffic loading, and the Overlay Tester (Texas DOT TEX 248F) that simulates horizontal movement at the PCC joint–crack due to environmental–temperature cycling (*1*). Examples of test results generated during these studies are shown in Figures 2 and 3. The test results also confirm the results reported by Bennert and Maher (*2*) regarding better fatigue resistance through the use of asphalt binders with lower low-temperature PG grades.

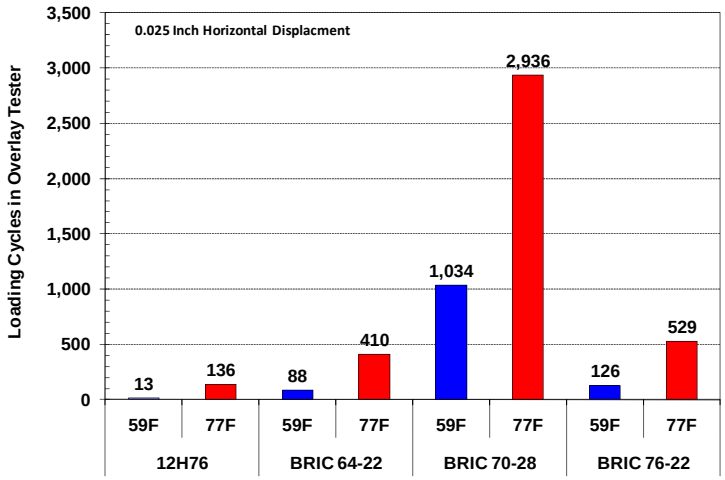

FIGURE 2 Overlay tester results for New Jersey BRIC mixture with different PG asphalt binders.

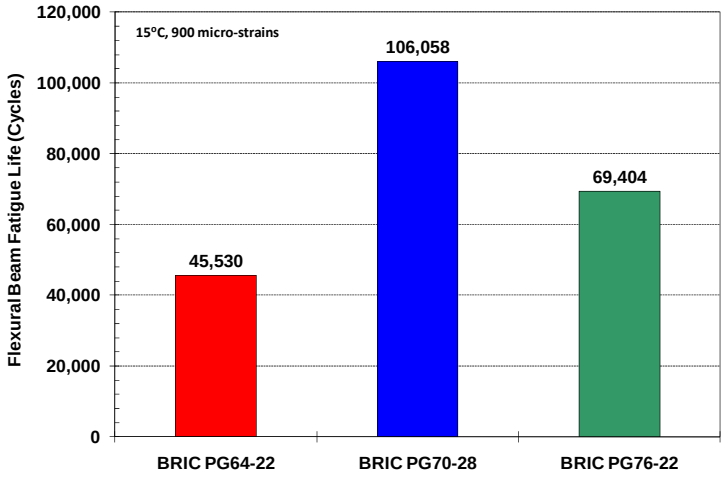

FIGURE 3 Flexural beam fatigue results of New Jersey's BRIC mixture with different PG asphalt binders.

To verify the performance of the BRIC, the mixture is required to be evaluated for rutting performance using the APA (AASHTO T340) and cracking resistance using the Overlay Tester (Texas DOT, TEX 248F). The performance requirements for the mixture design, test strip, and production material are as follows:

- APA (AASHTO T340):
  - 64°C, 100-lb wheel load, 100-psi hose pressure, and
  - Maximum rut depth of 6.0 mm at 8,000 loading cycles;
- Overlay tester (Texas DOT, TEX 248F):
  - 25°C test temperature, 0.025-in. horizontal displacement, 10-s loading frequency, and
  - Minimum of 700 cycles.

It should also be noted that the New Jersey is implementing the use of the BRIC mixture with a stone matrix overlay (SMA) being placed over it. This is to ensure that a fatigue-resistant asphalt mixture can withstand residual vertical and horizontal movement not "absorbed" by the BRIC mixture. The placement of stiff asphalt mixtures above or below a highly crack resistant mixture often results in a "crack jumping" phenomenon, where a crack forms above, and sometimes below, the more flexible mixture (Figure 4). New Jersey will be utilizing the SMA–BRIC on six projects during the 2011 paving season.

Bridge Deck Waterproofing Surface Course

The main purpose of New Jersey's BDWSC is to provide a rut- and fatigue-resistant and impermeable bridge deck overlay that can be placed using static compaction techniques. With an aging infrastructure, the New Jersey does not allow the use of vibratory compaction techniques when placing asphalt overlays on bridge decks. This has resulted in numerous bridge deck

FIGURE 4 Cracking above and below a highly fatigue-resistant mixture.

overlays compacted to low densities, creating a highly porous bridge deck overlay. Past attempts using an asphalt-treated membrane has not improved the general performance of the overlay, as infiltrated water has usually found a pathway to the bridge deck.

