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New Simple Performance Tests for Asphalt Mixes

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New Simple Performance Tests for Asphalt Mixes

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Chrysa Cullather, Editor; Jennifer Correro, Proofreader; Jennifer Weeks, Layout; Javy Awan, Production Editor

Foreword

Two concerns were raised at the 1995 meeting of the TRB General Issues in Asphalt Technology Committee about the new Superpave® volumetric mix design method. First, although the method includes a mixture analysis system, it is too complicated and time-consuming for use in routine mix design work and a simple, practical test should be substituted for validating hot-mix asphalt (HMA) performance; second, the Superpave volumetric mix design procedure does not define the effects of binder stiffness on mix performance. It essentially relies on limits on the compaction curve to define a good aggregate structure and minimum voids in mineral aggregate to provide durability. Committee members believed that a performance test to complement this procedure would allow the evaluation of the total mix, including the effects of binder. It would also allow the designer to determine how much binder a mix could tolerate before it becomes unstable. Indirectly, this test could aid in producing more durable mixes.

The objective of this circular was to identify practical and reliable laboratory tests that could be considered for ranking the rutting potential of HMA paving mixtures. A practical test usually can be performed in less than 24 hours (according to generally accepted industry standards) at a reasonable cost by laboratory personnel with a normal amount of training. This also means that the equipment should be affordable (less than $60,000) and not require elaborate setup. A reliable test must show a reasonable correlation to field performance (i.e., it could be used to establish realistic pass–fail criteria).

The Federal Highway Administration (FHWA) initiated research in 1996 to identify and evaluate a simple test method. In 1999, NCHRP Project 9–19, Superpave Support and Performance Models Management, continued the effort to examine “simple performance tests.” The research team was directed to evaluate “only existing test methods that measure hot-mix asphalt (HMA) response characteristics. The principal evaluation criteria were (*a*) accuracy (i.e., good correlation of the HMA response characteristics to actual field performance); (*b*) reliability (i.e., a minimum number of false negatives and positives); (*c*) ease of use; and (*d*) reasonable equipment cost.” Furthermore, the scope of the project was expanded to characterize not only rutting in HMA but also fatigue and low-temperature cracking. A summary of the work to identify the simple performance tests selected for field validation was published as *NCHRP Report 465: Simple Performance Test for Superpave Mix Design.*

In addition to the NCHRP-sponsored work, efforts to search for new tests that might satisfy the concerns of the Committee on General Issues in Asphalt Technology have continued. At the 2002 Transportation Research Board Annual Meeting, the committee sponsored a session to explore new tests that might meet these goals, that is, to find “a practical and reliable lab test which could be used to rank the rutting potential of an asphalt mix.” The presenters and their coauthors have prepared papers on their work, which are included in this circular. In addition, a paper by Brown and colleagues is included; it offers a summary of the literature on performance tests. Although all of these devices need some additional work (e.g., validation, standardization), they generally could meet the goal of being practical and potentially could be shown to be reliable.

Contents

Development of Mixture Creep Performance Tests Using a Dynamic Shear Rheometer

GERALD REINKE
STACY GLIDDEN
MTE Services, Inc.

A hot-mix asphalt (HMA) mixture performance test has been developed using a constant stress dynamic shear rheometer. The test is conducted using rectangular bars of mix, which are cut from either field cores or gyratory-compacted HMA specimens with a diamond-tipped saw. The bars are nominally 50-mm long, 12-mm wide, and 10-mm thick. Specimens are tested using the solids testing fixture provided by the rheometer manufacturer. Specimen test temperature is maintained by air circulation through a temperature-controlled oven that surrounds the specimen. Two types of creep tests were developed: a static creep test and a repeated creep recovery test. The static creep test uses the application of a rotational stress to one end of the long dimension of the specimen while the other end is held rigidly in place. As the stress is maintained at a constant value, the rectangular specimen undergoes a strain, which is monitored to failure. The repeated creep and recovery test uses the same equipment and specimen shape. A stress is applied to the specimen for 1 s followed by a 9-s rest period of zero stress during which time the specimen recovers some of the strain developed during the 1-s stress period. Repeated stress and relaxation cycles are applied to the test specimen up to 2,000 complete cycles or until specimen failure occurs. From these two creep test procedures, several types of results have been obtained, including time to 5% strain, time to failure, strain at failure, strain at 100 cycles, and mix viscosity at 100 cycles. Test data generated on specimens cut from field cores have been shown to correlate well (R^2 > 0.9) to the field rutting behavior of test track mixes placed at Minnesota Test Road at the Minnesota Department of Transportation research site and at the FHWA Accelerated Loading Facility test track in Sterling, Va. The time required to perform the creep tests varies with mix quality, stress level applied, and test temperature. It is believed, on the basis of work to date, that failure criteria data can be established that would enable this procedure to be used as a mix design tool. In addition, the test is easy and rapid enough to perform that quality control monitoring of HMA mixes being placed in the field could be performed and the results can be available within 24 h.

The Strategic Highway Research Program (SHRP) ended in 1993 without finalizing work on a predictive mixture performance test procedure. The SHRP shear tester (SST) and a suite of mixture performance tests described in AASHTO TP9 were provided; however, these tests were expensive to perform and the test procedures and analysis methods were not well defined. As a consequence, the models have had limited success in working with the materials and mixes that have evolved over the past 11 years since the SHRP research program ended. Within that period, a new FHWA contract was issued to reevaluate the SHRP models. That work has been ongoing for the past 6 years and has not been without controversy within the pavement research community. While the reevaluation of the SHRP performance models has been taking place, several research efforts have been mounted to develop what have generically been termed "simple performance tests."

The main goal of developing a simple performance test has been to provide the owner agencies and construction communities one or more tools for evaluating the performance potential of hot-mix asphalt (HMA) mixtures currently being placed. The SHRP research program did an excellent job in providing new tools and test methods for evaluating the performance potential of asphaltic binders. In addition, the volumetric mix design methodology using gyratory compaction provided by SHRP researchers has proven to be a significant improvement compared with the Marshall volumetric procedure. Although SHRP researchers made test procedures available to enable use of the SST to evaluate mixtures for performance potential, there were several problems with implementation:

1. The SST carried a $300,000 price tag; consequently, few units were sold, especially to state departments of transportation (DOTs).
2. The so-called Level 2 and Level 3 mix designs took from 14 to 30 days to complete, and the predictive results were controversial almost from the outset.
3. The test results obtained from the SST were not easily correlated to an understanding of how a specific result related to field performance. Several studies have attempted to provide this type of information (*1–4*); however, with a limited number of shear testers in service, there has not been much opportunity to have an effect on the paving community.
4. Level 2 and Level 3 mix designs were meant to predict long-term performance. However, DOTs and contractors wanted to know the answers to short-term questions, such as, "Is Mix A with Binder A likely to perform better or worse than Mix B with Binder B?" "What are the chances that this pavement will be acceptable a year (or 2 or 3 years) from now?"

The desire by user agencies for some basis on which to determine the likelihood that a given project would perform has led to several approaches for providing a "comfort level." Many states opted to add so-called SHRP Plus requirements to the performance-graded (PG) binder specifications they adopted. These additional binders tests, such as force ductility, elastic recovery, low-temperature ductility, and toughness and tenacity were holdovers from pre-SHRP polymer-modified asphalt specifications. Part of the rationale for inclusion of these tests has been that, in the absence of any kind of mixture performance test, the SHRP Plus requirements would force suppliers to provide binders with a known performance history. Through this reasoning, the binder specification has become a surrogate mixture performance specification. In addition to polymer-specific SHRP Plus specifications, some agencies began investigating the use of a variety of wheel tracking test devices. Among these are the French wheel tracking device, the Hamburg rut tester, and the asphalt pavement analyzer (APA). At this time, the APA has probably achieved the greatest following within the United States. This is not surprising because it is "homegrown," having evolved from Benedict's loaded wheel tester for slurry seal to the Georgia loaded wheel tester and now to a commercial piece of equipment. The goal of all these rut test devices or torture testers is to answer the question posed earlier—how well does one mix compare with another when tested under the same conditions.

In addition to comparing one mix to another, a useful mechanistic test should be able to measure fundamental strength or mechanical properties, which will distinguish mixes that perform well from those that do not. Ideally, a simple performance test should enable the operator to run the test at a given set of conditions and, from the results, to be able to predict the mixture's performance for a given set of climatic and loading conditions. Such a test should be relatively easy to perform, although a "simple" performance test need not be a simplistic test.

DEVELOPMENTAL BACKGROUND

The development of a mixture performance test using a dynamic shear rheometer (DSR) resulted from work being performed using other test procedures. For a number of years, we have performed mixture fatigue tests using a DSR following procedures developed by Goodrich (*5*) and Reese (*6*). More recently, data have been presented showing that frequency sweep tests performed on the DSR with mix slices are capable of producing the same complex shear modulus (G*) compared with temperature master curves as those obtained through frequency sweep testing performed on the SST (G. Reinke. PowerPoint presentation at FHWA Binder Expert Task Group meeting in Tampa, Florida, Nov. 14, 2000.). As a result of these successful efforts, there was confidence that rational creep test results could be obtained using small mixture slices. In January of 2001, our laboratory started to perform a repeated creep test on binders following a procedure developed by Bahia during the NCHRP 9–10 research program (*7, 8*). Bahia's repeated creep test was designed to determine the cumulative strain imparted to a binder after 100 cycles of testing with each cycle consisting of 1 cycle of stress application and 9 s of zero stress. The SST Repeated Shear at Constant Height (RSCH) test uses a 0.1-s loading period followed by a 0.6-s zero stress period, as does work recently reported by Witczak et al. in the development of a simple performance tester (*9*). It therefore appeared a natural extension of the repeated creep testing of asphalt binders to use the DSR to perform a repeated creep test on mixture specimens.

TEST DEVELOPMENT

After an initial series of tests to determine that the rheometer (a TA Instruments AR-2000) could in fact apply the desired stress level of 68 kPa without destroying the test specimens within the first minute, it became clear that 100 test cycles were inadequate for performing mixture tests. As an alternative, an investigation was launched into the use of a static creep test using the rheometer. At the same time, in cooperation with TA Instruments, a repeated creep test of up to 2,000 test cycles was developed. Either a 150-mm diameter gyratory pill or a 150-mm diameter core cut from an existing pavement was used to prepare test specimens. The gyratory specimen or core was mounted in a specially built holder (Figure 1), the top 5 to 6 mm were removed, and three 12-mm thick slices were cut from the gyratory specimen (or core) (Figures 2 and 3). From each 12-mm thick × 150-mm diameter slice, a 50-mm wide section (Figure 4) was cut. From that 50-mm wide × 150-mm long section, the 6-mm and 10-mm wide test articles (Figures 5 and 6) were cut. If the test articles were being cut from a gyratory pill, only the central 100-mm of the 50-mm × 150-mm slice were used. If the test articles were cut from a field core, the entire slice was used.

Whether a static creep or repeated creep and recovery test were conducted, the specimens were mounted in the solids testing fixture of the rheometer, and a rotational stress was applied (Figure 7). Initially, specimens 6-mm thick, 12-mm wide, and 50-mm long were used. However, as coarser mixtures began to be investigated, it became clear that a 10-mm-thick specimen was more practical. At this point the test has evolved to the point where only10-mm-thick

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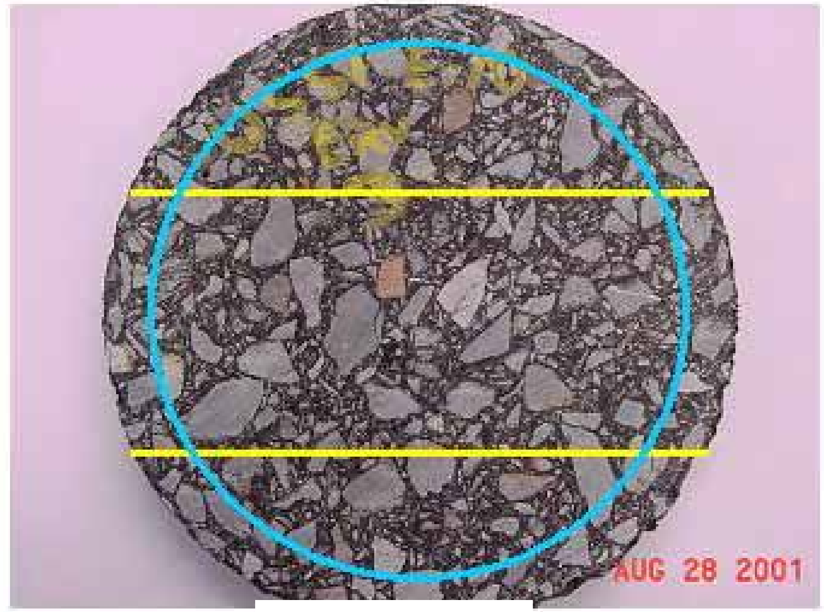

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FIGURES 1 through 4 Initial steps in cutting gyratory pill to obtain test specimens.

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FIGURES 5 and 6 Cutting test specimen from rectangular section.

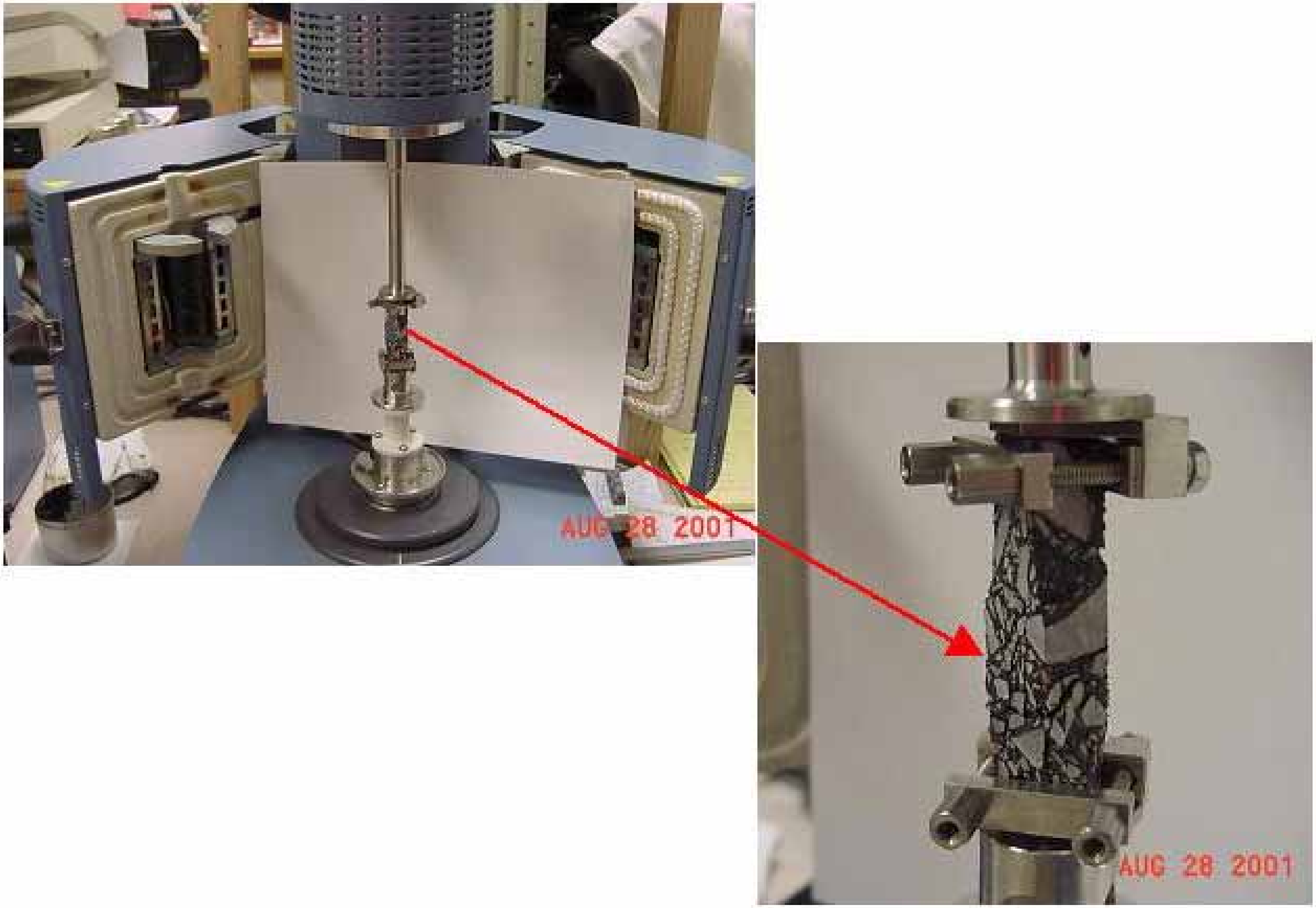

FIGURE 7 Typical test specimen mounted in rheometer and close-up showing specimen after testing.

specimens were used for both static and dynamic creep testing. The static creep test consists of applying and maintaining a constant rotational stress on a specimen, at the selected test temperature, until the specimen fails. Figure 8 shows a typical static creep test result; Figure 9 shows the same static creep curve with the different flow regions of the mix identified. The labels in Figure 9 have followed terminology defined by Witczak (*10*), although other researchers have reported on this failure profile of mixes undergoing creep testing (*2, 11*). Figure 10 shows a similarly shaped creep curve resulting from a dynamic creep test. Each cycle (see the inset test cycle in Figure 10) consists of 1 s of stress application followed by 9 s of zero stress. During this time, the specimen is able to recover a portion of the strain developed during the 1-s stress application period. Both test procedures have been used successfully, although the dynamic creep test does enable the specimen failures to be spread out over a wider time period. Figure 11 shows the test results for two samples of the same mix tested under static and dynamic creep conditions. The specimen tested under static creep conditions failed much more rapidly than did the specimen tested under dynamic creep condition. In addition, the static creep test was conducted at 15 kPa of applied stress rather than the 25 kPa used in the dynamic creep test. Not all specimens fail within the 2,000 cycles of the dynamic creep test; in such instances, having the static creep test available to achieve a failure result is valuable. Figure 12 shows several specimens that have been tested to completion. Note that "completion" may mean having been tested for 2,000 cycles without reaching tertiary flow. Some of the specimens in Figure 12 show very little strain, whereas others have been substantially strained. The weakest specimens fail catastrophically and actually come apart.

In defining the parameters for this test, SHRP guidelines were followed as closely as possible. Rather than test all mixes at a single, relatively cool temperature (for example 40°C or 50°C), Bahia's suggestion for cumulative binder testing in NCHRP 9–10 (*8*) were followed. Mixes were tested at the climatic service temperature of the pavement and not at the binder grade temperature. Therefore, regardless of the PG binder grade used in a mixture, if the climatic temperature for the mix location was 58°C, all the mixes were tested at 58°C. If the goal of mechanical testing is to understand the real impact of using a polymer-modified binder on permanent deformation characteristics in a mixture, it only makes sense to test the mixture at the service temperature at which it will be expected to perform. During test development and evaluation, we tried to conform to a 68-kPa stress for the dynamic creep test because that is the stress level used in SST. Some samples and especially field cores, such as the Minnesota Test Road (MnRoad) samples, do not have the strength to withstand a testing stress of 68 kPa. The stress level that a mixture can tolerate is itself an indication of the quality of that mixture.

