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Relationship Between Laboratory Cracking Tests and Field Performance of Asphalt Mixtures

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Foreword

This E-Circular captures the information exchanged during the 97th Annual Meeting of the Transportation Research Board in a session titled Relationship Between Laboratory Cracking Tests and Field Performance of Asphalt Mixtures. Thomas Bennert of Rutgers University presided over the session, which was sponsored by the Standing Committee on Critical Issues and Emerging Technologies in Asphalt.

The session explored a significant amount of work that has gone into a variety of laboratory asphalt mixture cracking test methods to improve the durability of asphalt mixtures. The IDEAL-CT method, or ideal cracking test from Texas, is a practical method using readily available equipment, was evaluated for its ability to be sensitive to mixture properties that control performance. It was compared to pavement test section performance, along with other laboratory cracking test methods. The semicircular bending and fracture method from Louisiana that measures a strain energy release rate was presented in detail and is used in conjunction with loaded wheel testing to provide mixes that balance two extreme performances: rutting and cracking. Implementation and training activities in the agency was shared. The Illinois Flexibility Index Test, or IFIT, was examined where the development of the analytical underpinnings was described. Validation comparisons between lab tests done on mixes taken from pavement sections and comparative round-robin testing for repeatability and reproducibility was presented. A disc-shaped fracture testing methodology was explored where a rich data set of field performance from Missouri, Illinois, and Minnesota emphasized practical testing thresholds that agencies could use in practice for performance specifications. Finally, a program has been developed in New Jersey that shares some commonalties with the Louisiana method of balancing a cracking test and wheel tracking to optimize mixture proportions. Practical asphalt mix volume proportions that are used currently in construction are incorporated with the new test methods to increase the assurance of performance.

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Development and Validation of the IDEAL Cracking Test

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INTRODUCTION

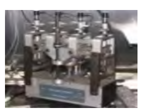
In the 1980s, rutting was a big problem for asphalt pavements. It gradually diminished with the implementation of Superpave specifications, the use of polymer-modified binders, the use of lower asphalt contents, or a combination of all of them. However, these measures resulted in early cracking problems (*1–3*), which has now become the primary mode of distress that results in the need for pavement rehabilitation. The cracking problem may get even worse, because the mixes are designed to lower costs with the increasing use of recycled materials [such as recycled asphalt pavement (RAP) and recycled asphalt shingles (RAS)] and binder additives (such as polyphosphoric acid and recycled engine oil bottom). Thus, there is an urgent need for a cracking test that is not only performance-related but also simple, repeatable, sensitive to asphalt mix composition, and practical enough for routine uses in the process of mix design, quality control (QC), and quality assurance (QA).

Various laboratory cracking tests have been developed in the literature. A critical review on these laboratory cracking tests was conducted under the National Cooperative Highway Research Program (NCHRP) 9-57: *Experimental Design for Field Validation of Laboratory Tests to Assess Cracking Resistance of Asphalt Mixtures* (*4*). Seven cracking tests (see Table 1) were finally selected by the NCHRP 9-57 panel members and invited experts for further laboratory evaluation and field validation, namely bending beam fatigue (BBF) test, Overlay Test (OT), disk-shaped compact tension (DCT) test, indirect tensile creep and strength test (IDT-CST) with full instrumentation, and three versions of Semicircular Bend (SCB) tests. Meanwhile, NCHRP 9-57 project identified seven desirable features for an ideal cracking test, as listed below:

1. Simplicity: no instrumentation, cutting, gluing, drilling, and notching to specimen.
2. Practicality: minimum training needed for routine operation.
3. Efficiency: test completion within 1 min.
4. Test equipment: cost less than $10,000.
5. Repeatability: coefficient of variation (COV) less than 25 percent.
6. Sensitivity: sensitive to asphalt mix composition (aggregates, binder, etc.).
7. Correlation to field performance: a good correlation with field cracking.

As presented in Table 1, the integration of all these seven features into one cracking test has never been done before. The objective of this study was to develop and validate such an ideal cracking test, named indirect tensile asphalt cracking test (IDEAL-CT). The IDEAL-CT is intended for routine uses for mix designs and QC/QA by contractors, departments of transportation (DOTs), and even researchers in academia.

TABLE 1 Seven Existing Cracking Tests Selected by NCHRP 9-57 and IDEAL-CT

Cracking Tests		Test Limitations and Equipment Cost
DCT		• Specimen prep: 3 cuts, 1 notch, and 2 holes. • Instrumentation: glue 2 studs, mount 1 clip gauge. • Equipment cost: $49,000.
SCB-AASHTO TP105		• Specimen prep: 3 cuts and 1 notch. • Instrumentation: glue 3 studs, mount 1 extensometer + 1 clip gauge. • Testing: 30 min. • Equipment cost: $52,000.
SCB-Louisiana Transportation Research Center		• Specimen prep: 9 cuts and 3 notches. • Testing: around 30 min. • Equipment cost: less than $10,000.
SCB-Illinois		• Specimen prep: 3 cuts and 1 notch. • Equipment cost: $10,000–$18,000.
IDT-CST		• Specimen prep: 2 cuts. • Instrumentation: Glue 8 studs, mount 4 extensometers. • Testing: 1–2 h. • Equipment cost: more than $50,000.
OT		• Specimen prep: 4 cuts, glue specimen to bottom plates. • Testing: 30 min–3 h. • Equipment cost: $40–50,000.
BBF		• Specimen prep: large slab, 4 cuts. • Instrumentation: glue 1 stud and mount 1 linear variable differential transformer.. • Specimen testing: 1 h to days. • Equipment cost: more than $100,000.
IDEAL-CT		• No cutting, notching, drilling, gluing, or instrumentation. • Test completion within 1 min. • Repeatable (or low variability) with COV<25%. • Practical for routine uses in DOTs and contractors' laboratories. • Low-cost test equipment (<$10,000). • Sensitive to asphalt mix composition. • Cracking performance-related.

IDEAL-CT: CONCEPT, TEST PROCEDURE, AND CRACKING INDEX

The IDEAL-CT is similar to the traditional indirect tensile strength test, and it is run at the room temperature with cylindrical specimens at a loading rate of 50 mm/min. in terms of cross-head displacement. Any size of cylindrical specimens with various diameters (100 or 150 mm) and thicknesses (38, 50, 62, 75 mm, etc.) can be tested. For mix design and laboratory QC/QA, the authors proposed to use the same specimen size as the Hamburg wheel-tracking test: 150 mm diameter and 62 mm height with 7±0.5% air voids, since agencies are familiar with molding such specimens. Either lab-molded cylindrical specimens or field cores can be directly tested with no need for instrumentation, gluing, cutting, notching, coring or any other preparation.

Figure 1 shows a typical IDEAL-CT: cylindrical specimen, test fixture, test temperature, loading rate, and the measured load versus displacement curve.

DEFINITION OF CRACKING TEST INDEX

After carefully examining the typical load-displacement curve and associated specimen conditions at different stages (Figure 1), the authors chose the post-peak segment to extract cracking resistance property of asphalt mixes. Note that with the initiation and growth of the macro-crack, load-bearing capacity of any asphalt mix will obviously decrease, which is the characteristic of the post-peak segment. Based on Paris' law (*5*) and the work done by Bazant and Prat (*6*), a cracking parameter named CT_{Index} was derived and listed in Equation 1.

$$CT_{Index} = \frac{G_f}{|m_{75}|} \times \left(\frac{l_{75}}{D}\right) \tag{1}$$

where

G_f = the energy required to create a unit surface area of a crack (see Figure 2);
$|m_{75}| = \left|\frac{P_{85}-P_{65}}{l_{85}-l_{65}}\right|$ = the secant slope is defined between the 85% and 65% of the peak load point of the load-displacement curve after the peak; and
l_{75} = deformation tolerance at 75 percent maximum load.

Generally, the larger the G_f, the better the cracking resistance of asphalt mixes. The stiffer the mix, the faster the cracking growth, the faster the load reduction, the higher the $|m_{75}|$ value, and consequently the poorer the cracking resistance. It is obvious that the mix with a larger $\frac{l_{75}}{D}$ and better *strain* tolerance has a higher cracking resistance than the mix with a smaller $\frac{l_{75}}{D}$.

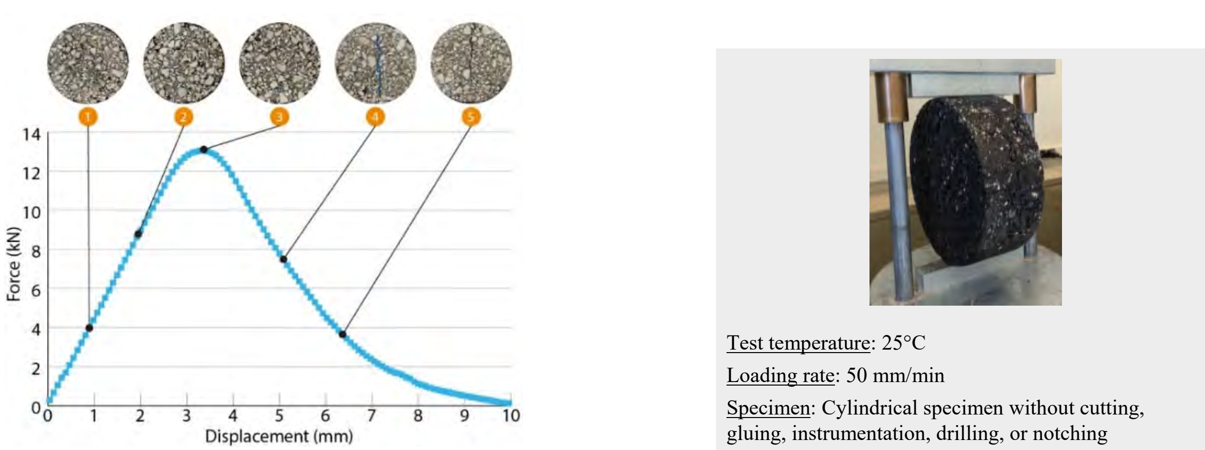


FIGURE 1 IDEAL-CT: specimen, fixture, test conditions, and typical result.

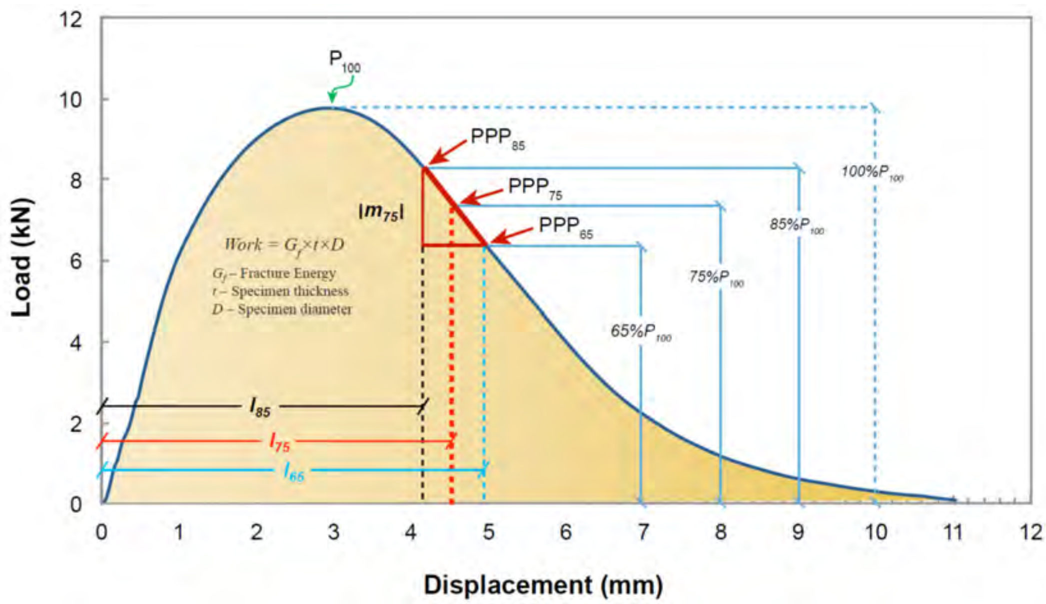


FIGURE 2 Illustration of the PPP75 point and its slope $|m_{75}|$.

As described previously, either lab-molded cylindrical specimens or field cores can be directly tested without cutting, notching, drilling, gluing, and instrumentation. Thus, the IDEAL-CT automatically meets top two features: simplicity and practicality. Furthermore, the IDEAL-CT is run at the loading rate of 50 mm/min., and the test is done within 1 min for one specimen. Thus, the third feature, efficiency, is met. Additionally, the same indirect tensile strength test equipment with a displacement measurement or any other loading frame (such as MTS, Universal testing machine, or Interlaken) can be used for the IDEAL-CT. Most of contractors and DOTs already have such equipment. Even if a new test machine is purchased, its cost is often less than $10,000. Therefore, feature No. 4 is met as well. The key to this entire study from now on is to evaluate and validate the IDEAL-CT sensitivity, repeatability, and correlation to field performance using CT_{Index}, which is discussed in the following sections.

IDEAL-CT SENSITIVITY

For any cracking tests to be used for mix design and QC/QA, it must be sensitive to the characteristics and volumetric properties of asphalt mixtures and aging conditions. A total of six variables were evaluated in this study, and they are RAP and RAS contents, asphalt binder type, binder content, air voids, and aging conditions. A series of laboratory-mixed and laboratory-molded specimens were utilized to evaluate the sensitivities of RAP and RAS contents, binder type, and binder content, which are much easier controlled in the laboratory than the field plant. A plant mix collected from one field test section was used in this study for sensitivities of air voids and aging conditions. Details are described below.

Sensitivity to RAP and RAS

The use of RAP and RAS in asphalt mixes has become a new norm. Any valid cracking test should be sensitive to influence of RAP and RAS on cracking resistance of asphalt mixes. To investigate the sensitivity of the IDEAL-CT to RAP and RAS, this study employed a virgin mix as the control mix. It is a typical 12.5 mm Superpave virgin mix with a PG 64-22 binder and limestone aggregates, and Figure 3 shows the gradation of the control mix. The control mix was designed according to the Texas Department of Transportation's (DOT's) Superpave mix design procedure, and its optimum asphalt content (OAC) was 5.0% at 4% design air voids. Then the control mix was modified to produce another two mixes: one with 20% RAP and the other with 15% RAP and 5% RAS:

1. 20% RAP mix: RAP binder was very stiff (PG103) and its binder content was 5%. It was expected that the 20% RAP mix would have worse cracking resistance than the virgin mix.
2. 15% RAP–5% RAS mix: The same RAP used in the 20% RAP mix was used here as well. The RAS was manufacturer waste shingles with extremely stiff binder (PG141) and its binder content was 20%. Again, it was expected that the 15% RAP/5% RAS mix would have the worst cracking resistance among the three mixes.

Note that neither the PG 64-22 binder nor the total binder content (5%) was changed for either modification. For the control mix, the 5% asphalt was 100% virgin binder; as is normal DOT policy for the modified mixes, some of the virgin binder was replaced with the binder from the RAP/RAS. Meanwhile, the aggregate gradations for all three mixes were kept as close as possible (see Figure 3).

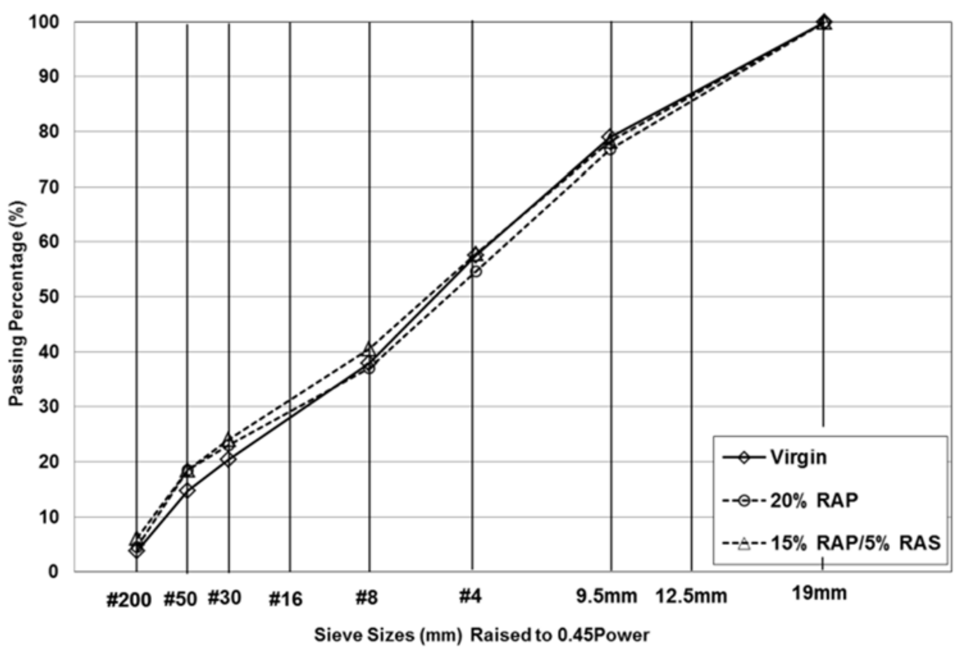


FIGURE 3 Aggregate gradations used for sensitivity analysis.

For each mix, three replicates of 150-mm diameter and 62-mm height specimens with 7±0.5% air voids were compacted using the Superpave Gyratory Compactor (SGC). Before the compaction, the loose mixes were conditioned in the oven for 4 h at 135°C. The IDEAL-CT was then run at a room temperature of 25°C and a loading rate of 50 mm/min. Figure 4 presents the IDEAL-CT results: CT_{Index} value for each mix. Note that CT_{Index} can vary from 1 to 1000, and a higher number indicates better crack resistance.

The CT_{Index} values in Figure 4 clearly show that the IDEAL-CT is sensitive to RAP and RAS. The additions of RAP and RAS significantly reduce cracking resistance of the asphalt mix. Thus, the IDEAL-CT is sensitive to the addition of RAP and RAS to asphalt mixes.

Sensitivity to Asphalt Binder Type

The 20% RAP mix with PG 64-22 binder was further modified with another two virgin binders, PG 64-28 and PG 64-34, to check the sensitivity of the IDEAL-CT to binder type. Among these three mixes, all variables (including virgin aggregates and gradation, RAP, and the total binder amount) were kept the same except the virgin binder type. Note that both PG 64-28 and PG 64-34 binders were SBS polymer-modified binders. Past experience indicated that the PG 64-34 binder generally had better cracking resistance than PG 64-28 binder, and PG 64-22 had the worst among the three (*7*). Thus, similar results were anticipated from the IDEAL-CT.

For each binder type, three replicates of 150 mm diameter and 62 mm height specimens with 7±0.5% air voids were compacted using SGC. Before the compaction, the loose mixes were conditioned in the oven for 4 h at 135°C. The IDEAL-CT was run at a room temperature of 25°C and a loading rate of 50 mm/min. Figure 5 presents the IDEAL-CT results: CT_{Index} value for each binder type. Obviously, the IDEAL-CT is sensitive to binder type. As expected, the 20% mix with PG 64-34 binder has the largest CT_{Index} value, followed by the one with PG 64-28 and then the one with PG 64-22. Thus, the IDEAL-CT is sensitive to asphalt binder type.

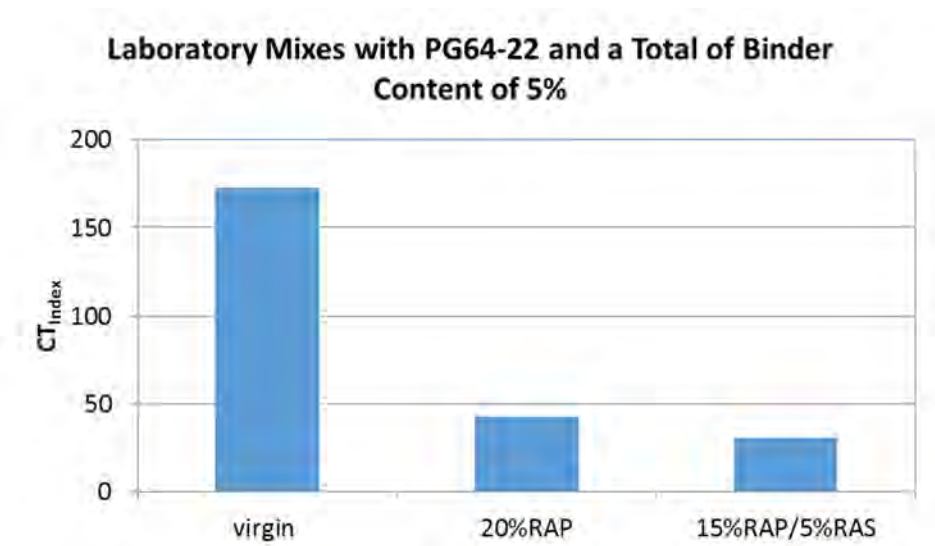


FIGURE 4 IDEAL-CT sensitivity to RAP and RAS.

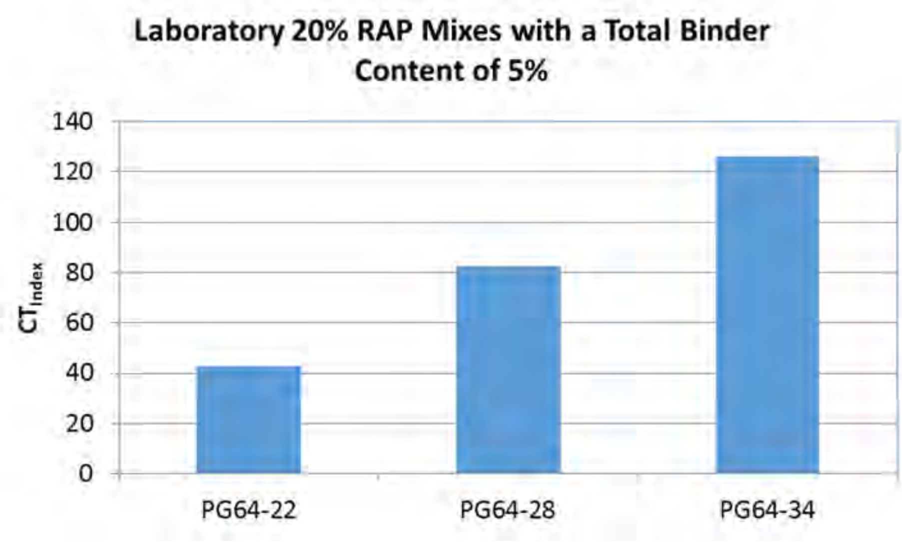


FIGURE 5 IDEAL-CT sensitivity to binder type.

Sensitivity to Asphalt Binder Content

Asphalt binder content is one of the key parameters for asphalt mix designs and has significant influence on asphalt mix cracking performance. Generally, the higher the binder content, the better the cracking performance in the field. To evaluate the sensitivity of the IDEAL-CT to the binder content, the control mix was modified through varying asphalt content only, ±0.5%. It was expected that this mix with +0.5% asphalt binder would have the largest CT_{Index} value, followed by the control mix, and then the one with –0.5% having the least CT_{Index} value.

For each binder content, three replicates of 150 mm diameter and 62 mm height specimens with 7±0.5% air voids were compacted using SGC. Before the compaction, the loose mixes were conditioned in the oven for 4 h at 135°C. The IDEAL-CT was run at a room temperature of 25°C and a loading rate of 50 mm/min. Figure 6 presents the IDEAL-CT results. As expected, the higher the binder content, the larger CT_{Index} value. Thus, the IDEAL-CT is sensitive to binder content.

Sensitivity to Aging Conditions

Aging makes the mixes brittle and less cracking resistant. To be a valid cracking test, the IDEAL-CT must be sensitive to aging conditions of asphalt mixes. In this study, three levels of oven conditioning at 135°C (4, 12, and 24 h before the compaction) were investigated with a plant mix collected from one field test section in Laredo, Texas. The plant mix was a 12.5 mm Superpave virgin mix with an asphalt binder content of 6.3%. For each level of aging condition, three replicates of 150 mm diameter and 62 mm height specimens with 7±0.5% air voids were compacted using SGC. The IDEAL-CT was run at a room temperature of 25°C and a loading rate of 50 mm/min. Figure 7 presents the IDEAL-CT results.

As expected, the longer the aging time in the oven, the poorer the cracking resistance. Thus, the IDEAL-CT is sensitive to aging conditions.

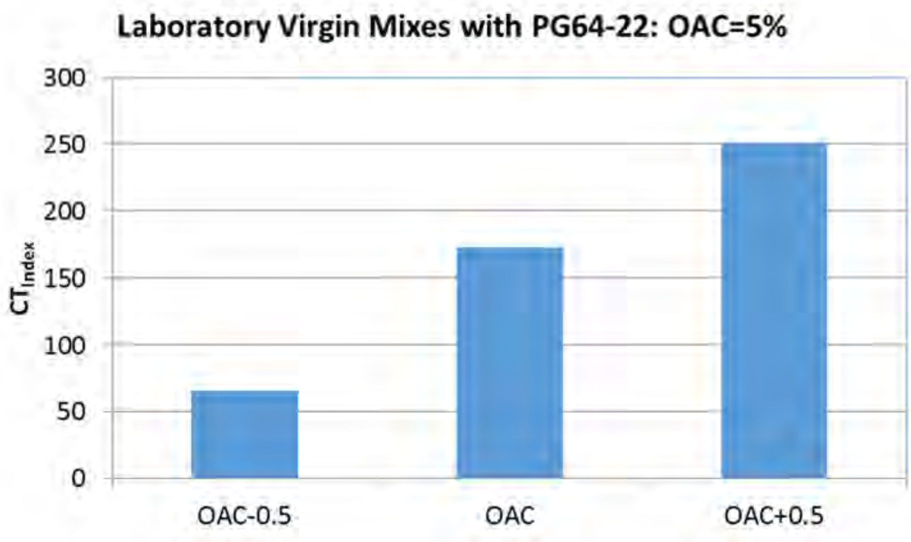


FIGURE 6 IDEAL-CT sensitivity to binder content.

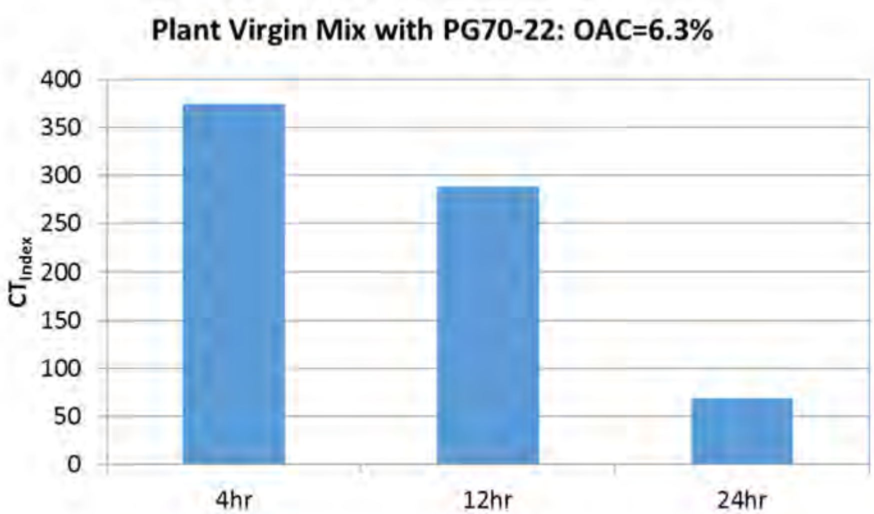


FIGURE 7 IDEAL-CT sensitivity to aging conditions.

Sensitivity to Air Voids

Air voids (or density) is another key volumetric property of asphalt mixes and plays critical roles in QC/QA. In this study, three levels of air voids (5, 7, and 9 percent) were investigated with the same plant mix used for evaluating the sensitivity to the aging conditions. For each level of air voids, three replicates of 150 mm diameter and 62 mm height specimens were compacted using SGC. Before the compaction, the plant mix was conditioned in the oven for 4 hours at 135°C. Similarly, the IDEAL-CT was conducted, and Figure 8 presents the test results.

Figure 8 clearly indicates that the IDEAL-CT is sensitive to air voids of asphalt mixes. The higher the air voids, the better the cracking resistance. It is worth noting that similar

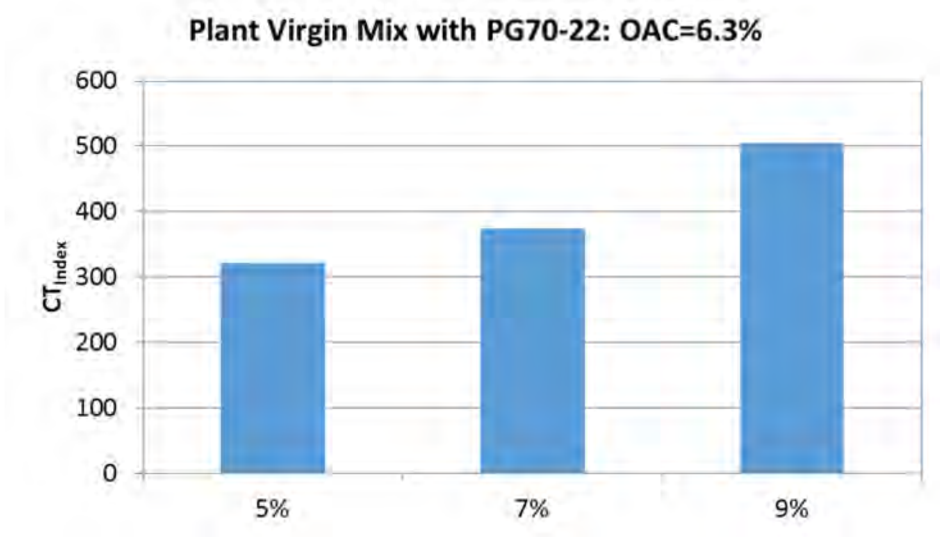


FIGURE 8 IDEAL-CT sensitivity to air voids.

findings have been reported by Barry (*8*) with the Illinois flexibility index test (I-FIT) and Zeiada et al. (*9*) with Simplified Viscoelastic Continuum Damage (S-VECD) fatigue test, although it is counterintuitive to what has been observed in the field. Thus, for the purpose of comparison among different asphalt mixes, all the specimens should be compacted to the same level of air voids (e.g., 7±0.5 percent). Also, a correction factor for air voids may be needed.