Since 2008, the New Jersey DOT has implemented the use of a BDWSC asphalt mixture to overlay and preserve its bridge decks. The BDWSC is a 9.5-mm NMAS, highly modified asphalt mixture purposely designed for low permeability. Tables 5 and 6 show the aggregate blend gradation and minimum asphalt content of the BDWSC and design and production volumetric of the BDWSC, respectively.

According to the specifications, the mixtures are recommended to be modified using either a polymer-modified asphalt binder or a concentrated thermoplastic polymeric asphalt modifier. Although the specification does provide PG recommendations of PG 76-28 to PG 82-34, it is the mixture performance that dictates final acceptance of the BDWSC.

Performance verification testing of the BDWSC consists of rutting potential measured in the APA (AASHTO T340) and fatigue cracking resistance measured in the Flexural Beam Fatigue (AASHTO T321). The performance requirements for the mixture design, test strip, and production material are as follows:

- APA (AASHTO T340):
  - 64°C, 100-lb wheel load, 100-psi hose pressure, and

TABLE 5 Aggregate Blend Gradation of New Jersey's BDWSC

Sieve Size	Percent Passing by Mass
1/2 in.	100
3/8 in.	80–100
#4	55–85
#8	32–42
#16	20–30
#30	12–20
#50	7–16
#100	3–12
#200	2.0–6.0
Minimum percent asphalt by mass of total mix	7.0

TABLE 6 Volumetric Requirements for New Jersey's BDWSC

	Required Density (% of Gmm)	Voids in Mineral Aggregate	Dust-to-Binder Ratio	Draindown AASHTO T 305
	N_{des} (50 gyr)	VMA		
Design requirements	99	≥18.0%	0.3–0.9	≤0.1%
Control requirements	98–100	≥18.0%	0.3–0.9	≤0.1%

  - Maximum rut depth of 3.0 mm at 8,000 loading cycles;
- Flexural Beam Fatigue (AASHTO T321):
  - 15°C test temperature, 10-Hz frequency, Sinusoidal waveform, 1,500 microstrains, and
  - Minimum of 100,000 cycles.

During construction, the BDWSC specification states to ensure that the paving surface is clean and apply the tack coat using the same tack coat material as required for the adjacent roadway paving on the project. However, for the ACROW steel deck, the tack coat application utilized a PG 76-22 and a sand grit to help reduce the potential of the BDWSC from sliding and shoving.

BDWSC Implementation: Route 80 ACROW Bridge

In November 2009, the New Jersey constructed and overlaid a temporary overpass–bridge on Route 80 (Figure 5). The steel paneled bridge deck was overlaid with 2.5 to 3.5 in. of a 12.5-mm Superpave mixture with a PG 76-22 asphalt binder. The bridge was open to westbound traffic on March 26, 2010. Within 2 weeks after the bridge was open to traffic, the New Jersey's contractor began patching the HMA due to excessive and rapid deterioration from cracking and shoving (Figure 6).

Approximately 1½ months after the asphalt overlay was opened to traffic, it was removed due to excessive failures and repeated patching. It was determined that the BDWSC would be placed on the ACROW bridge deck using a PG 76-22 asphalt binder as a tack coat and sand broadcasted onto the tacked steel panels to help mitigate potential sliding. The asphalt supplier had a pre-approved BDWSC mixture design, and therefore only needed to have material supplied during construction. Test results indicated average Flexural Beam Fatigue and APA to be 163,000 cycles and 1.8 mm of APA rutting, respectively. It should be noted that at the time of

FIGURE 5 Steel deck ACROW bridge on Route 80 in New Jersey.

this project, the New Jersey DOT was utilizing a flexural beam fatigue strain level of 2,000 microstrains, instead of the current 1,500 microstrains.

After construction, the bridge was opened immediately to traffic. As of April 2011, no visual distress was observed (Figure 7).

FIGURE 6 Patching of rapid deterioration on Route 80 ACROW bridge in New Jersey.

FIGURE 7 Route 80 ACROW Bridge with New Jersey DOT's BDWSC asphalt overlay.