Figures 13 and 14 show some of the creep results used to evaluate mixtures. Witczak (*9*) has defined the point of tertiary flow onset for a static creep test as "flow time" and for a dynamic creep test as "flow number"; we have continued to follow that same terminology. Because 5% is the upper limit of permanent strain measured in the SST repeated shear at constant height test, we have used the time to achieve 5% permanent strain as another test result to monitor. Many times, specimens will not achieve 5% permanent strain, in which case, to rationally compare different mixes or binder types there is a need to select the permanent strain at some other number of test cycles such as 100 or 200 test cycles. If one or more of the mixes being evaluated is weak, the number of cycles can be as low as 10 or 30. Because a DSR is used to generate the dynamic creep data, it is possible to determine a predicted value for the zero shear viscosity (η_0) of the mix specimen at a given test cycle (Figure 14). The TA Instruments software enables a

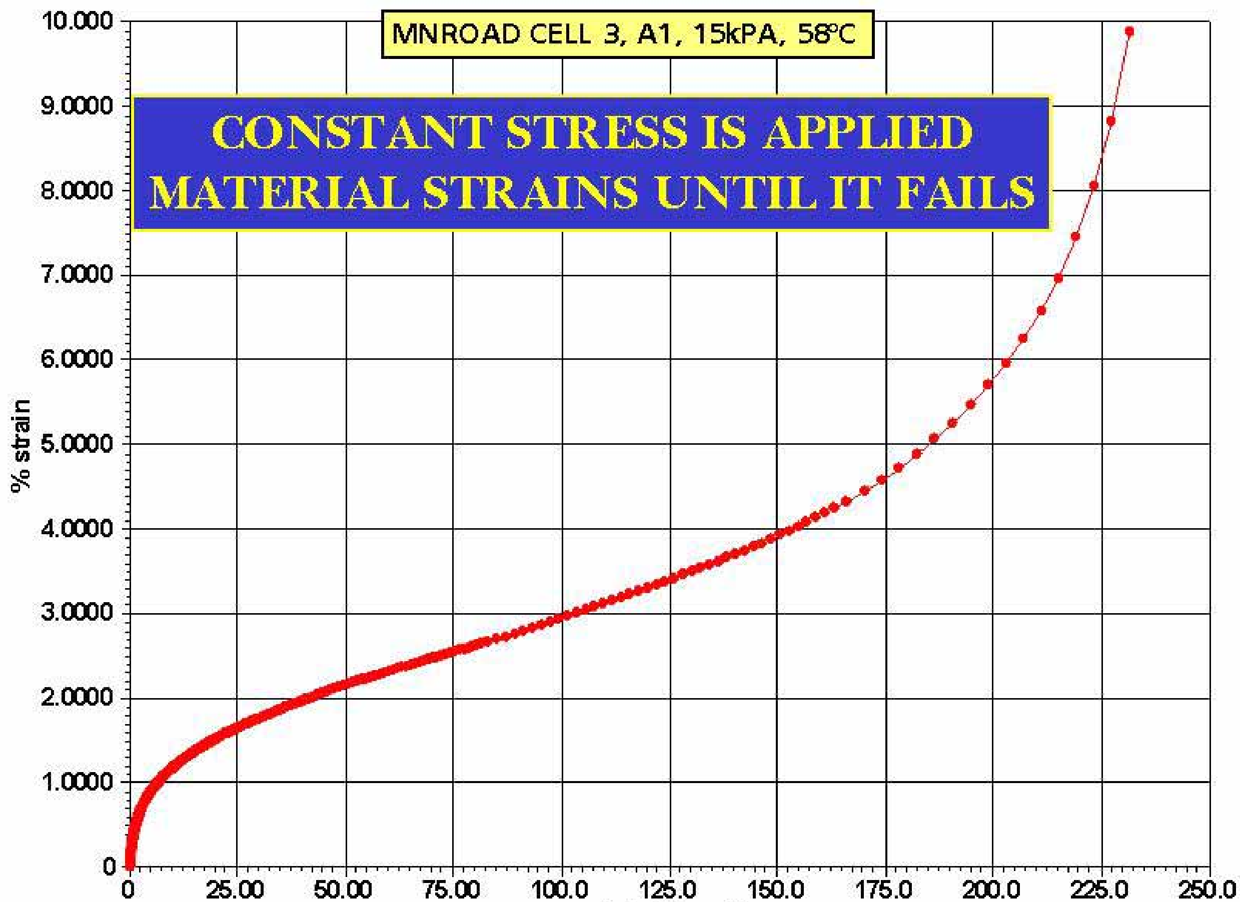

FIGURE 8 Test result of specimen tested under constant stress conditions.

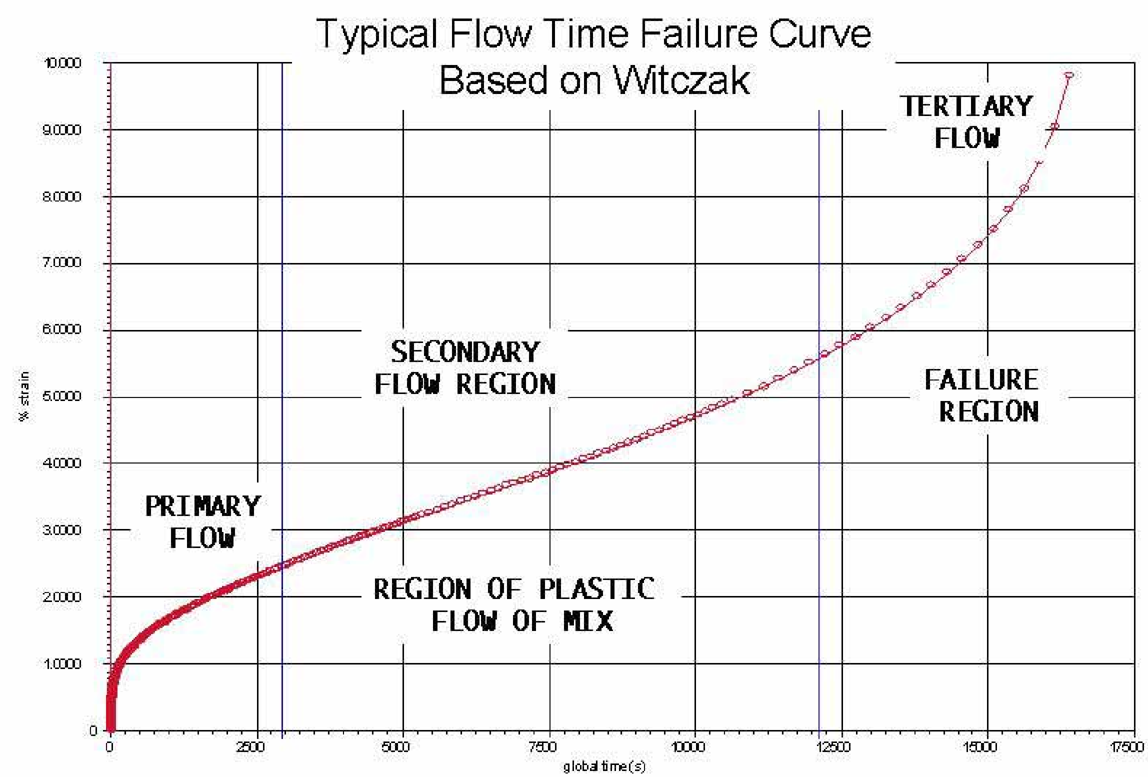

FIGURE 9 Plot showing difference portions of creep test failure curve [based on Witczat (*10*)].

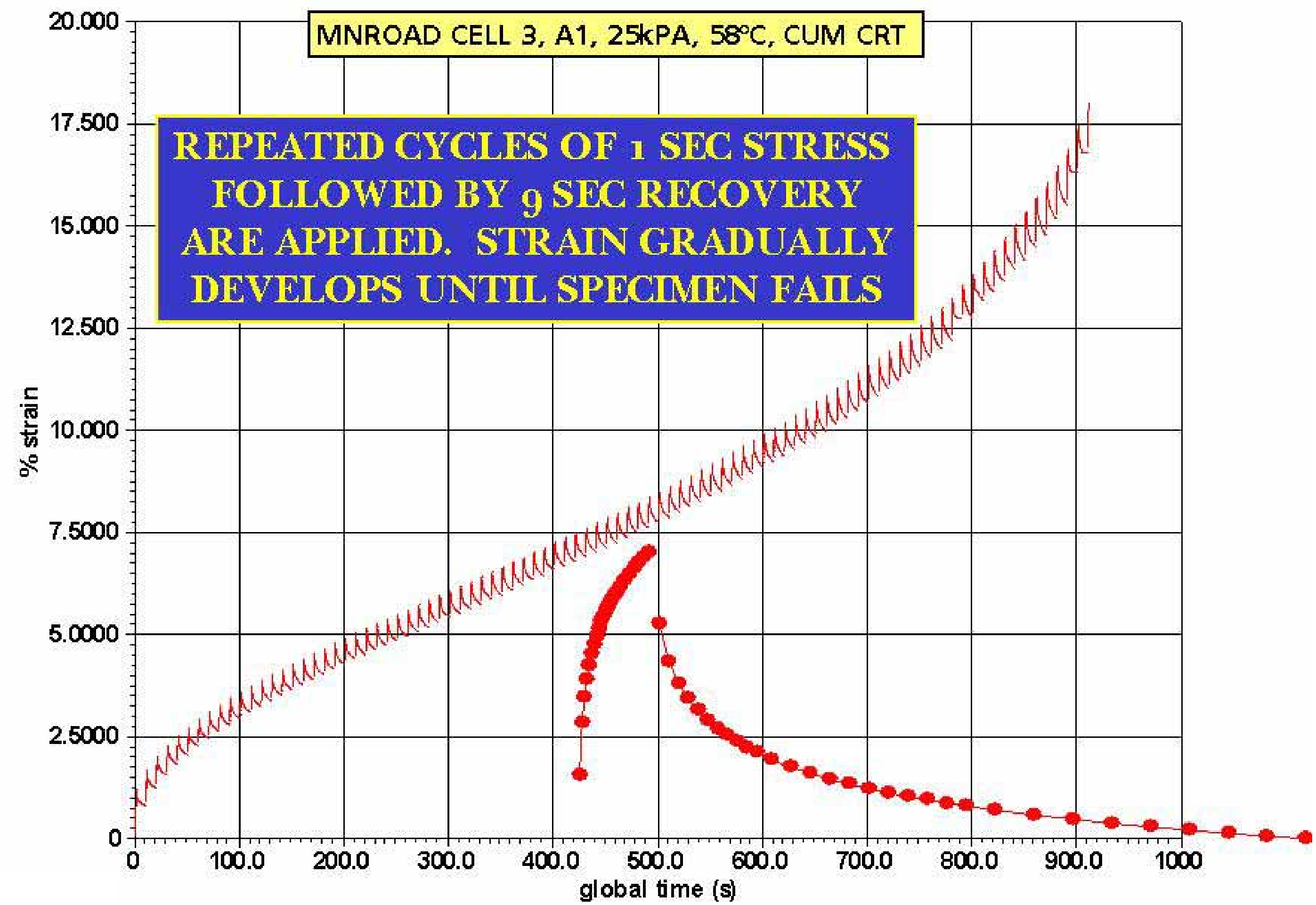

FIGURE 10 Failure curve generated by dynamic creep test (inset shows single creep and recovery cycle).

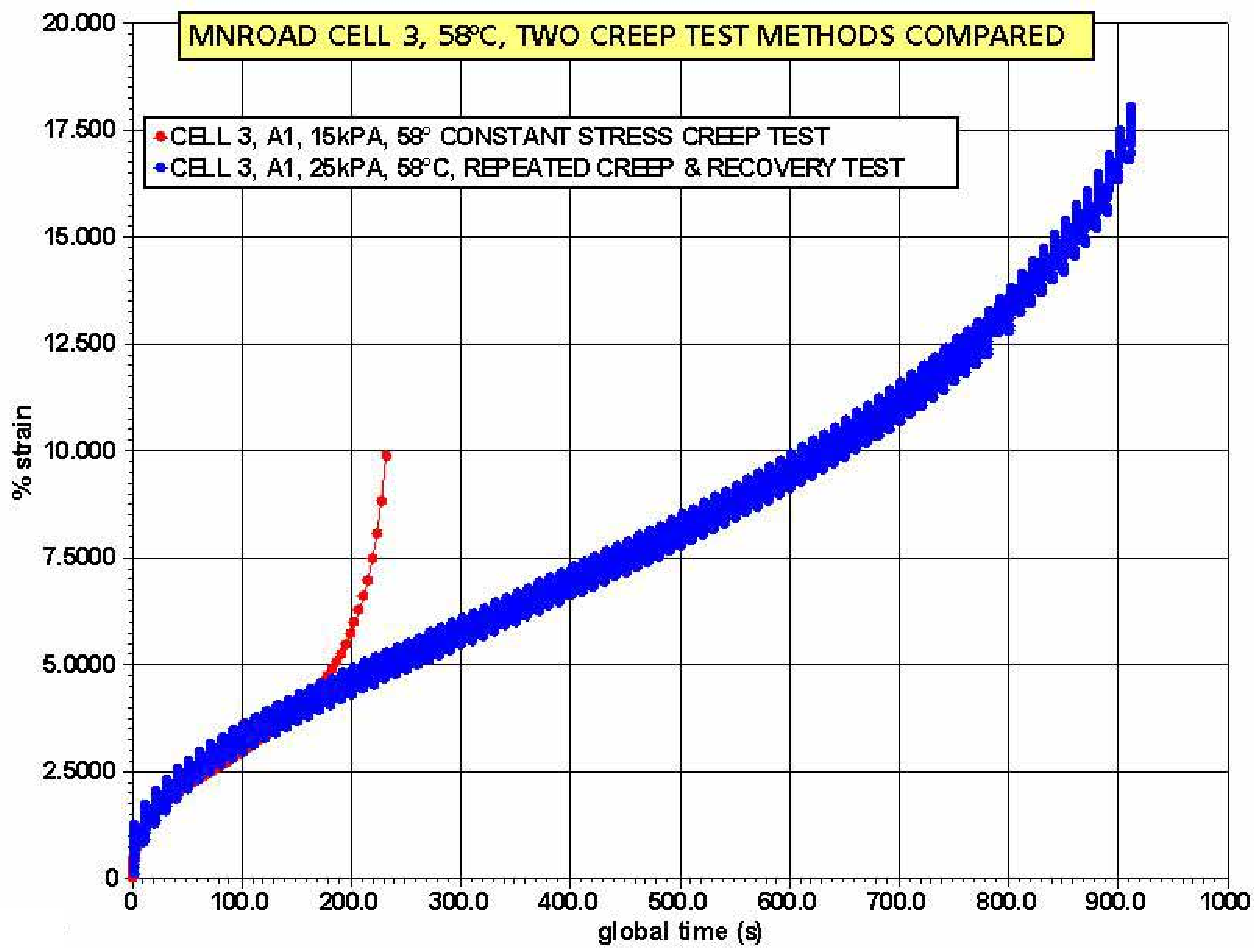

FIGURE 11 Comparative results for static and dynamic creep tests.

FIGURE 12 Typical failed test specimens from both 6 mm and 10 mm test articles.

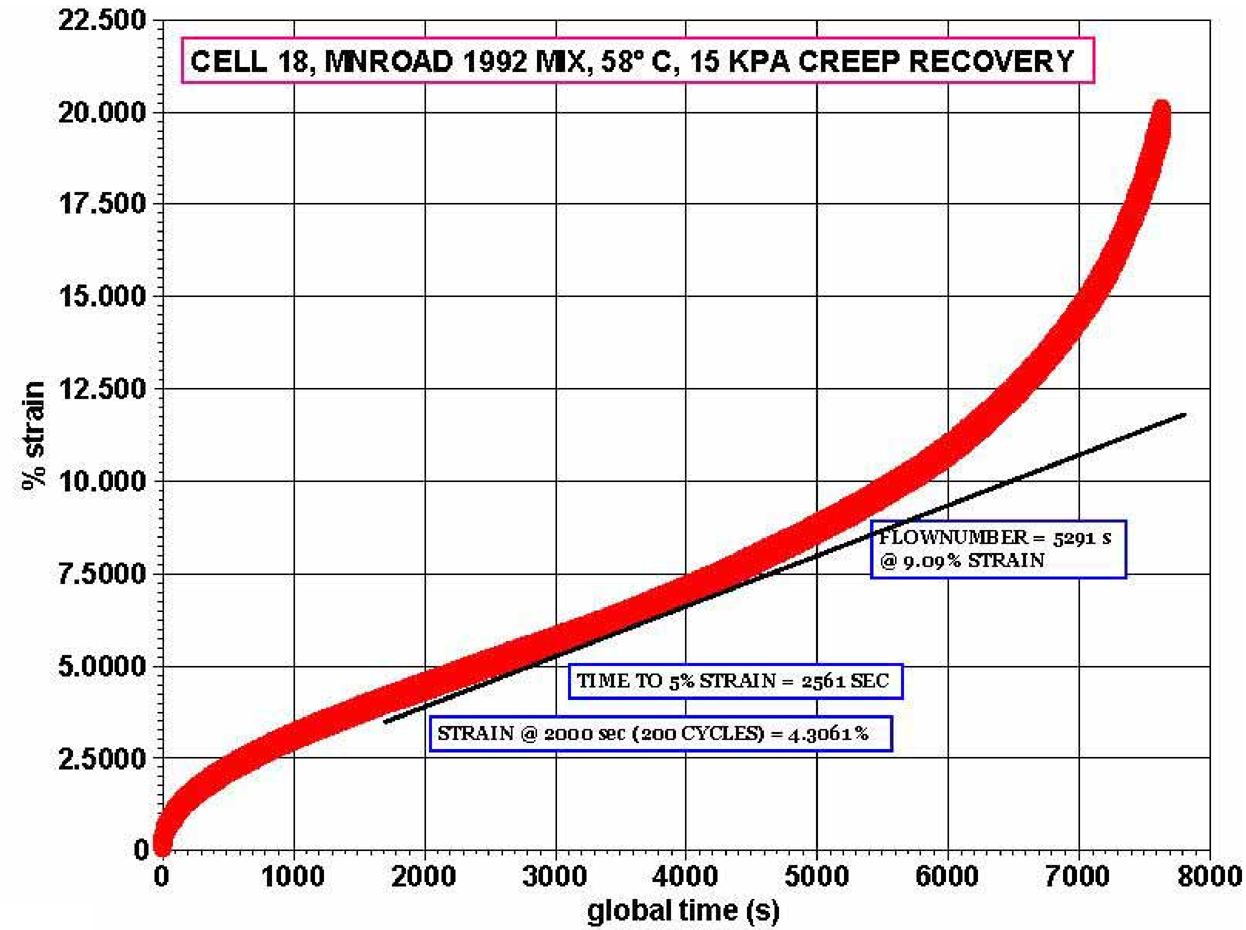

FIGURE 13 Typical DSR dynamic creep result detailing several parameters for investigation.