In summary, the IDEAL-CT results shown in Figure 4 through Figure 8 clearly indicate that the IDEAL-CT is sensitive to key asphalt mix components and volumetric properties: RAP and RAS, asphalt binder type, binder content, aging conditions, and air voids.

IDEAL-CT REPEATABILITY

The repeatability (or variability) of the IDEAL-CT is critical for being adopted by DOTs and contractors, because if the test has a high variability, not only more specimens will be needed, but it may also have difficulty in differentiating the poor from the good performers. There are different ways to evaluate repeatability (or variability) of a laboratory test. This report simply uses COV as an indicator for the repeatability. A smaller COV means the test is more repeatable.

Instead of testing new mixes, the authors simply analyzed the COVs of the IDEAL-CT results of the previous sensitivity study. Table 2 shows the average CT_{Index} value and associated COV for each mix. From Table 2, it can be seen that the maximum COV is 23.5% and most of them are less than 20%, which is much less than those of repeated load cracking tests including BBF test (*10*) and OT (*11, 12*). Additionally, the COVs of the IDEAL-CT are similar to or even better in some cases than those of the I-FIT SCB test (*13*).

TABLE 2 IDEAL-CT Repeatability

Asphalt Mixes			CT_{Index}	COV (%)
Laboratory mix	Sensitivity to RAP and RAS	Virgin	172.9	5.5
		20% RAP	42.8	23.5
		15%RAP/5%RAS	30.8	9.0
	Sensitivity to binder type	PG 64-22	42.8	23.5
		PG 64-28	82.4	13.8
		PG 64-34	126.2	1.8
	Sensitivity to binder content	OMC-0.5	66.0	1.7
		OMC	172.9	5.5
		OMC+0.5	251.0	20.5
Plant mix	Sensitivity to aging conditions	4 h	374.5	12.1
		12 h	287.6	20.0
		24 h	68.9	15.1

NOTE: OMC = optimum moisture content.

IDEAL-CT CORRELATION WITH OTHER CRACKING TESTS

As mentioned earlier, there are many cracking test methods in the literature. Among the various options, the Texas OT and Illinois flexibility index test (SCB test) were selected in this study to compare with the IDEAL-CT. Brief description on each test method is described as follows.

Texas OT

The Texas OT is used to represent the reflective cracking potential of the asphalt mixes. Detailed test procedure is described in Tex-248-F, *Test Procedure for Overlay Test*. The OT testing specimen is placed inside the environmental chamber of a mechanical testing machine for temperature equilibrium targeting the testing temperature of 25°C. The sliding block applies tension in a cyclic triangular waveform to a constant maximum displacement of 0.63 mm (0.025 in.). The sliding block reaches the maximum displacement and then returns to its initial position in 10 s. The time, displacement, and load corresponding to a certain number of loading cycles are recorded during the test.

Illinois Flexibility Index Test (I-FIT)

The I-FIT has been recently developed to quantify cracking potential of asphalt mixtures (*13*). This test suggested a testing temperature of 25°C with a loading rate of 50 mm/min. The I-FIT uses the so-called flexibility index (FI), as defined in Equation 2, to characterize cracking resistance of asphalt mixes. Typically, the FI values vary from 1 to 30 for the poorest to best-performing asphalt mixes.

$$FI = \frac{G_f}{|m|} \times A \tag{2}$$

where

G_f = fracture energy (J/m^2);
$|m|$ = absolute value of post-peak load slope (kN/mm); and
A = unit conversion and scaling factor equal to 0.01.

Materials, Asphalt Mixes, and Specimen Preparation

Local limestone aggregates, RAP, and RAS were collected from an actual field project in Texas to produce asphalt mixes for this correlation evaluation. The RAP binder content was 5% and its PG high-temperature grade was PG103. While the RAS binder content was 20% and its PG high-temperature grade was PG134. With these materials, four different dense-graded gradations for asphalt mixes were designed as shown in Figure 9.

The virgin mix with a PG 64-22 binder was first designed as the control mix in the laboratory following Texas DOT's Superpave mix design procedure. Its OAC was 5% corresponding to the target air voids of 4%. Then, this control mix was modified to produce its counterparts of four different mixes. Brief information on each mix is described as follows:

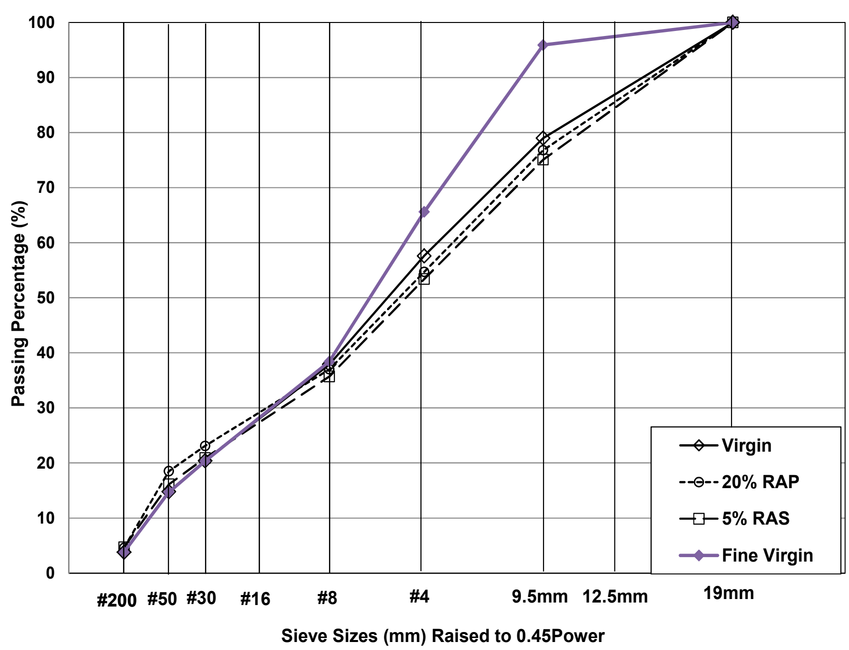


FIGURE 9 Aggregate gradations for asphalt mixes.

- Mix-1 (control mix): virgin mix with a PG 64-22 binder at OAC (5.0%).
- Mix-2: 20% RAP mix with the PG 64-22 binder at the total asphalt content of 5.0%.
- Mix-3: 15% RAP/5% RAS mix with the PG 64-22 binder at the total asphalt content of 5.0%.
- Mix-4: 20% RAP mix with a PG 64-28 binder. This mix is exactly the same as Mix-2 except the binder type.
- Mix-5: 20% RAP mix with a PG 64-34 binder. This mix is exactly the same as Mix-2 except the binder type.

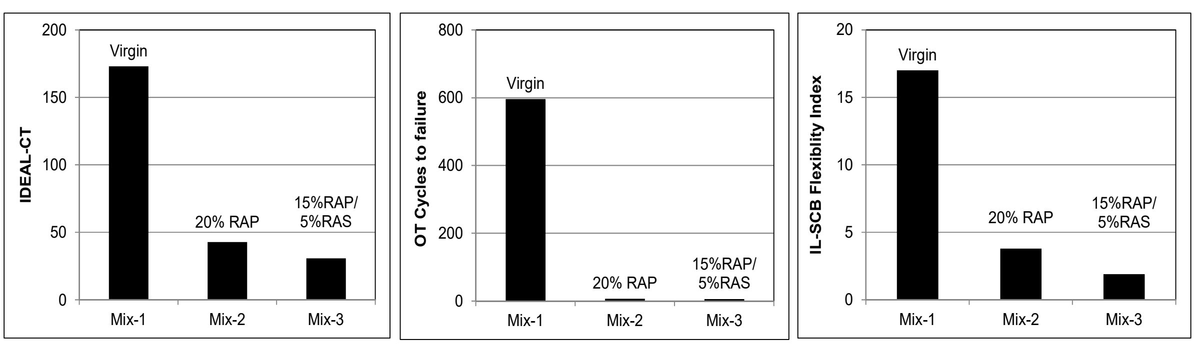
In addition to these above five mixes, five additional virgin mix samples were produced for further evaluation. The fine virgin mix with a PG 64-22 binder was designed following Texas DOT's Superpave mix design, and its OAC was 5.3% at the target air voids of 4%. Brief information on these five virgin mixes is described as follows:

- Mix-6: fine virgin mix with a PG 64-22 binder at OAC (5.3%).
- Mix-7: fine virgin mix with a PG 64-28 binder at OAC (5.3%).
- Mix-8: fine virgin mix with a PG 64-34 binder at OAC (5.3%).
- Mix-9: fine virgin mix with a PG 70-22 binder at OAC (5.3%).
- Mix-10: fine virgin mix with a PG 76-22 binder at OAC (5.3%).

For each mix, three IDEAL-CT, five OT, six I-FIT specimens were molded at 7±0.5% air voids after 4 h aging in the oven at 135°C. Then, all testing specimens were tested at 25°C.

Test Results and Discussion

Figure 10, Figure 11, and Figure 12 show the test results of the IDEAL-CT, OT, and I-FIT on different mixes. It can be seen that all cracking test methods indicate the overall same trend for all these mixes. Thus, the IDEAL-CT has a good correlation with the other two cracking tests.



(a) **(b)** **(c)**

FIGURE 10 RAP and RAS sensitivity identified by different cracking methods: (*a*) IDEAL-CT test, (*b*) OT test, and (*c*) I-FIT test.

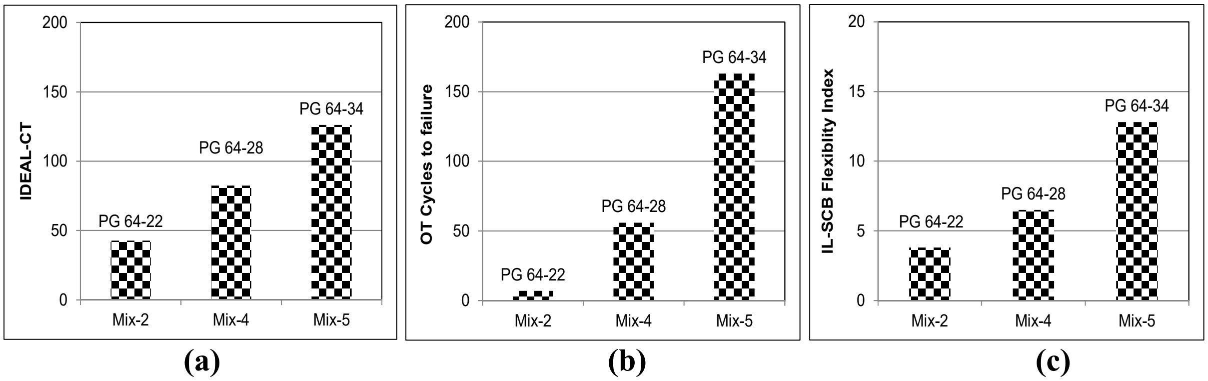


FIGURE 11 Binder type sensitivity identified by different cracking methods: (*a*) IDEAL-CT test, (*b*) OT test, and (*c*) I-FIT test.

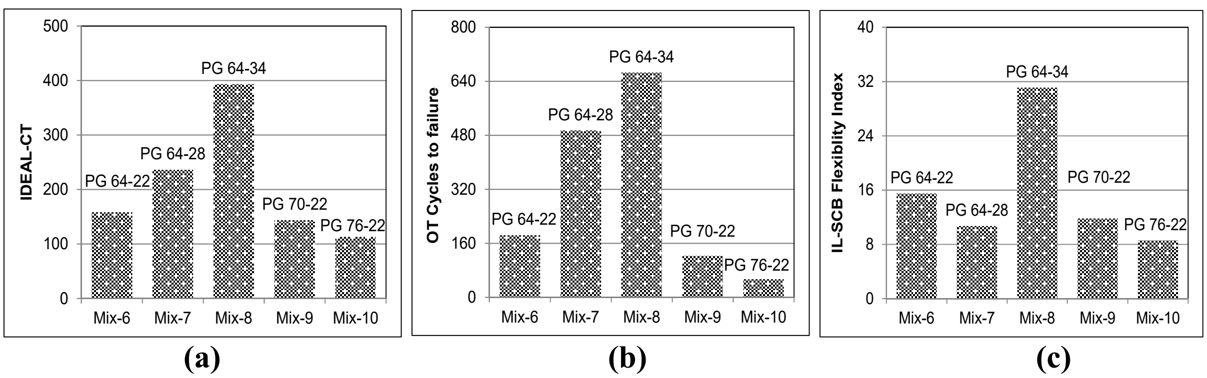


FIGURE 12 Binder type sensitivity identified by different cracking methods: (*a*) IDEAL-CT test, (*b*) OT test, and (*c*) I-FIT test.

IDEAL-CT CORRELATION WITH FIELD PERFORMANCE

This section focused on the IDEAL-CT correlation with field performance. For any test to be used for mix design, it must have good correlation with field performance. Field validation is a crucial step in the process of developing the IDEAL-CT. This study used the accelerated pavement testing data from the Federal Highway Administration's (FHWA's) accelerated loading facility (ALF), full-scale test road in Minnesota (MnROAD), and in-service roads in Texas to evaluate the correlation between the IDEAL-CT test and field performance.

FHWA ALF Test Sections: IDEAL-CT Versus Fatigue Cracking

In 2013, 10 test lanes were constructed at the FHWA ALF in McLean, Virginia, to evaluate fatigue performance of RAP and RAS mixes. The overall pavement structure is composed of 100-mm (4 in.) asphalt layer, 650-mm (26 in.) granular base, and subgrade. Both the base layer and subgrade are the same for all lanes (*14*). The only difference among the 10 lanes is the surface asphalt mix type, as shown in Table 3. All these mixes were 12.5-mm Superpave mixes with N_{design} = 65. The ALF testing was performed in the cooler seasons, and the testing

TABLE 3 FHWA ALF Experimental Design

ALF Lane	% Recycled Binder Ratio		Virgin Binder	Hot/Warm Mix	No. of ALF Passes for First Crack Observed	IDEAL-CT	
	RAP	RAS				CT_{Index}	COV (%)
1	0	—	PG 64-22	Hot mix	368,254	137.2	10.7
2	40	—	PG 58-28	Warm mix with water foaming	No result yet	123.5	23.2
3	—	20	PG 64-22	Hot mix	42,399	45.2	7.9
4	20	—	PG 64-22	Warm mix with chemical additive	88,740	115.5	5.6
5	40	—	PG 64-22	Hot mix	36,946	37.5	21.6
6	20	—	PG 64-22	Hot mix	125,000	93.9	19.2
7	—	20	PG 58-28	Hot mix	23,005	38.0	19.6
8	40	—	PG 58-28	Hot mix	No result yet	160.0	19.9
9	20	—	PG 64-22	Warm mix with water foaming	270,058	136.0	12.5
11	40	—	PG 58-28	Warm mix with chemical additive	81,044	69.5	23.9

temperature of 20°C at a depth of 20 mm beneath the surface was controlled through radiant heaters when needed. All lanes were loaded with a 425 super-single tire wheel (14,200 lb load and 100 psi pressure) at a speed of 11 mph with a normal distributed wander in lateral direction (*14*). Table 3 presents the number of ALF passes corresponding to the first crack observed.

One 5-gal bucket of plant mix from each test lane was obtained for the IDEAL-CT. For each plant mix, three replicates of 150-mm diameter and 62-mm height specimens with 7±0.5% air voids were molded. Before the molding, each plant mix was conditioned in the oven for 4 h at 135°C. The IDEAL-CT was performed at a room temperature of 25°C with a loading rate of 50 mm/min. The average CT_{Index} and COV for each plant mix are tabulated in Table 3 as well.

Figure 13 shows the correlation between the CT_{Index} values and the ALF passes to first crack occurrence. CT_{Index} correlates very well with field cracking observation. The higher the CT_{Index} value, the better the cracking performance in the field.

Texas Field Test Sections on SH15: IDEAL-CT Versus Fatigue Cracking

Different from the well-controlled FHWA ALF testing (fixed temperature and traffic loading), in-service pavements experience real world traffic and daily changing weather. This study used two more field test sections in Texas to validate the IDEAL-CT for fatigue cracking. A series of field test sections were constructed back to back on SH15 close to Perryton, Texas, in October 2013. The original objective of these field test sections was to investigate the approaches for improving cracking resistance of asphalt mixes with RAP. It was a milling and inlay job. A total of 62.5 mm (2.5 in.) asphalt layer was milled, and then was filled with 25.0 mm (1 in.) dense-graded Type F mix and 38 mm (1.5 in.) Type D surface mix. The Type F mix was used for the whole project. The focus of test sections was on the Type D surface mixes. Two of these test sections were selected for validating the IDEAL-CT:

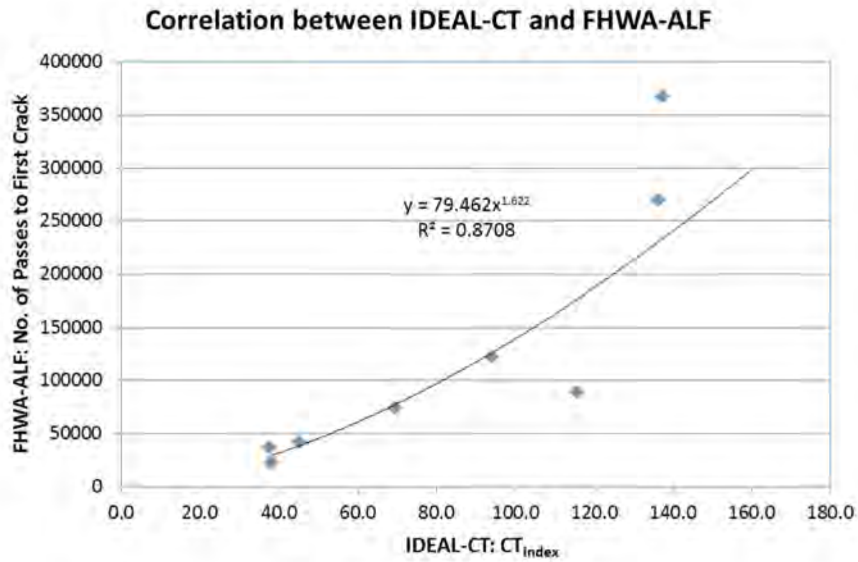


FIGURE 13 Correlation between IDEAL-CT and FHWA ALF full-scale testing.

- Section 1: a dense-graded Type D mix with a PG 58-28 virgin binder, 20% RAP, and the total asphalt binder content of 5.5%.
- Section 2: the same mix as Section 1 but a total asphalt binder content of 5.8%.

The only difference between these two test sections is the total asphalt binder content: 5.5 vs. 5.8 percent. Six field surveys have been conducted since traffic opening. No rutting was observed on either test section. No any cracking was observed on Section 1 until the last survey on March 3, 2016. As shown in Figure 14, significant low severity of fatigue cracking was observed on March 3, 2016. Section 2 with higher binder content still performed very well and no any cracking was observed, which was expected, since Section 2 has higher binder content.

Plant mixes were collected during the construction. For each plant mix, three replicates of 150-mm diameter and 62-mm height specimens with 7±0.5% air voids were molded. Before the molding, each plant mix was conditioned in the oven for 4 h at 135°C. The IDEAL-CT was performed at a room temperature of 25°C with a loading rate of 50 mm/min. Figure 15 presents the average CT_{Index} values of the two plants mixes. Comparing the data in Figure 14 and Figure 15, the CT_{Index} values match exactly what was observed in the field. The higher CT_{Index} values, the less fatigue cracking in the field.

Texas Field Test Sections on US-62: IDEAL-CT Versus Reflective Cracking

Reflective cracking is another major pavement distress, especially for asphalt overlays. Two 1,500-ft long field test sections were constructed on eastbound US-62 close to Childress, Texas, on October 3, 2013. The original purpose was to evaluate the impact of RAP–RAS on pavement performance. The existing pavement had multiple overlays and severe transverse cracking before the milling and inlay. The mill–fill pavement design called for milling the top 200 mm (8 in.)

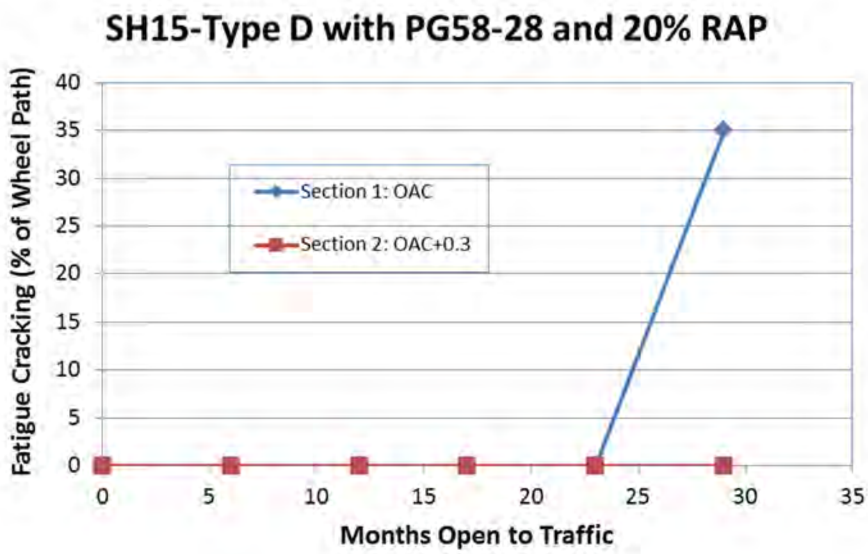


FIGURE 14 Fatigue cracking development observed on SH-15, Texas.

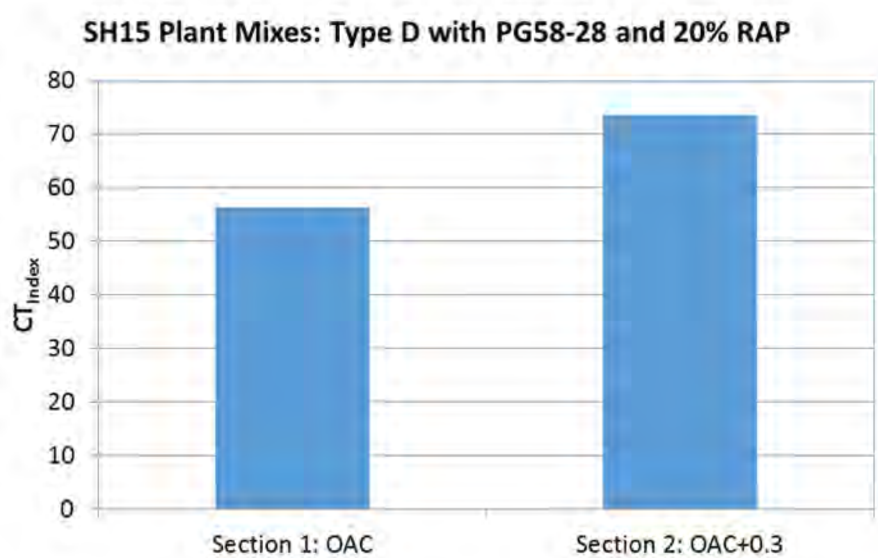


FIGURE 15 IDEAL-CT results of SH-15 plant mixes.

asphalt layer and then refilling with a 75 mm (3 in.) dense-graded Type B mix and 50 mm (2 in.) dense-graded Type D surface mix. The two test sections had the same Type B mix as the base course but the Type D surface course varied as follows:

- Virgin section: Type D virgin mix with PG 70-28 binder.
- RAP–RAS section: Type D with PG 70-28 binder and 5% RAP and 5% RAS.

The asphalt binder content of the virgin mix was 5.4%, and the total asphalt binder content of the RAP–RAS mix was 5.7% and recycled binder replacement was 23.6% from RAP and RAS. Performance survey results are shown in Figure 16. As seen in Figure 16, the virgin section performed much better.

Similarly, each plant mix collected during construction was compacted to obtain three replicates of 150-mm diameter and 62-mm height specimens with 7±0.5% air voids. Again, each plant mix was conditioned in the oven for 4 h at 135°C before molding the specimens. The IDEAL-CT was performed at a room temperature of 25°C with a loading rate of 50 mm/min. Figure 17 presents the average CT_{Index} values of the two plant mixes. Comparing the data in Figure 16 and Figure 17, clearly the IDEAL-CT has very good correlation with field reflective cracking observed on US-62. The higher CT_{Index} value means less reflective cracking in the field.

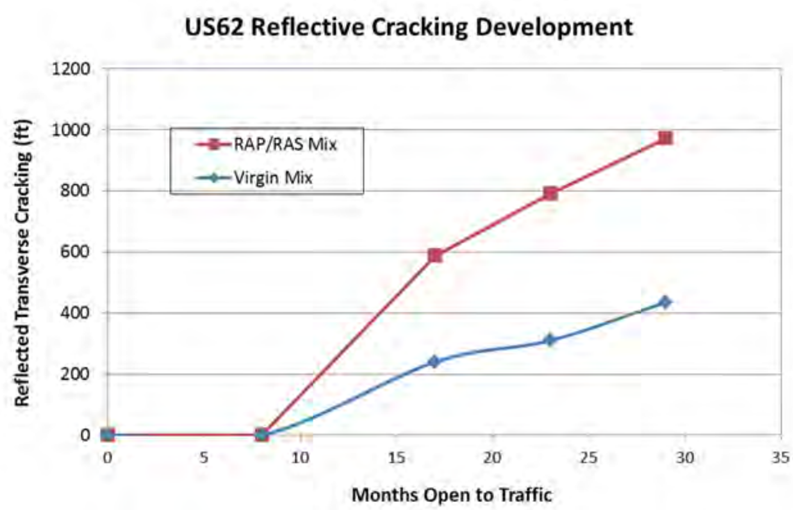


FIGURE 16 Cracking development observed on US-62.

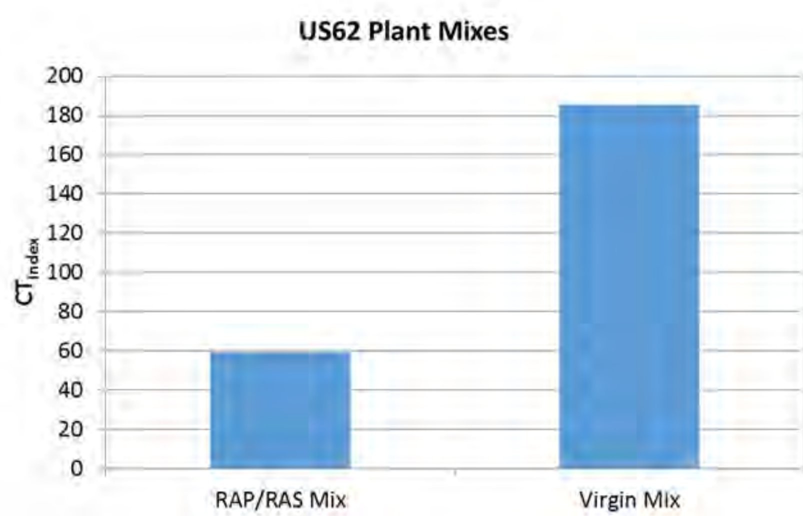


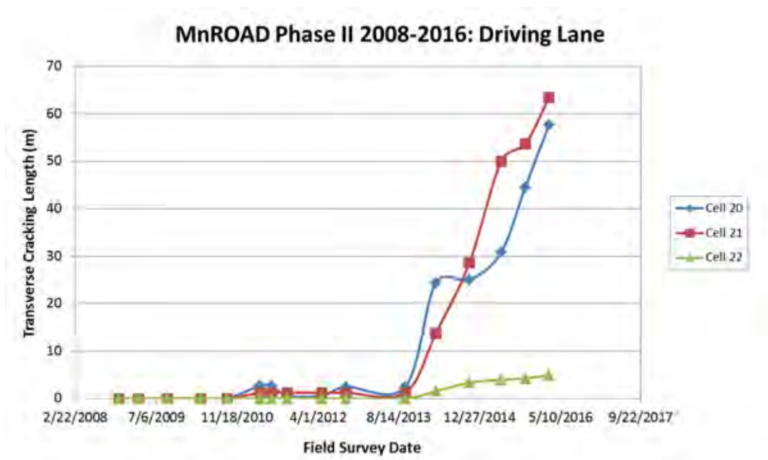
FIGURE 17 IDEAL-CT results of US-62 mixes.