Bottom-Rich Base Course

The main purpose of New Jersey DOT's BRBC is to provide a fatigue-resistant base course mixture that would allow for the design and performance of a perpetual pavement. In the classical perpetual pavement design (Figure 8), a flexible fatigue-resistant base course mixture is constructed at the bottom of the asphalt layer to provide adequate resistance from bottom-up cracking. The aggregate gradation, shown in Table 7, is consistent with New Jersey DOT's 19-mm Superpave specification. However, the target volumetric and design gyration level are modified in order to produce a mixture with a higher asphalt content than normally contained in New Jersey DOT's 19-mm Superpave mixtures (Table 8). The specification recommends an asphalt binder grade of a PG 76-28; although similar to the BDWSC, it is the final mixture performance of the BRBC that dictates its acceptance or not. Other asphalt binder grades are allowed if the required mixture performance criteria are achieved.

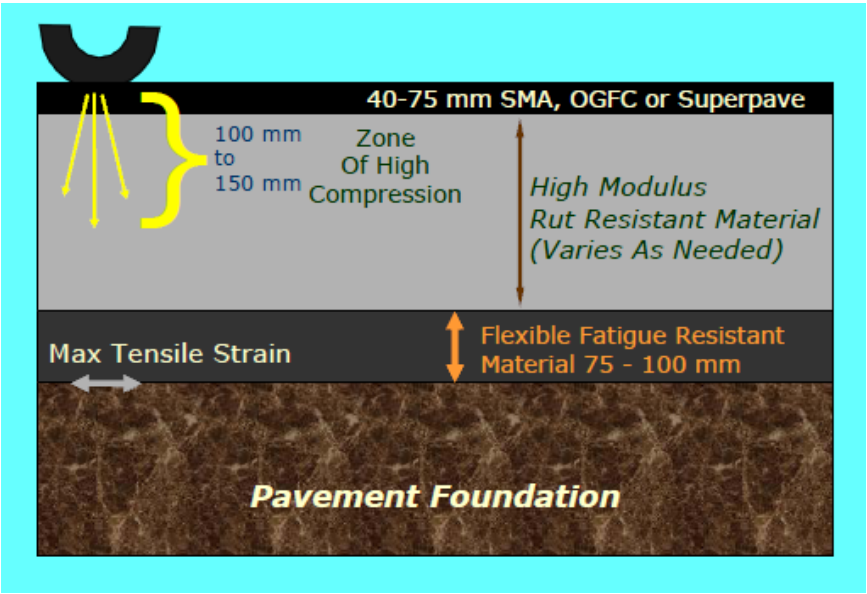

FIGURE 8 General schematic of a perpetual asphalt pavement (*3*).

TABLE 7 Aggregate Blend Gradation of New Jersey DOT's BRBC

Sieve Size	Percent Passing by Mass
1 in.	100
3/4 in.	90–100
1/2 in.	–90
#8	23–49
#200	2.0–8.0
Minimum percent asphalt by mass of total mix	5.0

TABLE 8 Volumetric Requirements for New Jersey DOT's BRBC

	Required Density (% of Gmm)	**Voids in Mineral Aggregate**	**Dust-to-Binder Ratio**	**Draindown AASHTO T 305**
	N_{des} (50 gyr)	VMA		
Design requirements	96.5	≥13.5%	0.6–1.2	≤0.1%
Control requirements	95.5–97.5	≥13.5%	0.6–1.2	≤0.1%

The performance tests and criteria for New Jersey DOT's BRBC are as follows:

- APA (AASHTO T340):
  - 64°C, 100-lb wheel load, 100-psi hose pressure, and
  - Maximum rut depth of 5.0 mm at 8,000 loading cycles;
- Flexural Beam Fatigue (AASHTO T321):
  - 15°C test temperature, 10-Hz frequency, Haversine waveform,
  - Minimum of six test specimens (three tested at 400 microstrains and three tested at 800 microstrains in accordance with NCHRP Project 9-38), and
  - Minimum of 100,000,000 cycles at 100 microstrains as determined using the method developed under NCHRP Project 9-38.