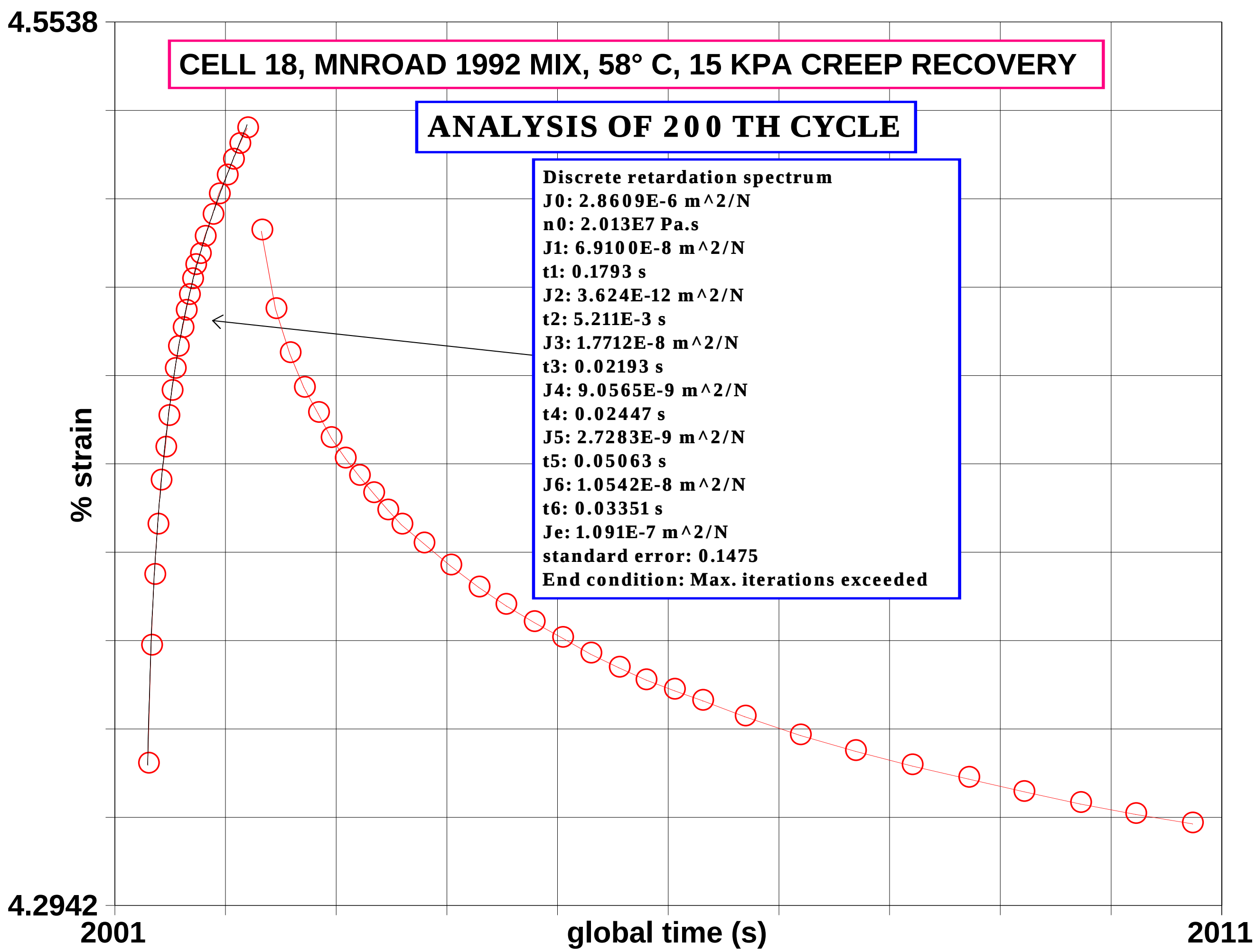

FIGURE 14 Single creep cycle from MnRoad DSR creep testing.

prediction of the η_0 value on the basis of the 1-s stress application period for a particular cycle. Depending on the specimen geometry, the stress applied and the amount of strain developed an estimation of η_0 can be made.

MNROAD TEST SECTIONS

The MnRoad specimens evaluated were taken from test sections constructed in 1992 at the MnRoad test site near Rogers, Minn. Among the test sections constructed were the ones listed in Table 1. In 1992, the binders used were pre-SHRP and the mix designs were pre-Superpave®, although one gyratory design was included. The specimens tested were cut from cores taken from the project in 2000.

The average rut depth data in the driving lane were reported by the Minnesota DOT from measurements taken in August of 2000. As testing the specimens cut from the MnRoad cores began, it became clear that stress levels of 15 to 25 kPa were all that the mixtures could tolerate. It is worth noting that in Witczak's testing (*10*) of 100-mm diameter cores from this same test site, he reported using a stress level equal to 68 kPa at a test temperature 4°C lower than we used with the DSR test method (Figure 15). An examination of Figures 16–18 shows the correlation of various test parameters obtained from the dynamic creep test performed on 6-mm slices taken from the MnRoad cores. Figure 16 shows the results of plotting rut depth as a function of permanent strain at the end of 200 test cycles. Figure 17 is a plot of rut depth as a function of total time required to achieve 5% of total permanent strain. Finally, Figure 18 is a plot of rut depth as a function of mix flow number (time to tertiary failure). For all of these mixture creep results, the correlation to the measured rut depth results in a R^2 greater than 0.90. Anyone versed in asphalt mixture technology would probably have ranked the rutting behavior of the mixes from these four test sections in the order in which they performed in the field. The fact that the DSR dynamic creep test procedure was able to discriminate between mixes produced with 120/150 and AC-20 is perhaps not surprising. However, the DSR test method also was able to discriminate between the 50-blow AC-20 mix and the 75-blow AC-20 mix. We would expect the 75-blow mix to contain less binder and to have a higher stiffness than the 50-blow mix and hence experience less rutting. The ability of the DSR dynamic creep test procedure to properly measure the differences between these two mixes is an important outcome.

TABLE 1 Rutting Data from MnRoad Test Sections

Cell #	Mix Type and Binder	Average Rut Depth, August 2000, mm
3	50-Blow Marshall, 120/150	6.21
4	Gyratory Design, 120/150	9.60
17	75-Blow Marshall, AC-20	5.15
18	50-Blow Marshall, AC-20	5.96

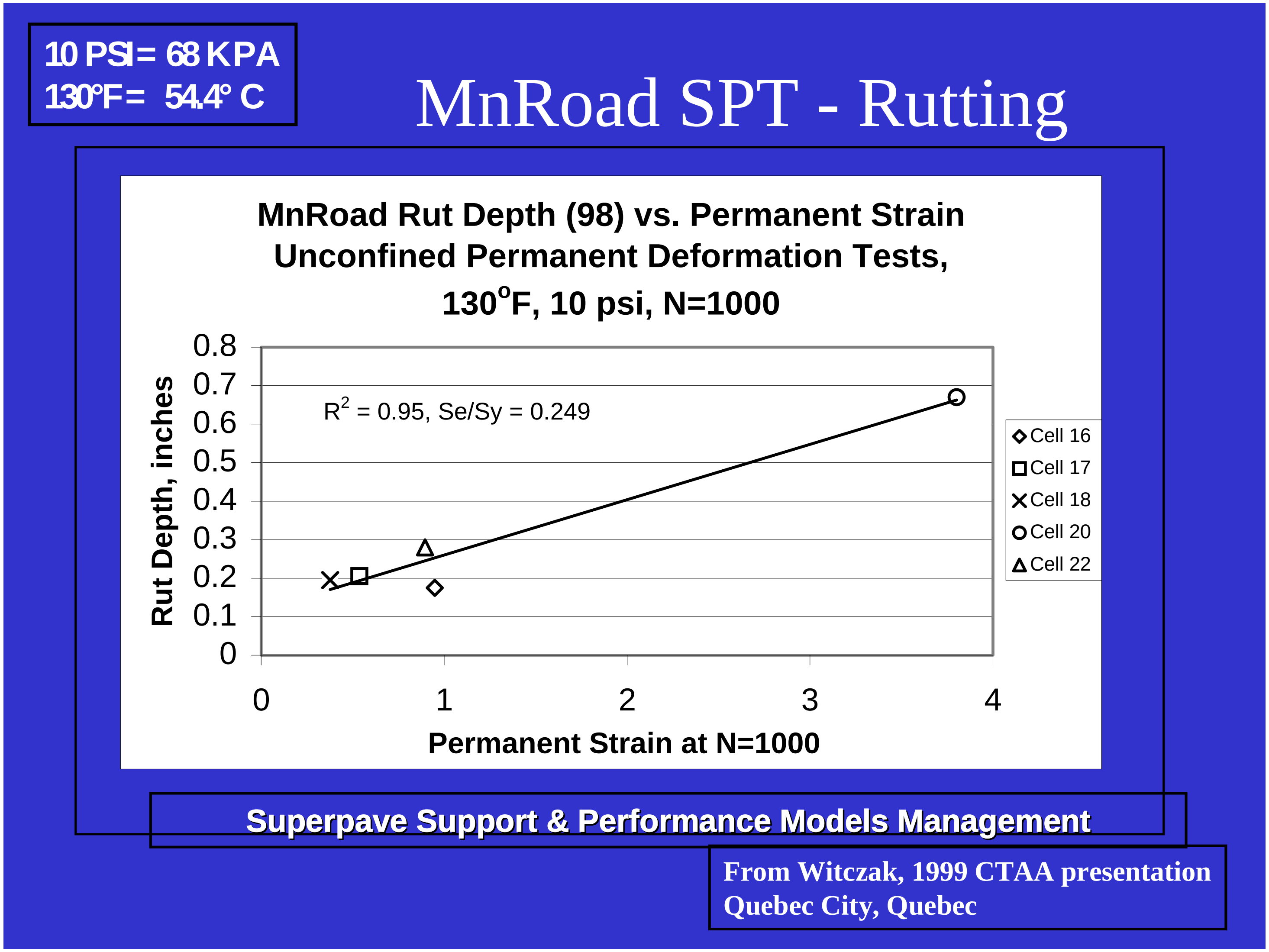

FIGURE 15 Plot showing one set of results developed by Witczak (*10*) from the MnRoad data.

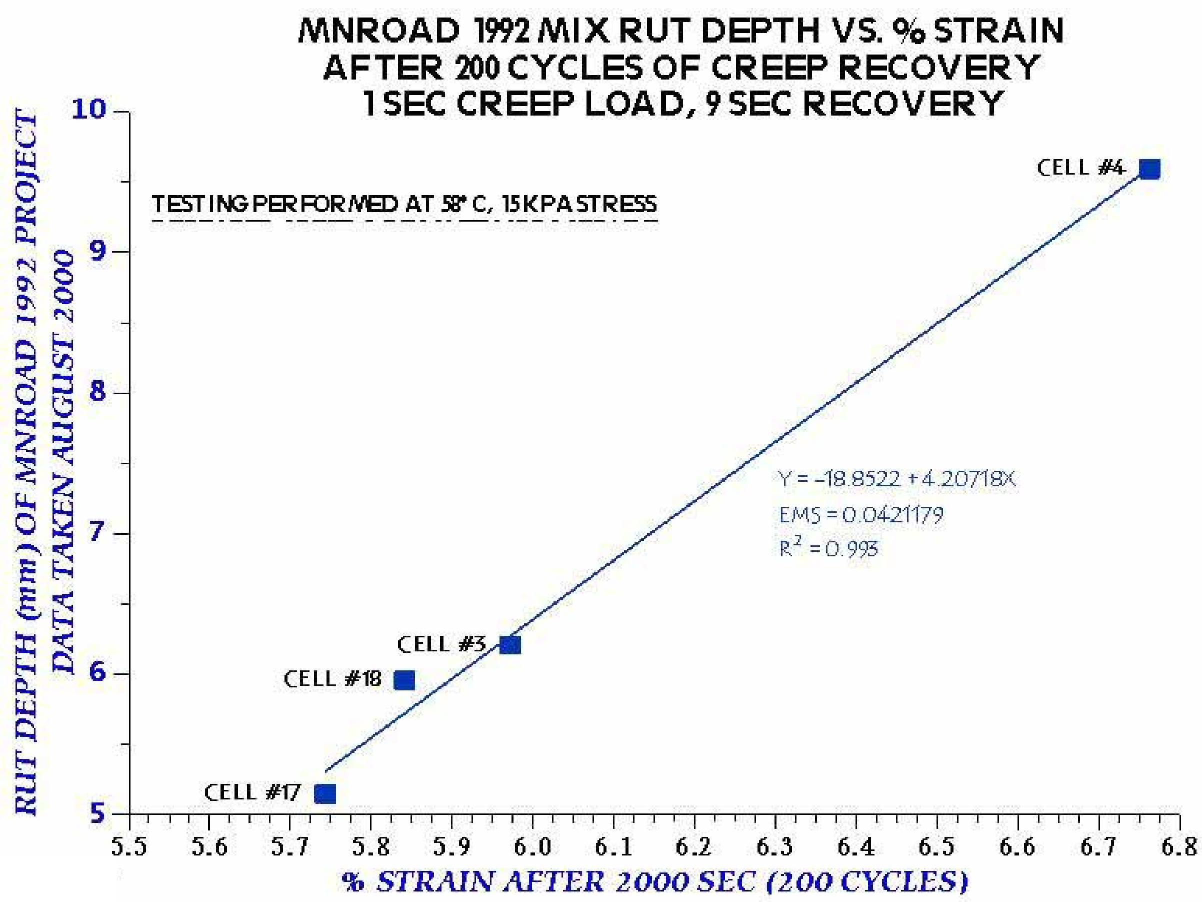

FIGURE 16 Plot of rut depth as a function of percent strain after 200 dynamic creep cycles for MnRoad mixes.

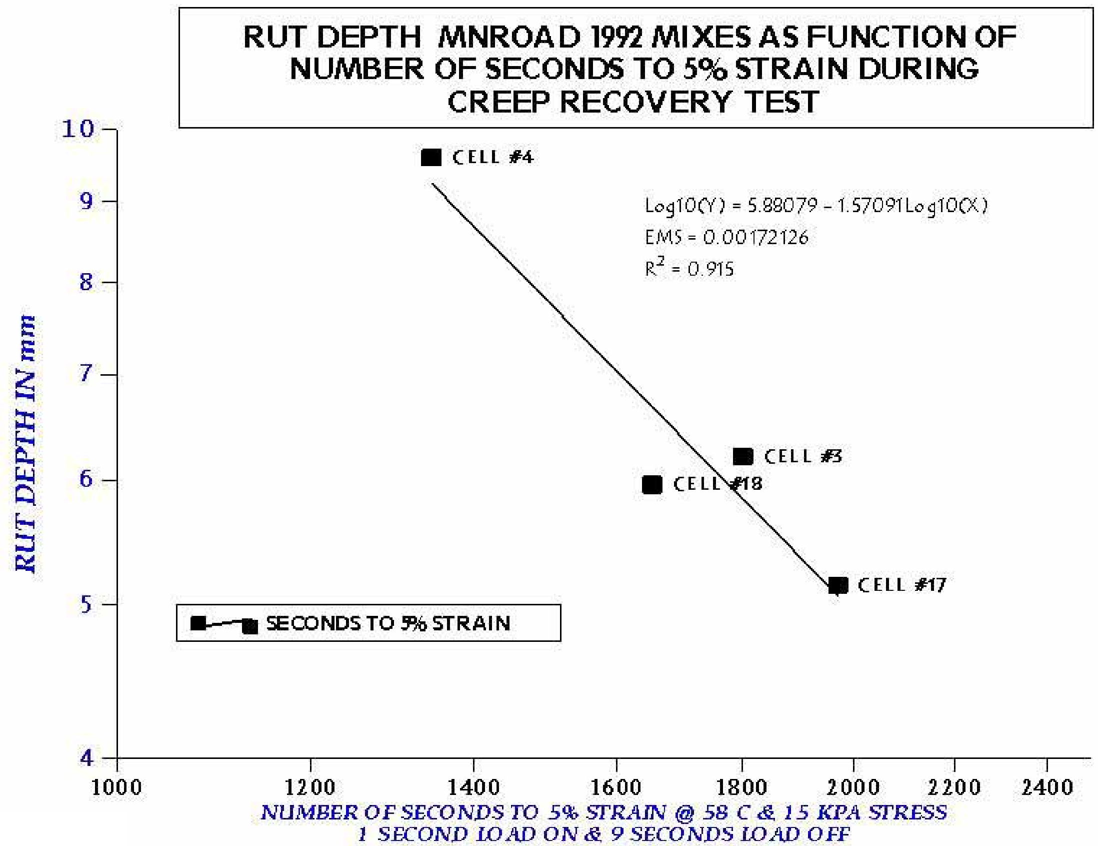

FIGURE 17 Plot of rut depth as a function of time to 5% strain from dynamic creep test for MnRoad mixes.

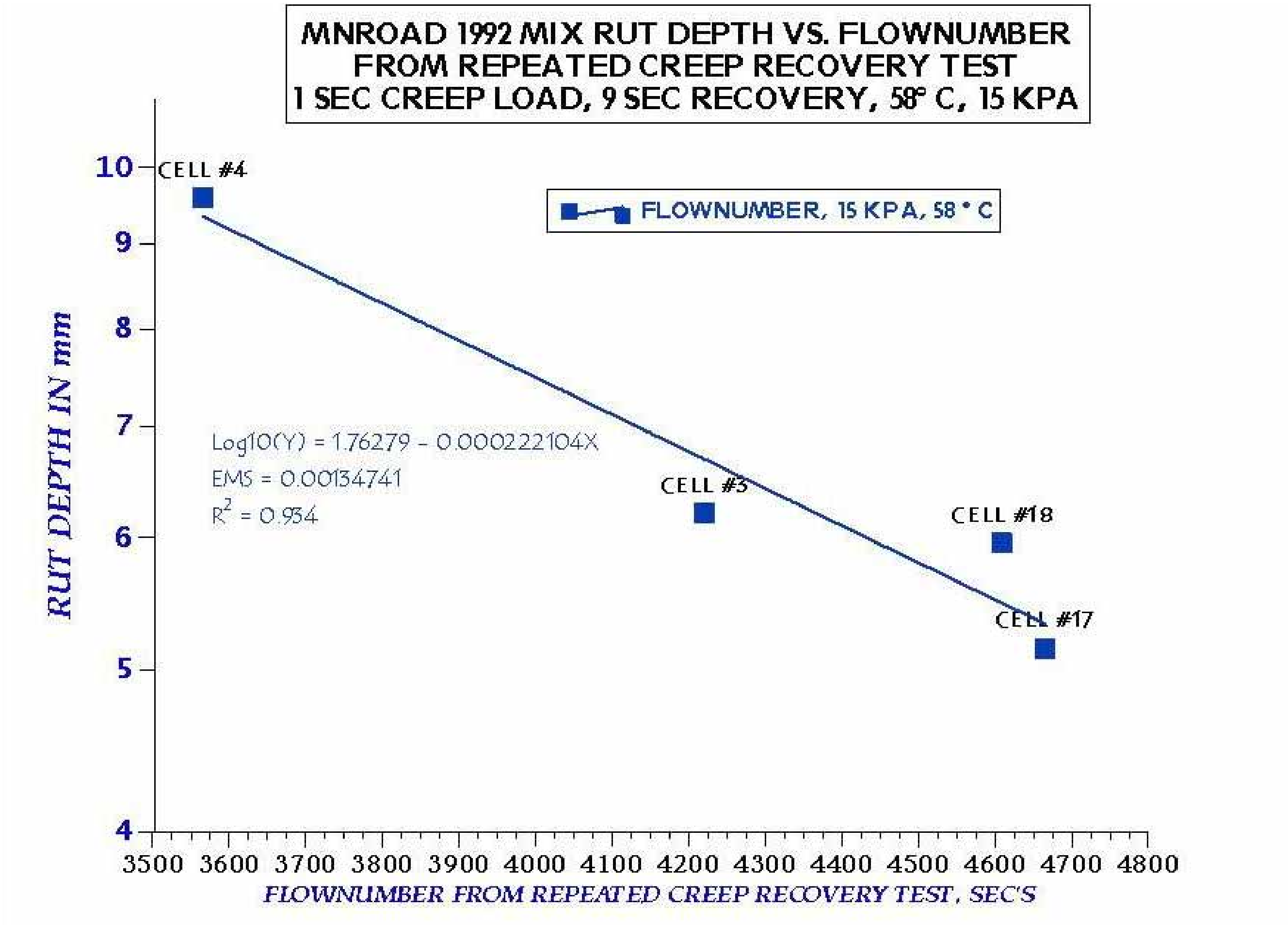

FIGURE 18 Plot of rut depth as function of flow number for MnRoad mixtures.