MnROAD Test Sections: IDEAL-CT Versus Thermal Cracking

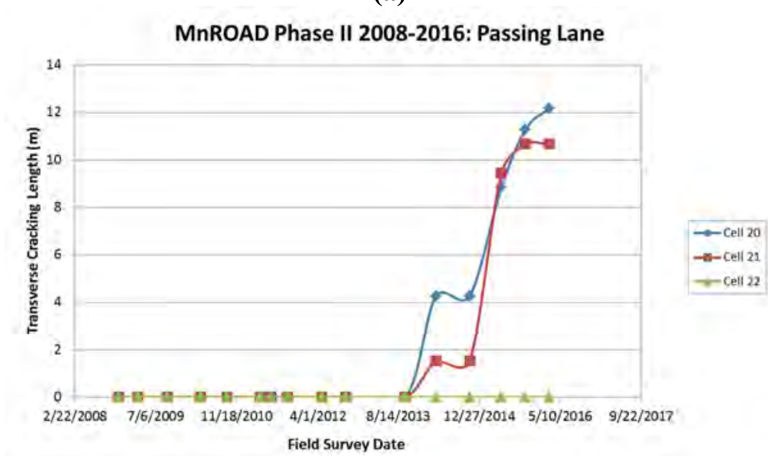
Various test sections (or cells) were constructed at MnROAD Phase II in 2008 (*15*). Three of them (Cells 20, 21, and 22) were designed for evaluating thermal cracking, which is the most common distress in cold climates. These three cells had the same pavement structure thickness, base materials, and subgrade, but the asphalt wearing course varied among the three cells, as listed below:

1. Cell 20: PG 58-28 virgin binder and 30% non-fractionated RAP.
2. Cell 21: PG 58-28 virgin binder and 30% fractionated RAP split on the ¼-in. screen.
3. Cell 22: PG 58-34 virgin binder and 30% fractionated RAP split on the ¼-in. screen.

MnROAD crews have been monitoring the three cells since the completion of construction in 2008. Figure 18 shows thermal (transverse) cracking development history for each cell in both driving and passing lanes. Three observations can be made from Figure 18:



(a)



(b)

FIGURE 18 Thermal cracking development history for Cells 20, 21, and 22: (*a*) driving lane and (*b*) passing lane.

1. Cell 22 performed much better than Cells 20 and 21, which is expected due to softer virgin binder (PG 58-34) in Cell 22.
2. Traffic loading had significant impact on thermal cracking development since the driving lane had more transverse cracking.
3. When reviewing the measured transverse cracking development on the passing lane, it seems that Cell 21 performed a little bit better than Cell 20, although there is no big difference.

Recently, the authors obtained the plant mixes of Cells 20, 21, and 22 collected during the construction. These mixes were tested under the IDEAL-CT. Figure 19 presents the average CT_{Index} values of the three plants mixes.

Figure 19 indicates that Cell 22 having the highest CT_{Index} value should perform the best, followed by Cells 21 and 20. Overall, the IDEAL-CT results match what has been observed in the field. More test sections are being constructed in the 2016 MnROAD, and the performance data will be used for further validating the IDEAL-CT.

In summary, various field test sections including FHWA's ALF, MnROAD, and Texas in-service roads were used to validate the IDEAL-CT. All test results indicate that the IDEAL-CT has good correlations with field fatigue cracking, reflective cracking, and thermal cracking.

SUMMARY AND CONCLUSIONS

Based on the work presented in the Stage I report, the following conclusions and recommendations are made:

- The IDEAL-CT is a simple (no instrumentation, cutting, gluing, drilling, and notching to specimen), practical (minimum training needed for routine operation), and efficient (test completion within 1 min) cracking test that can be performed with regular indirect tensile strength test equipment.

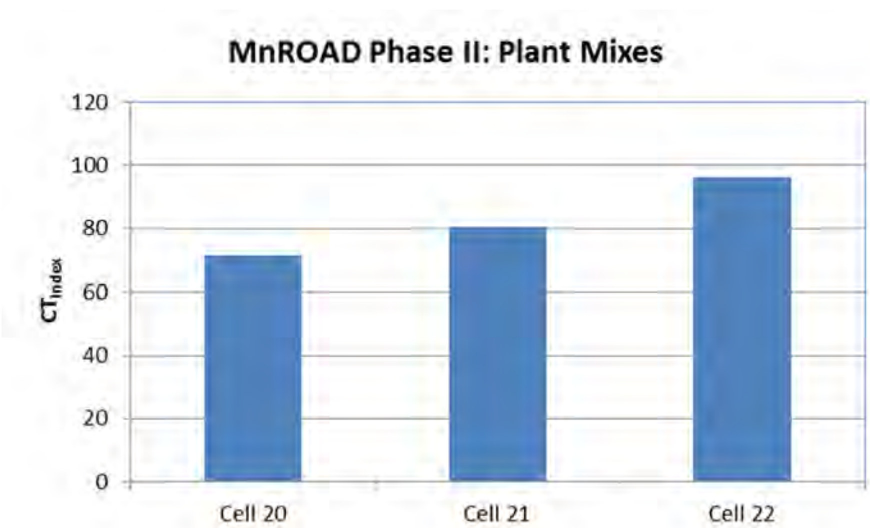


FIGURE 19 IDEAL-CT results of MnROAD Cells 20, 21, and 22.

- The IDEAL-CT is sensitive to key asphalt mix components and aging (RAP and RAS content, asphalt binder type, binder content, air voids, and aging conditions), and it also has much lower COV than traditional repeated load cracking tests. Most the IDEAL-CT results have COV less than 20%.
- The IDEAL-CT correlated well with two other cracking tests—Texas OT and Illinois I-FIT. All three tests had exactly the same rankings for 10 asphalt mixes in terms of cracking resistance.
- The IDEAL-CT correlated well with field performance in terms of fatigue, reflective, and thermal cracking.

Currently, several DOTs including Texas, New Jersey, Virginia, Minnesota, Washington, Oklahoma, and others, are either evaluating or considering the IDEAL-CT for potential adoption.

ACKNOWLEDGMENTS

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DISCLAIMER

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

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Implementation of Balanced Mixture Criteria During Asphalt Mixture Design

Louisiana's Experience

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INTRODUCTION

In 2016, the Louisiana Department of Transportation and Development (LADOTD) implemented laboratory performance testing for the approval of asphalt mixture designs. The background, methodology, results, and discussion related to the research that led to the implementation of LADOTD balanced mixture design was discussed in detail in *Transportation Research E-Circular E-C237: Application of Performance Tests During Asphalt Mixture Design*. (Cooper and Mohammad 2018). The intent of this report is to provide experience and observations made regarding the implementation of the balanced mixture design in the 2016 LADOTD specifications.

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

In 2016, the LADOTD specifications were modified to increase the effective asphalt content of the asphalt mixtures in an attempt to improve durability. The specifications were modified by reducing the number of gyrations at N_{design}, as well as increasing the minimum voids in the mineral aggregate (VMA) and voids filled with asphalt (VFA) requirements. Table 1 presents the 2016 LADOTD asphalt mixture specifications that were modified. This paper documents Louisiana's experience with the development of a balanced mixture design by complementing volumetric criteria with the Hamburg loaded-wheel tester (HLWT) and SCB tests for high- and intermediate temperature performance, respectively.

Cooper et al. (Cooper et al. 2014) conducted the preliminary research evaluating the impacts of specification modification for an improved balanced mixture design for LADOTD.A laboratory evaluation using pilot specifications for LADOTD to determine whether the mixtures designed would be balanced was conducted. The laboratory performance of 51 mixtures was evaluated using the HLWT and SCB test. Both laboratory and plant-produced mixtures were evaluated. Of the 51 mixtures, 11 projects were selected to utilize mixtures designed to meet the criteria of Louisiana Balanced Mixture Design methodologies as per 2016 LADOTD balanced mixture specifications. The remaining 40 mixtures were designed using conventional volumetric

TABLE 1 LADOTD Volumetric Specifications

Property	2016 LADOTD Specifications
N_{design}, Gyrations	65 – 75
Minimum VMA, %	10.5 – 13.0
VFA, %	69 – 80
Air Voids, %	2.5 – 4.5
HLWT Required	Yes
SCB Required	Yes

mixture design methodologies as per 2006 LADOTD specifications (LADOTD 2006). The research showed that the adjustments to the volumetric requirements resulted in an increase of balanced mixtures, when compared to previous specification criteria.

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

OBJECTIVES AND SCOPE

The objective of this report was to present the experience and results of LADOTD's implementation of laboratory performance testing into the asphalt mixture design specifications.

BACKGROUND

Selection of Mechanical Tests

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

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

Rutting Resistance

Numerous state transportation agencies use a version of the HLWT to evaluate rutting potential and moisture susceptibility of asphalt mixtures (Izzo et al. 1999, Cooley Jr. et al. 2000). This test has shown potential as a verification tool for mixture design as well as QC/QA practices. Since 2004, Texas DOT has successfully included the HLWT in their *Standard Specification for HMA Pavement* (Texas DOT 2004). Texas DOT specifications allow a maximum rutting value of 12.5 mm at 20,000, 15,000 and 10,000 passes for mixtures containing PG 76-22, PG 70-22, and PG 64-22 binders, respectively (Texas DOT 2004).

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

Intermediate Temperature Cracking Resistance

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

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

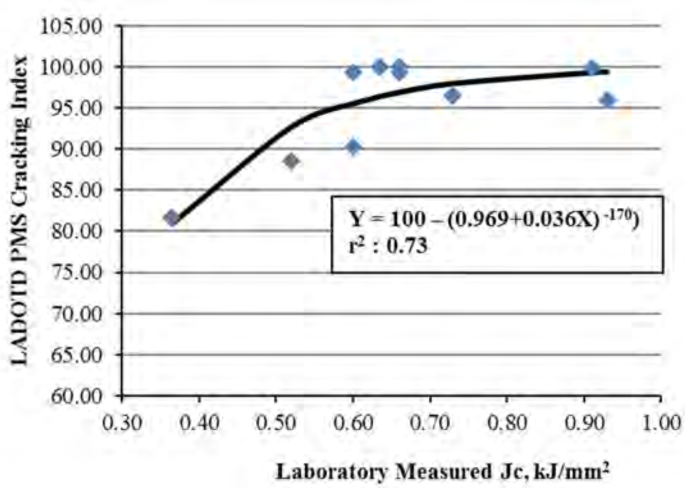

FIGURE 1 Measured J_c versus field performance (Kim et al. 2012).

IMPLEMENTATION

The LADOTD piloted the specification changes for a year in 2015 before fully implementing the performance testing into the standard specification. The intent of the pilot specification was to allow time for districts and contractors to mobilize and transition to the new requirements. Because the laboratory performance testing is conducted during design, contractors were required to purchase equipment for the SCB testing. Hot-mix asphalt (HMA) producers in Louisiana had previously purchased the equipment for HLWT in prior years. The HLWT requirement was implanted first, allowing for the contractors and districts to gain comfort with the performance testing requirements and specimen fabrication changes. LADOTD found that the main obstacle to implementation was user comfort level. Therefore, efforts were made to illustrate the need for the specification changes, as well as, the practical application of the test procedures. LADOTD implemented laboratory performance testing in the 2016 *Louisiana Standard Specifications for Roads and Bridges* for all mixtures intended for travel lanes (LADOTD 2016). The balanced mixture design procedure included HLWT and SCB testing for job mix formula (JMF) approval.

Training Workshop

The Louisiana Transportation Research Center hosted a statewide training workshop in April 2015. Contractors, agency representatives, and consultants attended. The full-day workshop agenda is presented in Figure 2. HLWT was implemented previously. Therefore, the workshop focused on the background, specimen preparation, and analysis of the SCB test.

SCB Test Job Mix Formula Approval

Contractors in Louisiana have been producing materials under the new balanced specification with little to no issues. Contractors have found increasing the effective asphalt content of the mixtures (not exclusively by increasing asphalt content) was the most significant change to pass the SCB specification. Figure 3 presents 11 mixtures submitted for approval under the 2016 specifications. Level 1 (low traffic) travel lane mixtures are required to have a minimum J_c value of 0.45 kJ/m^2. Level 2 (high traffic) travel lane mixtures are required to have a minimum J_c value of 0.55 kJ/m^2.

Innovations and Forensics

A major benefit of the specification change has been increased innovation within materials used in the asphalt mixtures. The specification allows substitutions of materials is traffic conditions are met and mixture testing is passed. Also, the new specification allows for increased use of reclaimed asphalt pavement (RAP) with mixture testing.

The inclusion and collection of HLWT and SCB data has also allowed for LADOTD to investigate premature failures. Figure 4 presents the SCB results of a mixture failure on a state highway in Louisiana. The pavement was exhibiting premature failure in the form of cracking, raveling and potholes. LADOTD was able to track the material and verify the J_c values as part of the forensic investigation. The failed section produced a J_c below the required threshold for approval. The mixture was produced prior to the full implementation of the 2016 specification. However, the data was collected during a trial period for the contractors to prepare for the

Semi Circular Bend (SCB) Test
Training Workshop
Agenda
April 16, 2015

8:00 – 8:30 am	Welcome and Announcements	Harold "Skip" Paul
8:30 – 9:45 am	Changes in the New Specification	Chris Abadie
9:45 – 10:00 am	Break	
10:00 – 11:30 am	SCB Training	
	a. SCB – History/Concept	Louay Mohammad (20 min)
	b. SCB - Research/Specification Review	Bill King (10 min)
	c. SCB – Testing	Sam Cooper III (60 min)
	i. Video	
	ii. Sample Prep	
	iii. Reporting	
11:30 – 12:30 pm	Lunch	Provided by LAPA
12:30 – 2:45 pm	Lab Demonstration of Test	Sam Cooper III/Lab Personnel
2:45 – 3:00 pm	Break	
3:00 – 4:00 pm	Open forum/Discussions/Questions	Chris Abadie/Bill King

FIGURE 2 SCB workshop agenda.

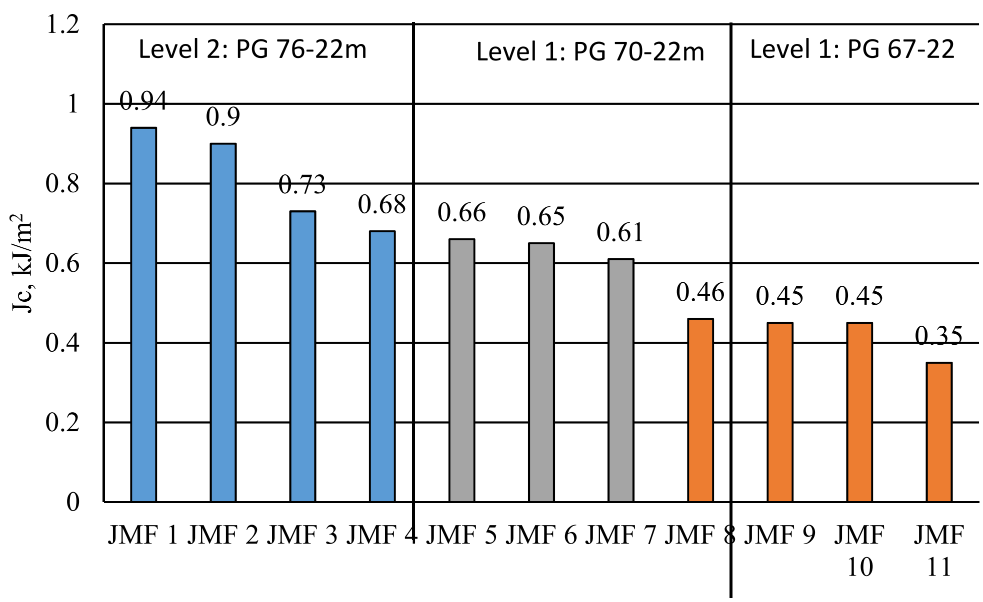

FIGURE 3 JMF approval–SCB results.

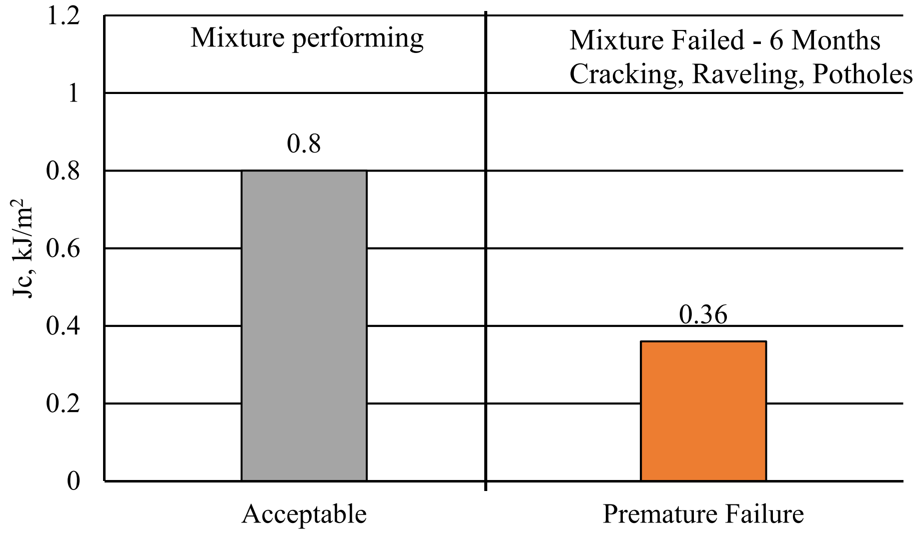

FIGURE 4 Forensic analysis–SCB results.

implementation. Further investigation revealed that the asphalt binder used in the failed section was not the correct material.

SENSITIVITY TO ASPHALT MIXTURE MATERIALS COMPOSITION

The use of SCB J_c to ascertain crack resistance in relation to asphalt mixture materials' composition were investigated. Asphalt mixtures presented were part of FHWA Project FHWA-PROJ-11-0070: *Advance Use of Recycled Asphalt in Flexible Pavement Infrastructure: Develop and Deploy Framework for Proper Use and Evaluation of Recycled Asphalt in Asphalt Mixtures*.

Ten mixtures were designed and constructed to incorporated RAP, RAS, warm-mix asphalt (WMA) technologies (water foaming and Evotherm), and different base binders (PG 64-22 and PG 58-28). Table 2 lists the material composition of the mixtures presented. The content of recycled materials was expressed in terms of recycled binder ratio (RBR), which is defined as the percentage of recycled asphalt binder in the total asphalt binder of the mixture. As shown in Table 2, the RAP provided RBR of 20% and 40% in the mixtures, while use of RAS yielded an RBR of 20% in the HMA mixtures. Details regarding the mixture design and production can be found elsewhere (Li and Gibson 2016).

Figure 5 presents the SCB J_c results for the 10 asphalt mixtures. The averaged COV ranged from 9.2% to 21.9% for each mixture, with an overall average of 14.5%, indicating a satisfactory test repeatability. The three HMA mixtures from L3, L5, and L7 containing 20% RAS or 40% RAP exhibited the lowest resistance to cracking. L1 control mixture and the two WMA mixtures from L2 (water foaming) and L11 (Evotherm) containing 40% RAP and the soft base binder (PG 58-28) yielded the highest crack resistance.

The effect of recycled materials (RAP–RAS) on the mixture property is ascertained through comparisons of mixtures L1, L3, L5, and L6; all are hot-mix asphalt mixtures with a base asphalt binder PG 64-22 (Figure 5). It is noted that the ranking of mixture based on J_c for

TABLE 2 Asphalt Mixtures Composition

Mix Designation	RBR (%)		Base Binder PG	HMA–WMA Process
	RAP	RAS		
L 1	—	—	64-22	HMA
L 2	40	—	58-28	Water foam
L 3	—	20	64-22	HMA
L 4	20	—	64-22	Evotherm
L 5	40	—	64-22	HMA
L 6	20	—	64-22	HMA
L 7	—	20	58-28	HMA
L 8	40	—	58-28	HMA
L 9	20	—	64-22	Water foam
L 11	40	—	58-28	Evotherm

NOTE: RBR = recycled binder ratio; RAP = reclaimed asphalt pavement; RAS = recycled asphalt shingles; PG = performance grade; — = not applicable.

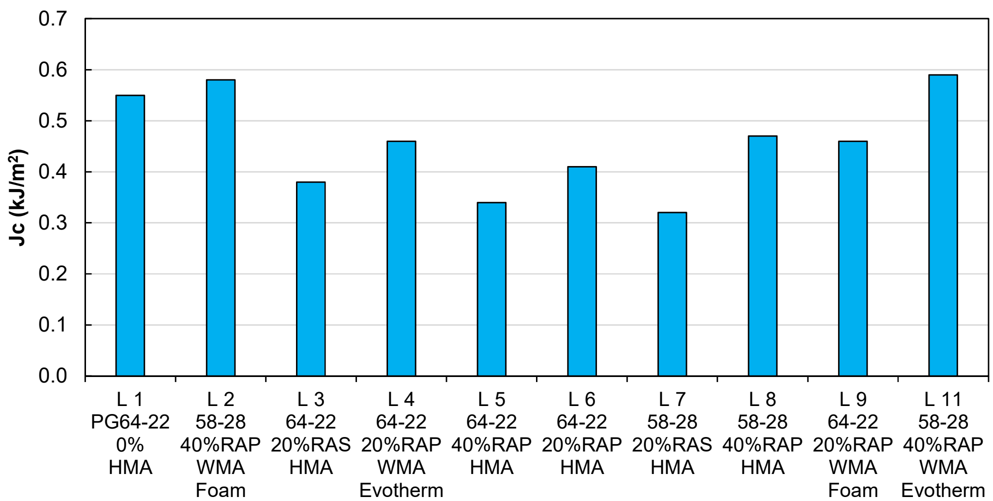

FIGURE 5 SCB critical strain energy release rates, J_c, test results.

crack resistance is L1 > L6 > L3 > L5. The control mixture L1 showed a significantly higher SCB J_c as compared to SCB J_c values of the remaining three mixtures (which were statistically similar to each other). Furthermore, a clear trend was observed in that the increase in the RAP–RAS content in hot-mix asphalt mixture did reduced SCB J_c values. In addition, the effect of 20% RAS (L3) on SCB J_c was between the 20% RAP (L6) and 40% RAP (L5) as a result of the moderate asphalt replacement with highly oxidized asphalt binder.

The effect of the use of soft PG base asphalt binder (PG 58-28) was evaluated through comparison of mixtures L3 with L7, and L5 with L8, all being hot-mix asphalt mixtures with the first group containing 20% RAS and the latter 40% RAP. It is noted that the use of soft binder slightly reduced crack resistance for hot-mix asphalt mixtures containing 20% RAS, as L7 showed lower J_c value, though not statically different, than L3, Figure 5. It is worth noting that the use of soft asphalt binder did significant improve crack resistance of mixtures containing 40% RAP by comparing SCB J_c results of mixtures L5 and L8.

The effect of warm-mix asphalt technology is presented through two sets of comparisons. The first set consists of mixtures L2, L8, and L11, all containing 40% RAP with base asphalt binder PG 58-28 (Figure 5). The two warm-mix asphalt mixtures (L2 and L11) showed statistically similar SCB J_c values and improvement over SCB J_c of hot-mix asphalt mixture of L8 (Figure 5). The second data set is from asphalt mixtures L4, L6, and L9, all containing 20% RAP with asphalt binder PG 64-22. The two warm-mix asphalt mixtures possessed higher SCB J_c values than the hot-mix asphalt mixture (Figure 5).

In summary, SCB J_c parameter was sensitive to the mixture materials' composition presented as well as the two warm-mix asphalt technologies, namely water foaming and Evotherm. The two warm-mix asphalt mixtures exhibited similar SCB J_c values when all other factors are similar. It is worth noting that when recycled materials were introduced into asphalt mixtures, use of warm-mix asphalt technology did improve cracking performance as measured by SCB J_c, especially for mixtures with high RAP content. This benefit can be attributed to the considerably reduced mixing and compaction temperatures and thus minimized short-term aging for warm-mix asphalt mixtures (Raghavendra et al. 2016).

SUMMARY AND CONCLUSIONS

The objective of this report was to present the experience, observations, and results of LADOTD's implementation of laboratory performance testing into the asphalt mixture design specifications. Mixtures were produced in accordance with newly implemented specifications to achieve a balance with respect to rutting and fatigue cracking. The following findings, observations, and conclusions may be drawn:

- LADOTD has implemented performance testing for JMF approval into the 2016 specification for all travel lane mixtures.
- Implementation is most successful when agency and industry are informed and trained together with transparency.
- Contractors in Louisiana have been producing materials under the new balanced specification with little to no issues. Contractors have found increasing the effective asphalt content of the mixtures (not exclusively by increasing asphalt content) was the most significant change to pass the SCB specification.
- SCB J_c parameter has shown to be sensitive to the mixture materials' composition observed.
- Benefits of the implementation have been illustrated by increased effective asphalt binder content of the mixtures, increased innovation of material usage, increased RAP content, potential lower asphalt binder grade substitutions, and failure investigations.

ACKNOWLEDGMENTS

This work was supported by the LTRC in cooperation with LADOTD and FHWA. The authors would like to acknowledge the efforts of William Gueho, Patrick Frazier, and Jeremy Icenogle at LTRC Asphalt Laboratory as well as the contributions of Engineering Material Characterization and Research Facility staff.

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Development of the Illinois Flexibility Index Test

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INTRODUCTION

Cracking in asphalt concrete (AC) pavements is among the major driving modes of pavement deterioration. Cracking mechanisms for AC materials can be generally grouped into two broad categories of load and non-load-related. Bottom-up, longitudinal wheelpath top-down fatigue cracking, near-surface cracking, and reflective cracking are considered in the load-related cracking category. Load-related cracks are commonly considered as indicators of issues related to pavement structural capacity, material properties, and truck loading. The second category is non-load–associated cracking, in which crack initiation and corresponding deterioration are driven primarily by environmental effects. Thermal cracking is considered in this category of cracking. Temperature fluctuations and material properties, along with structural characteristics of pavement, govern the initiation and growth of thermal cracks. Block cracking can also be considered in this category and is affected only by material properties and environmental conditions (temperature fluctuations, sunlight energy, etc.).

Understanding crack initiation and propagation in AC layers is critical for the design and construction stages of pavement life cycle. Once cracking mechanism in AC is understood, a more cost-effective pavement with adequate service life can be designed. Various cracking prediction models have been identified as a major need in the development of the mechanistic–empirical (M-E) pavement design methodology.

The cracking prediction models are needed to predict the extent and severity of cracking for a given pavement structure. Since cracking is a very complex phenomenon, advanced experimental characterization methods may be needed. The outcome of the test should provide inputs for pavement structural analysis and design to allow crack initiation prediction and the time required to propagate thorough the AC layer for given loading and environmental conditions. Such standard fracture tests were not available at the time of the M-E pavement design development; therefore, they were not incorporated into the current M-E pavement design process.

The asphalt paving industry has advanced the practice and is able to achieve high-quality AC mixes, including AC density targets using Superpave specifications (AASHTO M323 and R35). Currently, the standards and practice are capable of achieving and monitoring target field density precisely. This is an important milestone to improve construction quality and reduce the occurrences of premature failures; however, may not prevent problems associated with permanent deformation and cracking. Performance tests have been used to fill in this gap through better understanding the AC mixes response to traffic loading and environment. Tests used to characterize permanent deformation potential were successfully implemented. This results in reduced pavement rutting due to improvements in the AC quality (AASHTO T324).

The next challenge is to ensure that the produced and constructed AC mixes satisfy the intended pavement design life with respect to cracking. The quality of the material used in different layers of the pavement is the primary culprit for poor performance. This could be manifested as premature cracking and resulted in reduced service life. Thus, it is critically important to introduce characterization methods capable of predicting pavement performance.

In the last few decades, AC mixes were produced primarily using volumetric and tests to predict rutting potential. This resulted in sufficiently workable AC mixes that can be compacted to the target density with acceptable rut resistance. However, such AC mixes could be far from optimal in terms of their cracking potential. The situation was exacerbated with the inclusion of recycled materials. In addition, the decay in the virgin properties of asphalt binder results in more brittle AC mixes. Therefore, from the perspective of obtaining more balanced, application-specific AC mixes, additional characterization that includes cracking is needed.

Cracking characterization of AC mixes should complement the volumetric and rutting resistance goals. As a response to this almost universal need, a variety of test methods and criteria were developed to predict AC cracking potential. These methods include various testing geometries and testing conditions. DCT fracture energy (ASTM D7313), SCB fracture (AASHTO TP105 and TP124, Wu et al. 2005), Texas overlay fatigue (TEX-248-F), and IDEAL-CT (Zhou et al. 2017) are some of the most commonly used test methods to predict the AC mixes cracking potential.

In this paper, the development of the I-FIT is introduced. The steps followed include test development; identifying index parameter; algorithms to calculate the index; and test validation using the digital image correlation (DIC) technique. The results from various studies are presented to highlight the fact that the FI can be used to discriminate between AC mixes prepared with different constituents. Finally, correlation between FI and field results are discussed.