BRBC Implementation: I-295

The New Jersey DOT first implemented the BRBC on I-295 during the summer of 2010. The pavement consisted of a highly deteriorated PCC pavement that was long overdue for reconstruction. To alleviate future issues with reflective cracking in a composite pavement, the New Jersey DOT decided to rubblize the PCC pavement and apply an asphalt overlay. Initial pavement designs using the DARWIN system recommended an asphalt thickness of approximately 12 in. thick. Unfortunately, an HMA layer thickness of 12 in. would require large undercuts areas under the 20 overpasses along I-295 to maintain existing clearance. In addition the full directional closure production schedule required extremely high mix production rates to meet the completion date. The reduction in thickness reduced the amount of HMA required for the project by one-third (170,000 tons) and eliminated 64,000 yd^2 of PCC pavement removal and undercutting. Engineers at Rutgers University and the New Jersey DOT decided to utilize an elastic layer analysis program (JULEA) to evaluate the maximum tensile strains at the bottom of the HMA layer with varying asphalt layer thicknesses. It was determined through the sensitivity analysis that the HMA layer could be reduced to 8 in. while resulting in a maximum tensile strain at the bottom of the HMA layer of 82 microstrains. Therefore, the New Jersey DOT decided upon the final pavement structure:

- Surface course: 2 in. of SMA with PG 76-22 asphalt binder;

- Intermediate course: 3 in. of 19-mm dense-graded HMA (Superpave) with PG 76-22 and 25% RAP; and
- Base course: 3 in. of New Jersey DOT BRBC.

Three HMA suppliers submitted mixture designs for the BRBC with varying success. One supplier was able to achieve the performance requirement with their first design while another supplier had to make three revisions in order to pass the mixture design performance testing phase. In all cases, it was found that the rutting criteria was easy to meet with the flexural fatigue requirement of 100,000,000 cycles at 100 microstrains [as determined using the methodology established by NCHRP Project 9-38 (*4*)] being the harder of the two to pass. An example of the graphical output of the NCHRP 9-38 analysis is shown in Figure 9. The graph shows the test results for production data of the BRBC and also the intermediate course, New Jersey DOT 19M76. The comparison of the results in Figure 9 indicates that the BRBC can achieve the 100,000,000 cycles at strain levels twice the magnitude of New Jersey DOT 19M76. It should be noted that in most pavement structures in New Jersey, the M compaction level (75 gyrations) is commonly used for surface, intermediate, and base course mixtures.

Unlike the other performance-based "Specialty" mixes that the New Jersey DOT uses, the performance testing required for the BRBC takes approximately 1 week to complete, as opposed to 2 days like the other mixes. This is due to the time required to complete the beam fatigue testing. Therefore, for production purposes, it was decided that it would only conduct the flexural beam fatigue tests at the 800 microstrain level, except for Lot #1 where the full set of beam fatigue tests would be conducted. The assumption made was that the general slope of the fatigue life line shown in Figure 9 should not change dramatically due to slight changes with the asphalt mixture, only shift up or down based on the magnitude of the fatigue life measured at 400 and 800 microstrains. Therefore, if it is assumed that the slope will not deviate drastically, it was concluded that as long as the fatigue life at 800 microstrains was equal to or greater than that achieved in Lot #1, the 400 microstrain level would not be required as the final extrapolated

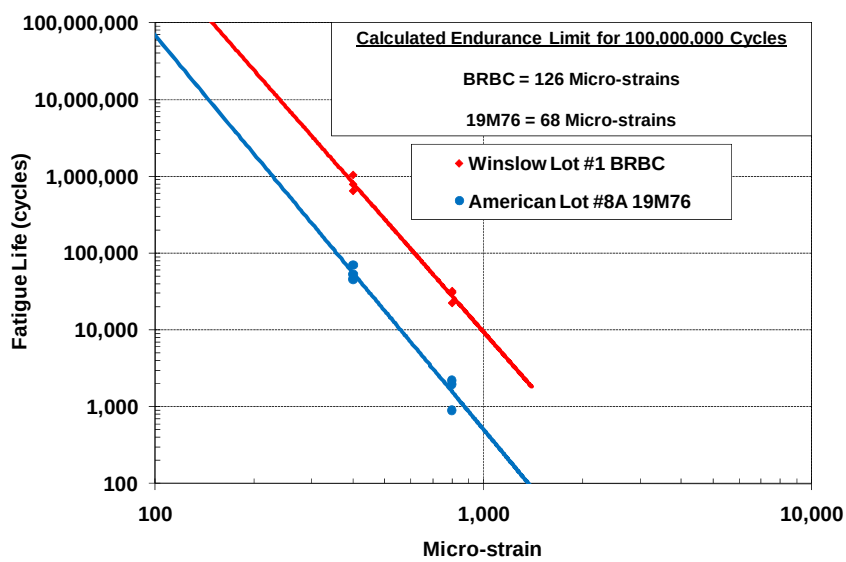

FIGURE 9 NCHRP 9-38 endurance limit graphical output.

endurance limit would always be greater than the Lot #1 material. Only if the fatigue results at 800 microstrains were lower than those determined from Lot #1 would it require that the 400 microstrain testing be necessary.