FHWA ACCELERATED LOAD FACILITY TEST TRACK SPECIMENS

The ability of the DSR dynamic creep test to correlate to the rutting behavior of a conventional mix at the MnRoad test site was encouraging. That mix, however, was a dense graded sandy mix low in coarse aggregate (Figure 19). An opportunity to use the DSR dynamic creep test on a coarser mix was provided by FHWA. Cores taken from five of the mix sections paved in 1993 at the Turner–Fairbank Highway Research Center Accelerated Loading Facility (ALF) were provided for testing. FHWA published a detailed report describing the experimental design and rutting behavior of the mixes placed on the ALF site (*12*). The different lanes (test sections) tested and the binders used in those lanes are summarized in Table 2. Measurements of rutting behavior reported in the FHWA report also have been summarized in Table 2. One advantage of efforts to validate the usefulness of the DSR dynamic creep test procedure was the use of polymer-modified binders to construct two of the test sections (Lanes 7 and 8). In addition to the use of polymer-modified binders in two test sections, the aggregate used in the mixes placed at the ALF site was much coarser (Figure 20) than the aggregate used at the MnRoad site (Figure 19). The cores tested were cut in 2000, and the dynamic creep testing was performed during the summer of 2001.

Because the ALF mixes used a coarse, angular aggregate, a 68-kPa stress level was chosen for testing. The tests were performed at 58°C—the same temperature at which the ALF testing had been performed for the rutting data reported in Table 2. Figure 21 shows a comparison of one test for Lanes 5, 9, and 10 at 58°C and 68 kPa. Because of the rapid failure of these conventional asphalt specimens, the test stress would normally have been reduced, as was done for the MnRoad specimens. However, the data traces in Figure 22 for polymer-modified Lanes 7 and 8 showed that at testing periods up to 1,800 cycles, the mixes had not failed. Rather than reduce the stress level and obtain a reduced response from the polymer-modified mixes, the decision was made to perform all mixture tests at 68 kPa, realizing that for the lower stiffness binders the failure times would be very short.

Although the number of cycles to failure for the mixes containing non–polymer-modified binders were low, the predictive results of the DSR dynamic creep test were quite good. Figure 23 shows the relationship between the number of ALF wheel passes required to reach a rut depth of 15 mm as a function of the mix permanent strain at 10 test cycles. At 10 test cycles, the strain

TABLE 2 Rutting Data from FHWA ALF Test Sections

Lane No.	Binder Grade	1994 Rut, mm	Rutting at 2,730 Wheel Passes, mm	Wheel Passes to 15-mm Rut Depth	Rutting at 10,000 Wheel Passes, mm
5	AC-10, 58–28	27	23.2	946	39.3
7	Styrelf 82–22	18	8	5.55 E4	12
8	Novaphalt, 76–22	9	3.5	1.75 E6	4.4
9	AC-5, 52–34	22	37.4	340	48.1
10	AC-20, 64–22	36	20.1	980	36.3

NOTE. All ALF testing performed at 58°C.

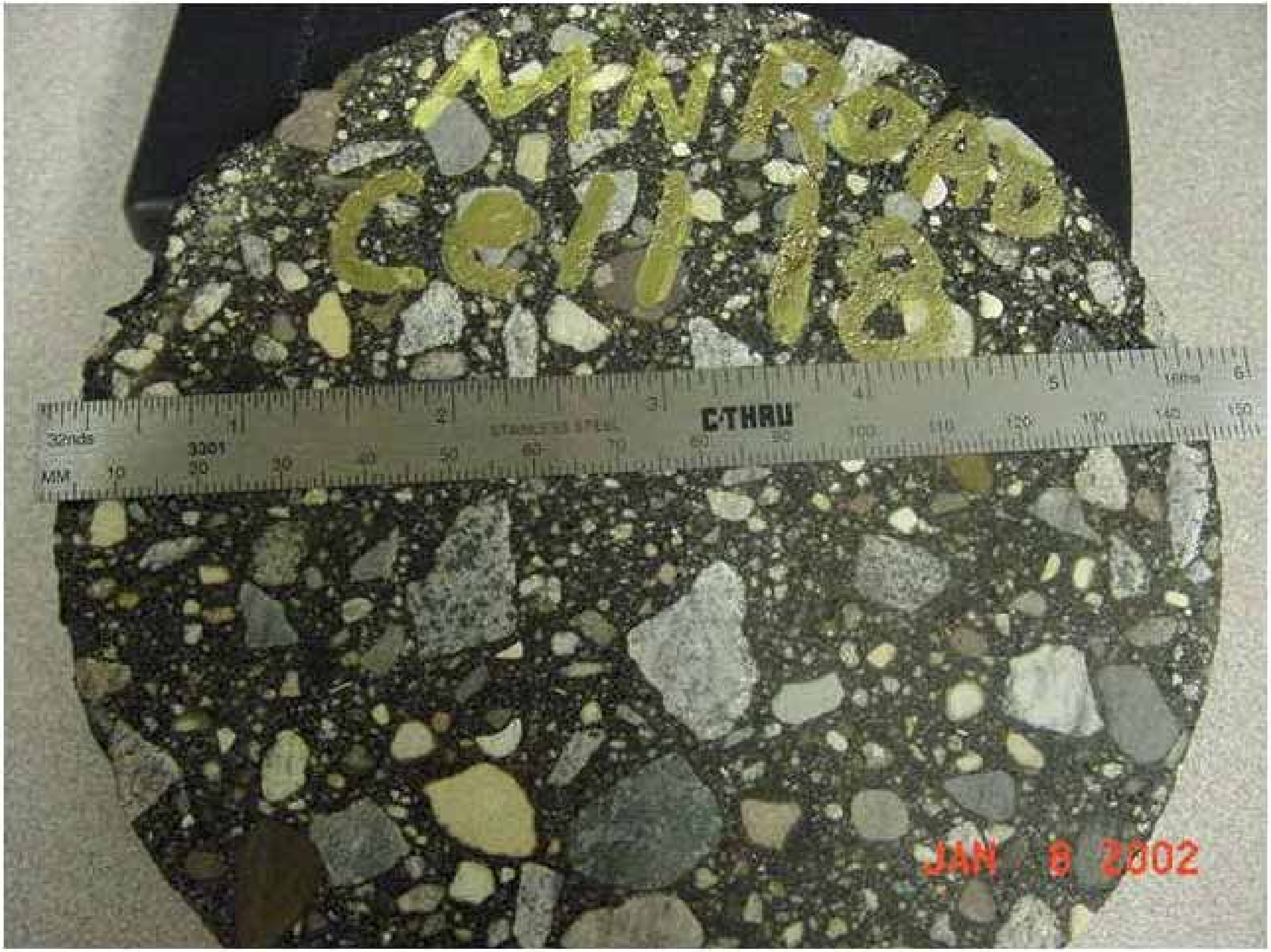

FIGURE 19 Aggregate structure MnRoad 1993 paving mixture.

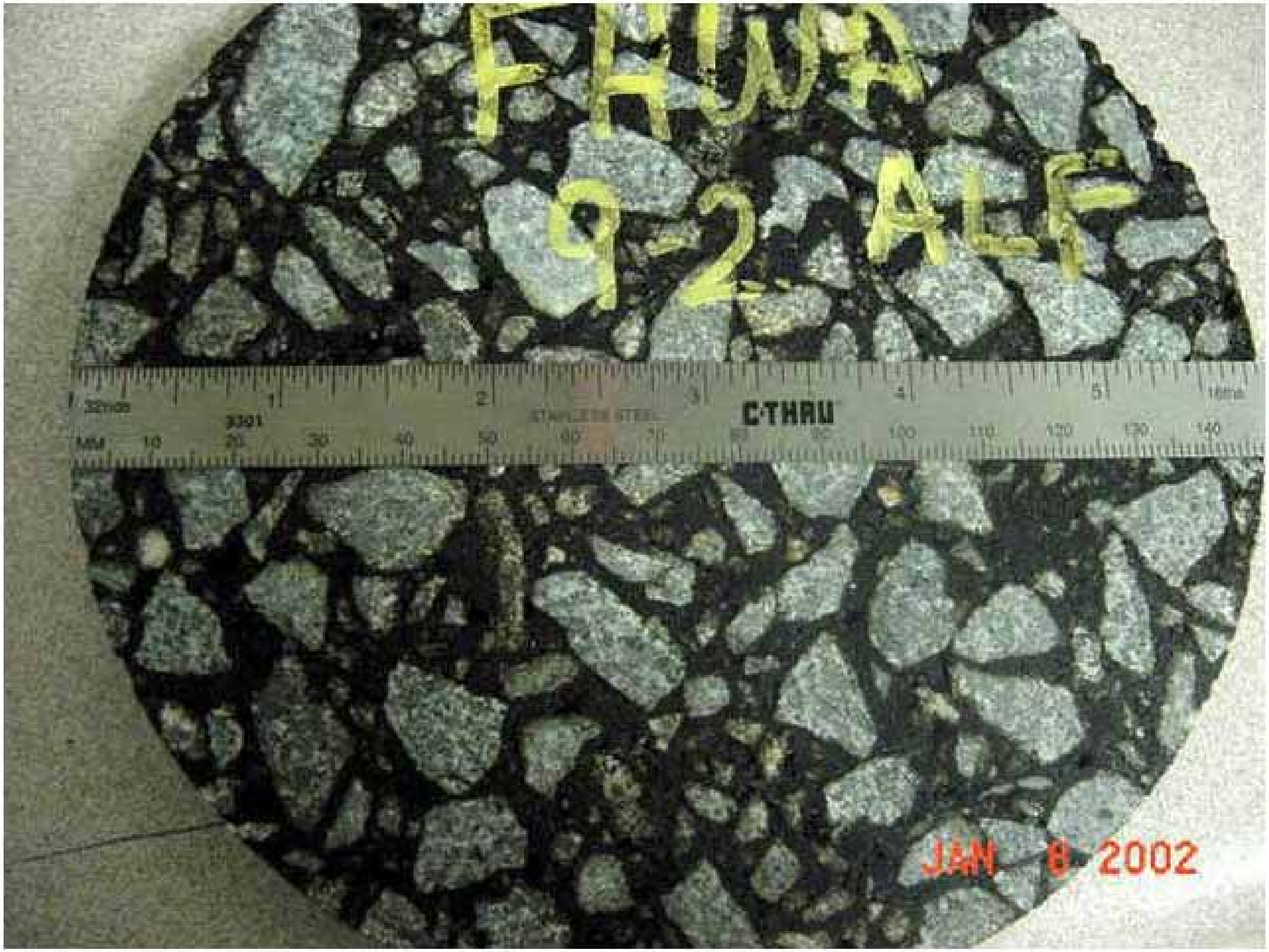

FIGURE 20 Aggregate structure of FHWA ALF mixture.

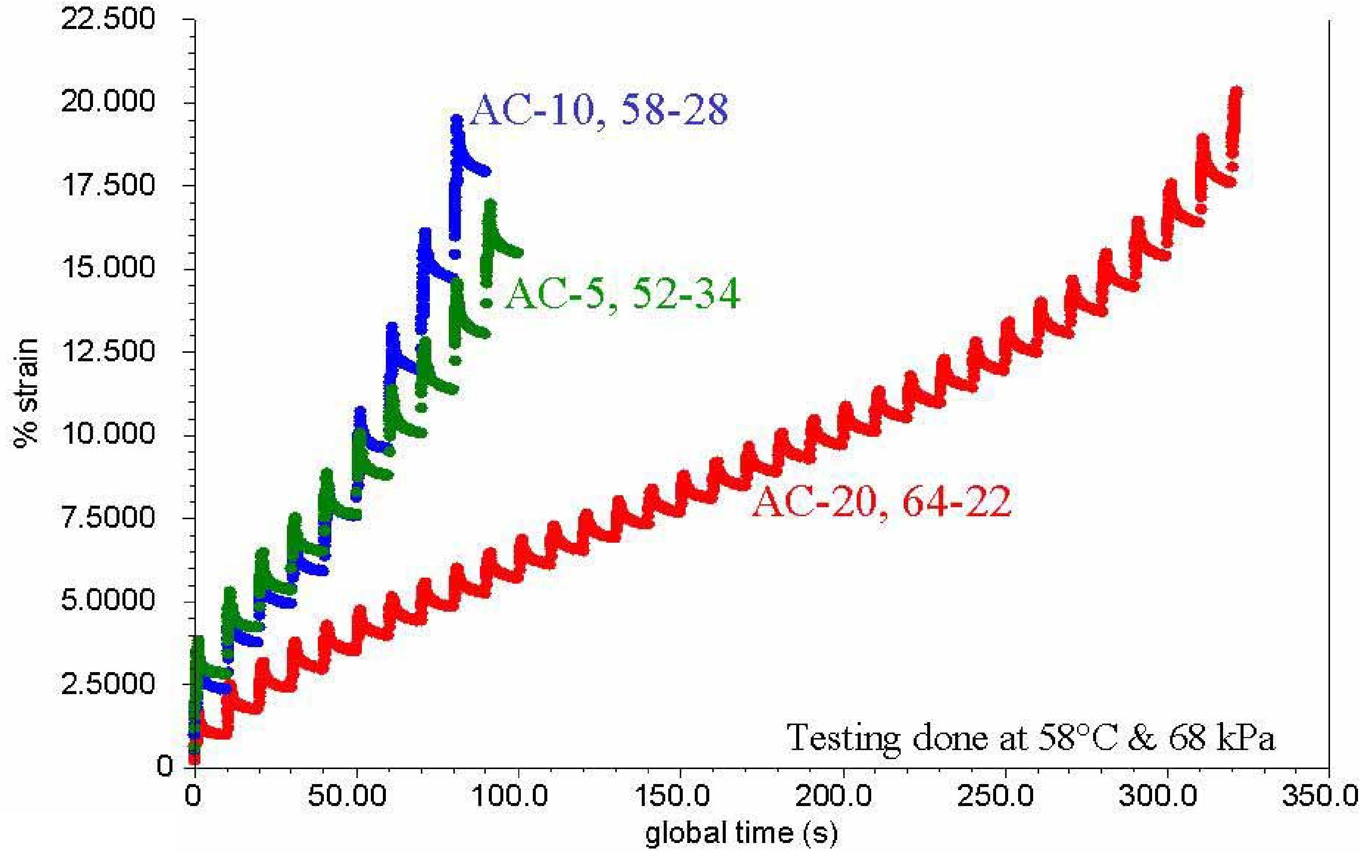

FIGURE 21 Example creep data of PG 52–34, 58–28, and 64–22 ALF mixes at 58°C and 68 kPa stress.

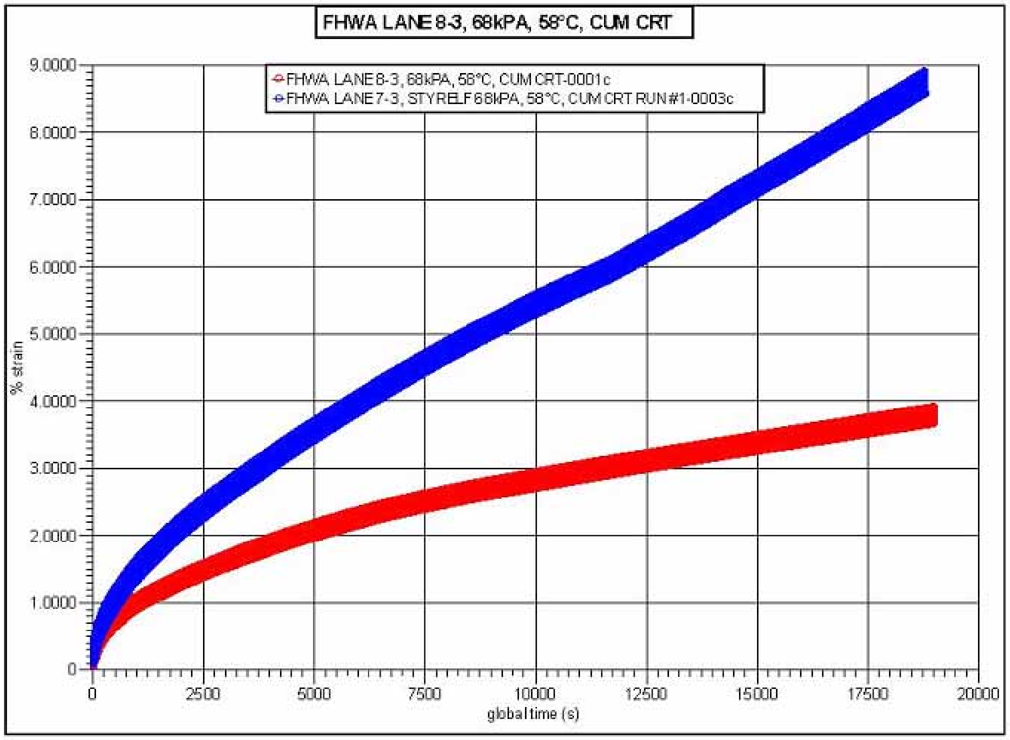

FIGURE 22 Creep data of Novaphalt (Lane 8) and Styrelf (Lane 7) at 58°C and 68 kPa stress.