DEVELOPMENT OF THE ILLINOIS FLEXIBILITY INDEX TEST

Key Features

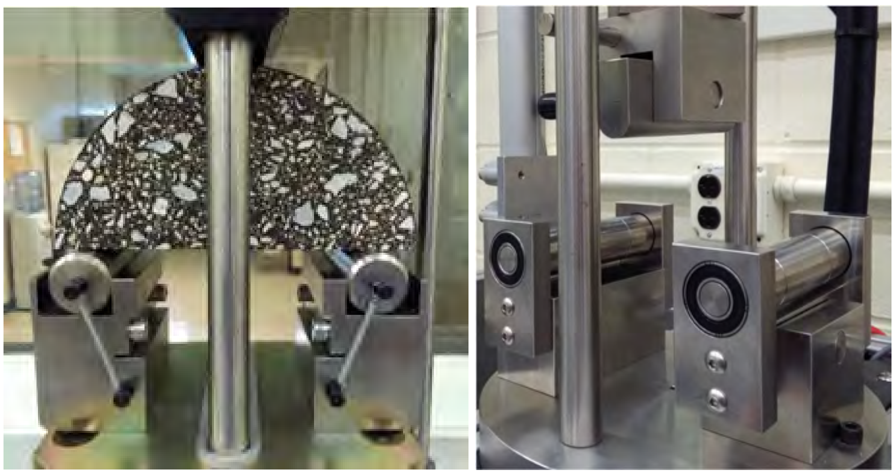
The I-FIT protocol, was developed to evaluate the overall resistance of AC mixtures to cracking-related damage (Al-Qadi et al. 2015, Ozer et al. 2016a, 2016b). The test was conducted at an intermediate temperature (25°C) using a custom-designed SCB fixture geometry placed in a servo-hydraulic AC testing machine (AASHTO TP124). The test was conducted using load-line displacement control at a displacement rate of 50 mm/min. The major test criterion obtained from the I-FIT is the FI. The FI was developed as an index for discriminating between AC mixes with respect to their cracking-related potential damage. The FI is calculated using the slope of the post–peak curve at the inflection point. The FI was shown to be able to discriminate between various AC mixes, regardless of preparation method—plant-produced lab-compacted, lab-produced lab-compacted, or field cores. In addition, FI had good correlation with pavement field performance (Al-Qadi et al. 2015). The FI is calculated using the following equation:

$$FI = A \times {}^{G_f}\!\Big/_{abs(m)} \tag{1}$$

where G_f is fracture energy, reported in joules/m^2; m is slope, reported as kN/mm. Coefficient A is a unit conversion factor and scaling coefficient. A was taken as 0.01 in this study. Fracture energy was calculated using the work of fracture method by finding the area under the load-displacement curve and dividing by the crack propagation area. The two commonly used test configurations are shown in Figure 1. The typical load-displacement results and fracture energy results for the two lab-designed mixes are shown in Figure 2.

Flexibility Index Formulation

An index parameter that can describe the overall patterns of load-displacement curves, shown in Figure 2, is needed to discriminate the cracking potential of AC mixes. The results demonstrate distinctive reactions from the specimens primarily due to binder grade and asphalt binder replacement (ABR). Empirical correlations between candidate indices and the speed of crack propagation (or approximate crack propagation velocity) were obtained from the SCB–geometry experiments. The form of the index parameter was inspired by the rate of crack growth definition



(a) (b)

FIGURE 1 I-FIT configurations: (*a*) bearing rollers and (*b*) spring rollers.

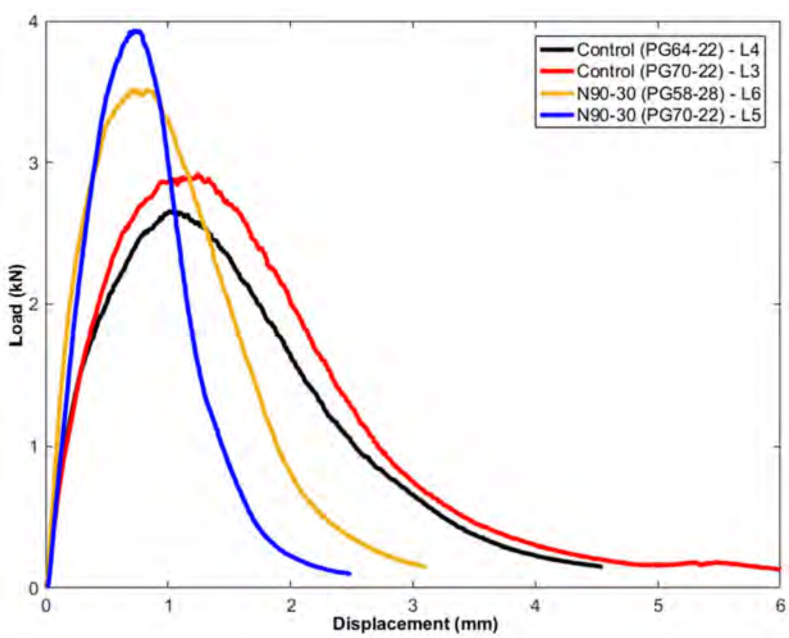


FIGURE 2 Typical load-displacement curves for two lab-designed AC mixes.

provided by Bazant and Prat (1988) for concrete materials to explain the effect of temperature and humidity on crack growth at a reference temperature.

$$\dot{a} = v_c (\frac{G}{G_f})^{n/2} \tag{2}$$

where

v_c = a constant,
G = energy release rate ($G = K_I^2/E$ with K_I is stress intensity factor), and
n = a material and geometry factor.

$$\dot{a} = v_c \frac{1}{(EG_f)^{n/2}} (K_I)^{n/2} \tag{3}$$

The stress intensity factor is related to the geometry and loading which can be assumed to be constant for the SCB–geometry; the other factors are proportional to material properties that can accelerate or decelerate crack growth. As fracture energy and modulus decrease or stress intensity increases, crack growth accelerates. An empirical correlation between brittleness (inverse of flexibility) and crack growth is exploited to formulate the index parameter. Therefore, Equation 5 is simplified in the following form (Equation 4), including a function for the FI:

$$\dot{a} = \frac{1}{FI_o} (K_I)^{n/2} \tag{4}$$

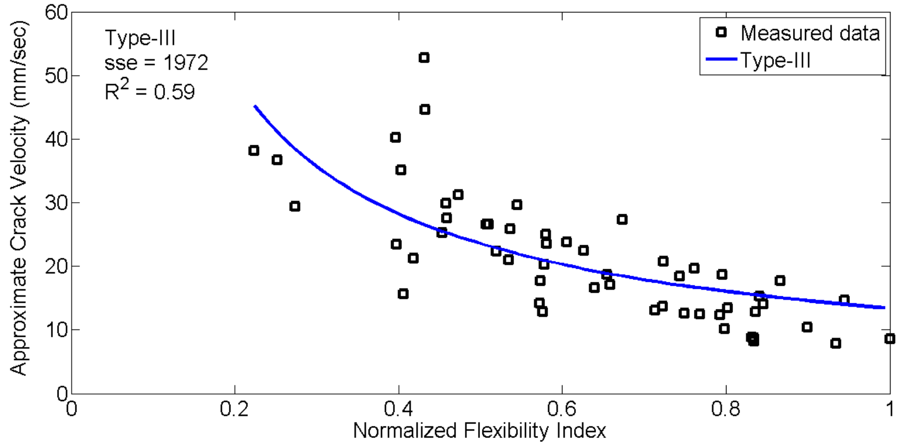
where three versions of FI (Type I, II, and III) were considered, respectively:

$$FI_I = {G_{fa}}\Big/{abs(m)} \tag{5a}$$

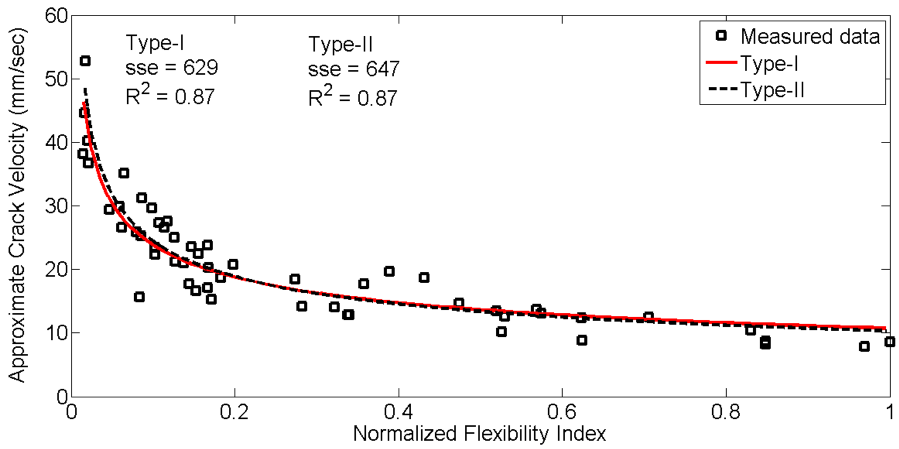
$$FI_{II} = {G_{fa}E}\Big/{(abs(m)f_t^2)} \tag{5b}$$

$$FI_{III} = G_{fa} \tag{5c}$$

An approximate crack velocity is used as proxy to the speed of crack propagation in Equation 4. The approximate crack velocity was calculated directly from the experimental data by assuming constant crack propagation speed. As the acceleration of crack propagation becomes more significant (true for some stiff mixes), accuracy of this approximation is reduced. Nevertheless, this can be considered as a proxy parameter as a first order approximation to crack velocity and used in correlating to various forms of FI, as shown in Figure 3. The research team has investigated more than 20 potential parameters for applicability to AC before selecting the aforementioned three types of FI (Al-Qadi et al. 2015). The form of the FI with fracture energy



(a)

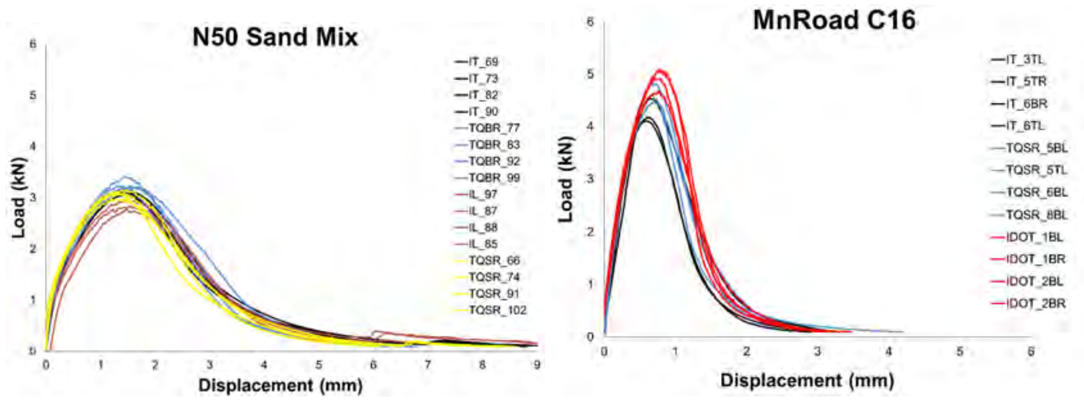


(b)

FIGURE 3 Correlation between normalized FI parameters (Types I, II, and III, corresponding to Equations 5a through 5c, respectively and approximate crack velocity experimentally measured (Ozer et al. 2016a).

and post–peak slope (Type I) is chosen as the final form because of its simplicity and good correlation to crack propagation growth.

In various projects conducted for Illinois DOT (Al-Qadi et al. 2015, Ozer et al. 2017, Al-Qadi et al. 2017), many types of AC mixes were evaluated using this protocol. Figure 4 illustrates typical load-displacement curves obtained from various machine configurations for two AC mixes that can be considered at the extreme end of flexibility spectrum.



(a) (b)

FIGURE 4 Typical results obtained for AC two mixes using various testing configurations: (*a*) sand mix with FI of 19 to 23 and (*b*) brittle surface mix with FI of 3 to 4 (Ozer et al. 2017.

Flexibility Index Algorithm

An accurate calculation of post–peak slope is critical for the calculation of FI. An algorithm was developed to process the post–peak segment of the load-displacement curve to calculate the area under the curve as well as the inflection point and slope at the inflection point, as shown in Figure 5. In the development of the FI, it was observed that the post–peak segment of the load-displacement curves was very sensitive to changes in characteristics such as binder type, content, and recycled materials content. Therefore, the shape of the post–peak curve as a crack propagates can reveal useful information describing crack propagation speed in the I-FIT specimen. In order to standardize the test output processing, a software package was developed and made publicly available by the Illinois Center for Transportation (ICT).

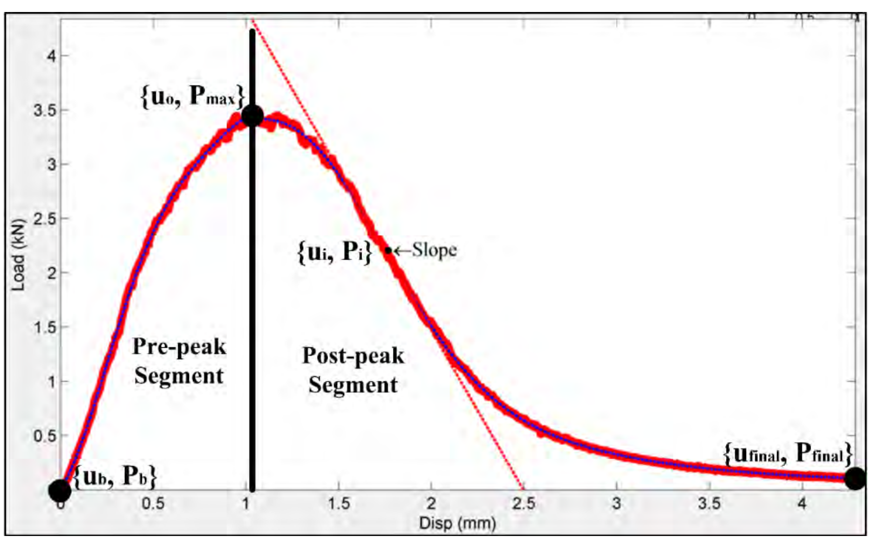


FIGURE 5 Demonstration of key parameters of a load-displacement curve obtained from I-FIT as obtained from the I-FIT software (I-FIT 2018).

The equations proposed for the tail part of the fracture curve were *n*th-order Gaussian function. The following function is used to fit the post–peak segment of the load-displacement curve for displacements (u) after the peak load (P_{max}) to the cut-off displacement (u_{final}):

$$P_2(\mathrm{u}) = \sum_{\mathrm{i}=1}^{\mathrm{n}} \mathrm{d_i} \exp\left[-\left(\frac{\mathrm{u}-\mathrm{e_i}}{\mathrm{f_i}}\right)^2\right] \tag{6}$$

where d, e, and f are polynomial coefficients and n is the number of exponential terms (n is taken as 4 in the final version of the software).

Then, the inflection points at which the second derivative of the fitted equation becomes zero are extracted, and the first derivatives indicating the slopes (m_i) are computed at the extracted inflection points (u_i).

$$m = \left(\frac{\partial P_2(u)}{\partial \mathrm{u}}\right)_{u=u_i} \tag{7}$$

It is common that the fitted equation may produce more than one slope when there is more than one root found in the previous step (Figure 6). There is only one slope consistent with the definition of the test; the remaining slopes are spurious and need to be eliminated. To determine the most representative slope and eliminate the unrealistic slope(s), three visual-based criteria were implemented to identify the right slope and eliminate the others. The algorithm was tested and refined by a significant number of mixes after release of the software.

Figure 7 presents the first derivatives for some of the laboratory-designed AC mixes to illustrate the changes between AC mixes and the sensitivity of post–peak behavior to AC mix design properties. The first derivatives can be interpreted as follows: The crack begins to propagate with a steady-state speed and slows down as it approaches the zone under the loading

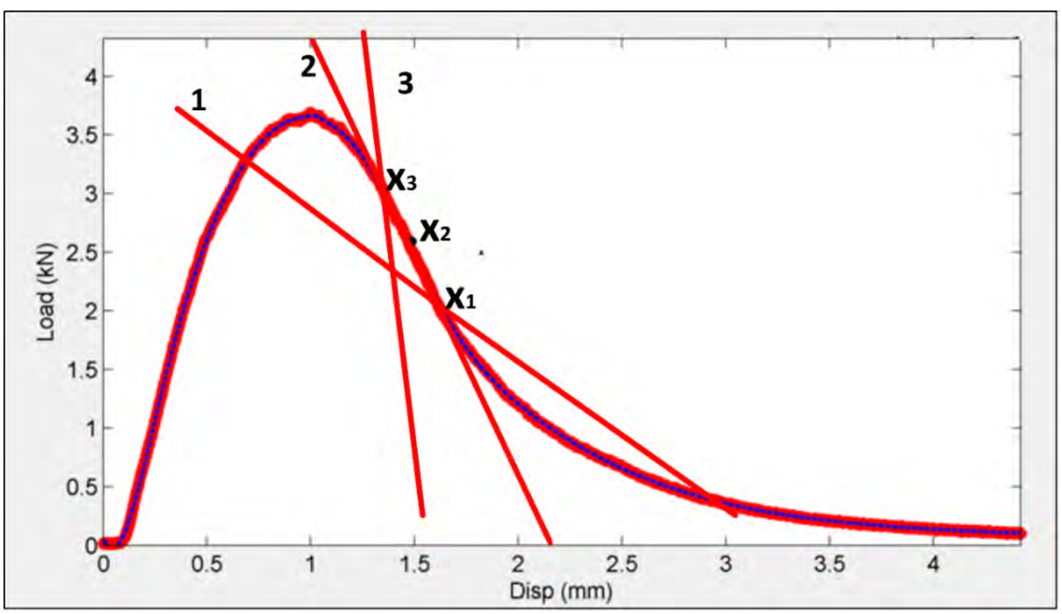


FIGURE 6 Candidate slopes in the post–peak load-displacement curve.

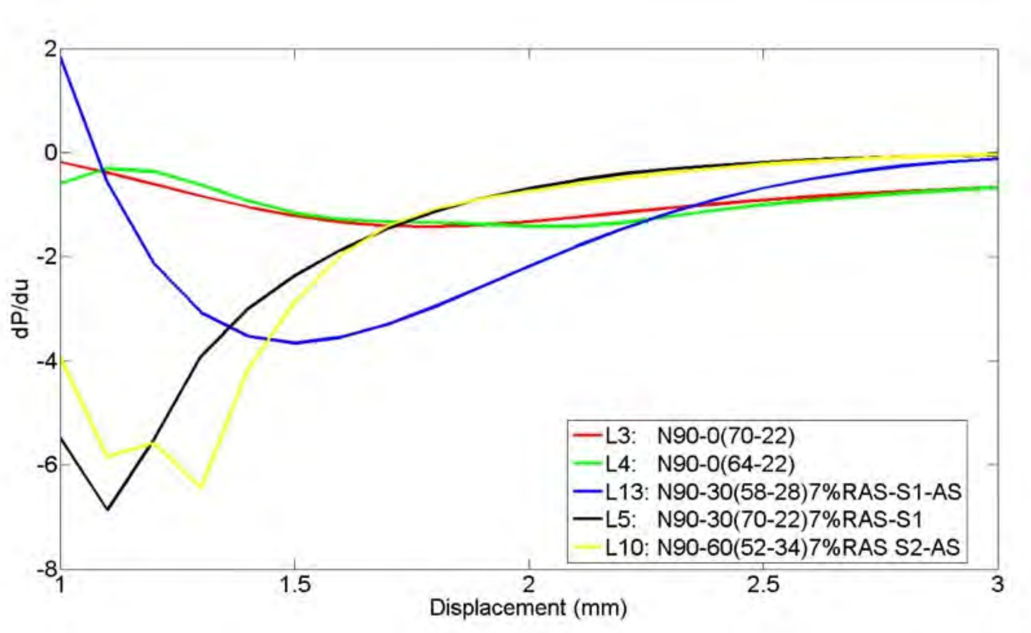


FIGURE 7 First derivatives of post–peak curve for an N90 AC mix with increasing RAP and RAS contents up to 60% (Ozer et al. 2016b).

head governed by compressive stresses. The global minimum point of the first derivative represents the maximum slope at the inflection where the crack slowed down. The data for L4 and L10 AC mixes show more than one inflection where first derivative becomes zero. As the AC mix becomes more brittle, the global minimum point migrates to the lower left with increasing magnitude and faster propagation. For an AC mix with no recycled content, the minimum point has a smaller value and occurs at greater displacement, indicating slower crack propagation.

Mechanics of the I-FIT Using Digital Image Correlation

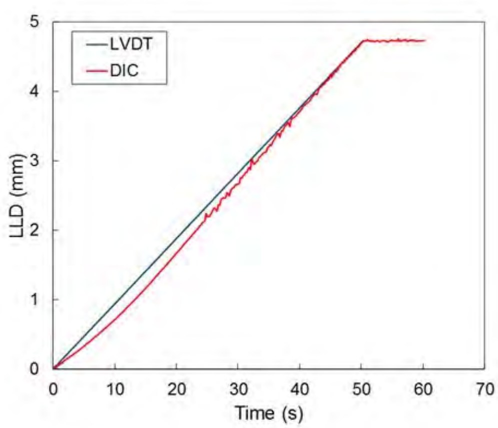
DIC is an optical method providing popular and versatile means of measuring surface strains and displacements on a deforming specimen separately from the single-point load and displacement records (Sutton et al. 2009). The DIC technique was used in various stages of test development for the following objectives:

1. Evaluate fidelity of the single-point global load-line displacement readings recorded by the loading devices.
2. Evaluate fracture process zone as a function of temperature and mixture constituents.
3. Quantify the impact of non-fracture related energy dissipation mechanisms to the fracture energy.
4. Calculate viscoelastic fracture parameters driving crack initiation and propagation.

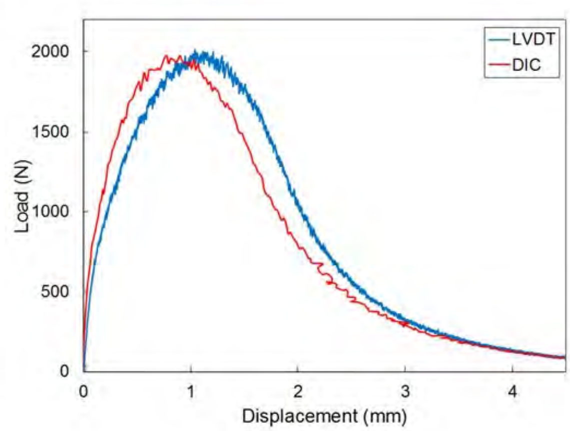
During the tests, pictures were recorded using the software VicSnap (from Correlated Solutions, Inc.). Data analysis was done by the DIC software Vic2D (from Correlated Solutions, Inc.) using the cross-correlation coefficient. Two different CCD cameras were used for imaging during the experiments—a Point Grey Gazelle 4.1MP Mono (2048x2048 pixels, 150 frames per second-fps), and an Allied Vision Prosilica GX6600 (6576x4384 pixels, 4 fps) with a Tokina AT-X Pro Macro 100 2.8D lens. The camera chosen for a particular experiment depends on the spatiotemporal resolution required, with the Prosilica GX6600 generally used for the higher magnification (i.e., zoomed-in) experiments meant to study process zone evolution and the Gazelle is used for experiments at a larger length scale where the materials can generally be considered homogeneous. Details of the measurements and analysis were introduced elsewhere (Berangere et al. 2017a and 2017b).

One of the first experiments were conducted to meet the first objective. Figure 8 shows the measurements recorded by the device used to conduct the I-FIT experiment and the digitally collected using a point on the specimen right beneath the loading head. Since the linear variable differential transformer (LVDT) measurement shown in the figure is supposed to represent the load-line displacement the specimen receives, it has to match the measurements recorded by the DIC on the specimen. There is a small shift between the two measurement points due to compliance of the testing fixture configuration.

The DIC system, coupled with the high-resolution camera, was used to evaluate the effect of changes in the microstructure on cracking at the crack front as part of the second objective. Figure 9 shows the evolution of the DIC-measured strain field at 8 microns/pixel as a function of load for an AC mix. The measurements were superimposed on the aggregate structure for this case. The upper left figure shows the load-displacement curve for this experiment and the red dots mark the loads at which selected DIC results are shown in Figure 9 for the measured horizontal strain. It was clearly observed that most of the strain was concentrated in the matrix material with the aggregate having almost no strain at all, even as load increases to peak value. Large strain values are observed closer to the notch as expected, even though the area of the strained matrix increases considerably as load is increased. Concentrations with particularly high strain are also visible near aggregate corners or at junctions or "triple" points.



(a)



(b)

FIGURE 8 Load-line displacement: (*a*) LLD versus time and (*b*) load versus displacement measurements with the load frame (using LVDT) and with DIC at the subset location right beneath the loading head (Berangere et al. 2017a).

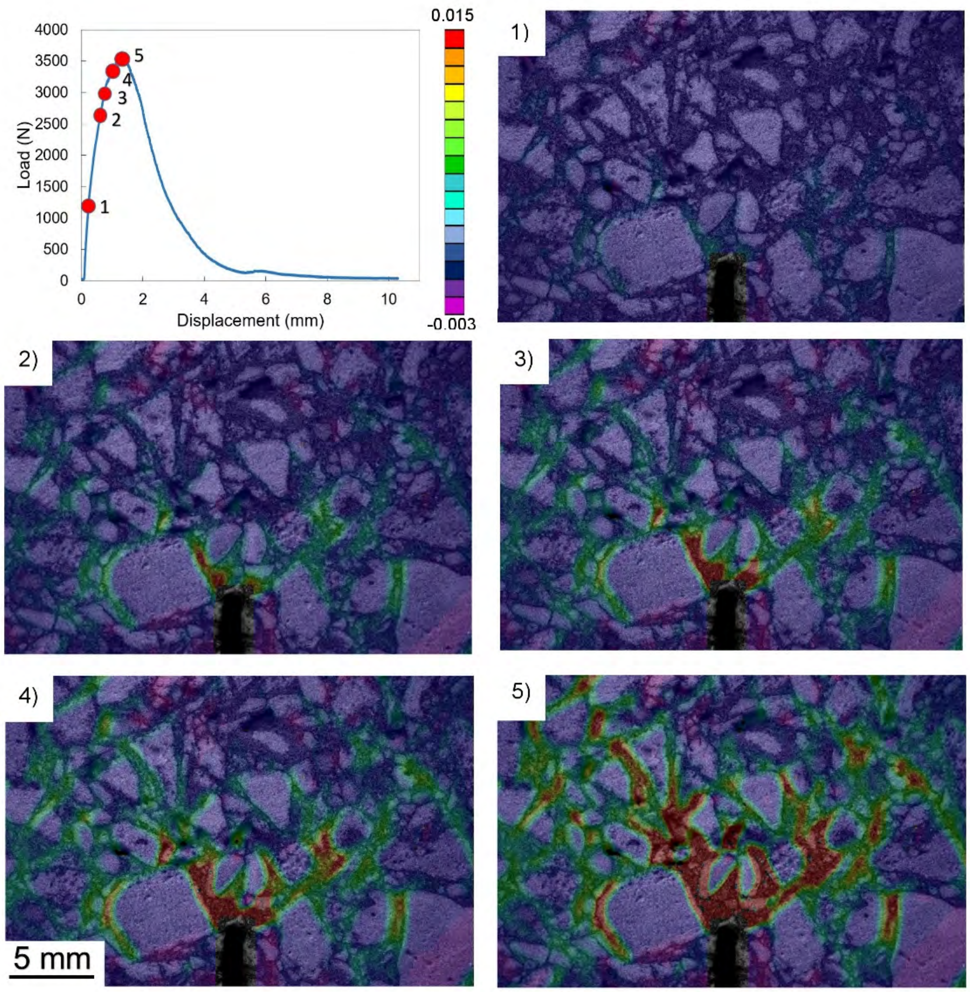


FIGURE 9 Load-displacement curve (upper left), as well as five contour plot results of DIC-measured horizontal strain at loads indicated for a mix with a spatial resolution of 8 microns/pixel (Berangere et al. 2017b).

The zoomed-in measurements were conducted for AC mixes with different ABR and binder type. The goal was to evaluate the effect of changes in AC mixture constituents on crack front strains and damage. Figure 10 shows the strain field at peak load for three AC mixes: control (0% ABR); mix with 30% ABR (7%RAS) and PG 58-28; and mix with 30% ABR (7%RAS) and PG 64-22. Figure 10 offers a comparison of the extent of straining for different temperatures, rates, and ABR contents. Temperature had a very noticeable effect on strains: at low temperature, the strain level was about 10 times smaller than at room temperature, and the strains were much more localized at the notch tip. This is consistent with the embrittlement of asphalt binder at low temperature. The embrittlement may be associated with the shrinking fracture process zone, thus indicating less distributed or less diffused microstructural damage and additional localization of strain and stress fields. The comparison between the two AC mixes also shows an embrittlement with an increase in RAS content. The strain level is smaller and the strains are more localized for the mix with 7% RAS.