Figure 10 shows the beam fatigue test results at 800 microstrains for the sampling intervals determined by the New Jersey DOT. The test results indicate that all lots produced after Lot #1 achieved the required level of fatigue performance. The figure also shows the superior fatigue resistance of the BRBC when compared to what is commonly utilized by the New Jersey DOT in their base course applications (19M76). Additionally, it should also be known that while achieving the required level of fatigue performance, the BRBC also maintained the required rutting resistance (Figure 11).

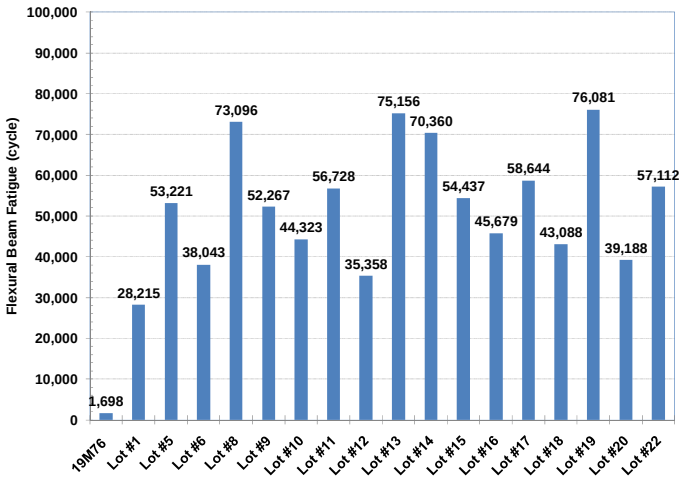

FIGURE 10 Flexural fatigue performance at 800 microstrains for New Jersey DOT's I-295 BRBC mixture.

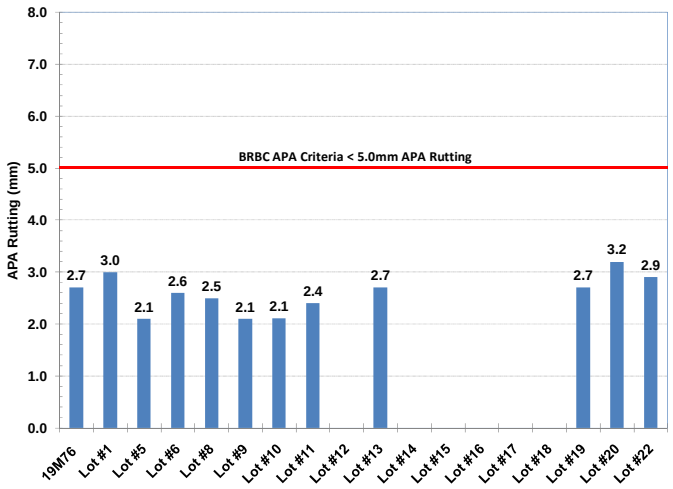

FIGURE 11 APA performance for New Jersey DOT's I-295 BRBC mixture.

SUMMARY AND CONCLUSIONS

With a deteriorating transportation infrastructure, decreasing transportation funding, and an increasing traffic conditions, the New Jersey DOT has begun to implement a performance-based asphalt mixture design system for their specialty mixtures. These specialty mixtures, comprised of approximately 5% of the total asphalt tonnage placed in the state, are selected based on the extreme needs of the pavement structure in question. For example, New Jersey DOT utilizes a BDWSC as a low permeable, rut- and fatigue-resistant overlay on all bridge deck structures.

Each of these specialty mixes is required to undergo performance testing during the mixture design, test strip, and project construction phase to ensure the final mixture achieves the desired performance to the specific pavement structure.

Although New Jersey DOT has only begun to implement the performance-based specialty mixtures since 2008, monitored field performance of these mixtures has indicated that these materials are all performing exceptionally well, and in some cases (i.e., ACROW bridge on I-80), are performing far beyond what conventional New Jersey DOT asphalt mixtures are capable of. While New Jersey's HMA suppliers–contractors were skeptical and somewhat reluctant to begin this new age of performance-based asphalt mixtures, they understand New Jersey's need for these mixtures and have embraced their use. As the specialty mixtures have become more widely accepted and the methodology of design and production becomes more efficient, it is hoped that New Jersey will be able to implement some form of performance-based for all asphalt mixtures.

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