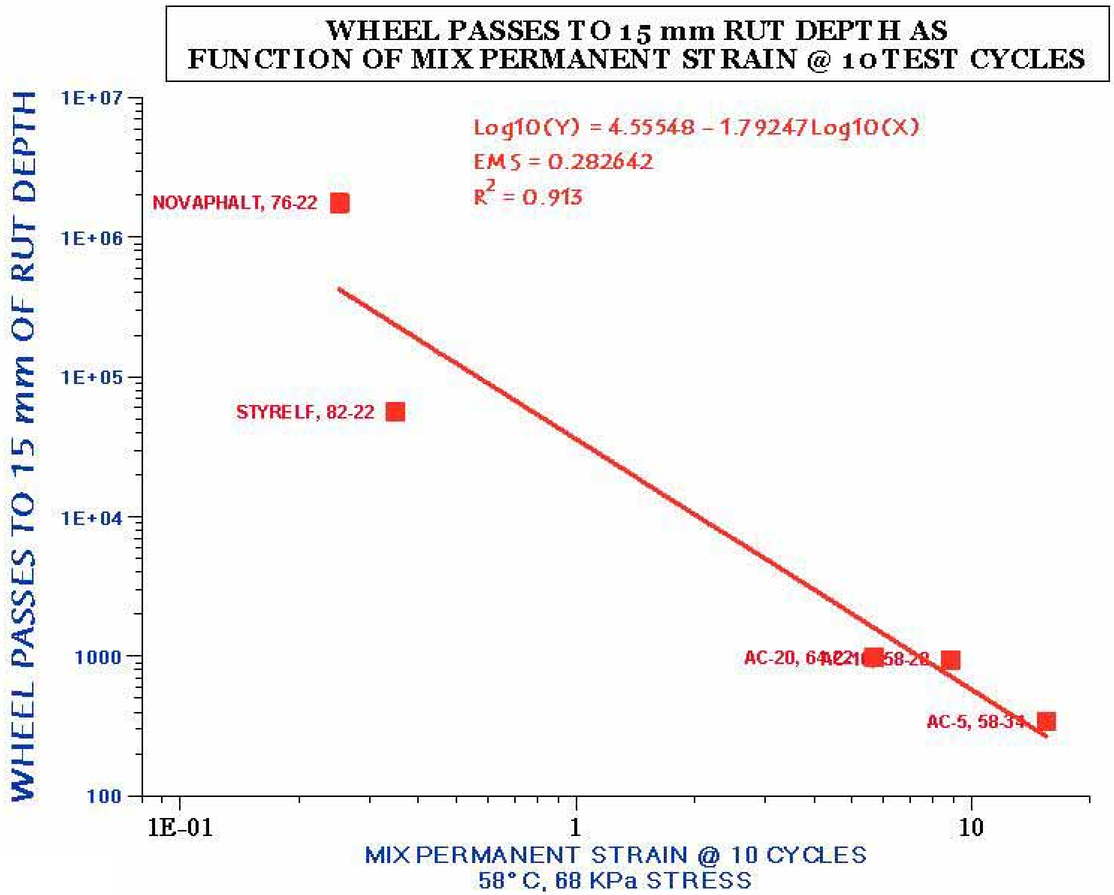

FIGURE 23 Plot of wheel passes to 15 mm rut depth as function of mix percent strain at 10 creep cycles.

values of the modified mixes were very low; however, the R^2 was still 0.91. In Figure 24, the same rut depth data (wheel passes to 15 mm of rut depth) are plotted as a function of the mix zero shear viscosity at 10 cycles. The R^2 for this relationship is 0.99. A similar result occurs when the rut depth after 10,000 wheel passes is plotted as a function of permanent strain after 10 cycles (Figure 25, R^2 = 0.989) or as a function of mix zero shear viscosity after 10 cycles (Figure 26, R^2 = 0.98). Finally, the plot of rut depth after 2,730 wheel passes as a function of mix zero shear viscosity after 10 test cycles is shown in Figure 27 (R^2 = 0.96). All of these DSR dynamic creep results are good predictors of field rutting of the ALF mixes. It appears that the mix zero shear viscosity value is a somewhat better predictor than the total strain at 10 cycles. This is probably because several of the test specimens failed at or near 10 cycles. At the point of failure, the strain values were increasing rapidly, but the predicted zero shear viscosity values had not become erratic.

Figure 28 shows a plot of rut depth at 10,000 wheel passes as a function of the SHRP specification parameter, G*/sin(δ), at both 10 radians/s (rad/s) and 2.25 rad/s. In addition to the SHRP specification test frequency of 10 rad/s, a test frequency of 2.25 rad/s was selected by FHWA because that frequency matched the speed of the wheel on the ALF track. The results plotted in Figure 28 show that the binder stiffness is a reasonably good predictor of mix rutting, although not as good as the results obtained with the DSR dynamic creep approach. The binder DSR results for the AC-20, AC-10, and AC-5 binders are very good predictors of the rutting behavior of the mixes produced from those binders. However, G*/sin(δ) is not a good predictor of the field rutting behavior of the Styrelf or the Novaphalt. The data in Figure 27 show that the Styrelf had a higher G*/sin(δ) stiffness than the Novaphalt, yet the ALF test results showed that the Styrelf section exhibited more rutting than did the Novaphalt. This apparent contradiction led some researchers to speculate that there were mixture- or construction-related problems between the two modified blends, the assumption being that G*/sin(δ) should provide a good indication of field rutting performance. The DSR dynamic creep results shown in Figures 23 through 27, however, show that, despite the G*/sin(δ) results, the Styrelf mix does fail more rapidly than the Novaphalt mix at 58°C does. The DSR creep test results rank the Styrelf and Novaphalt in the same order as the ALF test track. It could be assumed that the DSR creep results are merely confirming the speculation regarding mixture-related problems. However, the data traces in Figure 29 and the results shown in Figure 30 argue against that conclusion. Using mix slices from the same cores tested at 58°C, dynamic creep tests were performed on the Styrelf and Novaphalt mixes at 70°C using a 68-kPa stress. Figure 29 shows the results of duplicate test runs for both materials. It is clear from these data traces that at 70°C the Novaphalt mix fails more quickly than the Styrelf mix does. During the FHWA study, test lanes of the ALF track containing Styrelf and Novaphalt mixes were tested at 70°C. A comparison of the rutting results for the two mixes at 58°C and 70°C are plotted in Figure 30. At 70°C, the Novaphalt mix did exhibit more rutting than the Styrelf mix did in contrast to the results at 58°C. Because specimens from the same core were tested at both temperatures, variability in the mixes between the sections tested at the ALF facility as the cause for the differences in the DSR creep results can be ruled out. Apparently, a change occurs in the mixture resistance to repeated stress applications between 58°C and 70°C that the DSR dynamic creep test can identify. An examination of Table 3 shows that G*/sin(δ) does not predict this switch in rutting results (*12*).

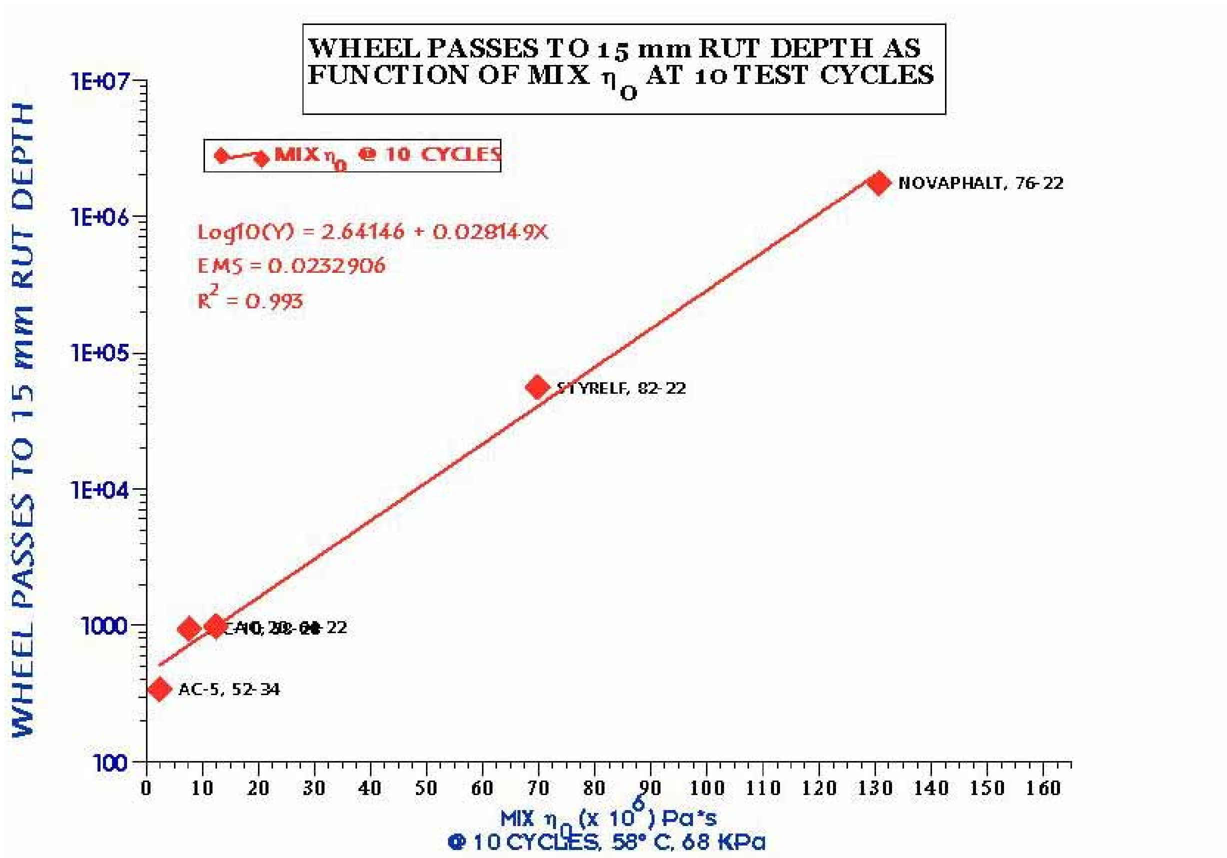

FIGURE 24 Plot of wheel passes to 15 mm rut depth as function of mix zero shear viscosity at 10 creep cycles.

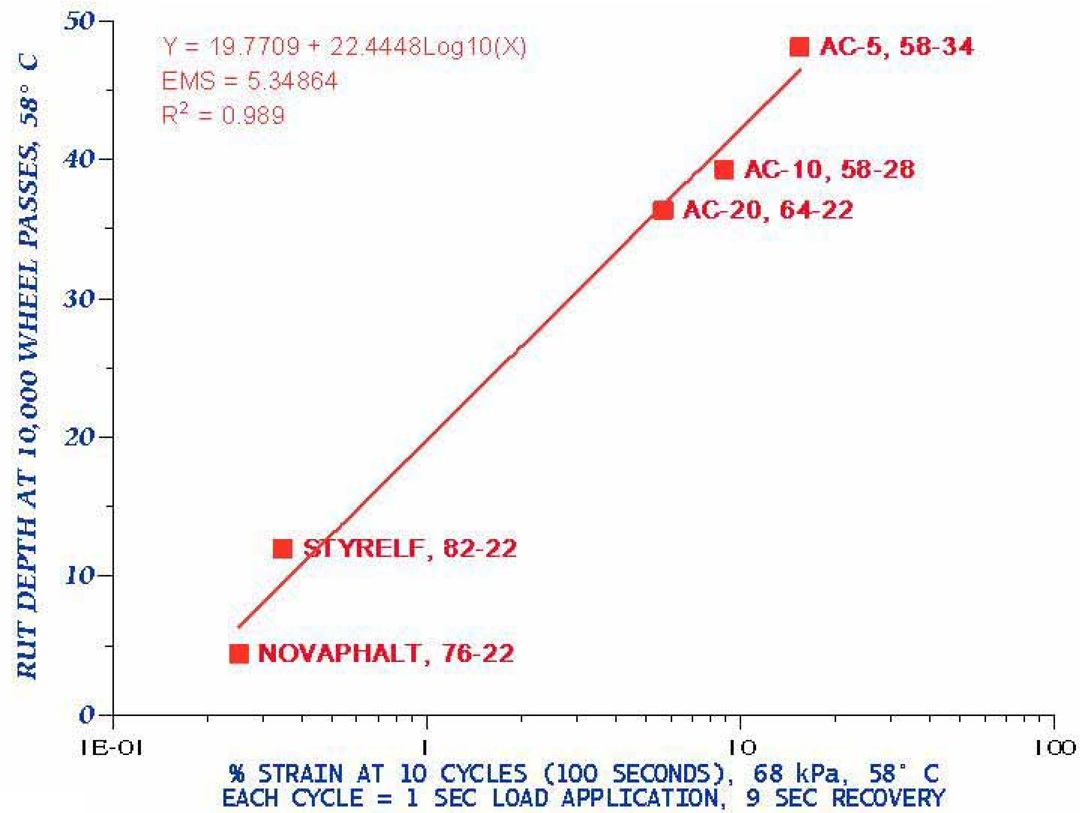

FIGURE 25 Plot of rut depth at 10,000 wheel passes as a function of percent strain at 10 creep cycles.

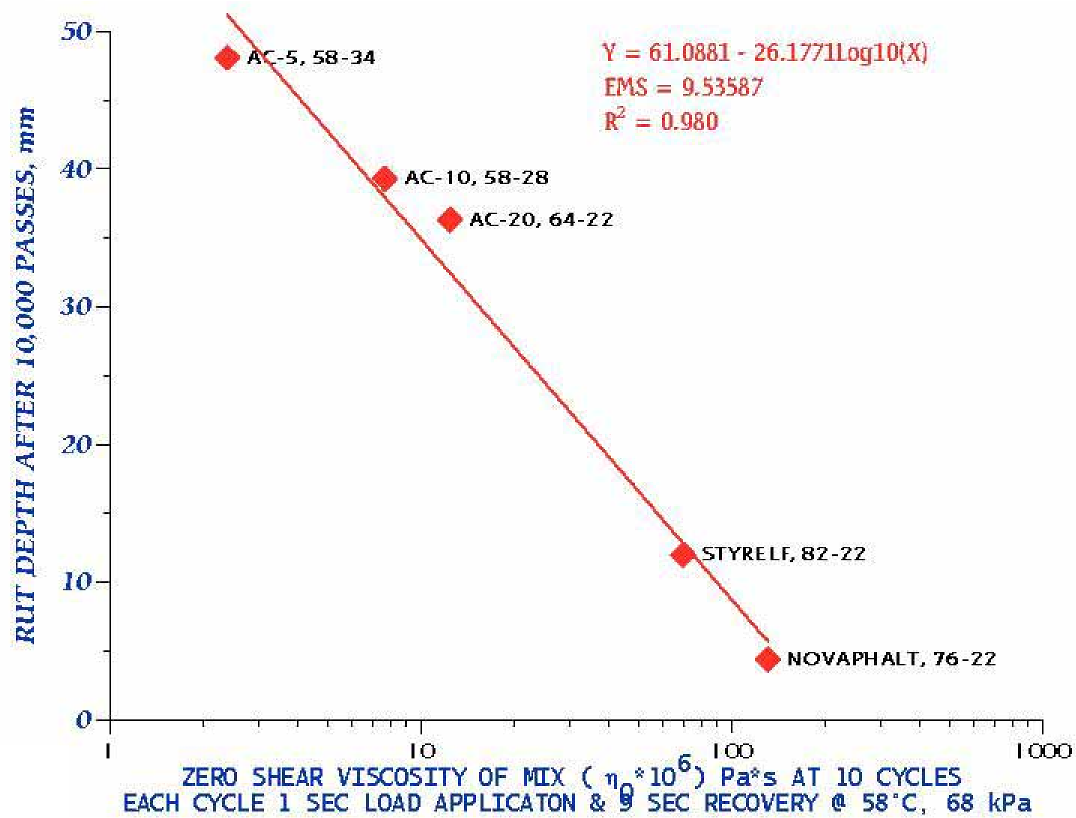

FIGURE 26 Plot of rut depth after 10,000 wheel passes as a function of mix zero shear viscosity at 10 creep cycles.

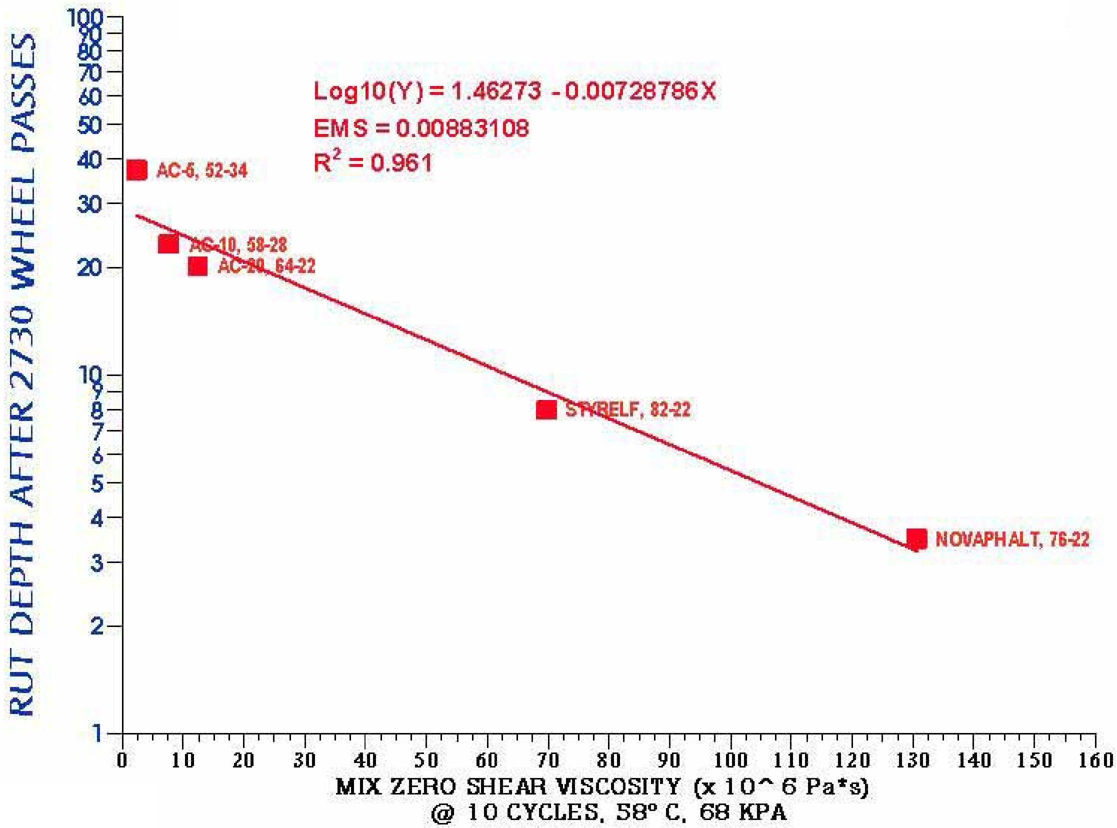

FIGURE 27 Plot of rut depth after 2,730 wheel passes as a function of mix zero shear viscosity at 10 creep cycles.

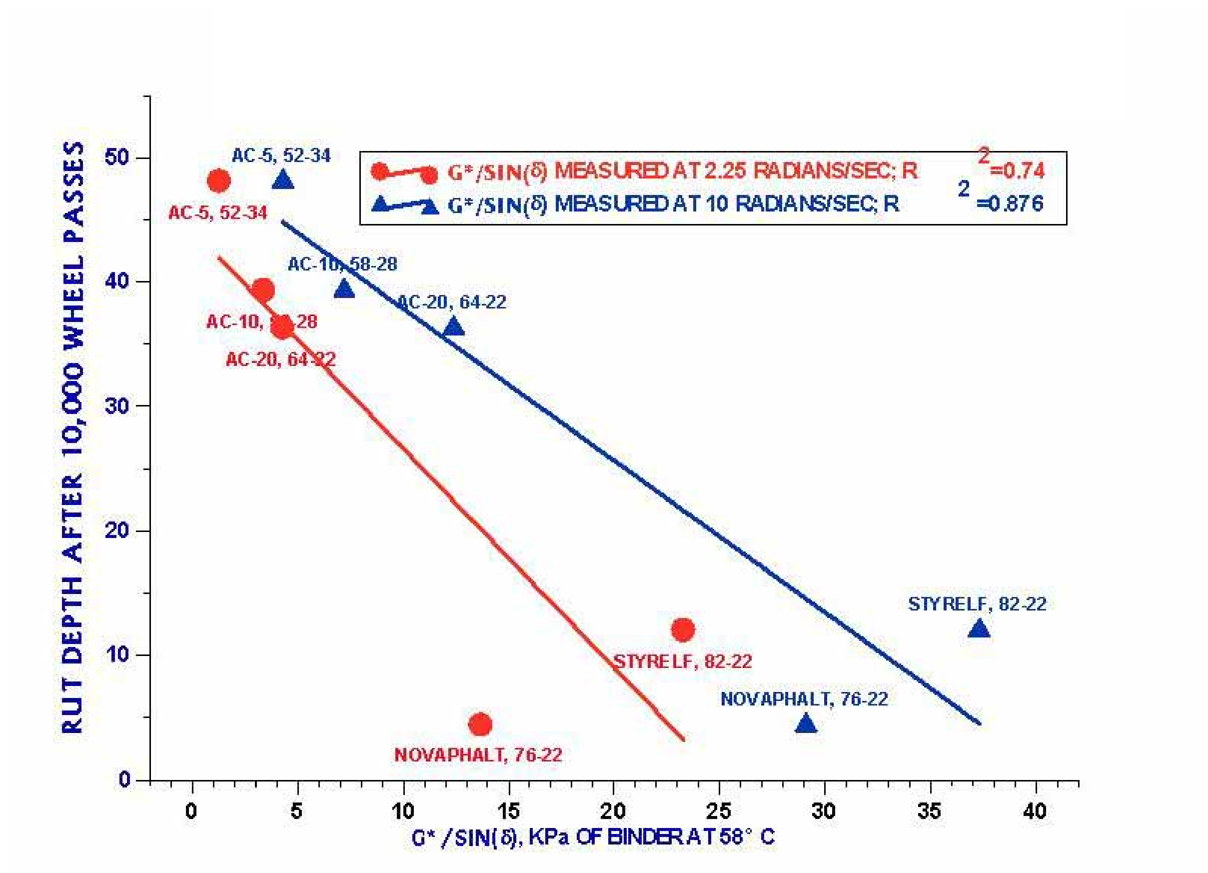

FIGURE 28 Plot of rut depth at 10,000 wheel passes as a function of G*/sin(δ) for ALF mixes.