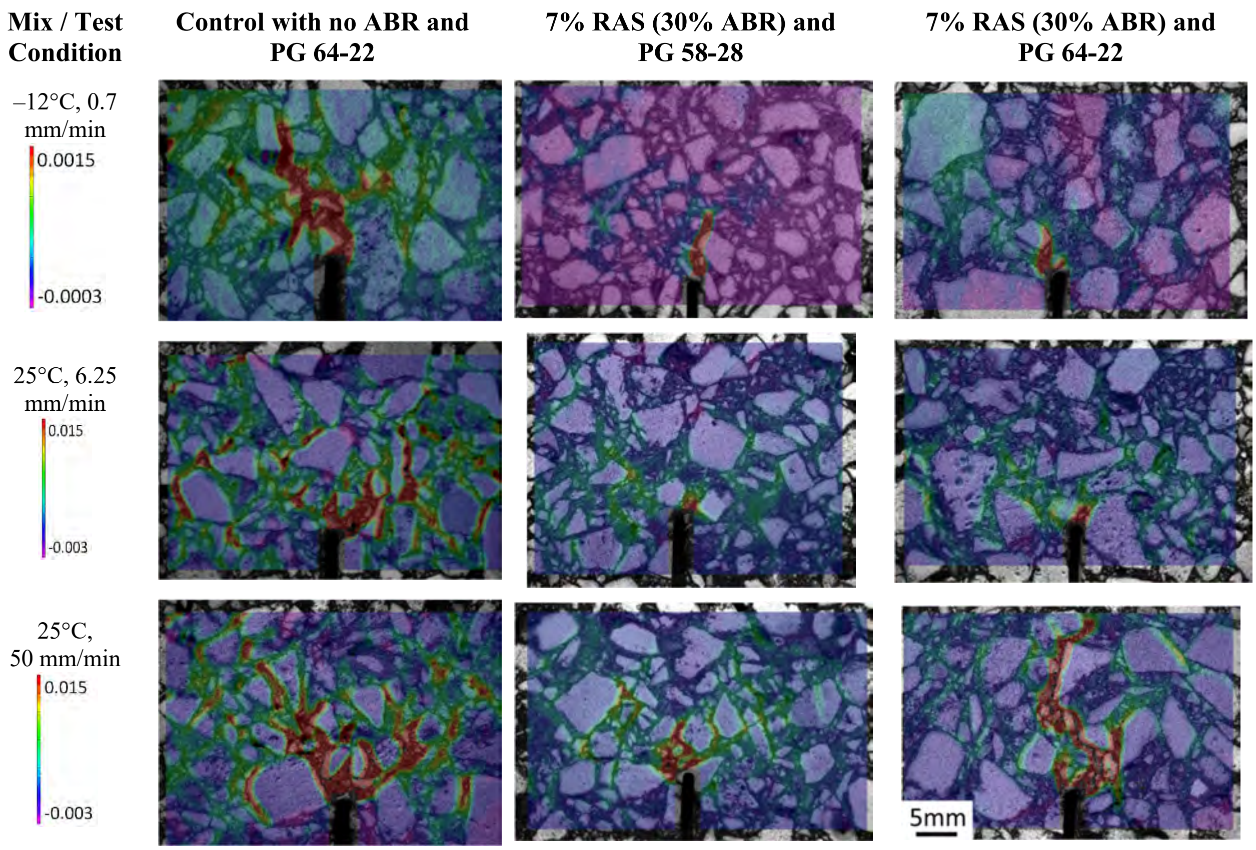


FIGURE 10 Strain field superimposed on the aggregate structure for AC mixes L4 and L6 at –12°C 0.7 mm/min, 25°C 6.25 mm/min, and 25°C 50 mm/min at peak load in each case (Berangere et al. 2017b).

Finally, non-fracture related energy dissipation mechanisms were investigated using the DIC system. In any fracture or strength, there is always a possibility of damage or creep-related energy dissipation reducing and impacting the available amount of energy for the actual crack of interest. Loading head and supports are potential spots for high stress concentrations and strains resulting in damage of the specimen. This effect was investigated using the high-resolution camera and using specimens with increasing notch depth (10, 15, 20, and 35 mm), as shown in the results in Figure 11. Blue-colored strains indicate compressive strains accumulating under the loading head. It is very interesting to note the growth of compressive strain field with increasing notch depth and bridging to cover the entire notch depth, while at smaller notch depths the effect of loading head appears to be localized with much smaller magnitude of strain levels. Therefore, it was concluded SCB geometries with deep notches are not ideal for fracture characterization due to the extent of the impact of the boundary condition.

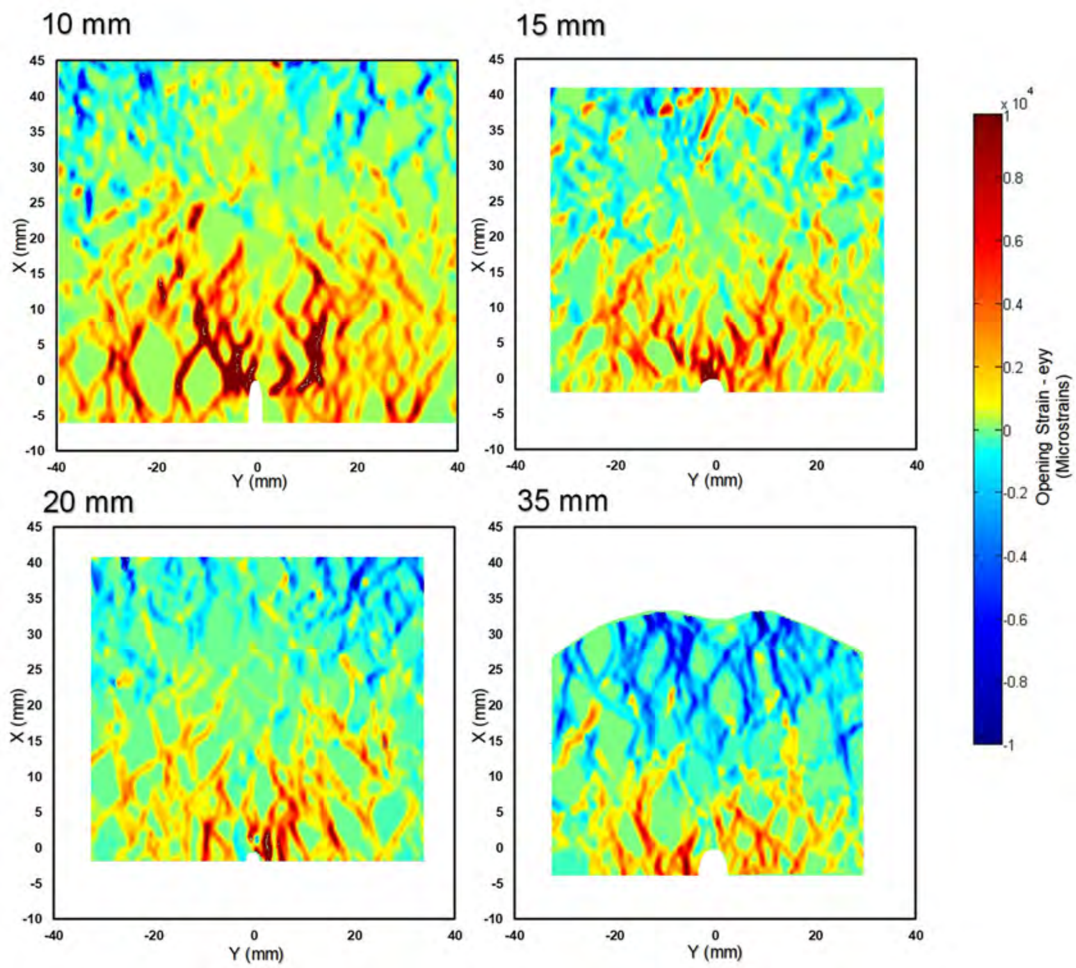


FIGURE 11 Horizontal strain fields for specimen geometries with notch depths of 10, 15, 20, and 35 mm at the peak load (Rivera 2017).

EVALUATION OF VARIOUS MIXES USING FLEXIBILITY INDEX

Laboratory-prepared and plant-produced AC mixes were evaluated in various ICT studies throughout the development as well as the deployment of the I-FIT protocol (Al-Qadi et al. 2015, Al-Qadi et al. 2017, Ozer et al. 2017). In one of these studies, laboratory mixtures were designed with varying proportions of RAP and RAS. A total of 11 AC mixtures were designed consistent with Illinois DOT specifications. The AC mixes were N90-design surface courses with VMA target of 15.3±0.1%. The range of AC mix designs included AC mixes with varying amount of RAS and RAP, only RAS, high ABR, and with and without antistripping agent. ABR was calculated based on 100% contribution from RAP and RAS to the mix design. The AC mixture designs were prepared using the Bailey design method (Vavrik et al. 2001). The volumetric properties of the AC mixes were kept identical to allow a comparison of AC mixes without any bias. VMA and binder content were kept the same for all AC mixes with variations in RAP and RAS content. Table 1 contains details of the AC mixtures referred in this paper.

TABLE 1 Laboratory AC Mix Designs Used for FI Evaluation (Ozer et al. 2016b)

Mix ID[6]	Mix Name	Binder Grade	RAP (%)	RAS (%)	ABR (%)	AC[5] (%)	VMA (%)
L3	N90-0	70-22 SBS	—	—	—	6.0	15.3
L4	N90-0	64-22	—	—	—	6.0	15.3
L5	N90-30 7% RAS S1[3]	70-22	—	7	29.8	6.0	15.3
L6	N90-30 7% RAS S1[3]	58-28	—	7	29.8	6.0	15.3
L7	N90-20 5% RAS S1[3]	58-28	—	5	21.2	6.0	15.3
L8	N90-10 2.5% RAS S1[3]	64-22	—	2.5	10.5	6.0	15.3
L9	N90-30 5% RAS S2[4] AS[1]	58-28	11	5	30.5	6.0	15.2
L10	N90-60 7% RAS S2[4] AS[1]	52-34	40	7	60.8	6.1	15.2
L11	N90-0 AS[1]	64-22	—	—	—	6.0	15.3
L12[2]	N90-30 7% RAS S2[4] AS[1]	58-28	—	7	30.6	6.0	15.2
L13[2]	N90-30 7% RAS S1[3] AS[1]	58-28	—	7	29.8	6.0	15.3

[1] AS indicates AC mixture with 1% Pavegrip 550 antistrip added to virgin binder.
[2] These AC mixtures have different RAS sources but similar mix design.
[3] RAS source (S1).
[4] RAS source (S2).
[5] AC = Asphalt content including virgin binder and asphalt binder replacement (ABR) from RAS and RAP (100% blending between virgin and recycled binder was assumed).
[6] Mix designation L = laboratory-prepared mixes followed by numeric value representing a specific mix type as designed.

The FI values and fracture energy of the AC mixes introduced in Table 1 are shown in Figure 12. The values were normalized with respect to the control AC mix with PG 70-22. The overall pattern with the FI was a consistent reduction with increasing ABR. The reduction was much more pronounced when compared to fracture energy values obtained at the same temperature. Some of the additional key findings from the comparison of FI values for various AC mixes are as follows. L5 (N90, 30% ABR with PG 70-22) and L10 (N90, 60% ABR with PG 52-34) AC mixes resulted in the lowest FI values. The changes in the binder grade had a clear impact on FI values. For example, AC mixes with the same ABR and a similar RAS type and content but with stiffer binder, L5 (N90, 30% ABR with PG 70-22), showed significant lower FI the same AC mix with a softer binder, L6 (N90, 30% ABR with PG 58-22). Such variation may not be obtained from fracture energy results.

COMPARISON TO FIELD PERFORMANCE

AC Overlay Performance in Illinois

The performance of various overlay projects in Chicago area was monitored closely: three total-recycle asphalt (TRA) pavement sections and a comparison section let on April 26, 2013 and five projects let June 13, 2014, by the Illinois DOT (Figure 13, Table 2); two of these projects were constructed in 2014 and three in 2015. Most of these projects were documented in detail, including existing condition evaluation, data and material gathering during construction,

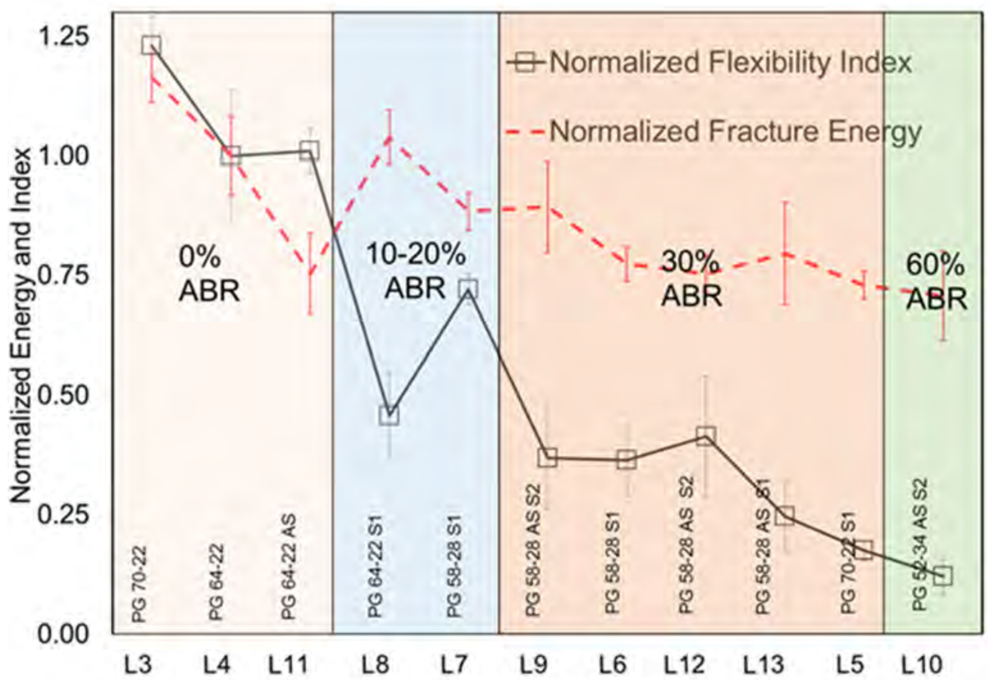


FIGURE 12 Comparison of normalized fracture energy with normalized FI using I-FIT test results [range of FI values are from 1.5 (L10) to 15.7 (L3) with reference AC mix L4 having FI of 12.8] (Ozer et al. 2016b).

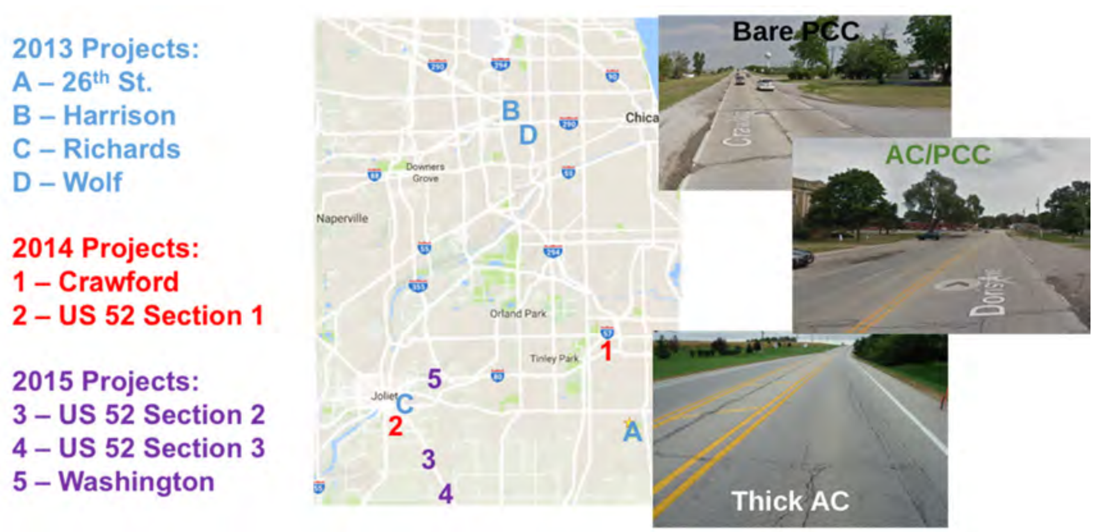


FIGURE 13 Various overlays constructed in 2013, 2014, and 2015 monitored (Al-Qadi et al. 2017).

TABLE 2 AC Mixes and Volumetrics Used in Various Overlay Projects in Illinois (Al-Qadi et al. 2017)

Map ID	Project/Mix Name	Binder Grade	RAP (%)	RAS (%)	ABR (%)	AC (%)	FI
A	26th Street / N50 TRA	PG 52-28	51	4.6	60	6.7	3.8
B	Harrison Street / N50 TRA	PG 52-28	53	5.0	56	6.5	0.9
C	Richards Street / N50 TRA	PG 58-28	27	—	37	5.8	4.1
D	Wolf Road / N70 Mix D	PG 58-28	30	—	20	5.9	—
1S	Crawford Ave. / N70-30	PG 58-28	9.9	5.0	29	5.7	3.4
1N	Crawford Ave. / N70-15	PG 64-22	4.9	2.5	15	5.6	4.8
2E	US-52 Section 1/ N70-30	PG 58-28	20	3.1	30	5.5	6.3
2W	US-52 Section 1/ N70-30	PG 58-28	34	-	29	6.0	11.9
3	US-52 Section 2/ N70-TRA	PG 52-34	39	5.0	48	6.0	5.4
4	US-52 Section 3/ N70-TRA	PG 52-28	39	5.0	48	6.3	7.1
5W	Washington Street / N70-30	PG 58-34	20	3.1	30	6.6	10.4
5E	Washington Street / N70-30	PG 58-34	34	—	30	6.0	10.6

post-construction surveys, and coring. The goal was to evaluate effect of construction, structure, and material on crack development and AC overlay performance. A total of 12 AC mixes, with ABR ranging from 15% to 60%, were evaluated. AC mixture properties were determined for the plant mix at production and then in situ through pavement coring during the 2- to 4-year life span of the pavements, depending on when the project was constructed.

Mixes identified as A to C and 3 and 4 are dubbed as TRA mixes with all recycled aggregates including RAP, RCA, and steel slag resulting in high ABR values. These mixes had the lowest FI. In the rest of the mixes, ABR gradually increased from 15% to 48% and different techniques were used to compensate for the inclusion of RAP and/or RAS. Mixes 1S and 1N were produced with binder contents of 5.7% and 5.6%, resulting in relatively low FIs, 3.4 and 4.8, respectively. As the binder content increased, there was clear improvement in the FI value (e.g., Mix 2W, 5W, and 5E) given that ABR level was moderate and no RAS was used. The use of RAS reduced the FI when not accompanied by an increase in binder content or by the use of softer binder grade. For example, the FI value of Mix 5W (3.1% RAS) was among the highest and remained close to its counterpart Mix 5E (no RAS) due to additional 0.6% binder and softer binder grade PG 58-34. On the other hand, Mix 2E (3.1% RAS) experienced a sharp reduction in the FI value from 11.9 (for Mix 2W with no RAS) to 6.3 due to 0.5% binder deficit and no binder grade adjustment.

Overall, FI was found to be responsive to mix design adjustments such as binder content, binder grade, ABR level, and/or RAS content.

Distress survey data were collected on the sections using established distress criteria (Illinois DOT 2012). The datasets consist of pre-construction, post-construction, and springtime surveys in 2014, 2015, 2016, and 2017.

The correlation of FI values to transverse cracking is shown in Figure 14. Correlation improves after third and fourth winters as cracking became more apparent on the surface, more frequent, and its extent and severity increased. The slope and trends of the relationship indicate

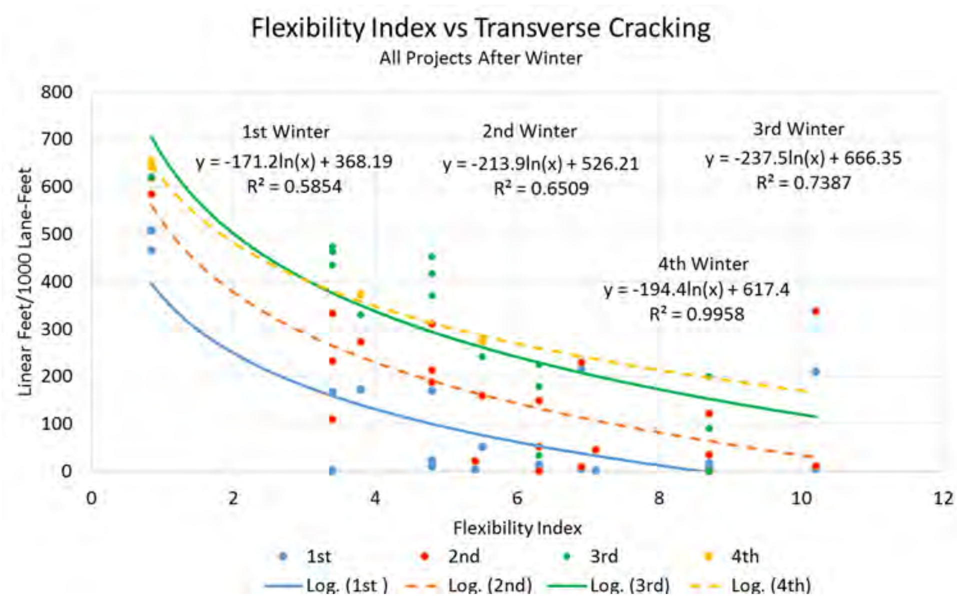


FIGURE 14 FI relationship to transverse cracking on all projects (Al-Qadi et al. 2017).

that, with respect to surface AC mix, FI values in the 8 to 10 range provide the greatest benefit in reducing transverse cracking. It is important to note that there were structural factors affecting transverse cracking including thickness of AC lifts (new and old) and underlying pavement type [portland cement concrete (PCC) versus AC] and PCC joints. Sometimes, structural factors may mask the material contribution to transverse cracking. The correlation may not offer direct comparison of AC mixtures with varying flexibility values used in the same or similar structures.

Correlation with FHWA ALF Experiment

The test sections constructed at the FHWA Turner–Fairbanks ALF in McLean, Virginia, provided a unique opportunity to correlate the FI along with other cracking tests to performance results obtained from these sections with (ideally) identical structures (4-in. AC over 9-in. granular base). The ALF test sections were designed primarily to evaluate the effects of varying AC mixture design characteristics on fatigue cracking. Experiments were conducted at 20ºC. One parent AC mixture design was used to develop variety of AC mix designs containing different levels of RAP and RAS.

The ABR level in the AC mixes varied from 0 percent to 40 percent. Efficiency of binder grade bumping were evaluated at 20% and 40% ABR levels. According to the FI results shown in Figure 15, it is evident that the best-performing AC mix is the control (Lane 1) followed by AC mixes with 20% ABR with and without using WMA technologies (Lanes 4, 9, and 6). On the other hand, AC mixes with RAS and 20% ABR (Lanes 3 and 7) and 40% ABR (Lane 5) with PG 64-22 (no binder grade bumping) were the three most poorly performing AC mixes. The intermediate-performing group consisted of the AC mixes with 40% ABR and PG 58-28 (Lanes 2, 8, and 11), which illustrates the significance of the binder grade bumping that improved the performance of AC mixes from poor to intermediate. The results again support the ability of I-FIT to predict cracking of various mixes when used in a full-scale controlled tests.

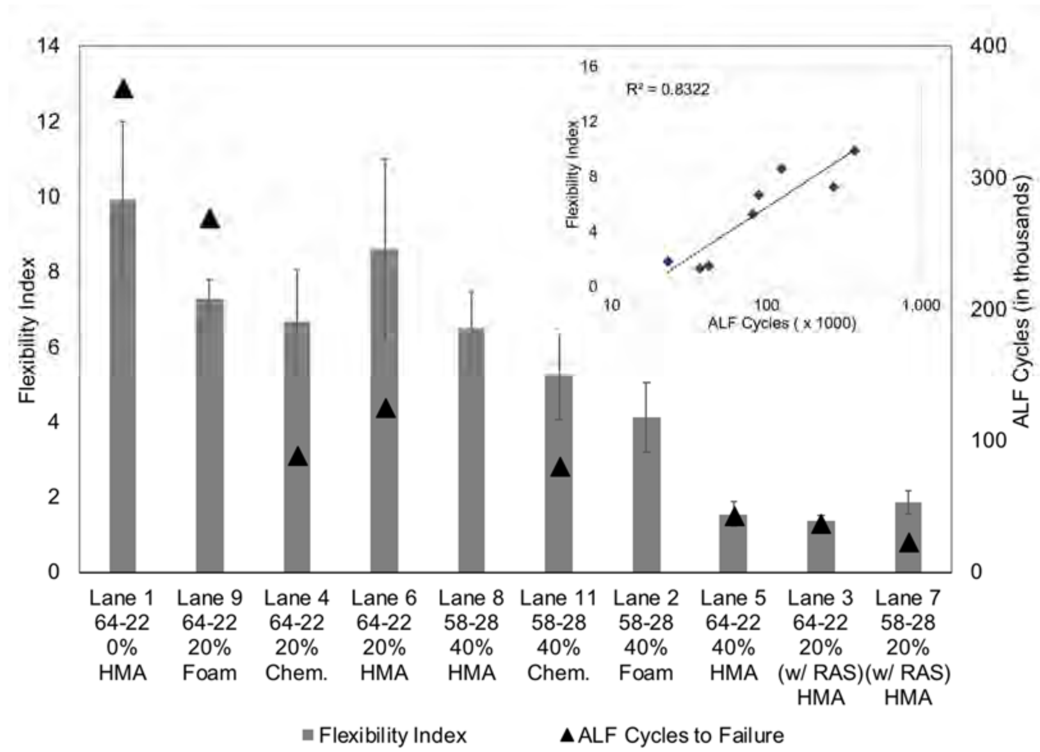


FIGURE 15 FI values and number of cycles to critical fatigue cracking recorded in the ALF experiment for AC mixes with varying ABR content from 0% to 40% (Ozer et al. 2018).

ROUND-ROBIN TESTING

Recently, two round-robin studies were completed in 2017 and 2018 by Illinois DOT. Thirty labs participated in 2017 including 10 Illinois DOT, 15 private, and five other states and universities. Participation increased to 34 in 2018 with the addition of two other states and private labs. The data collected from the round-robins included six commonly used testing equipment. Each year's testing consisted of three rounds to include different height of gyratory compacted pills (160, 150, and 115 mm). One mix was sent to the labs each year. The mixes used are N50 surface type of mixes with 31% ABR in 2017 and 10% ABR and polymer-modified binder in 2018. A brief summary of all tests conducted in 2017 and 2018 is provided in Table 3. It was shown that individual lab COV is between 11% and 16%. No consistent effect of gyratory specimen height was observed.

CONCLUDING REMARKS AND FUTURE WORK

Illinois Flexibility Index TEST (I-FIT) Protocol was introduced in 2015 as an outcome of an ICT project. The test protocol was accepted as AASHTO provisional specification TP 124. The I-FIT protocol offers various advantages to the industry due to its simplicity and affordability while providing meaningful results to discriminate between AC mixes. The results obtained from

TABLE 3 Summary of Round-Robin Testing Conducted by Illinois DOT

	Average FI			Individual FI COV %			Population FI COV%		
	Round 1	Round 2	Round 3	Round 1	Round 2	Round 3	Round 1	Round 2	Round 3
2017	4.7	4.5	5.9	11.1	13.6	10.9	25.5	22.2	29.3
2018	23.3	22.0	22.8	15.7	14.7	13.0	24.5	26.9	20.9

various ICT projects by testing numerous lab-produced and plant-produced AC mixes indicate consistent trends with changes in AC mix constituents and I-FIT results.

The results from two multiple projects to correlate FI values to field performance validate the ability of I-FIT to predict potential cracking. In one of the projects, consistency was demonstrated between the FI and transverse cracking for the AC mixes used in 12 overlay projects. In another experiment, there was a very good correlation between fatigue cracking at a full-scale ALF and FI values obtained by testing the plant-produced AC mixes collected during the production.

Currently, Illinois DOT has been following a roadmap to implement I-FIT as part of performance specifications in Illinois. Pilot projects are underway allowing data collection from contractors and testing in the districts. Development of a mixture long-term aging protocol is subject of an ongoing study. Once the aging protocol is developed, according to the roadmap, I-FIT will be part of performance specifications in 2020.

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Relating DC(T) Fracture Energy to Field Cracking Observations and Recommended Specification Thresholds for Performance-Engineered Mix Design

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INTRODUCTION

Thermal cracking is a major distress in cold regions of the United States. During pavement cooling cycles, thermally induced (or non-load–associated) tensile stress begins to build-up in asphalt pavement surfaces. From a local or microscopic viewpoint, when the accumulated thermal stress exceeds the local tensile strength of the material, separation in the form of micro-cracks occur. Coalescence and propagation of this material tearing leads to a visible, highly oriented crack pattern. Thermal cracks typically appear as transverse cracks in the pavement (*1*), since the pavement is generally restrained in the longitudinal direction while it can contract in the transverse direction. Long-term aged pavements sometimes display block cracking, possibly in combination with thermal cracking. This paper focuses mainly on transverse cracking. However, two of the sections investigated were dominated by block cracking and a technique was developed to assess these sections in terms of their transverse cracking content. Over the past decade, considerable laboratory and field investigations have linked fracture energy as measured in the DC(T) to transverse cracking. This paper provides a comprehensive summary of data from 52 projects having both DC(T) fracture energy results and field cracking measurements. A discussion of results and implications to performance-engineered mix design (PEMD) are provided, along with currently recommended specification thresholds and future directions.

Mechanical Testing of Binders and Mixtures: Up to and Including Superpave

Early efforts to control transverse cracking as reported in the literature were focused on binder testing (*2*, *3*). The penetration test and the ductility test provided some degree of cracking control, based on limiting the binder temperature susceptibility and its ability to withstand thermally induced strain. Early mechanical mixture tests tended to be torture-type empirical tests, generally performed at room temperature or above, with testing criteria independent of structure type or layer thickness. Some examples include the Marshall stability and flow tests and the tensile strength ratio test (AASHTO T-283). However, neither of these tests were intended to control thermal or block cracking. undamental work by Monismith and colleagues provided an early fundamental look at the use of asphalt mixture viscoelastic tests and models to control thermal cracking (*4*), but were not adopted in routine practice.

Obtaining reliable field data for the purpose of selecting, calibrating, and validating mixture performance tests requires the use and documentation of appropriate evaluation techniques. This equates to considerable time and resources, which should be carefully considered in research planning and budgeting. One of the first major undertakings in correlating field transverse cracking to binder and mixture tests was the Ste. Anne's test road project, undertaken jointly by Shell Canada Limited and Manitoba Department of Highways (*5–7*). The project included 29 test sections intended to incorporate a wide range of variables pertinent to the study of transverse cracking: subgrade type (which has an effect on frictional drag between the pavement surface and support layers during heating–cooling cycles, and thus, thermal stress level); different sources and types of asphalt binders; different pavement structures; and so on. Both binder and mixture time-dependent properties (stiffness as a function of loading time) were measured and related to field performance in the St. Anne test road study. The study led to several important conclusions, which included classifying transverse cracking as a predominantly temperature-related distress, establishing a correlation between binder grade and cracking rate in Canada, and highlighting the strong relationship between asphalt mixture stiffness at long loading times, for instance, 2 h (*1*, *6*, *7*). Despite their importance in moving technologists towards new and powerful fundamental tests and models, these approaches were not widely adopted to assist in the control thermal cracking in the United States.