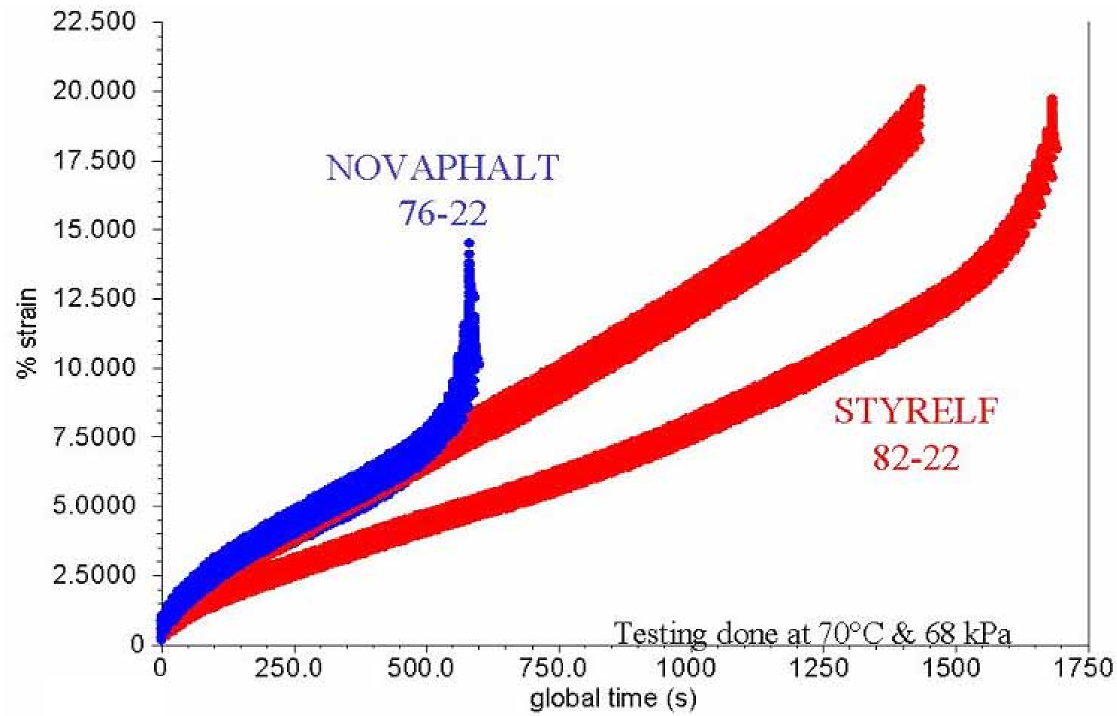

FIGURE 29 Data plots of Novaphalt and Styrelf at 70°C and 68 kPa test conditions.

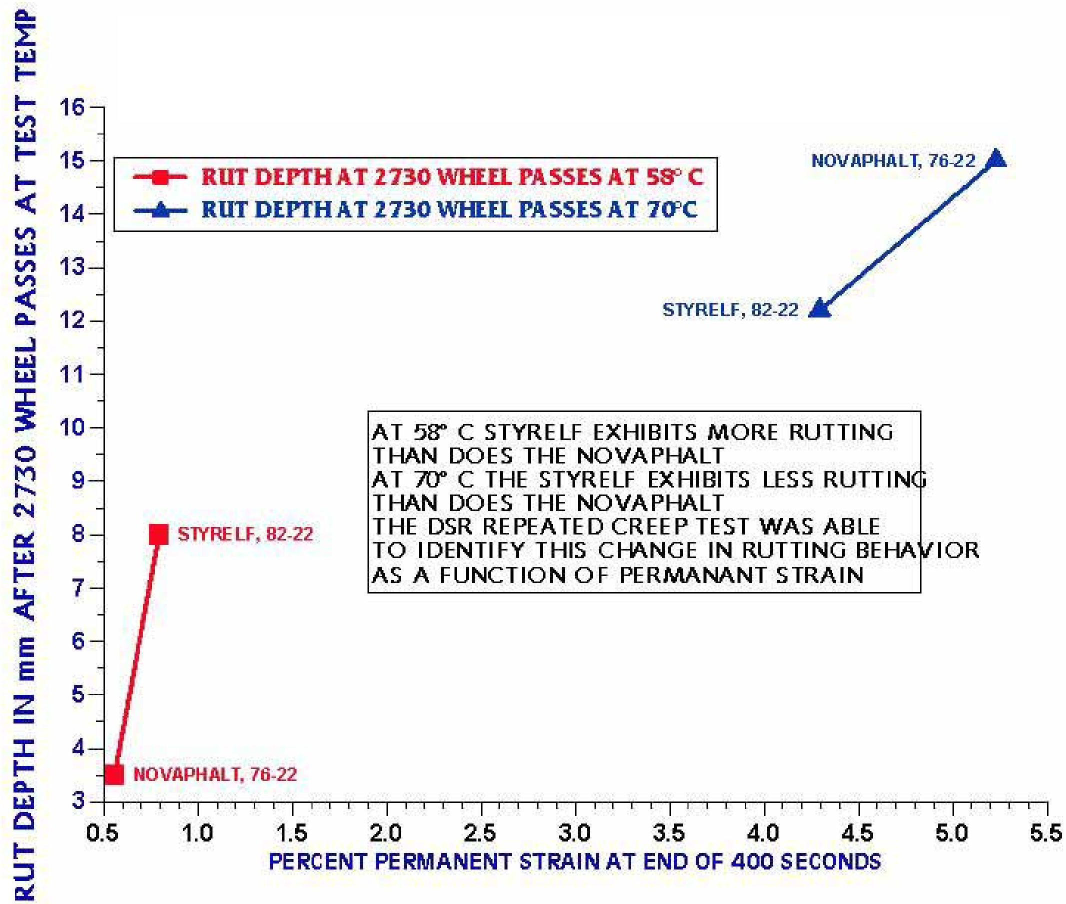

FIGURE 30 Reversal of rutting performance between Novaphalt and Styrelf at 58°C and 70°C.

TABLE 3 G*/sin(δ) Test Results for Extracted Binders from FHWA Report Table 12

Test Temperature	Test Frequency (rad/s)	Novaphalt G*/sin(δ), Pa	Styrelf G*/sin(δ), Pa
58°C	10	21,090	35,170
58°C	2.25	6,826	13,710
70°C	10	4,965	11,380
70°C	2.25	1,306	4,435

DISCUSSION OF TESTING VARIABILITY

Testing variability is always a concern, especially when a bituminous mixture failure test is being examined. Efforts have begun to systematically track the standard deviation and coefficient of variation on recent sets of samples. Initially, the goal was to develop the test procedure and then to determine whether the results of the test could in fact be used to predict actual field rutting behavior. The authors are now satisfied that the DSR dynamic creep test procedure is able to identify nuances in mixture performance related to binder type, test temperature, air voids levels, and aggregate structure. The data shown in Tables 4 and 5 are a summary of dynamic creep test results for a series of mixes placed at Mathy Construction Company's main office facility and on two nearby projects on state trunk highways (STHs). For the work placed at the office (identified as MPL), two aggregate gradations (an E-3 and an E-10) were used.[1] The E-3 aggregate structure is used for projects with traffic volumes of 1 to 3 million equivalent single-axle loads (ESALs) and the E-10 gradation is to be used for projects with traffic volumes of 3 to 10 million ESALs (Figure 31). With these two aggregate gradations, three different binders were used. Sections with the E-3 aggregate using an unmodified PG 58–28, a polymer modified PG 64–34, and a polymer modified PG 70–28 were constructed. Sections with the E-10 gradation using the PG 58–28 and the PG 70–28 also were constructed. (See the summary in Table 4 for details.)

Mix samples were taken from the hot mix plant and were compacted in the laboratory using a Pine Gyratory compactor to target voids levels of 3.5% and 7%. These mixes were then sliced and tested using the DSR dynamic creep test procedure. Because these projects had just been constructed, there are no field rutting data. Therefore comparative information for these

TABLE 4 Mix Design Data for Field Projects Used to Evaluate DSR Creep Test

	MPL E-3	MPL E-10	STH 72 E-1	STH 54 E-3
Binder(s)	PG 58–28, PG 64–34, PG 70–28	PG 58–28, PG 70–28	PG 64–34	PG 58–28
Gyrations @ N_{design}	75	100	60	75
ESAL range × 10^6	1 to < 3	3 to < 10	0.3 to < 1	1 to < 3
Asphalt Content,%	6.0	5.4	6.0	5.2
Voids Filled with Asphalt,%	72.6	71.8	71.9	71.5
Voids in Mineral Aggregate,%	14.6	14.2	14.2	14.0

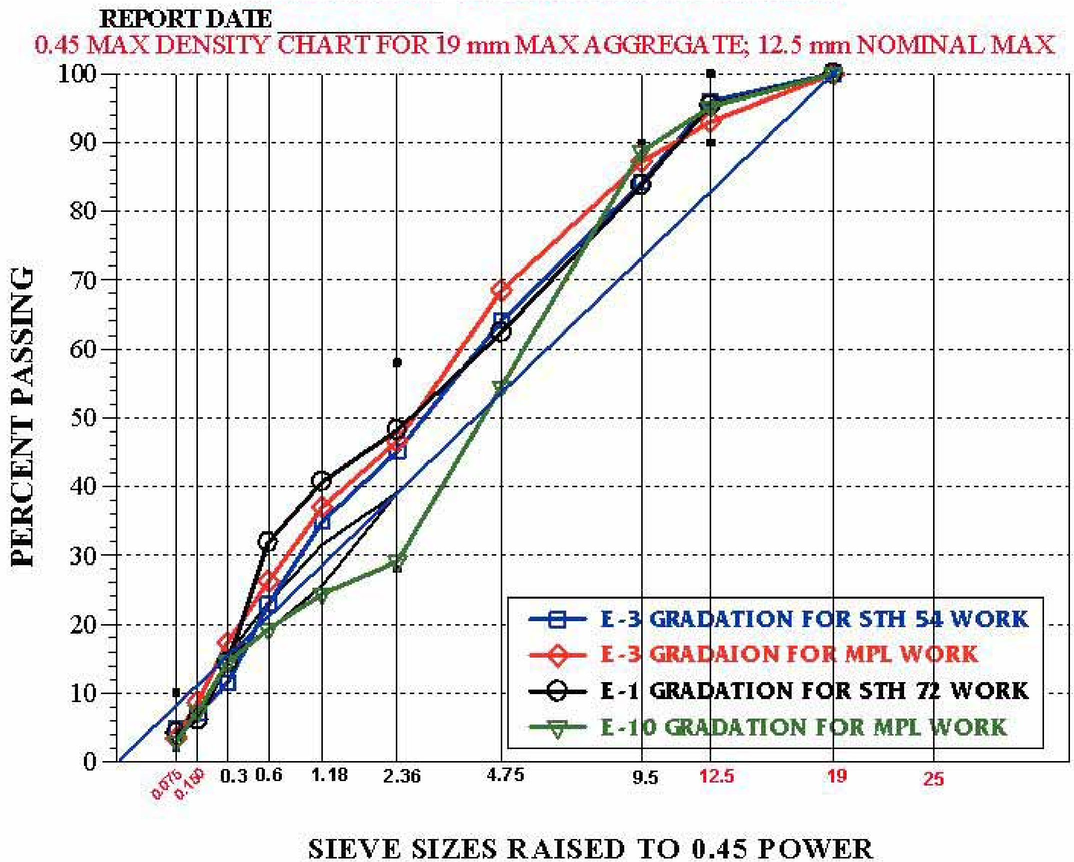

FIGURE 31 Aggregate gradations: E-10, E-3, and E-1 gradations.

mixes has been compiled. In addition, laboratory-compacted mixes from two other paving projects also were tested. One of these projects used an E-1 aggregate gradation (0.3 to 1 million ESALs pavement) with a PG 64–34 polymer-modified binder and the other project used a different E-3 aggregate gradation with a PG 58–28 unmodified binder. Those gradations also are shown in Figure 31.

Tables 4 and 5 show the statistical analysis results for DSR creep testing work performed on all of these mixes at both the 3.5% and 7% air voids levels. All tests were conducted at 58°C using a 68-kPa stress level. Some of the mixes failed very rapidly, but many did not fail over the course of the test. Because of this range of test results, the following data were collected and analyzed: time to secondary flow and percent strain at secondary flow in addition to percent strain at 5 and 30 test cycles. Secondary flow is defined as the point in the creep test at which the initial rapid nonlinear change in strain of the mix enters the period of linear or plastic flow (refer to Figure 9). As can be seen from an examination of the statistical data, there is a range of variability, depending on the sample being tested. The coefficient of variation ranges from the 5% to 6% range to as high as 50%, in some cases. Generally, the coefficients of variation are in the mid-10 to mid-20 in value. Destructive tests generally have greater variability than do nondestructive tests do. In addition, these tests are being conducted on bituminous mix samples which themselves are heterogeneous. The real test of whether the variability of the DSR creep test is acceptable is the predictive ability of the test results. The information presented in this paper demonstrates that for both the MnRoad and ALF mixes, the DSR creep results did an excellent job of predicting rutting behavior.

CREEP EVALUATION OF BINDER AND AGGREGATE VARIABLES

The next challenge was to determine whether the DSR creep test could be used to test newly paved mixes and to provide some insight into the potential performance of those mixes. Test results for the E-3 and E-10 mixes are summarized in Figures 32 through 34. Figure 32 shows a data plot for each of the 3.5% target air voids mixes to show how these different kinds of mixes behave relative to each other. Except for the PG 58–28 E-3 mix, all of the binder and mix combinations show very good resistance to deformation. Figure 33 summarizes the creep results for all of the 3.5% air voids mixes. The polymer-modified E-3 and E-10 mixes all exhibited very little deformation. When the PG 58–28 E-3 and E-10 mixes were compared, the coarser E-10 mix exhibited less than half the deformation of the E-3 mix. Because these two mixes were produced with the same PG binder, the difference in performance must be caused primarily by the difference in aggregate structure. Keep in mind that the E-10 mix contained 0.6 percentage points less binder than did the E-3 mix. This lower binder content and lower voids filled with asphalt also would contribute to a resistance to creep deformation for the E-10 mix. The E-1 mix produced with the PG 64–34 binder had approximately the same percent strain as the PG 58–28 E-3 mix. How much of the STH 72 E-1 mix performance is due to the polymer-modified binder relative to the unmodified PG 58–28 is difficult to ascertain. In the case of the MPL E-3 mix and the STH 72 mix, the binder content for both mixes was 6.0%. An evaluation of the same E-1 aggregate with the PG 58–28 would be needed to sort out the binder effect from the aggregate effect. The E-3 mix used on STH 54 exhibited substantially higher strain compared with all the mixes. The two E-3 aggregates were from different sources, and therefore could be expected

TABLE 5 Statistical Data for 3.5% Air Voids Mix Samples

Mathy Parking Lot, 3.5% Air Voids, 68 kPa, 58°C, DSR Creep Test of Mix				
	Time to Secondary Flow	**% Strain at Secondary Flow**	**Permanent % Strain at 30th Cycle**	**Permanent % Strain at 5th Cycle**
MPL 58–28, E-3, 6 Tests				
Average	219.2	2.8013	2.7866	1.0878
Standard Deviation	74.41	0.33467	0.34431	0.14560
Coefficient of Variation	33.94%	11.947%	12.356%	13.384%
MPL 58–28, E-10, 4 Tests				
Average	2254	3.2294	1.2425	0.4624
Standard Deviation	1175	0.22335	0.44515	0.18950
Coefficient of Variation	52.16%	6.916%	35.826%	40.980%
MPL 64–34, E-3, 2 Tests				
Average	3406	1.1564	0.3975	0.1745
Standard Deviation	1124	0.23112	0.03775	0.03830
Coefficient of Variation	33.01%	19.987%	9.498%	21.954%
MPL 70–28, E-3, 4 Tests				
Average	3464	1.1542	0.4198	0.1820
Standard Deviation	235.1	0.27973	0.11665	0.05310
Coefficient of Variation	6.789%	24.236%	27.785%	29.183%
MPL 70–28, E-10, 4 Tests, 3 Tests Used for Average				
Average	3741	1.2630	0.4193	0.1794
Standard Deviation	357.6	0.17410	0.08669	0.03642
Coefficient of Variation	9.560%	13.785%	20.677%	20.300%
STH72 64–34, E-1, 5 Tests				
Average	262.9	3.2350	2.7172	1.1270
Standard Deviation	31.94	0.41920	0.41315	0.21982
Coefficient of Variation	12.15%	12.958%	15.205%	19.505%
US-35 and US-54, 58–28, E-3, 3 Tests				
Average	30.93	2.39	8.51–1 Test	1.96
Standard Deviation	6.743×10^{-7}	0.125635	—	0.100009
Coefficient of Variation	2.180×10^{-6}%	5.2472%	—	5.0901%

to behave differently. It would appear, from this work, that the mix placed on STH 54 has a greater potential for rutting than the other mixtures do. The STH 54 mix used only 5.2% asphalt cement (AC), but it also used the aggregate blend closest to the maximum density line (Figure 31). One would expect a mixture with lower AC content to have greater resistance to deformation, yet this mix did not. Seeking explanations on the basis of component factors of a mix design may prove to be an academic exercise. If aggregate gradation, aggregate angularity, binder content, and binder properties (i.e., mix design factors) could always define performance, there would be no need for a mechanistic mixture test. The results in Figure 34 show a comparison between the percent strain results for the 3.5% air voids and 7% air voids mixes (Table 6). As one would expect, the 7% air voids mixes all exhibit higher levels of strain than do the 3.5% air voids mixes. Except for this difference, the behavior of the mixes compacted to 7% air voids mirrors the behavior of the mixes compacted to 3.5% air voids. It is clear that for each

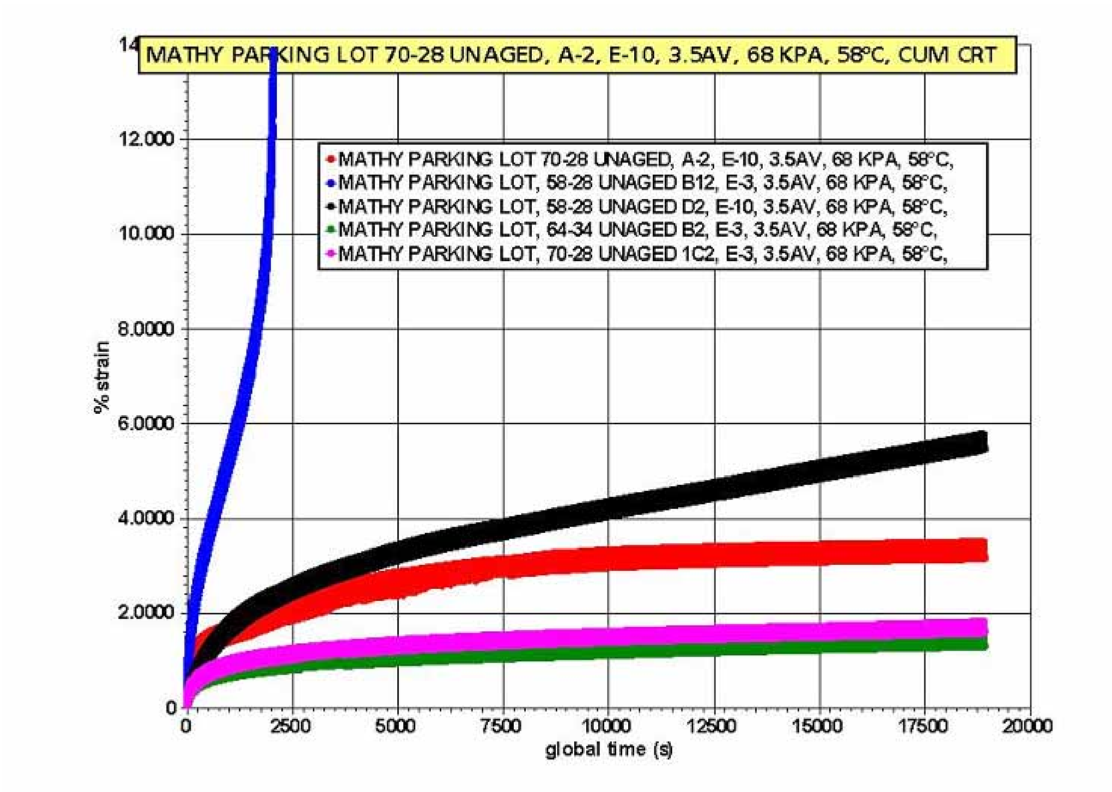

FIGURE 32 Data traces for E-10 and E-3 mixes using PG 70–28, 64–34, and 58–28.

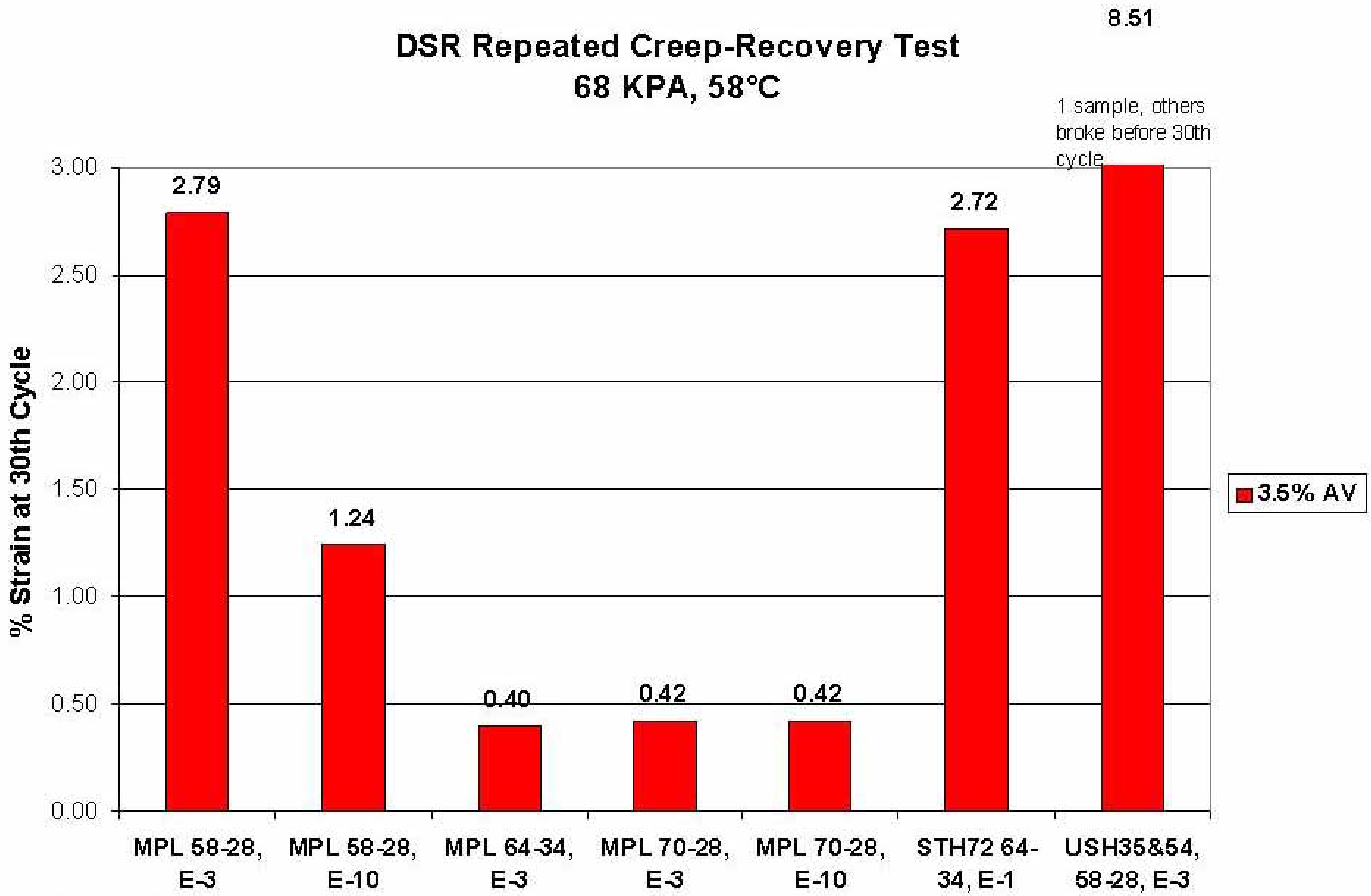

FIGURE 33 Dynamic creep test results for field mixes at 3.5% air voids.

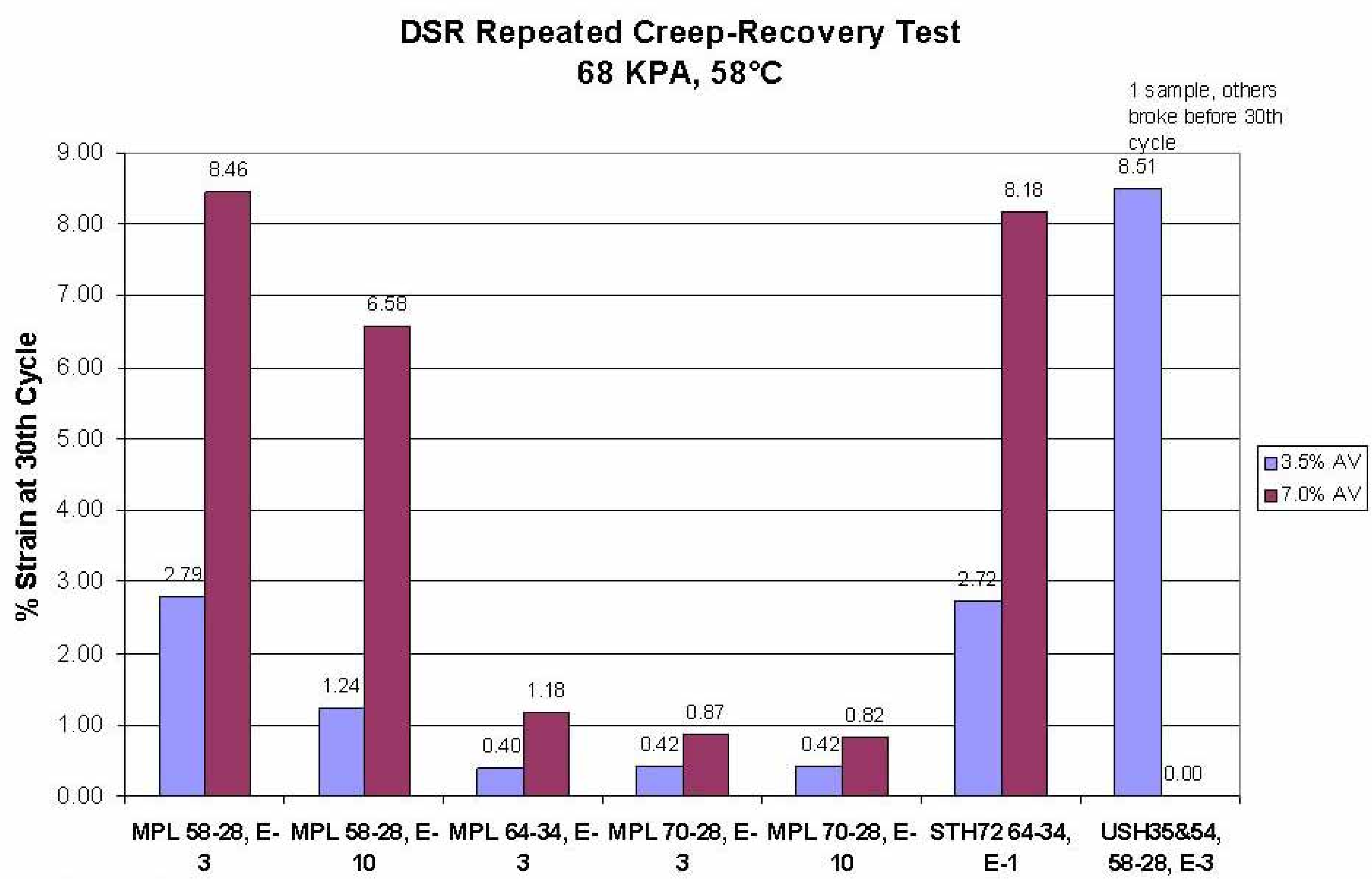

FIGURE 34 Dynamic creep test results of field mixes for both 3.5% and 7% air voids.

TABLE 6 Statistical Analysis for 7% Air Voids Mix Samples

Mathy Parking Lot, 7.0% Air Voids, 68 kPa, 58°C, DSR Creep Test of Mix				
	Time to Secondary Flow	**% Strain at Secondary Flow**	**Permanent % Strain at 30th Cycle**	**Permanent % Strain at 5th Cycle**
MPL 58–28, E-3, 3 Tests				
Average	57.60	3.1288	8.46–2 Tests	2.1588
Standard Deviation	5.774	0.87438	1.5686	0.59618
Coefficient of Variation	10.02%	27.946%	18.530%	27.617%
MPL 58–28, E-10, 3 Tests				
Average	60.85	2.8905	6.5763	1.9922
Standard Deviation	19.99	0.41612	0.77666	0.18456
Coefficient of Variation	32.84%	14.396%	11.810%	9.264%
MPL 64–34, E-3, 3 Tests				
Average	3288	3.1391	1.1783	0.52918
Standard Deviation	910.0	0.17307	0.10857	0.05163
Coefficient of Variation	27.68%	5.514%	9.214%	9.756%
MPL 70–28, E-3, 3 Tests				
Average	3041	2.4829	0.86938	0.36016
Standard Deviation	622.3	0.13980	0.04070	0.01879
Coefficient of Variation	20.46%	5.630%	4.682%	5.217%
MPL 70–28, E-10, 3 Tests				
Average	4441	2.7938	0.82073	0.34422
Standard Deviation	424.6	0.39841	0.14593	0.06494
Coefficient of Variation	9.56%	14.261%	17.781%	18.865%
STH72 64–34, E-1, 5 Tests				
Average	22.93	3.4560	8.18–1 Test	3.1242
Standard Deviation	13.04	0.42133	—	0.96218
Coefficient of Variation	56.86%	12.191%	—	30.797%
US-35 and US-54, 58–28, E-3, 4 Tests				
Average	18.43	2.5830	—	2.7477
Standard Deviation	5.00	0.69639	—	0.63189
Coefficient of Variation	27.13%	26.960%	—	22.997%

mix type the DSR creep test is reproducibly testing the aggregate structures regardless of voids level. For example, the relative relationship between the E-3 PG 58–28 MPL mix and the E-1 PG 64–34 STH 72 mix is the same for both air voids levels. If the DSR creep test was not able to accurately respond to the aggregate structures and binder contents, the authors do not believe that this level of agreement would be seen between the 3.5% and 7% air voids test results.

CONCLUSIONS

Static and dynamic creep tests for bituminous mixtures were developed using a DSR as the testing device. The work reported here concentrated on the dynamic or repeated creep test.

Recommendations for performing the test are as follows:

1. Use 10-mm-thick slices to minimize variability, especially for coarse mixtures.
2. Perform the test at the appropriate climatic temperature for the job location and pavement layer. Resist the temptation to test at the PG grade temperature. The beneficial effect of polymer-modified binders can be properly ascertained in a mix structure only if the mixture is being tested at an appropriate service temperature. Conversely, comparing polymer-modified and unmodified mixtures at inappropriately high temperatures does not inform the designer as to the potential for success of the conventional binder.
3. Use a 68-kPa test stress, although lower stress values may need to be used. So far, only limited work has been performed at 34 and 17 kPa, and the effect of changing stress levels on the understanding of mix performance normalized to 68 kPa is not known.
4. Test laboratory specimens for mix design purposes. Testing field cores yields much lower response values than testing laboratory-compacted specimens even though air voids are similar.

On the basis of this work, the following conclusions can be drawn:

1. The dynamic creep test can identify the effect of aggregate structure, mix design, binder grades, and service temperatures.
2. For the two projects evaluated (MnRoad and FHWA ALF), the results of the DSR dynamic creep test correlate extremely well to rutting behavior of the mixes in the field.
3. The DSR dynamic creep test could be suitable as a mix design tool; however, additional work needs to be performed to identify levels of test response needed for mixture performance in the field. These levels of response need to be related to the design ESAL count of the pavement in question.
4. Several test response parameters should be investigated to determine which ones are the most useful candidates for a mix design parameter. Some candidates are time to 5% strain, flow number, percent strain, or zero shear viscosity at some number of cycles before the onset of tertiary flow. This typically could be 50 to 200 cycles, but it needs to be at a point when the integrity of the test specimen is still intact.
5. The DSR dynamic creep test is suitable as a HMA quality control tool. Volumetric quality control specimens could be prepared for creep testing within a 6-hour time period. Thus, creep response data could be available on the same day the mix was placed.
6. Equipment costs for the rheometer and the saws are approximately $75,000. A decision would need to be made as to whether such creep analysis of production samples should be performed in the field or at a central laboratory location.

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NOTES

1. The Wisconsin Mix "E" notation refers to pavement ESAL design level. The mix is defined primarily by the N_{design} values at which mixture volumetrics must be achieved.

Indirect Tension Strength as a Simple Performance Test

Donald W. Christensen
Ramon Bonaquist
Advanced Asphalt Technologies, LLC

David A. Anderson
Pennsylvania State University

Salil Gokhale
Applied Research Associates

The use of the indirect tension (IDT) strength test at high temperature was demonstrated as a simple test for evaluating the rut resistance of asphalt concrete. The data presented in this paper were gathered in a study on triaxial strength tests on asphalt concrete in which mixture cohesion and internal friction were determined for a variety of mixtures and were related to their rut resistance. The IDT strength tests were performed as part of a simplified protocol to determine mixture cohesion and internal friction. It was discovered that the IDT strength not only provided an excellent measure of mixture cohesion but also correlated very well both to laboratory indicators of rut resistance and measured rutting rates in actual pavements. The IDT strength test is an extremely simple and inexpensive procedure that is ideally suited to quality control tests and potential forensic studies.

This paper describes research that suggests that the indirect tension (IDT) strength test, performed at high temperatures, is potentially a simple and accurate test for evaluating the rut resistance of asphalt concrete mixtures. The research described in this paper was originally undertaken to evaluate the relationships of triaxial strength test data, mixture cohesion, and internal friction, and rut resistance (*1*). The original objective was to determine whether the triaxial strength test might be effective as a simple performance test for inclusion in the Superpave® system of mixture design and analysis. The IDT strength was included in this study as part of a simplified procedure for determining mixture cohesion and internal friction. However, after analyzing the data, surprisingly strong correlations were observed between IDT strength at high temperature, laboratory tests of rut resistance, and field rutting rates. These correlations suggested that the IDT test in and of itself might be an extremely simple and effective test for evaluating the rut resistance of asphalt concrete mixtures (*1*). This paper therefore deals in part with triaxial strength testing, but it emphasizes the relationship between IDT strength and rut resistance. The results of this research are presented in substantially greater detail in the project report submitted to the Pennsylvania Department of Transportation (PennDOT) (*1*).

BACKGROUND

The Superpave mixture design and analysis system—unlike its predecessor, the Marshall mix design method—originally included no strength or stiffness test as a final step in evaluating paving mixtures. Many practicing pavement engineers and technicians were uncomfortable with the lack of a "proof" test. Therefore, much attention has been given over the past 5 years to developing a simple performance test, primarily for the purpose of evaluation the rut resistance of asphalt concrete mixtures designed using the Superpave system. The triaxial strength test was one of the tests recently evaluated by NCHRP Project 9–19 as a potential simple performance test. However, the evaluation described in this paper is substantially different from the one performed as part of NCHRP 9–19. The test conditions were much different—the strength tests were performed at much higher temperatures and slower loading rates in NCHRP Project 9–19, minimizing the contribution of binder consistency to the mixture response. Furthermore, in the research discussed in this paper, a greater effort was made to analyze and interpret the results of these strength tests. This study also included the IDT strength test as part of a simplified or abbreviated protocol. As a result, and of most practical importance, the IDT strength test was identified as a potentially economical and effective performance test (*1*).

Triaxial Strength Testing and Mohr-Coulomb Failure Parameters

Triaxial strength testing is essentially a method for evaluating the effects of confining pressure on the strength of granular materials. This technique was originally developed by soils engineers and scientists and is closely associated with the Mohr-Coulomb failure theory (*2*). In a typical triaxial strength test, a soil specimen is prepared for testing with an aspect ratio of 2 to 1 (height to diameter). The specimen is encased in a latex membrane and placed inside a specially designed pressure vessel, called a triaxial cell, in which pressure can be applied to the specimen while applying a compressive load. Triaxial cells for soil testing also have provisions for keeping a specimen saturated and controlling the internal pore water pressure during the test (called "back pressure"), although this is not an essential part of triaxial strength testing for paving materials.

In analyzing triaxial strength tests, Mohr-Coulomb failure theory usually is applied. This theory is simply a way of mathematically representing the relationship between a confining stress and failure stress for granular materials. Because of space limitations, complete details of this theory cannot be presented here; for more information, refer to an introductory text on soil mechanics (*1*) for this information and to the final project report (*2*).

From a practical point of view, triaxial testing and the ensuing analysis produce two parameters of interest to materials and pavement engineers: internal friction ϕ and cohesion c. Internal friction is an important parameter for granular materials; it indicates the degree of interaction among particles. Granular materials consisting of strong, cubicle aggregates will have a high value for ϕ, indicating a strong dependence of strength on confining stress. Materials consisting of smooth, spherical particles will have small values for ϕ, indicating little or no increase in strength with confining stress. The cohesion theoretically represents the shear strength at zero confining pressure. For purely granular materials, c equals 0. For materials containing clay or other plastic fines, the cohesion will have some positive value. In the case of asphalt concrete mixtures, the cohesion is a function of the quantity and consistency of the asphalt cement binder. Under confinement, the strengths of materials that contain both plastic fines (or asphalt binder) and granular materials will depend on both the cohesion and the internal friction.