The Strategic Highway Research Program (SHRP) led to the development of a large suite of sophisticated, fundamental binder tests, mixture tests, and pavement performance prediction models. Quoting from the abstract of the SHRP A-370 report (*8*):

> **Binder Characterization and Evaluation Volume 4: Test Methods** describes the development of test methods for the characterization of asphalt cement. These test methods may be used in specifications and for developing correlations between physical and chemical properties. To understand how the properties of asphalt cement and asphalt concrete mixtures relate to one another, <u>fundamental material properties expressed in engineering units were required</u>. This information was used to develop models that relate the properties of asphalt cement to mixture properties and, in turn, to pavement performance.

Over 25 years ago, SHRP researchers understood the importance of using fundamental tests, expressed in engineering properties, in advancing asphalt and pavement technology. Those tests were generally performed at test temperatures and stress/strain levels that were akin to a particular distress type being addressed (prevented), and with proper age conditioning. For rutting, dynamic shear rheometer tests were performed on the asphalt binder at the high in-service pavement temperature on unaged and short-term aged samples. For low-temperature cracking, a bending beam rheometer was used to obtain creep compliance (although expressed as the inverse of creep compliance, deemed as creep stiffness) and *m*-value, or the slope of the log creep stiffness–log time curve as obtained by fitting a power-type model.

These developments in advancing fundamental binder tests notwithstanding, the SHRP A-370 researchers acknowledged the need for fundamental asphalt mixture tests, and models, in order to accurately predict and control key categories of pavement performance.

The Superpave IDT and TCModel

The SHRP program clearly advanced the state-of-the-art in the characterization and specification of newly produced virgin binders. Despite the clear importance of the asphalt binder in dictating asphalt mixture stiffness and relaxation ("stress shedding") characteristics, binder tests cannot fully capture the transverse cracking resistance of the complete asphalt mixture, especially in light of modern recycling practices. The SHRP A-357 project led to the development of the Superpave indirect tensile test (IDT), as a mixture-based test for evaluating transverse cracking potential of asphalt mixtures (*9*, *10*). Using the Superpave IDT, mixture tensile strength, creep compliance (and subsequent master curves), and mixture coefficient of thermal expansion–contraction can be used in predicting asphalt mixture behavior at low temperatures.

In an effort to validate the performance-graded (PG) binder specification (*11*), SHRP researchers developed a computer-based thermal cracking prediction tool called TCModel to help link binder properties, along with mixture data collected in the Superpave IDT, to field cracking performance in a range of climates across the United States and Canada (*10*, *12*). TCModel was an early portable computer-based Fortran program developed to predict transverse cracking at different depths on an hourly basis. In the model, a phenomenological crack propagation model (Paris law) using IDT tensile strength and the slope of the log mixture compliance versus log time curve (i.e., the mixture *m*-value). TCModel was released at the end of the SHRP program in 1993 and subsequently revised and appended (Buttlar et al., 1998) and implemented as part of the NCHRP 1-37A project (*13*). To this day, TCModel is integrated in Pavement ME (*14*), although it requires substantial local calibration in order to match field performance (*15*, *16*) due to its limited physical representation of asphalt and pavement fracture (*17*).

Illi-TC and TCAP

Researchers at the University of Illinois at Urbana–Champaign (UIUC) addressed some of the key limitations in TCModel in a new thermal cracking simulation code entitled Illi-TC, primarily incorporating DC(T) fracture energy as an input and by implementing a more fundamental and accurate crack propagation model. This work was done as part of the FHWA Pooled Fund Study on Low-Temperature Cracking (#776), as described in Marasteanuet al. (*18*). The Illi-TC used a 2-D, finite-element–based pavement cracking model instead of the 1-D phenomenological model used in TCModel. Illi-TC implemented a cohesive zone crack modeling approach, viscoelastic bulk material modeling, and most importantly, utilized asphalt mixture fracture energy as opposed to mixture tensile strength as the primary mixture cracking resistance input (*19*). The tool combines mixture properties with hourly pavement data obtained from the Enhanced Integrated Climatic Model to predict transverse cracking on a given pavement structure for selected geographic locations. Illi-TC was validated in the second phase of the Low-Temperature Cracking Pooled Fund Study (*5*) and was found to relate to thermal cracking at MnROAD with reasonable accuracy after only minor model calibration. More recently, Illi-TC was validated by Dave and Hoplin (*20*) using five pavement sections. The results indicated a high accuracy of Illi-TC in predicting thermal cracking performance of asphalt pavements.

TCAP, a transverse cracking prediction tool developed under the Asphalt Research Consortium work element E2d, likewise made recent strides by directly incorporating pavement aging in cracking analysis. The tool also moved away from the traditional IDT strength test and used the Uniaxial Thermal Stress and Strain Test to determine the mixture tensile strength under thermal loading and aging-dependent mixture coefficient of thermal contraction (CTC) (*21–23*).

Limited Adoption of Advanced Tools (Or "Why is Nobody Using This Stuff?")

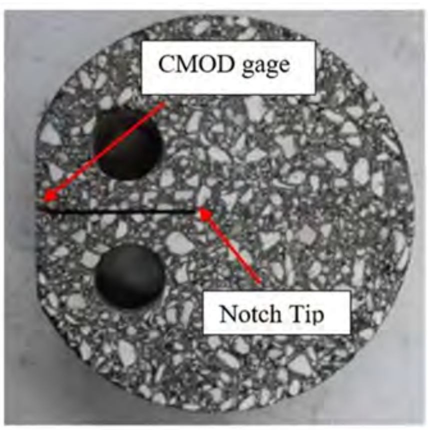
While these new tools, and others (*24*), have correlated well to field results, none have resulted in widespread application by practitioners. This is likely due to the number and complexity of mixture tests required, the expense required to develop and maintain user-friendly software working across multiple computing platforms and remaining stable over time with advances in computer operating systems. Moreover, there is a strong desire in the asphalt industry to have a simple approach to control of asphalt cracking in various stages of pavement design, asphalt mixture design, and asphalt mixture production and acceptance. Pavement modeling is viewed by many practitioners as too cumbersome and unreliable for the purpose of everyday QC, acceptance, and pay-for-performance type specifications. Furthermore, it is fair to conclude that the majority of personnel responsible for asphalt mixture design and QC have far more educational training and field experience in the areas of measurement and evaluation, and in mechanical testing and mechanical production, as compared to computer modeling, simulation, and pavement mechanics. This implies that these personnel are more likely to (*a*) correctly operate, interpret, troubleshoot, and ensure the quality data resulting from mechanical testing, as compared to their ability to and (*b*) properly handle simulation inputs and outputs, and to install, maintain and troubleshoot sophisticated computer-based material evaluation systems.

In the aforementioned Pooled Fund Study on Low-Temperature Cracking, one of the focal points of this study was to evaluate existing lab binder and mixture tests and to identify or develop new ones and to evaluate their correlation to field observations. The study focused on fracture mechanics-based lab tests, such as a low-temperature, fracture energy-based SCB test, and later in the study, the newly developed DC(T) test. Indeed, rather than the software simulation programs developed in this study, it was the mechanical mixture tests and the linkages of the fundamental properties obtained and field performance that were ultimately adopted by practitioners. Details of the tests and specification thresholds developed are now reviewed.

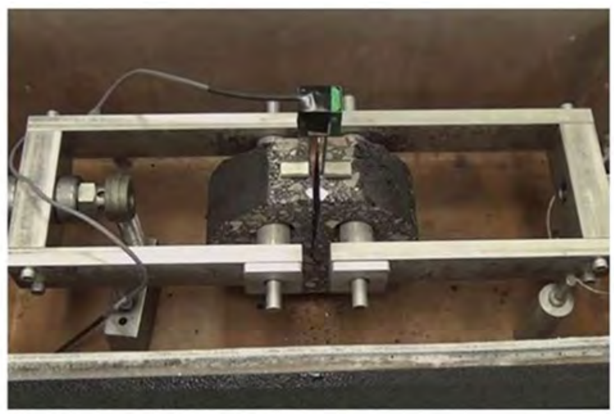
Development of the DC(T)

Thermal cracking in asphalt pavements is dominated by Mode-I, or pure opening fracture, as thermal cracks in the field are predominantly oriented perpendicular to the direction of the thermal-induced stresses in the pavement, and thus perpendicular to the direction of traffic. Motivated by an NSF study on reflective cracking, Wagoner et al. (2005) reported on the development of a DC(T) geometry for AC specimens, based on ASTM E399 as a starting point. It was observed that the peak load and fracture energy values obtained with the new specimen geometry could be replicated with a favorably low COV (*25*). The test temperature generally specified for the DC(T) test is 10°C higher than the Superpave PG low-temperature grade of the binder used in the asphalt mixture for a given geographical location. However, many states, such as Illinois, use this test at a specific temperature corresponding to the low-temperature climate of the state obtained from LTPPBind tool at various reliability levels, for example, for Illinois, –12°C is the common DC(T) test temperature, while a testing temperatures in range of –18°C to –24°C are commonly used in Minnesota.

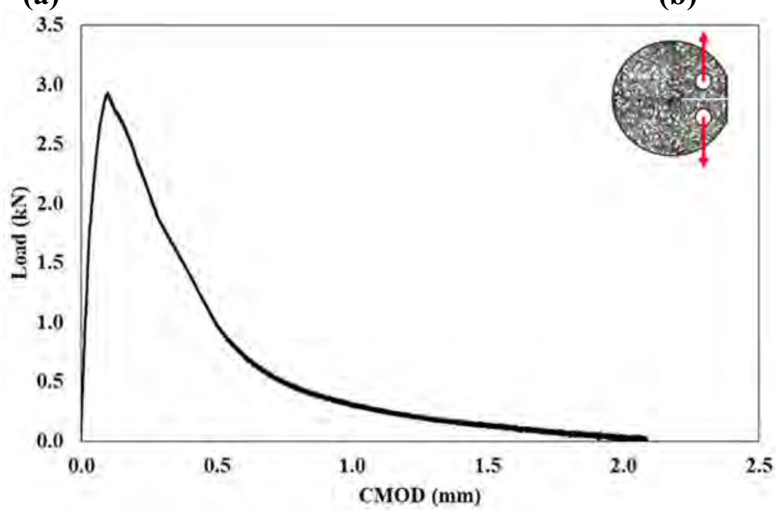
Figure 1 displays a typical DC(T) specimen, loading apparatus, and the typical load versus crack mouth opening displacement (CMOD) curve obtained from the test. Once steel dowels are placed through the drilled holes, the CMOD gage is affixed on knife-edged gage points, and temperature is stabilized, a seating load of 0.1 kN is applied. The specimen is then pulled at a constant (CMOD) rate of 0.017 mm/s or 1 mm/min. After the peak load is reached, a fracture process zone is fully developed and a coalesced Mode-I crack begins to propagate outward from the notch tip, similar to a propagating thermal crack in the field. The test is stopped when the post-peak loading reaches a nominal level of 0.1kN. The area under the curve, normalized by the fractured ligament area of the specimen, is reported as the fracture energy of the asphalt mixture specimen. The method of testing is outlined in ASTM D7313-13 standard (*26*), and has been modified by some agencies, such as Minnesota DOT, Iowa DOT, and Wisconsin DOT (*27-29*). This parameter is sometimes referred to as the 'total fracture energy', as the area under the load–CMOD curve contains both the pre- and post-peak work of fracture, along with a small amount of viscous dissipation resulting from the bending of the "arms" of the specimen, i.e., the region between the notch tip and location of the CMOD gage (Figure 1). As a



(a)



(b)



(c)

FIGURE 1 DC(T): (*a*) specimen, (*b*) loading scheme, and (*c*) a typical load–CMOD plot obtained from DC(T) test.

result, it is a matter of debate whether or not experimentally determined fracture energy is a fundamental parameter or not. It can perhaps be considered an engineering parameter, as it derives from fundamental principles of fracture mechanics testing, provides a commonly used fracture modeling parameter, and is expressed in engineering units. It also provides a straightforward-to-interpret parameter that consists of the total work of fracture needed to create the fractured area, which can be easily visualized (viewed) after testing. This in turn can be visualized in the pavement structure—higher fracture energy in the mix equates to more resistance to the propagation of a large, channeling crack, driven by temperature fluctuation, traffic or their combined effects.

Stated otherwise, the DC(T) test has the ability to determine if the mix has sufficient fracture energy to mitigate thermal cracking, or if the selected binder–aggregate combination (or other mix design or construction effects) have resulted in a brittle, cracking prone mixture (*30*). More details regarding fracture energy thresholds for varying design reliability–traffic levels are presented in the following section. Researchers have shown that the DC(T) test can differentiate between different asphalt mixture designs and aging levels. Behnia et al. (2011) showed that increased RAP contents tended to decrease the fracture energy of the mixture (*31*). Buttlar et al. (2016) found similar results for mixture containing recycled roofing shingles, but reported that suitable DC(T) fracture energies could be achieved with these mixes with the appropriate choice of softer binders, including polymer-modified binders (*32*). Hill et al. (2013) used DC(T) fracture energy test to evaluate the effects of various WMA additives on low-temperature cracking behavior (*33*). Zegeye et al. (2012) used DC(T) fracture energy test to compare the low-temperature fracture properties of polyphosphoric acid-modified asphalt mixtures (*34*).

Importance of Aging and Practical Consideration in a Simple Specification

Braham et al. (2009) found that DC(T) fracture energies are also affected by aging levels (*35*), in an, arguably, complicated fashion. On one hand, the slope of the post-peak portion of the load–CMOD curve was found to become steeper with aging level in all cases. On the other hand, the total fracture energy measured in the DC(T) when following the procedures outlined in ASTM D7313 has the general tendency of first increasing at early levels of mixture aging, followed by a peak and then a decrease in total fracture energy with further laboratory aging. This peak roughly occurs between 4 and 12 h of loose mix oven aging at 135°C (*35*). This phenomenon was accounted for when developing fracture energy thresholds in the Pooled Fund Low-Temperature Cracking Study (*5*), as described in the following section. The Technical Review Panel for this study did not recommend the development of a detailed long-term mixture aging protocol as part of the study. Instead, fracture properties from long-term aged field cores were compared to mixture properties of retained, original materials, which were then aged in the laboratory using different aging methods and aging levels. The results of this investigation led to the establishment of fracture energy criteria on short-term aged and compacted mixtures (2 h in a forced-draft oven between mixing and compaction at the compaction temperature), which were established to account for the typical loss of fracture energy expected in the field. It was agreed by the project panel that long-term aging should be revisited for possible inclusion of future version of the DC(T) fracture energy-based specification (*36*). After calibration and evaluation in a blind study, the DC(T) test was selected by the project Technical Review Panel based on its closer correlation to field performance as compared to the SCB test (*5*).

Recent Trends in Mixture Performance Testing and Scope of New Data Presented Herein

Although the early DC(T)-based thermal cracking specification developed in the PFS recommended a combination of fracture energy, creep compliance, and Illi-TC simulation results to control thermal cracking for high-traffic–volume test facilities, in practice, agencies simply implemented the DC(T) test and the fracture energy threshold-based specification (*18*). None of the agencies seriously considered the creep compliance or simulation modeling requirements proposed in the original specification. These agencies include Minnesota DOT, the Illinois Tollway, Chicago DOT, Iowa DOT, Wisconsin DOT, and Pennsylvania DOT.Explanations of why this might have occurred were covered in an earlier section.

Over the past decade, numerous simple performance tests have been developed for AC. In some cases, the tests and associated analysis procedures have been predicated on fracture mechanics principles developed in other engineering disciplines (*25*, *37*). These methods also pay attention to strict temperature control requirements, and were standardized only after the procedure led to an acceptable COV, which can be computed as the standard deviation of test replicates divided by the mean value of the measured parameter. In the asphalt industry, the most highly repeatable binder tests generally yield COV values in the range of 2% to 10%, while the most repeatable mix tests fall in the range of 5% to 20%.

Empirical, index-type tests have also been developed (*38–40)*, in some cases without strict temperature control requirements (for example, allowing tests at room temperature without a climate control chamber), and often with relatively high values of COV (some routinely exceeding 30% or even 40%) (*41*). This can lead to erroneous results when fewer numbers of replicates are tested and disputes between parties in a contractual setting (*42*). The number of replicates required for an equivalent reliability in parameter determination increases with the square of COV. Thus, tests with double the COV of the typical range would, in theory, require four times the number of test replicates to achieve similar reliability. The other disadvantage of index-type tests is the inability to directly use the test results in detailed validation studies, where pavement modeling is required to check the veracity of model calibration and/or correlations of lab data to field performance.

For these reasons, agencies should carefully weigh the pros and cons of fundamental, repeatable lab tests as compared to simpler index-type tests, especially in the context of project criticality. Agencies with high project criticality, such as the Illinois Tollway, Minnesota DOT, and O'Hare Airport, were early adopters of DC(T) specifications, in large part because the relative cost of including DC(T) testing in the design phase was negligible as compared to the potential savings in avoiding early cracking failures. In addition, these agencies also considered the economic and environmental benefits of achieving high levels of recycling while maintaining confidence regarding pavement durability. The use of the DC(T) and Hamburg tests as low and high-temperature PEMD tests has been shown to help agencies in achieving these sustainability objectives in a number of published studies (*32*, *33*, *41*, *43*, *44*).

Finally, in the early development of new tests, results are often compared to existing tests, and sometimes compared to measurements taken during accelerated pavement testing programs. However, a new performance test must ultimately be vetted based on its ability to predict or control one or more modes of pavement distress.

The remainder of this paper presents both early and recent field performance data, focused on thermal and block cracking, along with fracture energy measurements obtained with the DC(T). Fifty-two mixtures and associated field performance data sets, covering a wide range of mixture types, service life ages, and covering four Midwestern states in the United States are presented. Existing fracture energy thresholds used in several agency specifications are then validated using this new data.

EARLY CORRELATIONS BETWEEN LOW-TEMPERAURE FRACTURE ENERGY AND FIELD CRACKING DATA

Original Correlation Between DC(T) Fracture Energy and Thermal Cracking

Figure 2 (*5*) shows the correlation between transverse cracking and fracture energies for field sections in the Pooled Fund Low-Temperature Cracking Study tested with the DC(T). As fracture energy of the asphalt mixture drops, the observed transverse cracking in the field increases, with an apparent kink in the curve in the range of 400 J/m^2. An important factor that was not included in this first-of-its-kind plot was the pavement age of the various sections. As asphalt pavement ages, it loses its ability to relax the thermal stresses over time due to oxidative stiffness and thus is more prone to cracking. Ideally, a plot of this nature would be composed of similarly aged field sections, but in practice, a range of differing aged pavement surfaces is typically available for study. In addition, the plot consisted of a relatively few number of sections, mostly residing in Minnesota, with a few sections from northern Illinois and Wisconsin. Fortunately, dozens of additional sections have now been tested over a wider geographical region, and across many different mix types and traffic levels.

Those details notwithstanding, a preliminary performance-based thermal cracking specification was developed, as shown in Table 1. Fracture energy thresholds for different levels of project criticality were included, along with thresholds for thermal cracking levels predicted

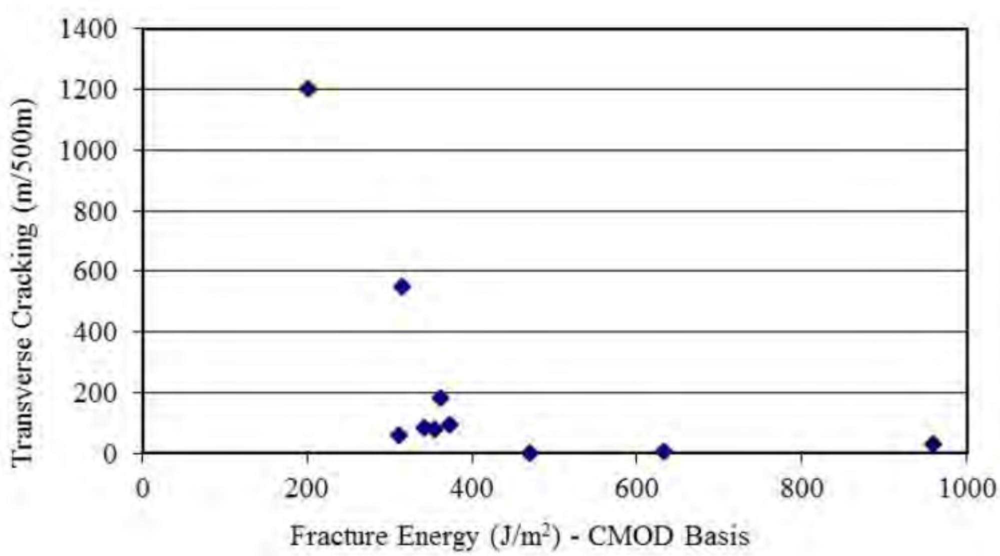


FIGURE 2 Transverse cracking (m/500 m) versus DC(T) fracture energy (*5*). [Note: Tests conducted at temperature equal to the Superpave PG low-temperature (PGLT) grade, rounded to the nearest grade at or exceeding a reliability level of 98%, plus 10°C.

TABLE 1 Original DC(T) Fracture Energy Thresholds from PFS #776 (*5*)

Contents	Project Criticality–Traffic Level		
	High (>20M ESALs)	Moderate (10–30M ESALs)	Low (<10M ESALs)
Fracture energy, min. (J/m^2), PGLT + 10°C	690	460	400
Predicted thermal cracking using Illi-TC (m/km)	<4	<64	Not required

NOTE: M = million; ESAL = equivalent single-axle loads.

using the Illi-TC simulation software, which in turn required creep compliance data and data regarding project location, such as geographical location and surface layer thickness. To develop those thresholds, it was originally determined that long-term aged fracture energy levels of 350 J/m^2, 400 J/m^2, and 600 J/m^2 would be required to achieve, low, moderate, and high reliability levels for thermal cracking protection. However, for practical purposes, it was decided to adjust the thresholds to allow short-term oven aged specimens to be used. Based on the results by Braham et al. (*33*), the values required in Table 1 were increased to account for the expected difference between field core fracture energy levels near the end of service life and short-term oven aged specimens (*35*). A shift factor of approximately 15% was used, resulting in the levels shown in Table 2. Anecdotally, in the early years of implementation of this specification, the high project criticality threshold requirement (typically applied to stone mastic asphalt (SMA) mixtures) of 690 J/m^2 is viewed by some as overly conservative, and generally requires the use of high-quality, hard aggregates and a sufficiently ductile binder–mastic system at low temperature. These values will be re-evaluated in the near future in light of additional performance data, such as that presented later in this paper.

Some Details Regarding Assembly of New Cracking Performance Data

Before presenting the updated cracking versus fracture energy plot, a few details regarding the assembly of the new database are first reviewed. Recent research studies have begun to incorporate the variable of pavement age while considering the transverse cracking performance of a pavement by defining various transverse cracking measures. Dave et al. (*42*) defined five transverse cracking measures (reproduced in Table 1) to better differentiate between pavements with seemingly similar performance. For example, two pavements could have same length of transverse cracking at the end of their service life, but one of them cracks in the first year and is stable thereafter, while the other only cracks towards the end of its service life. Both the pavements have similar performance if compared with the old method of measuring transverse crack lengths at the end of service life, but clearly, the latter (pavement that cracks substantially towards the end of its service life) has demonstrated better performance than the former. Each of the five measures defined by Dave et al. merits a different aspect of pavement performance in terms of transverse cracking (*44*).

TABLE 2 Description of Transverse Cracking Measure (*44*)

Maximum Total Transverse Cracking Amount (MTCTotal)	Maximum transverse cracking amount (low + medium + high) of all survey years for a pavement section normalized against number of years for which pavement section has been in service.	% cracking/year
Maximum Total Transverse Cracking Rate (MTCRTotal)	Maximum increase in total transverse cracking amounts (low + medium + high) between any two consecutive years of service.	% cracking/year
Average Total Transverse Cracking (ATCTotal)	Sum of total transverse cracking (low + medium + high) for every survey year of a pavement section normalized against number of years for which pavement section has been in service.	% cracking/year
Weighted Average Total Transverse Cracking (WATCTotal)	Total transverse cracking (low + medium + high) for every survey year of a pavement section is first normalized against the corresponding survey year. The sum of these values is then normalized against number of years for which pavement section has been in service.	% cracking/year/year
Total Transverse Cracking (TCTotal)	Sum of the total transverse cracking (low + medium + high) work over the service life. Total area is then normalized against the number of years for which pavement section has been in service.	% cracking

UPDATED CORRELATIONS BETWEEN DC(T) FRACTURE ENERGY AND FIELD CRACKING DATA

A comprehensive array of field section data was gathered in this study, as presented in Table 3. The sections encompass data from four different states: Illinois, Minnesota, Missouri, and Wisconsin. Inclusion of field sections from different states gives this study a broader geographical view on the transverse cracking performance of the pavements. Further, as seen from the table, the field sections include a variety of variables such as pavement layer configuration, binder type, aggregate source, and so on. More details on the field sections have been published by various agencies and researchers (*5*, *18*, *41*, *43–45*).

Building on the work of Marasteanu et al. (*5*, *18*) and Dave et al. (*45*), Figure 3 presents DC(T) fracture energy (J/m^2) versus transverse cracking (m/500 m), with the diameter of the plot points representing the age of the asphalt mixture. Table 4 provides details of the plotted mixture/pavement data. In this fashion, the age of the pavement section investigated (at the time of coring and distress surveying) can be visualized as the weight of the data point (displayed as semi-transparent 'bubbles'). Initially when the asphalt mixture is placed, the fracture energy of the section might increase to a small extent owing to the consolidation of the pavement under traffic, and for reasons discussed in an earlier section. As the pavement continues to age, the fracture energy drops and the propensity to incur thermal cracking increases. Hence, the age of the pavement has a direct relation with the reliability of the transverse cracking data (or the weight that the analyst should give to that data point). This implies that the smaller 'bubbles' have a higher probability to 'bubble' upwards with further

TABLE 3 Description of Field Sections

Section	Additional Mixture and Pavement Configuration Details
I-88 EB GTR PG 58-28	SMA[a], 33.9% ABR[c], 12.5 NMAS, lane mix
I-88 EB GTR PG 46-34	SMA, 33.9% ABR, 12.5 NMAS, lane mix
I-88 EB GTR PG 46-34 high ABR	SMA, 46.8% ABR, 12.5 NMAS, lane mix
I-88 EB ECR PG 58-28	SMA, 33.9% ABR, 12.5 NMAS, lane mix
I-88 EB ECR PG 46-34	SMA, 33.9% ABR, 12.5 NMAS, lane mix
I-88 EB ECR PG 46-34 high ABR	SMA, 46.8% ABR, 12.5 NMAS, lane mix
I-88 EB RMA PG 58-28	SMA, 33.9% ABR, 12.5 NMAS, shoulder mix
I-88 EB RMA PG 46-34	SMA, 33.9% ABR, 12.5 NMAS, shoulder mix
I-88 EB RMA PG 46-34 high ABR	SMA, 47.0% ABR, 12.5 NMAS, shoulder mix
TH10-PG 58-28	87.5 mm M/O[b]
CH10-PG58-28	37.5 mm on existing HMA
CH10-PG58-28	37.5 mm M/O
TH28-PG 58-34	75 mm M/O
TH28-PG 58-34	112.5 mm M/O
CH30-PG 64-34	150 mm M/O
TH220-PG58-28	75 mm M/O
I-294 NB, North of Cermak Toll	Mid ABR (31%) - HMA, Quartzite, PG 70-28 SBS, 50 mm surface
TH9-PG58-34	75 mm O/L[d] on FDR[e]
TH9-PG58-34	75 mm O/L on FDR
WI STH 73 PG58-28	subbase stabilized with asphaltic base course, 50 mm milled HMA base, 75 mm surface
I-90 WB Rt. 25	Mid ABR (33%) - HMA, Quartzite, PG 70-28 SBS, 45 mm surface
TH6-PG58-28	37.5 mm M/O
TH27-PG 58-28	75 mm M/O
TH27-PG 58-28	75 mm M/O
TH210-PG58-28	50 mm O/L on existing concrete
MnROAD 33 PG58-28	silty clay subgrade (1994), 300 mm crushed granite base (class 5), 100 mm HMA surface (1999)
US50_1	ABR (24.6%), PG64-22, 12.5 NMAS, 50 mm HMA surface (2011)
MnROAD 34 PG58-34	silty clay subgrade (1994), 300 mm crushed granite base (class 5), 100 mm HMA surface (1999)
MnROAD 35 PG58-40	silty clay subgrade (1994), 300 mm crushed granite base (class 5), 100 mm HMA surface (1999)

NOTE: SMA = stone matrix asphalt; M/O = mill and overlay; ABR = asphalt binder replacement; O/L = overlay; FDR = full-depth reclamation; and GTR = ground tire rubber.

Continued on next page.

TABLE 3 (*continued*) Description of Field Sections

Section	Additional Mixture and Pavement Configuration Details
I35-PG 64-28	100 mm M/O on existing concrete
MO 52_1	ABR (33.5%), PG64-22, 12.5 NMAS, 50 mm HMA surface (2010)
I-90 WB Rockford	Low ABR (14%) - HMA, Gravel, PG76-22 GTR[f], 50 mm surface
TH1-PG58-34	100 mm O/L on FDR
TH1-PG58-28	37.5 mm O/L on existing HMA
TH53-PG 58-28	37.5 mm M/O
TH212-PG70-34	100 mm SMA new construction
I-90 EB near Newburg Rd.	Mid ABR (36%) - HMA, Quartzite, PG 76-22 SBS, 50 mm surface
US63_2	ABR (29.9%), PG64-22, 12.5 NMAS, 50 mm HMA surface (2008)
TH113-PG58-28	37.5 mm O/L on existing concrete
TH113-PG58-34	125 mm O/L on FDR
Taxiway E, Greater Peoria Regional Airport	150 mm AC overlay on 190 mm existing AC pavement (P201) and 510 mm aggregate base (P154), Strip-type inlay system used for base isolation
MN 75 2 PG58-28	sand-gravel subgrade (1955), 300 mm crushed base (class 5), 70 mm recycle mix (32B), 50 mm recycle mix (42B), 50 mm HMA surface
MN 75 4 PG58-34	sand-gravel subgrade (1955), 300 mm crushed base (class 5), 62 mm recycle mix (32B), 2 50 mm HMA lifts
TH10-PG 64-28	100 mm M/O (sealed cracks)
TH10-PG 64-28	100 mm M/O (cracks not sealed)
US54_8	ABR (8.6%), PG70-22, 12.5 NMAS, 50 mm HMA surface (2006)
TH6-PG58-34	37.5 mm on existing HMA
TH6-PG58-34	112.5 mm O/L on FDR
TH2-PG58-34	100 mm O/L on existing HMA
MnROAD 19 PG64-22	silt clay subgrade constructed in 1992, crushed subbase (class 3), 200 mm HMA (AC-20)
MnROAD 03 PG58-28	silty clay subgrade constructed in 1992, crushed base class 5, 160 mm HMA (120/150)
US54_7	ABR (0%), PG64-22, 12.5 NMAS, 50 mm HMA surface (2003)
IL I-74 (AC-20)	lime stabilized subgrade, 390 mm 19mm mix AC-20 base, 38 mm AC-20 surface

NOTE: SMA = stone matrix asphalt; M/O = mill and overlay; ABR = asphalt binder replacement; O/L = overlay;
FDR = full-depth reclamation; and GTR = ground tire rubber.

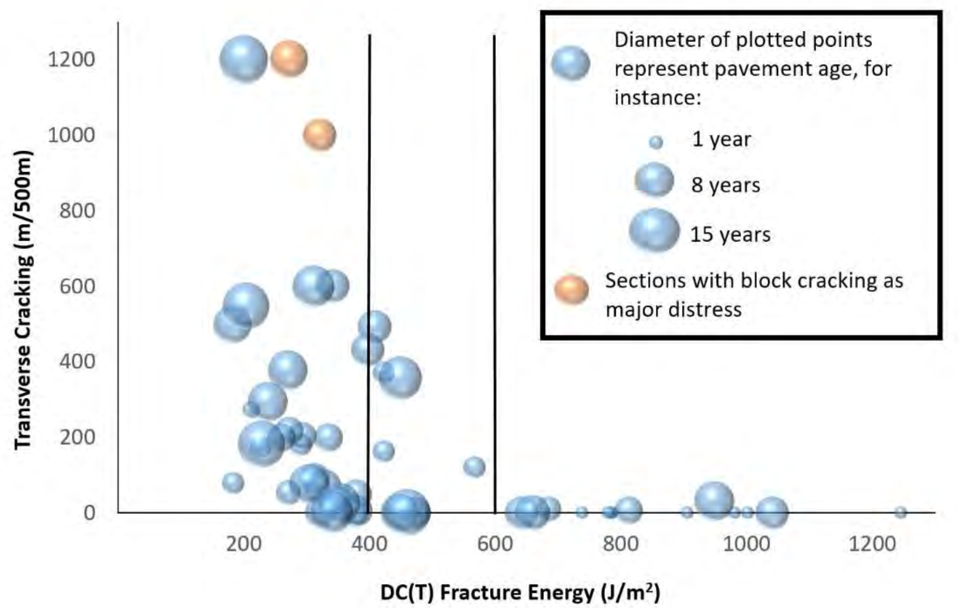


FIGURE 3 DC(T) fracture energy versus transverse cracking (field section I-88 GTR PG 46-34, having a fracture energy value of 2073 J/m^2 and zero transverse cracking, is not shown in the plot).

aging, while the larger bubbles are becoming stabilized in terms of long-term aged fracture energy and field cracking level.

It is important to mention here that the transverse crack length values include all severity levels. In the future, new analysis techniques could be developed so that the severity levels can be considered separately, by devising a system to convert different severity levels into a single parameter. In absence of such a system, all the cracks were counted towards the transverse crack length measure irrespective of severity levels for simplicity. Studies in the past have shown that this is a fair assumption considering the complexity of consolidating all the different levels of data into one single plot (*5*). Another assumption that should be checked in future research is the trajectory of the bubbles on the plot. By coring and testing field sections every year or two, it will be possible to study the progression of each field section on the plot. Current observations suggest that the small bubbles in the region of 700 J/m^2 will tend to stay anchored on the x-axis with time; e.g., their fracture energies may increase slightly at first, then drop with age, but the pavement section itself will not be expected to experience thermal cracking. On the other hand, the smaller bubbles that start at lower fracture energy levels will likely grow and 'float' upwards with time.

As seen in Figure 3, similar to the original data set displayed in Table 4, there is a clear trend in DC(T) fracture energy and transverse cracking. The data in the updated plot has a rectangular hyperbola shape; mixtures with fracture energy above certain threshold have low-to-medium transverse cracking, while mixtures with low fracture energy values tend to have thermal cracking levels that 'bubble upwards' with age. Marasteanu et al. (2012) recommended DC(T) fracture energy values according to traffic levels, as previously displayed in Table 1. Thresholds dividing the three reliability (traffic) levels for short-term aged specimens were set at

TABLE 4 DC(T) FE and Transverse Cracking Details of Field Sections

Section	Test Temperature (°C)	Fracture Energy (J/m²)	Transverse Cracking (m/500 m)	Age (in years)
I-88 EB ECR PG 46-34	-12	980	0	1
I-88 EB ECR PG 46-34 High ABR	-12	905	0	1
I-88 EB ECR PG 58-28	-12	785	0	1
I-88 EB GTR PG 46-34	-12	2073	0	1
I-88 EB GTR PG 46-34 High ABR	-12	1245	0	1
I-88 EB GTR PG 58-28	-12	785	0	1
I-88 EB RMA PG 46-34	-12	1001	0	1
I-88 EB RMA PG 46-34 High ABR	-12	779	0	1
I-88 EB RMA PG5 8-28	-12	738	0	1
TH10-PG 58-28 (H)	-24.2	212	273	2
CH10-PG 58-28 (F-1)		423	372	3
CH10-PG 58-28 (F-2)		423	162	3
CH30-PG 64-34 (L)		567	120	3
TH220-PG 58-28 (R)		183	78	3
TH28-PG 58-34 (K-1)		291	180	3
TH28-PG 58-34 (K-2)		227	174	3
I-294 NB, North of Cermak Toll	-12	685	10	4
TH9-PG 58-34 (E-1)		271	54	4
TH9-PG 58-34 (E-2)		352	36	4
I-90 WB Rt. 25	-12	812	6	5
TH210-PG 58-28 (P)	-24.8	293	202	5
TH27-PG 58-28 (J-1)		335	199	5
TH27-PG 58-28 (J-2)		272	216	5
TH6-PG 58-28 (C)	-24.2	260	199	5
WI STH 73 PG 58-28	-24.7	375	0	5
I35-PG 64-28 (M)		379	48	6
MnROAD 33 PG 58-28	-23.8	312	91	6
MnROAD 34 PG 58-34	-23.8	380	5.5	6
MnROAD 35 PG 58-40	-23.8	473	0	6
US-50_1	-12	322	1	6
I-90 WB Rockford*	-12	642	0	7
MO 52_1	-12	321	1000	7
TH1-PG 58-28 (A-2)	-26.3	342	600	7
TH1-PG 58-34 (A-1)	-26.3	408	493	7
TH212-PG 70-34 (Q)	-20.7	1040	0	7
TH53-PG 58-28 (N)	-25.7	397	432	7
I-90 EB near Newburg Rd.	-12	659	0	8
Taxiway E, Greater Peoria Regional Airport	-10	448	0	9
TH113-PG 58-28 (O-1)	-23.7	182	499	9
TH113-PG 58-34 (O-2)	-23.7	326	67	9
US-63_2	-12	272	1200	9
MN 75 2 PG 58-28	-24.4	304	76	10

Continued on next page.

TABLE 4 (*continued*) DC(T) FE and Transverse Cracking Details of Field Sections

Section	Test Temperature (ºC)	Fracture Energy (J/m²)	Transverse Cracking (m/500 m)	Age (in years)
MN 75 4	-24.4	948	30	10
TH10-PG 64-28 (G-1)	-24.2	270	378	10
TH10-PG 64-28 (G-2)	-24.2	238	294	10
TH6-PG 58-34 (D-1)	-24.2	311	600	11
TH6-PG 58-34 (D-2)	-24.2	352	24	11
US-54_8	-12	340	2	11
TH2-PG 58-34 (B)	-24.4	449	356	12
MnROAD 03 PG58-28	-23.8	228	182	14
MnROAD 19 PG64-22	-23.88	204	547	14
US-54_7	-12	459	1	14
IL I-74 (AC-20)	-16.4	200	1200	15

(yellow)	Data obtained from MnDOT PMS system.
(grey)	Data obtained via field observations.
(blue)	Data obtained from field images.
(green)	Data obtained from Google Maps images dated same/within a year of coring.
*	Google Map images from 2012, cored in 2015.

690 J/m^2 and 460 J/m^2. However, the authors noted that these values were for the short-term aged, lab-compacted specimens and that for field cores, the thresholds for acceptable long-term fracture energy levels would be lower (*5*). The thresholds for long-term aged, field-cored specimens were suggested to be 600 J/m^2 and 400 J/m^2. In the updated plot (Figure 3), mixtures with fracture energies in excess of 600 J/m^2 are indeed found to have very low transverse cracking.

Fifteen field section datasets fall in this threshold limit and none of them have substantial transverse cracking, indicating that mixtures with high DC(T) fracture energy correlate to higher low-temperature cracking resistance. Even as these pavements reach the end of their service lives, they should be expected to exhibit little to no transverse cracking. Most of the mixtures in this portion of the data set are newer SMA mixtures used on the Illinois Tollway. Even though many of these mixtures contain up to 50% binder replacement by the inclusion of RAP, RAS, and GTR, most were designed with the DC(T) test criterion to ensure a sufficiently soft and modified virgin binder grade to counteract the stiffening effects of the recycled materials. The mixtures below 600 J/m^2 but above 400 J/m^2 generally begin their service life with little to no transverse cracking. With pavement age, these mixtures begin to show non-negligible levels of transverse cracking, particularly those points closer to the lower threshold of 400 J/m^2. Accordingly, a wedge-shaped locus of points makes up the 400 to 600 J/m^2 fracture energy domain. Mixtures below the lower threshold of 400 J/m^2 tend to display higher levels of transverse cracking, and faster cracking rates. This portion of the data also contains some of the most aged sections in the dataset, primarily because the older pavements investigated were dense-graded Superpave mixtures containing RAP from the early 2000s, which tended to have lower asphalt contents and little to no binder bumping to account for the effects of RAP. Some of the other data points in this region were mixtures designed to pass the Hamburg wheel-track tests, but were not required to pass a mixture cracking test requirement.

Penn State researchers developed thermal cracking thresholds under the SHRP A-357 project during the development of TCModel. The classification according to the SHRP A-357 report is shown in Table 5 (*10*).

The cracking thresholds in Table 5 allow the data to be represented in a two-dimensional table according to categories of fracture energy and cracking level, as shown in Table 6. This analysis shows that only five of 30 sections having fracture energy values less than 400 J/m^2 have experienced zero cracking at the time of the survey. However, only two of those sections had service lives of more than 8 years. The remaining 25 of 30 sections in this category have low fracture energy and very high transverse cracking levels or cracking rates. Conversely, for the asphalt mixtures in the very high fracture energy category of more than 600 J/m^2, 14 of 15 sections exhibit zero cracking. At this time, only one of the 14 sections has a service life of more than 8 years. Continued monitoring will be performed in order to confirm that these sections continue to resist thermal cracking as the current trend strongly suggests.

It is not a surprise that mixtures with fracture energy ranges between 400 and 600 J/m^2 exhibit some scatter in terms of total cracking when viewed on this style of plot. One must consider the myriad of assumptions and simplifications that accompany a simple, single parameter based performance design approach such as this. These factors include differing pavement age (although the scaled bubble plot assists the analyst in accounting for this factor); lumping together cracking severity levels; differing climatic conditions; the statistical randomness of critical cooling events combined with the fact that different sections have different starting years and may or may not have experienced an unusually cold winter; the wide range of aggregates, binders, and recycled materials used [although the DC(T) appears to capture most of these variables reasonably well]; construction quality variability; and pavement section effects, including the possibility for the hybridization of cracking data with cracks caused primarily by the reflective cracking mechanism rather than true thermal cracking, etc.

TABLE 5 Classification of Thermal Cracking Thresholds (*10*)

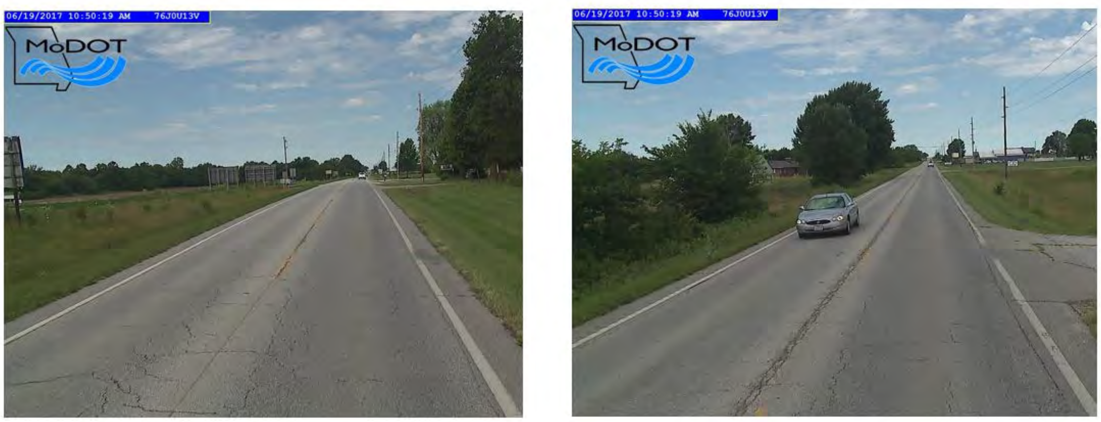
Zero Cracking	0 to 25 ft cracking per 500 ft section (<1 crack per 250 ft or <1 crack per 75 m)
Low Cracking	25 to 75 ft cracking per 500 ft section (from 1 crack per 250 ft to 1 crack per 85 ft or 1 crack per 75 m to 1 crack per 25 m)
Medium Cracking	75 to 150 ft cracking per 500 ft section (from 1 crack per 85 ft to 1 crack per 40 ft or 1 crack per 25 m to 1 crack per 12 m)
High Cracking	greater than 150 ft cracking per 500 ft section (>1 crack per 40 ft or >1 crack per 12 m)

TABLE 6 Grouping Data According to Cracking Thresholds (*10*)

	Number of Sections with			
Fracture Energy (J/m^2)	**Zero Cracking**	**Low Cracking**	**Medium Cracking**	**High Cracking**
<400	5(2)	4(1)	3(1)	18(8)
400-600	3(2)	0(0)	1(0)	4(1)
>600	14(1)	1(1)	0(0)	0(0)

NOTE: Numbers in parentheses indicate number of sections > 8 years of age.

The orange-colored plot points represent the two sections that contained block cracking as the major distress, i.e. sections US-631 and MO-521. The inclusion of block cracking sections is a new area of study, and herein, a rough engineering estimation was used to analyze block cracking distress in order to estimate the amount of transversely-oriented cracks contained. Some of the transverse cracking in these sections is clearly due to classic thermal cracking (as can be observed in Figure 5), while the majority of crack length comes from the portion of block cracking oriented approximately in the transverse direction. Note that the transverse cracking details for these sections were obtained from field images collected by an automatic road analyzer van, examples of which are shown in Figures 4 and 5. The MO-52 section was found to contain severe block cracking throughout. Conservatively, a 1.8 m (6 ft) transverse crack spacing was assumed in the calculations to emphasize the wider cracks present, although much of the area had block cracking dimensions between 0.3 to 1.2 m (1 to 4 ft), when medium and low crack severity levels were also included. The entire section investigated was measured to be 1,704 m (5,590 ft), and a total cracking level of 1,000 m/500 m was computed.



(a) **(b)**

FIGURE 4 MO-52 section with block cracking.

(a) **(b)**

FIGURE 5 US-63 section with block cracking.

US-63, by observation, had a higher amount of block cracking than the MO-52 section. From the previous estimation, it can be conservatively estimated that US-63 has experienced at least 1,200 m/500 m of transverse cracking. These numbers, by no means, reflect the exact amount of cracking. Rather, they provide a conservative starting point for including block cracking-dominated sections to be included in establishing DC(T) limits. Although more work is clearly needed, these two data points shown as orange bubbles on Figure 3 agree with the thermal cracking data—fracture energy levels well under 400 J/m^2 have led to high surface cracking levels. Continued data collection on additional sections will be pursued to determine if the DC(T) thermal cracking limits can be viewed as appropriate for the control of block cracking, or if different limits or different testing methods are needed to control this undesirable and unsightly distress mode.

CONCLUSIONS AND RECOMMENDATIONS

Data from 52 field sections located in the Midwestern United States were analyzed and summarized to evaluate the relationship between fracture energy as measured in the DC(T) to transverse cracking in asphalt pavement surfaces. The new data appears to validate the recommendations provided in 2012 in the final report of the National Pooled Fund Study on Low-Temperature Cracking. Based on the findings of this study, the following conclusions can be drawn:

1. Fracture energy measured at low temperature with the DC(T) is strongly correlated to low-temperature cracking;
2. The fracture energy thresholds recommended in the PFS for various traffic levels have been validated, and represent a good starting point for asphalt mixture design specification requirements.

Local calibration of these thresholds is recommended, as the highly simplified practice of linking a single, fundamental parameter to field performance may require regional calibration and other local considerations for implementation. These considerations would include differences in climate (number annual critical cooling events, cooling rate, aging of the asphalt surface), differences in pavement structure and traffic effects, and availability and effects of varying aggregate, binder, and recycled materials (although these affects appear to be largely captured by the DC(T) test). Some agencies are investigating the possibility of lowering the fracture energy threshold for high traffic, especially in areas where it is difficult to obtain aggregates with very high strength. The development of a statistically derived, reliability-based approach for various traffic levels is the focus of current research, and will assist agencies in this regard once completed. Research on block cracking and the inclusion of this distress mode in a DC(T) test specification is also underway, and appears to be promising based on the results presented herein, although currently limited to two observations.

Implementation of the DC(T) test along with the Hamburg wheel-tracking test in a PEMD is underway or has been recently completed by several agencies, including the Illinois Tollway. Validation and adjustment of the specification based on reliability, pavement layer type, and pavement layer depth is the focus of ongoing work, as is the evaluation of other mixture performance tests, aimed at controlling other distress modes.

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

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INTRODUCTION

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

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

BALANCED MIXTURE DESIGN

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

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

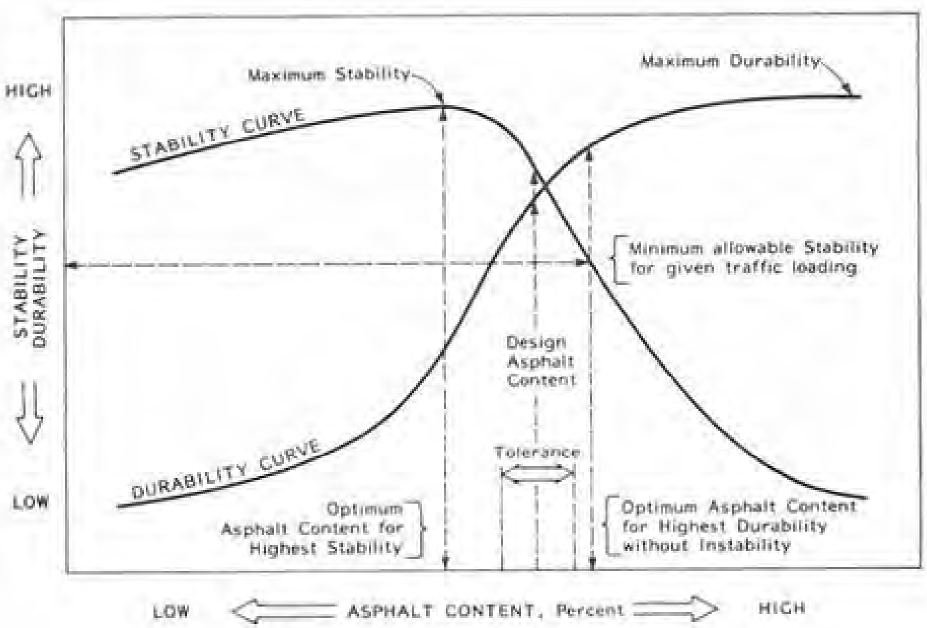

FIGURE 1 Importance of balancing stability and fatigue durability (*1*).

In 2006, the Texas Transportation Institute (TTI) re-introduced the concept of a BMDin their report, *Integrated Asphalt (Overlay) Mixture Design, Balancing Rutting and Cracking Requirements* (*2*). The researchers utilized the wet Hamburg wheel-tracking device to index rutting resistance while indexing the fatigue cracking performance of asphalt mixtures with the overlay tester (OT). Over the past few years, the Texas DOT and TTI had generated a significant database of laboratory test performance that had been correlated to observed field performance with these laboratory tests and believed they could be utilized to verify asphalt mixtures during design. Their general methodology was as such (Figure 2).

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

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

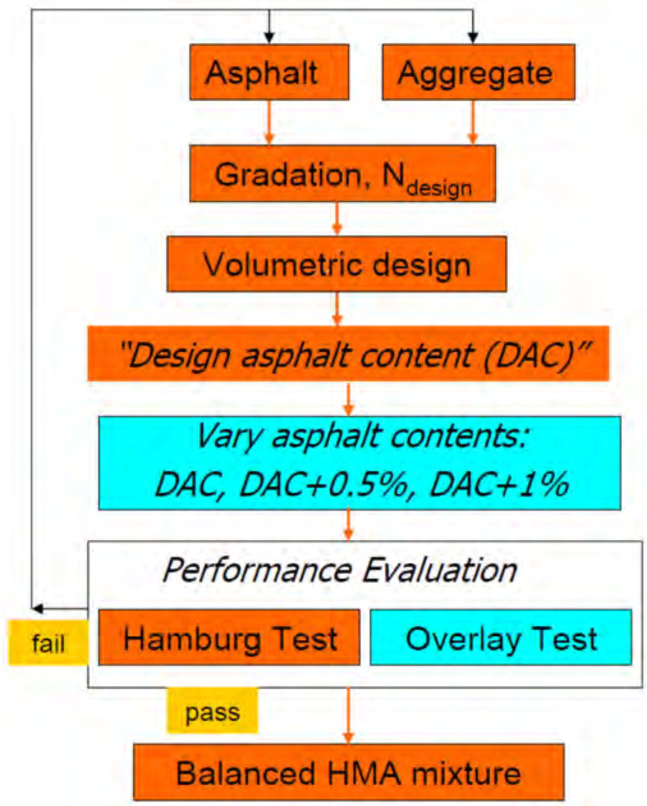

FIGURE 2 Texas DOT BMD concept (*3*).

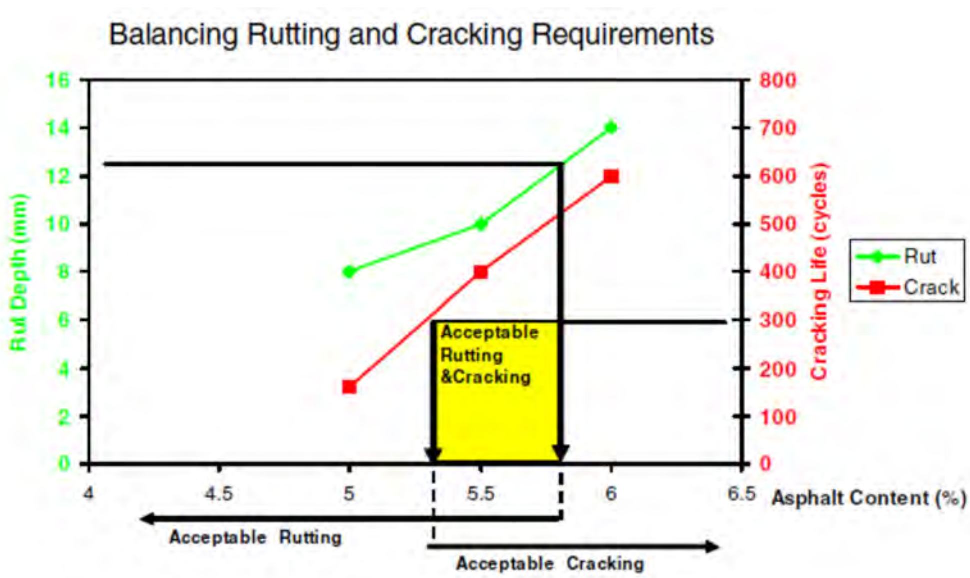

FIGURE 3 BMD results (*2*).

NEW JERSEY'S BALANCED MIXTURE DESIGN APPROACH

In the Texas DOT BMD procedure, Texas DOT prefers to utilize the wet Hamburg wheel-tracking device to assess rutting potential. However, the New Jersey DOT has had a long history of using the asphalt pavement analyzer (APA) as a test to evaluate rutting potential, and therefore, it is utilized in the New Jersey DOT's BMD. The OT was also selected for the New Jersey DOT balanced design due to its ability to trend with field performance, especially when recycled asphalt pavement (RAP) is used.

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

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

Materials–Mixture Design

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

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

Table 1 and Table 2 show the aggregate gradations and OAC for the New Jersey DOT-approved mixtures.

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

TABLE 1 New Jersey DOT-Approved 9.5-mm NMAS Mixtures

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

TABLE 2 New Jersey DOT-Approved 12.5-mm NMAS Mixtures

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

Tilcon, Mt. Hope Mixtures

9.5-mm Nominal Maximum Aggregate Size Mixtures The resultant mixture performance for the 9.5-mm NMAS mixtures are shown in Figure 4 for the 9.5M64 and Figure 5 for the 9.5M76 mixtures, respectively. For the 9.5M64 mixture, the balanced design shows that the optimal range in asphalt content to achieve both good rutting and fatigue cracking properties is 5.2% to 5.9% asphalt content. Meanwhile, the balance design results for the 9.5M76 asphalt mixture indicates that an asphalt content range of approximately 5.1% to 5.6% would result in an asphalt mixture with good rutting and fatigue resistance. Both of the balanced design results indicate that the volumetric-based design results in under-asphalting the asphalt mixture.

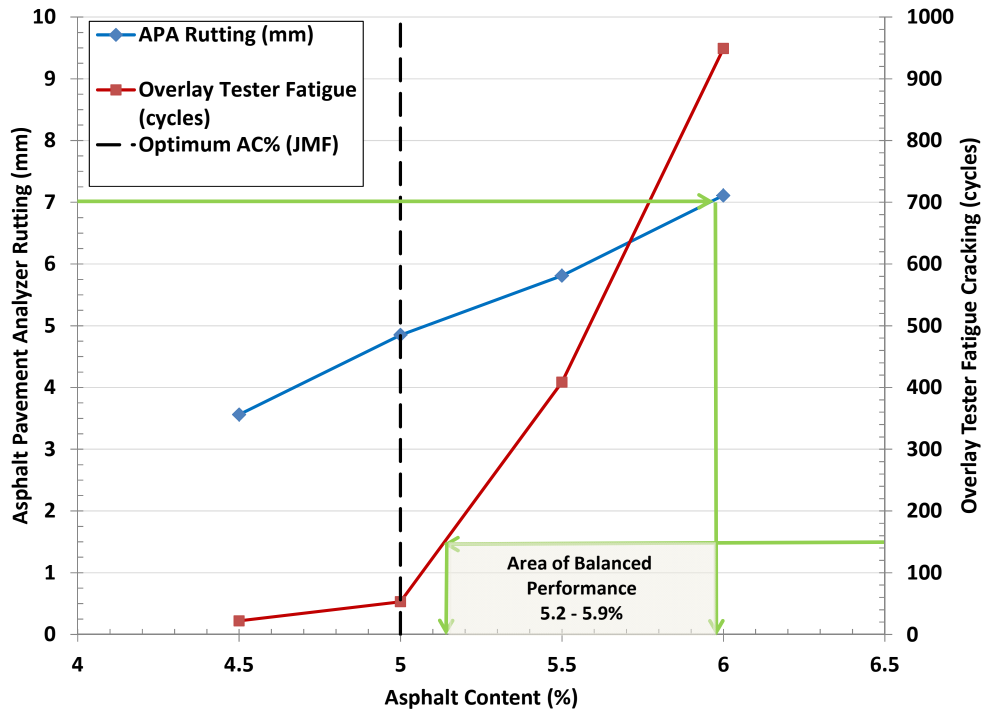

FIGURE 4 Tilcon Mt. Hope 9.5M64 balanced performance versus asphalt content. (JMF = job mix formula.)

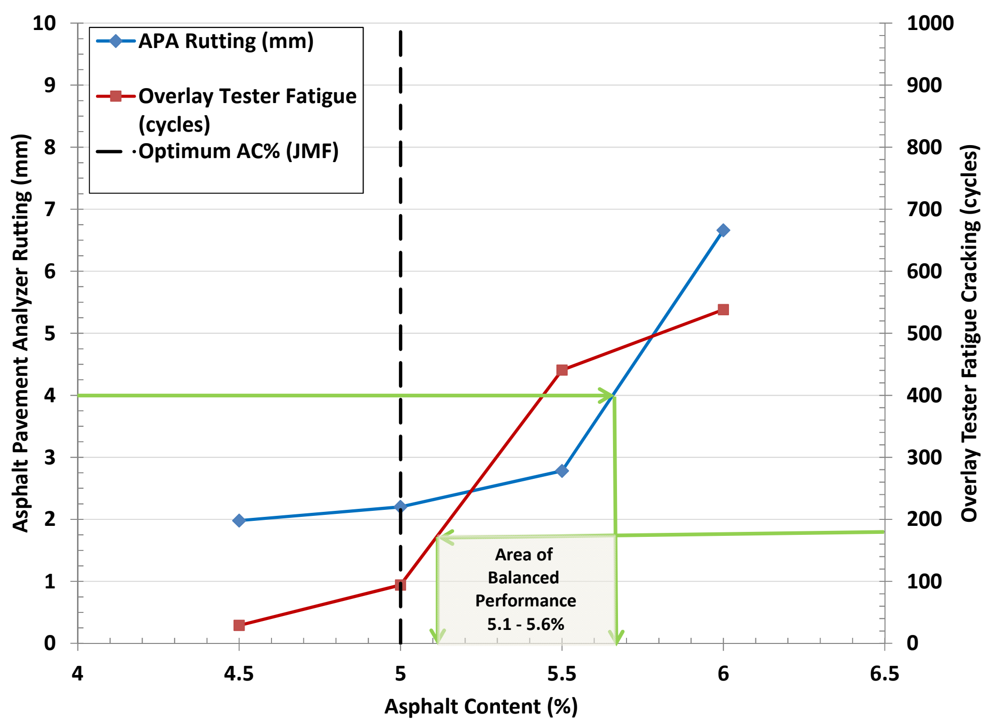

FIGURE 5 Tilcon Mt. Hope 9.5M76 balanced performance versus asphalt content.

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

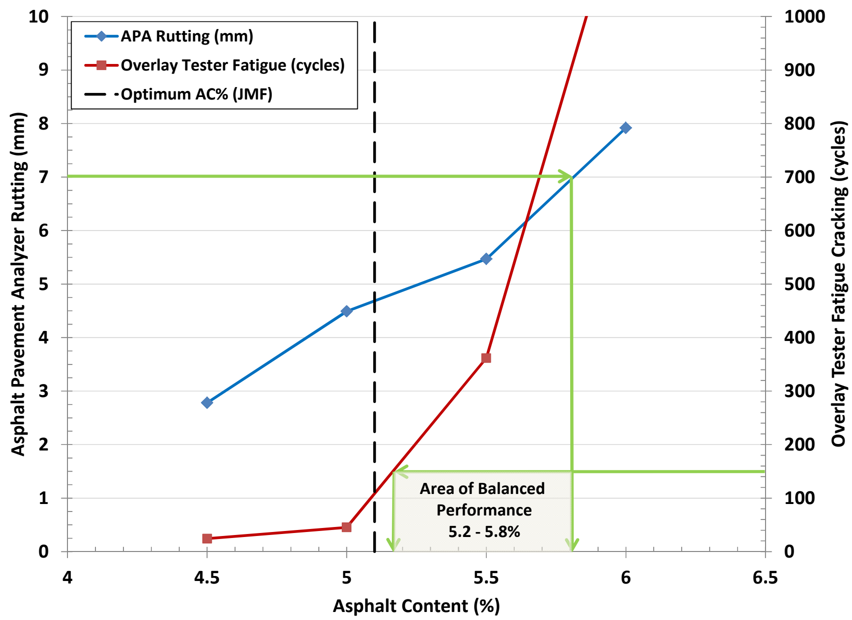

FIGURE 6 Tilcon Mt. Hope 12.5M64 balanced performance versus asphalt content.

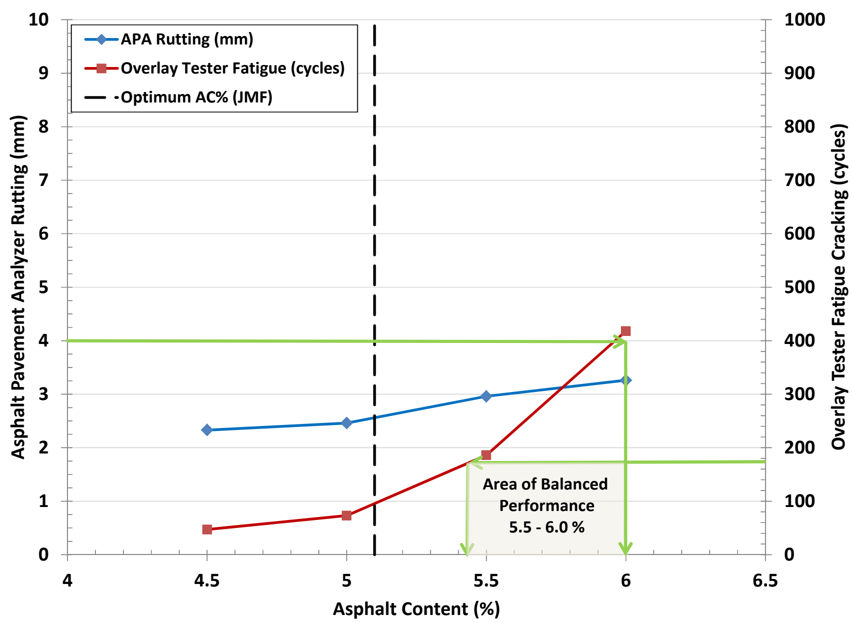

FIGURE 7 Tilcon Mt. Hope 12.5M76 balanced performance versus asphalt content.

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

Trap Rock Industries Mixtures

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

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

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

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

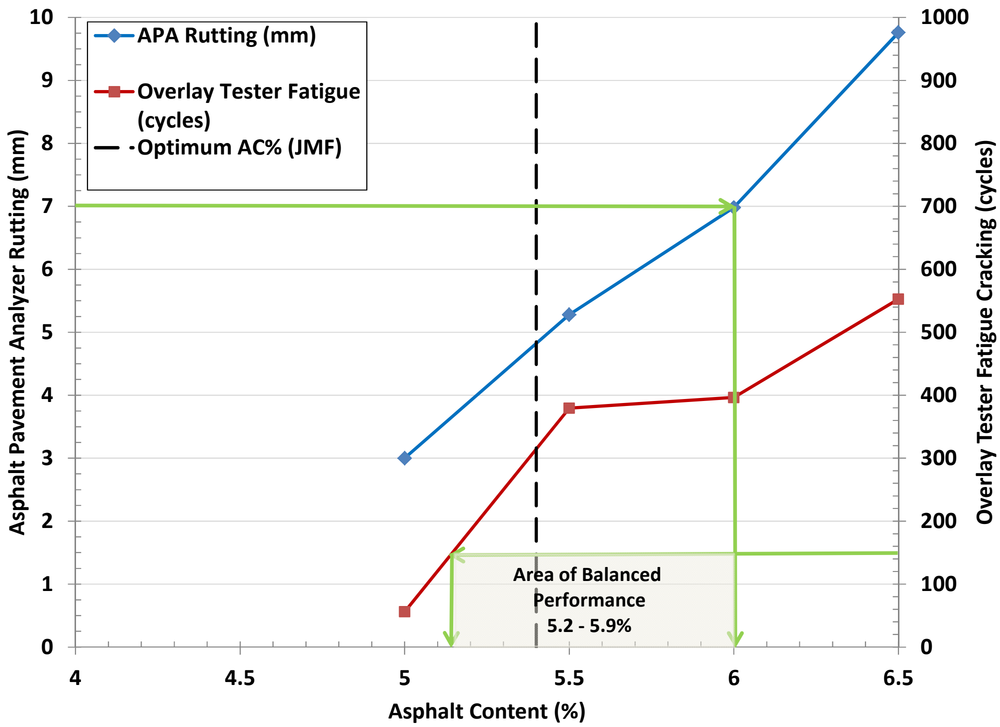

FIGURE 8 TRI 9.5M64 balanced performance versus asphalt content.

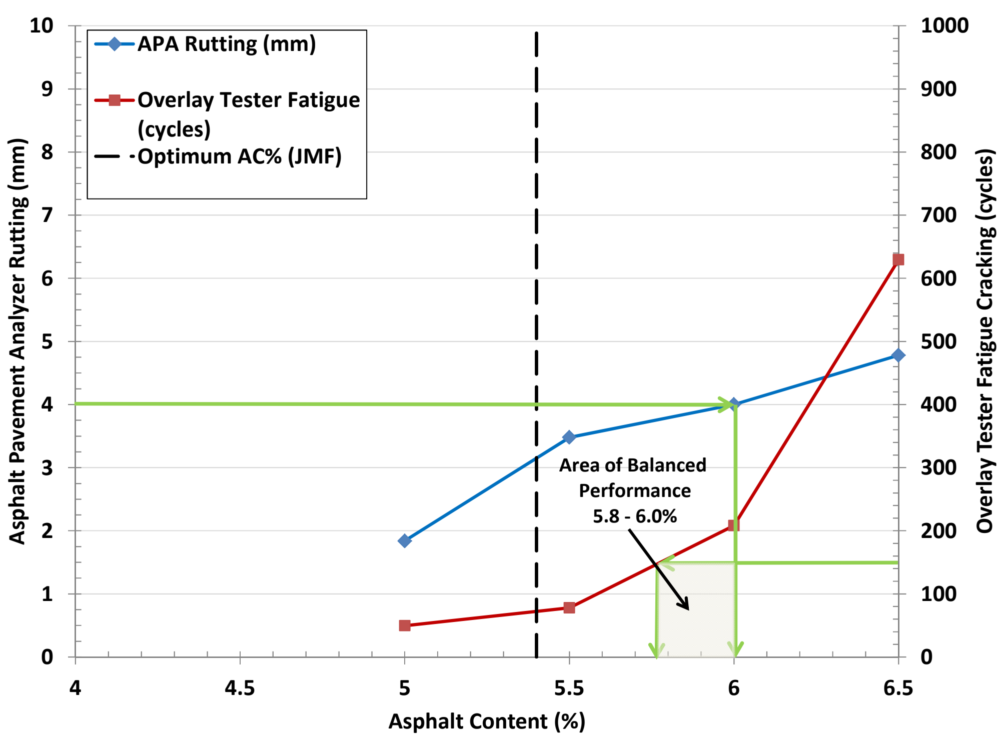

FIGURE 9 TRI 9.5M76 balanced performance versus asphalt content.

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

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

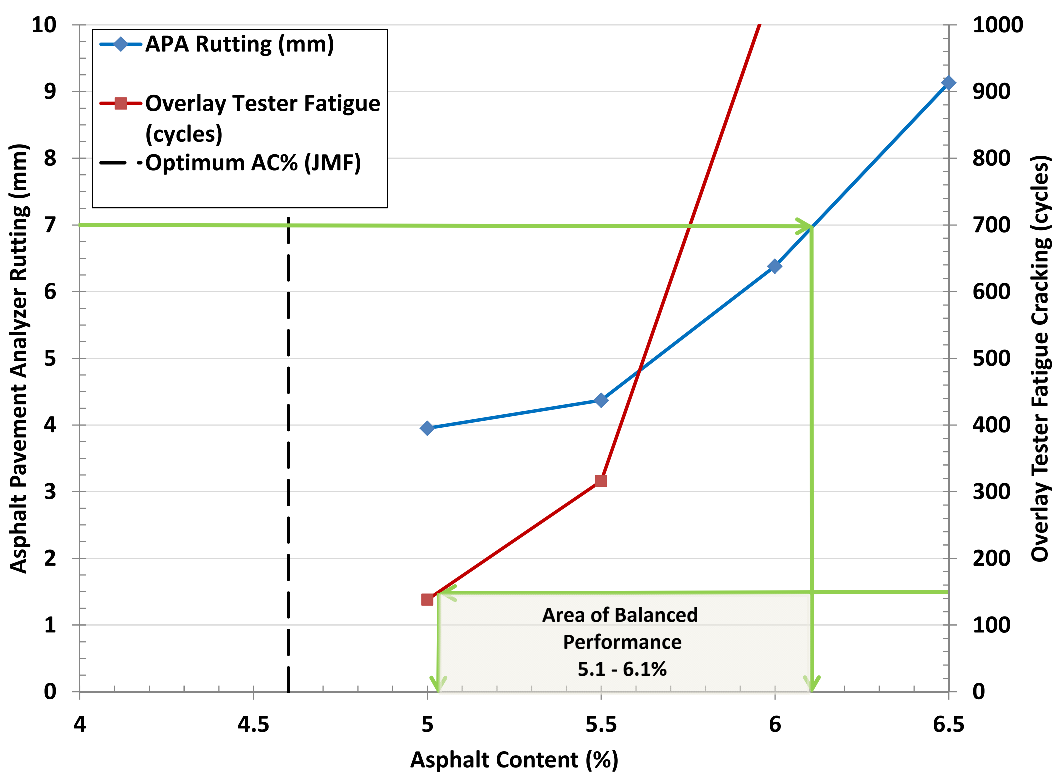

FIGURE 10 TRI 12.5M64 balanced performance versus asphalt content.

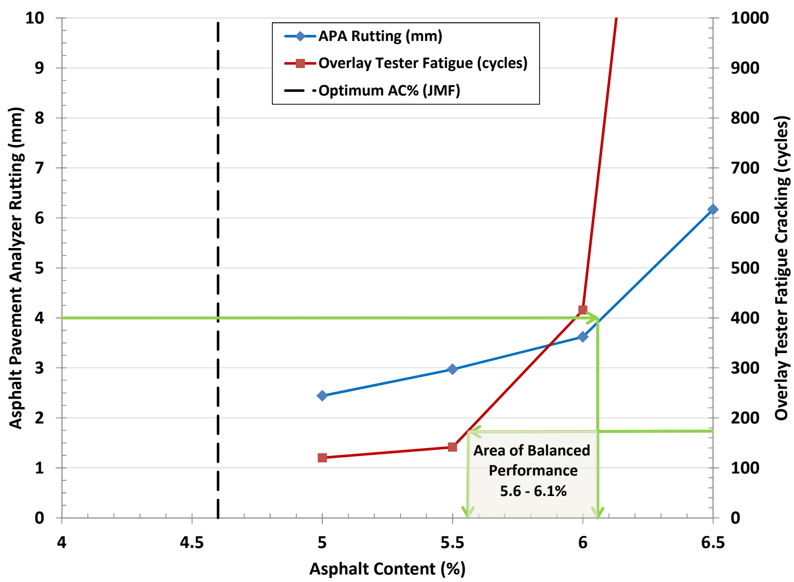

FIGURE 11 TRI 12.5M76 balanced performance versus asphalt content.

TABLE 4 Summary of Determined Asphalt Contents for TRI Asphalt Mixtures

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

USING BALANCED MIXTURE DESIGN TO IMPROVE VOLUMETRIC SPECIFICATIONS

Durability in asphalt mixtures is a very broad characteristic, but it generally covers the asphalt mixtures' ability to resist cracking, raveling, and brittle-type failures. For years, pavement engineers and asphalt material technicians have utilized the volumetric property voids in mineral aggregate (VMA) as a general "durability index" parameter. In general, the higher the VMA at design, the greater the amount of effective asphalt (by volume), the better the durability of the asphalt mixture. Finer aggregate gradations, with increased surface area, require higher levels of VMA to ensure adequate asphalt film thickness around the aggregates occur. The current New Jersey DOT volumetric requirements for asphalt mixture design are shown in Table 5. Unfortunately, since the VMA is comprised of both effective asphalt content and air voids, established criteria for VMA only have meaning during mixture design and asphalt plant quality control (QC) testing. Therefore, instead of utilizing VMA as an indicator of durability, it was proposed in this study to look at the effective binder content by volume (EBCV). Since the EBCV does not change like VMA due to varying air voids, the EBCV is a much more stable parameter and can be easily evaluated and compared to during a BMD approach.

TABLE 5 New Jersey DOT Asphalt Mixture Design Volumetric Requirements

Table 902.02.03-3 HMA Requirements for Design

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

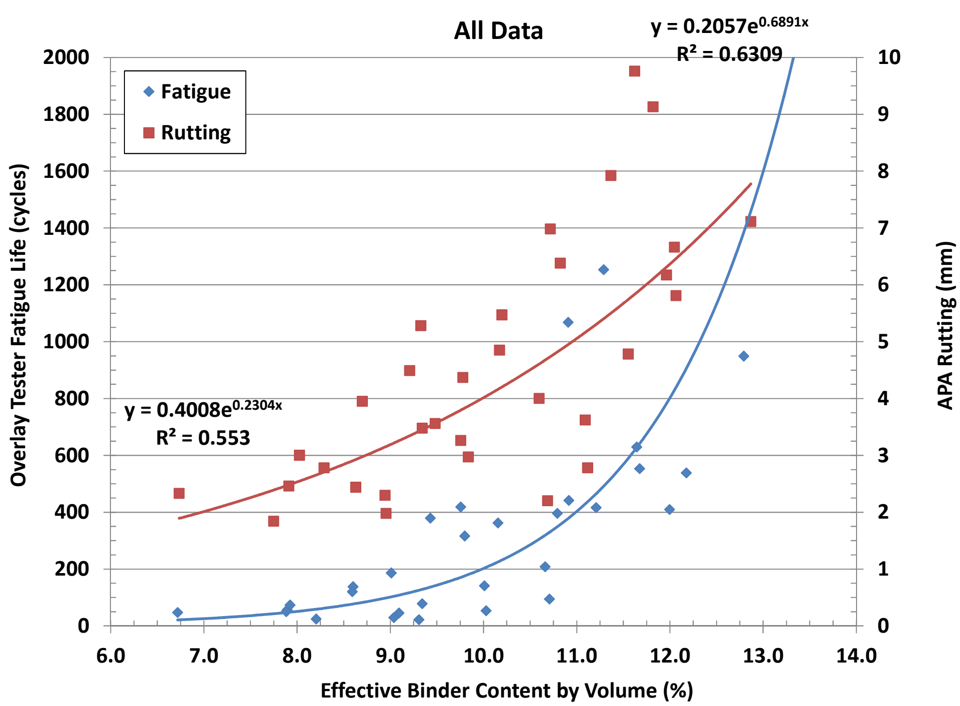

FIGURE 12 EBCV (% versus balanced design performance—all test data.

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

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

Both the 9.5-mm NMAS and 12.5-mm NMAS mixtures were separated out, along with the PG grade of the asphalt binder in Figures 13 and 14, to illustrate where the performance of

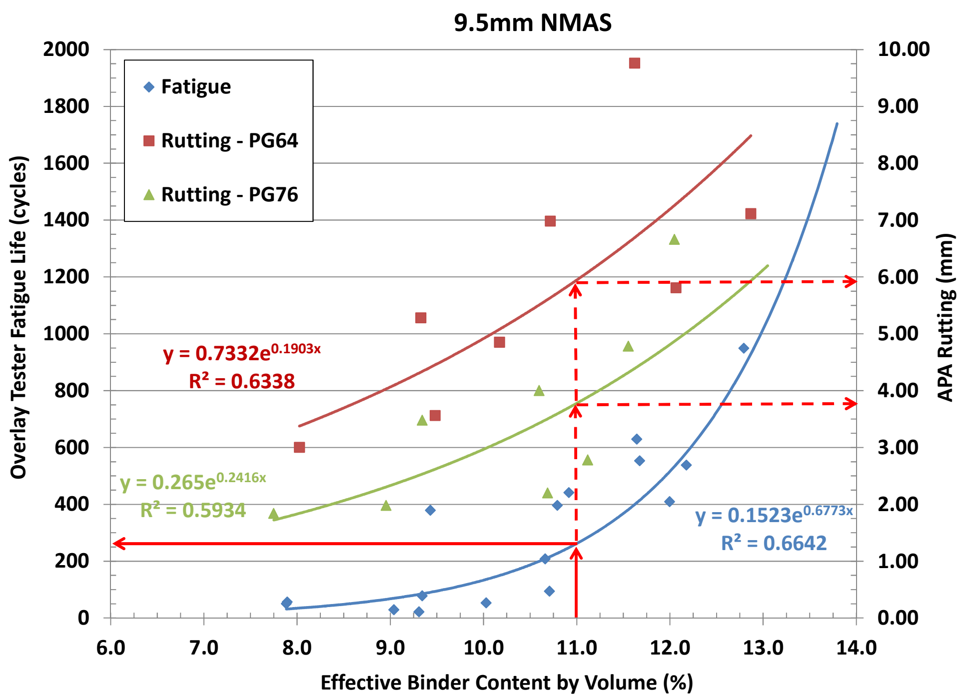

FIGURE 13 EBCV (%) versus balanced design performance—9.5-mm NMAS with current New Jersey DOT design VMA criteria.

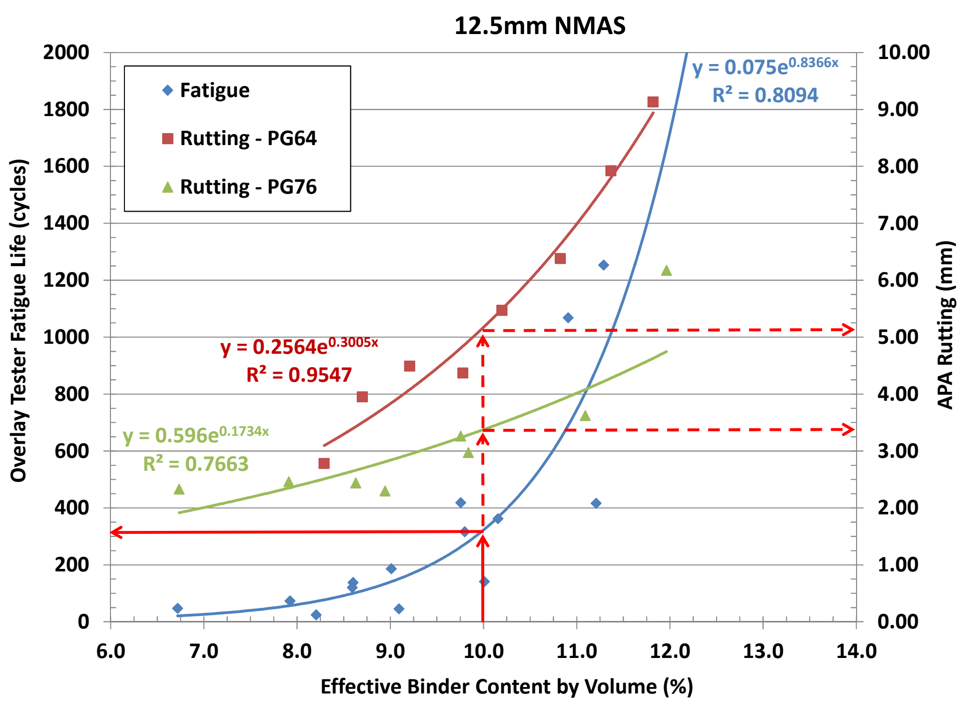

FIGURE 14 EBCV (%) versus balanced design performance—12.5-mm NMAS with current New Jersey DOT design VMA criteria.

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

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

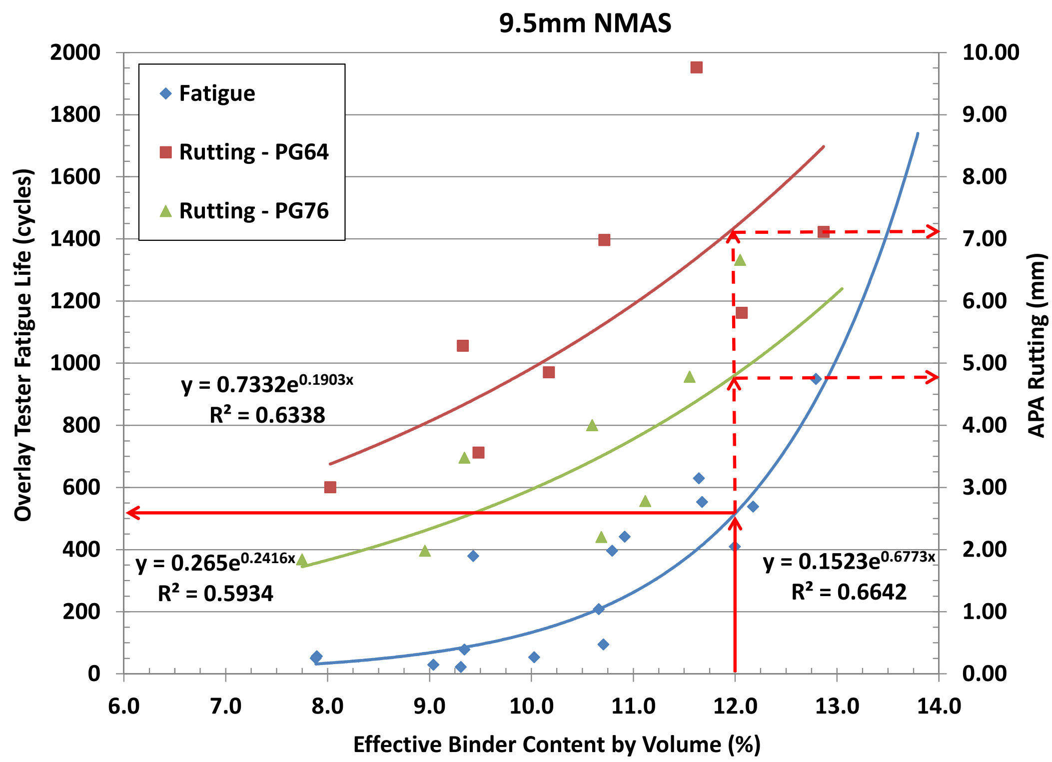

FIGURE 15 EBCV (%) versus balanced design performance—9.5-mm NMAS with proposed New Jersey DOT design VMA criteria.

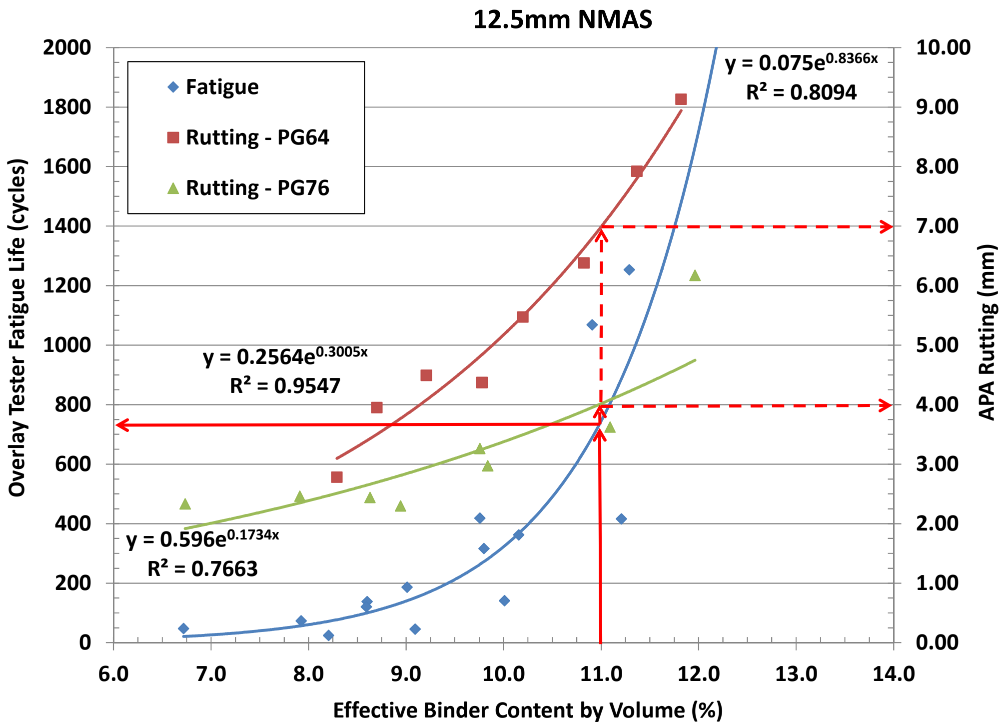

FIGURE 16 15 EBCV (%) versus balanced design performance—12.5-mm NMAS with proposed New Jersey DOT design VMA criteria.

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

SUMMARY OF BALANCED MIXTURE DESIGN WORK FOR NEW JERSEY

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

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

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

FUTURE NEEDS TO IMPLEMENT BALANCED MIXTURE DESIGN

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

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

Regarding production tolerances, would a state agency target the center of the balanced range and use the balanced range as tolerances? Should current agency production tolerances be incorporated using the "balanced" asphalt content?

A few things that agencies need to consider before moving towards BMD. However, the methodology does seem to provide an improvement over current volumetric procedures, especially when considering how to enhance durability and fatigue resistance of asphalt mixtures.

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