The values of c and ϕ are normally determined from triaxial tests by first converting the test results into p and q coordinates (*2*):

$$p = \frac{\sigma_1 + \sigma_3}{2} \tag{1}$$

$$q = \frac{\sigma_1 - \sigma_3}{2} \tag{2}$$

Where q is the minimum principal stress at failure, and σ_3 is the maximum principle stress at failure for a given test. For an unconfined strength test, for example, p is equal to q, and both are equal to the failure stress divided by 2. In practice, when analyzing triaxial strength data, failure points for each of several tests are converted to p, q coordinates using Equations 1 and 2. Then linear regression analysis is used to determine the intercept, a_0, and slope, a_1, of the resulting line, called the K_f-line. Next the values for internal friction and cohesion are calculated from the following parameters (*2*):

$$\phi = \sin^{-1}(a_1) \tag{3}$$

$$c = \frac{a_0}{\cos\phi} \tag{4}$$

Mohr-Coulomb failure parameters also theoretically can be determined by performing one set of tests in unconfined compression and a second set in either simple tension or split tension. The latter approach is more common because soils and granular composites such as concrete are difficult to test in pure tension. Consider an indirect tensile test with a material having a Poisson's ratio of $\nu = 0.5$. The magnitude of the compressive stress at failure for such a material would be three times the tensile stress, $\sigma_y = 3\sigma_x$. Thus, the p value would be $(\sigma_y - \sigma_x)/2 = (3\sigma_x - \sigma_x)/2 = \sigma_x$ (remembering that the IDT strength σ_x is negative). In other words, the p value is simply equal to the absolute values of the IDT strength. Similarly, the q value would be $(\sigma_y + \sigma_x)/2 = (3\sigma_x + \sigma_x)/2 = 2\sigma_x$, or, simply put, q is equal to twice the IDT strength. The advantage of this approach is that it is potentially much quicker and simpler compared with the standard protocol and does not require a triaxial cell or any other special equipment.

Although this discussion was limited to soils, as will be discussed in the following sections, much of this theory is directly applicable to asphalt concrete pavement at intermediate to high temperatures. Under these conditions, the mechanical behavior of asphalt concrete is, in many ways, similar to a partly cohesive, granular soil. Several researchers have used triaxial testing and the Mohr-Coulomb failure theory to characterize asphalt concrete, including Nijboer (*3*) and Hewitt (*4*). Nijboer found that the results of triaxial testing were both time and temperature dependent. He also concluded that stiffer binders tended to decrease internal friction while increasing cohesion. In general, increasing binder content leads to a decrease in internal friction. Increasing coarse aggregate content will generally increase internal friction in a mixture (*3*). Hewitt developed a complete mixture design system based on shear strength, relying primarily on triaxial strength testing and related analyses (*4*). In general, his findings support Nijboer's work.

Determination of Appropriate Test Conditions for Strength Testing of Asphalt Concrete Mixtures

Because the mechanical response of asphalt concrete is both time and temperature dependent, the temperature and loading rate used in strength tests intended to evaluate resistance to permanent deformation must at least approximately duplicate conditions in a pavement at high temperature under traffic loading. Theoretically, the best approach would be to perform the test at the maximum 7-day average high pavement temperature (T_{max}) at a very high rate of loading. However, such rapid loading would be difficult to control and would cause transient loads and other dynamic effects that would be difficult or impossible to measure and analyze. The approach taken here was to approximately apply time–temperature superposition to establish a somewhat lower test temperature and a reasonably slow loading rate that would be approximately rheologically equivalent to traffic loading at the critical pavement temperature for rutting.

In the analysis as described in detail in the final project report, a typical pavement structure of 150-mm asphalt concrete over 150-mm granular base material was assumed, with properties typical for a hot summer day (*1*). A layered elastic analysis indicated that a dual wheel inflated to 690 kPa would cause a strain of 0.0053 at 50-mm depth. A vehicle traveling at 48 kph would have an equivalent triangular pulse time of 0.06 s, so the approximate strain rate under these conditions would be 0.0053/(0.06/3) = 0.18 s^{-1} (*5, 6*). For a 150-mm high gyratory specimen, this is equivalent to a loading rate of 1.6 m/min (*1*).

Krutz and Sebaaly found typical failure strains in compression of approximately 2%. To obtain a reasonable failure time of approximately 20 s, this would translate to a loading rate of approximately 9 mm/min for a 150-mm high specimen (*7*). Applying an Arrhenius shift function with a typical activation energy for asphalt concrete of 200 kJ/mol-°K (*8*), the required temperature to obtain rheological equivalence to 1,600 mm/min at 53°C at a 9-mm/min loading rate would be 31.6°C, a difference of 21.4°C. For simplicity, and because this analysis is only approximate and uses typical values, the suggested protocol is to test at a temperature 20°C below T_{max} at a rate of 7.5 mm/min. This latter rate is suggested to maintain consistency with the current ASTM specification for compressive strength of asphalt concrete (ASTM D-1074). A related analysis for IDT strength tests indicated that the appropriate loading rate should be somewhat lower—3.75 mm/min (*1*).

In the verification study, Gokhale performed a more rigorous viscoelastic analysis of strains in several representative pavement structures and determined that an equivalent typical vertical loading rate for a 150-mm high specimen would be 445 mm/min rather than the 1,600 mm/min estimated in the original study (*9*). To maintain a 7.5 mm/min loading rate, this would suggest a test temperature approximately 5°C higher than originally recommended. However, to maintain consistency with the original study, Gokhale performed tests at the same rate used in the original study (*9*).

MATERIALS, METHODS, AND EXPERIMENT DESIGN

Two groups of mixtures were used in this research project. Six mixtures were based on materials used in the PA-11 project in Pennsylvania, and four were based on mixtures used in the New York Superpave implementation effort and studied by the Asphalt Institute (*10–12*). These mixtures were selected for a variety of reasons. The PA-11 mixtures were included because of the relatively thorough documentation of the properties of these materials and their performance in the field, as well as the range of aggregates and binder used. The N_{design} mixtures were included to evaluate the effect of level of compaction on the triaxial strength data and resulting Mohr-Coulomb failure parameters. [N_{design} is the design number of gyrations for Superpave asphalt concrete (AC) compaction.] These projects and the materials used during their construction are described in detail in the following sections of this paper.

PA-11 Intersection Study

This project was completed in October of 1991. The test section is located on PA-11 in Cumberland County between segment 0660/offset 2815 and segment 0680/offset 0704 (*10, 11*). This section of pavement was receiving extremely heavy traffic, with a high proportion of trucks, and included numerous intersections and traffic lights. The project involved an overlay of existing pavement, which was milled out to a depth of 4 to 5 in. In the south end of the test section, the milling removed pavement down to the original portland cement concrete (PCC) material; however, at the north end, there was still 3 to 4 in. of bituminous material remaining over the original PCC after milling. The overlays were all placed over 2 in. of ID-2, heavy-duty binder course material. Eight different test materials were included in this test section. ID-3 and ID-2 wearing course mixtures were used, with four different binder types: two unmodified and two modified with common commercial polymer modifiers. The ID-2 wearing course mixture was a Marshall heavy-duty mix design, using 9.5-mm, nominal-maximum size aggregate. The ID-3 mixture was also a Marshall heavy-duty mix design but with a 19-mm nominal maximum aggregate size. Two AC-40 mixtures were placed on the southbound passing lane; two AC-20 mixtures were placed in the northbound passing lane. Two ethyl vinyl acetate (EVA) mixtures were placed in the southbound traffic lane, whereas two styrene butadiene (SB) mixtures were placed in the northbound traffic lane (*10, 11*).

Unfortunately, it was discovered after construction of the project that there were dramatic differences in the traffic level in the traffic and passing lanes. Traffic counts performed over a 2-day period in 1992 gave traffic levels of 2,957 and 44 equivalent single-axle loads (ESALs)/day for the northbound travel and passing lanes, respectively, and 2,653 and 114 ESALs/day for the southbound travel and passing lanes, respectively (*10, 11*). Therefore, the mixtures made using the unmodified binders received much less traffic than the mixtures made using the modified binder. This makes interpretation of the rut depths somewhat complicated.

PennDOT engineers measured rut depths in the test section over a 4-year period after construction. The results are summarized in Table 1 (*10*). Although there does not appear to be much difference in the performance of the binders, keep in mind that the traffic on the pavements made with the EVA- and SB-modified binders was approximately 25 to 60 times greater than that on the pavements made with the AC-40 and AC-20 binders. A linear regression was performed on the log–log transforms of rut depth versus traffic level (R^2 75% to 98%), and the resulting equations were used to estimate the rut depth for each mixture at a traffic level of 1 million ESALs—a level roughly intermediate between the total traffic for the travel and passing lanes. These values are included in Table 1.

Six of the eight PA-11 mixtures were selected for inclusion in this study: both ID-3 and ID-2 wearing course mixtures using the AC-20, EVA-modified, and SB-modified binders. In addition, two ID-3/AC-20 mixtures were included with excess mineral filler—one with 1% excess (AC20/MF+) and one with 2% excess mineral filler (AC20/MF++). These last two mixtures were included because certain parts of the test section were thought to have rutted very quickly because of excessive mineral filler content (*10, 11*).

Suppliers of the original binders and aggregates were contacted. They agreed to supply either the same material as was used for these mixtures or a reasonably close substitute. Table 2 is a summary of the mix designs used to produce the mixtures for this study. Aggregate gradations were based on substantial quality control records, rather than the original mix designs, to reflect the as-placed material. For preparation of the laboratory mixtures, samples of aggregates were obtained from the quarries used during the field project. Samples of the binders

TABLE 1 Rut Depth Measurements for PA-11 Study

Year	Rut Depth, mm							
	EVA		AC-40		AC-20		SB	
	ID-3	ID-2	ID-3	ID-2	ID-3	ID-2	ID-3	ID-2
1992	1.3	0.8	0.5	0.3	2.5	1.3	1.1	1.9
1993	1.7	1.3	1.1	0.8	2.6	2.1	1.8	2.2
1994	1.6	1.9	1.6	1.4	3.3	2.6	2.6	3.3
1995	2.4	1.7	1.9	1.4	4.0	2.8	3.1	3.8
	Estimated Rut Depth at 1Million ESALs							
	1.3	0.9	11.8	13.9	9.2	14.0	1.0	1.7

TABLE 2 Mixture Design Data for PA-1 Mixtures

Property	ID-2	ID-3	ID-3 MF+	ID-3 MF++
Size	*Percent Passing*			
25 mm	100	100	100	100
19 mm	100	95	95	95
12.5 mm	100	78	78	78
9.5 mm	96	68	68	68
4.75 mm	62	52	52	52
2.36 mm	42	36	37	37
1.18 mm	25	21	22	22
0.600 mm	15	12	14	14
0.300 mm	9	8	9	9
0.150 mm	5	5	6	7
0.075 mm	3.8	3.8	4.8	5.8
Asphalt Content (%)	6.3	4.9	Not Determined	
Marshall Stability (lbs)	3225	3864		
Flow (1/100 in.)	11.2	11.7		
Air Voids (%)	3.9	4.0		
Voids in Mineral Aggregate (%)	15.9	13.5		
Voids Filled with Asphalt (%)	75.5	70.5		

used in the construction of the test sections were not available in sufficient quantities for this study. The supplier of binders during the original construction of the project provided an unmodified PG 64–22 and laboratory blended SB-modified binder that were similar to the binders used during construction. The SB-modified binder graded as a PG 76–28. In the interest of brevity, details of the binder test data are included in the final project report (*1*).

New York N_{design} Study

Four Superpave 12.5 mm nominal size mixtures from New York were included in the study. Each of the New York mixtures was designed by the paving contractors using the Superpave volumetric mixture design method. Table 3 summarizes pertinent mix design properties for these mixtures; details for the N_{design} = 126 mix design unfortunately could not be procured. Because

TABLE 3 Design Properties for New York Mixtures

Property	NY-316	NY-12	I-81 (109)	I-81 (126)
Size	*% Passing*			
19.0 mm	100	100	100	100
12.5 mm	100	100	99	99
9.5 mm	88	90	87	90
4.75 mm	55	53	47	46
2.36 mm	32	32	28	28
1.18 mm	23	20	19	18
0.600 mm	15	12	13	11
0.300 mm	8	7	8	7
0.150 mm	5	5	5	5
0.075 mm	4.0	3.2	3.9	3.9
Asphalt Content (%)	5.1	5.1	5.5	5.5
Binder Grade	PG 58–28	PG 58–28	PG 64–28	PG 64–28
N_{design}	76	96	109	126
Air Voids (%)	4.0	3.8	4.0	—
Voids in Mineral Aggregate (%)	14.5	14.8	15.2	—
Voids Filled with Asphalt (%)	72.5	74.6	72.8	—
Filler/Effective Asphalt Ratio	0.7	0.6	0.8	—
% G_{mm} at $N_{initial}$	84.7	84.8	84.2	—
% G_{mm} at $N_{maximum}$	97.7	97.8	97.6	—
Coarse Aggregate Angularity	100/100	100/100	96/92	—
Fine Aggregate Angularity	45.7	48.3	46.4	—
Flat and Elongated	1.0	0.1	0.3	—
Sand Equivalent	66.9	58.0	67.8	—

NOTE. G_{mm}, the maximum theoretical specific gravity; $N_{initial}$, the initial number of gyrations for Superpave AC compaction; $N_{maximum}$, the maximum number of gyrations for Superpave AC compaction.

quality control data were not available for the New York mixtures, mix designs were based on the job mix formula (*12*). For the N_{design} = 126 mixture, the parameters for the N_{design} = 109 mixture were used as a guideline.

Samples of binders of the same grade as those used in construction were obtained from the suppliers. The NY-316 and NY-12 projects used a PG 58–28 binder from one supplier; the two I-81 projects used a PG 64–22 binder from a second supplier. As with the PA-11 binders, details of the binder grading can be found in the project report (*1*).

Verification Study

Because some of the findings of the initial research reported in this paper were so surprising, a second study was undertaken soon after its completion to verify and extend the results to other materials (*9*). Space limitations prohibit detailed discussion of the materials and methods used in this study. Nine different mixtures were included in the second study: four were mixtures included in a PennDOT validation study performed by the Pennsylvania State University for PennDOT (*13*); the other five were materials used in the FHWA Accelerated Loading Facility (ALF) rutting study (*14*). These mixtures were made using five different binders—an AC-5, AC-10, AC-20, and two polymer-modified binders with excellent high temperature properties (*14*). The inclusion of the ALF mixtures was extremely important because the ALF study was well controlled and included substantial rut depth information as a function of loading. The specimen preparation and test methods used by Gokhale were essentially identical to those used in this study, which are summarized as follows (*9*).

Specimen Preparation

All specimens were prepared using an Interlaken gyratory compactor following procedures as outlined in AASHTO TP4 except that the mass of the batches was adjusted to obtain specimens with required heights for the various tests. After mixing, the material was short-term oven aged in accordance with AASHTO PP2 at a temperature of 135°C for 4 h. The mixtures required different levels of compactive effort to produce specimens within the specified air void tolerances. The triaxial specimens, which were 140-mm high × 70-mm in diameter, were cored from the center of the gyratory specimens using a standard electric coring drill. A special stand was fabricated to hold the drill and specimen in alignment during coring. A double-bladed saw was used to saw all specimens to ensure parallel specimen ends. The compressive strength and indirect tensile strength tests used specimens directly from the gyratory compactor. The compressive strength specimens were 150-mm high, whereas the IDT specimens were 100-mm high; both were 150 mm in diameter. The repeated shear at constant height (RSCH) specimens were sawn from gyratory specimens and were 50-mm thick × 150-mm diameter. The target air void content for the final test specimens was 4.0% with a tolerance of ± 0.5%.

Test Procedures

Triaxial strength tests were performed on a servohydraulic testing system. The tests were performed using a standard soil triaxial test cell for 70-mm diameter specimens purchased from a commercial laboratory supplies vendor. The tests were performed unconfined and with 207 kPa confining pressure, at a loading rate of 7.5 mm/min. All tests were preformed at 33°C.

The tests using the abbreviated protocol were performed on the same servohydraulic system. The compression specimens were capped using reusable capping sets as used in testing PCC cylinders (ASTM C 1231, AASHTO T22). This system consists of two steel retainers, each containing a Neoprene pad. The system was modified by using 40-durometer neoprene, which is somewhat softer than the 50-durometer generally used for testing ordinary PCC. The compression tests were performed using a loading rate of 7.5 mm/min at a temperature of 33°C. The IDT strength tests were performed at a loading rate of 3.75 mm/min using a standard Lottman breaking head for 150-mm diameter specimens (ASTM D 4123, AASHTO T283). As with the other strength tests, the test temperature was 33°C.

The RSCH test was performed on a Superpave shear test (SST) system, according to procedures described in AASHTO TP7–94. The test was conducted at the maximum 7-day average pavement temperature for south-central Pennsylvania (T_{max}), 53°C. The main data produced from this test are the maximum permanent shear strain (MPSS) values, which increase with decreasing rut resistance.

RESULTS OF THE INITIAL STUDY

Results of the unconfined and confined compressive strength tests (standard protocol) and unconfined compressive and IDT strength tests (abbreviated protocol) are given in Table 4. This table also includes the results of the RSCH tests performed on the SST. Multiple regression analyses were performed to determine the intercepts and slopes of the functions relating *p* and *q* values for strength data from both the standard triaxial tests and the abbreviated protocol. In general, the standard deviations for the parameters are quite high, indicating a high degree of variability in the data. To try to develop better estimates, regression analyses were performed using both sets of data simultaneously. Including all data in the regression analysis significantly improved the precision of the parameter estimates. For example, the standard deviation for the cohesion estimates (intercepts) using this method ranged from approximately 40 to 50 kPa, whereas the standard deviation values using the standard triaxial data ranged from approximately 130 to 180 kPa. Standard deviations for cohesion estimates using the abbreviated protocol data were all approximately 120 kPa. A similar significant improvement in precision was apparent in the internal friction (slope) estimates. This improvement in precision is probably caused by an increase in the number of data points and an increase in the overall range of the data when using the full data set. The estimates using the complete data set should therefore be considered most reliable. Using *p* and *q* values from all three sets of data, *c* and ϕ values were determined for all mixtures. These are summarized in Table 5, which includes the slope and intercept values used in the calculations (see Equations 3 and 4).

DISCUSSION OF RESULTS

Several observations can be made from Tables 4 and 5 concerning the data generated using the two methods.

In general, the agreement between the standard triaxial tests and the abbreviated protocol is good. However, the compressive strengths for the 150 × 150-mm gyratory specimens tend to be slightly lower than for the 70 × 140-mm cores. The most likely cause for this difference is the higher air void content at the ends of the untrimmed gyratory specimens, which unfortunately was not quantified and cannot be used to develop a correction factor. The analyses using abbreviated protocol data, in most cases, produced slightly lower cohesion values and slightly higher internal friction values compared with analyses using data from the standard triaxial procedure. As discussed previously, the abbreviated protocol produced more precise parameter estimates, even though, in many cases, significantly fewer replicate measurements were made.

TABLE 4 Results of Triaxial Tests and Repeated Shear Tests

Mixture	Compressive Strength or IDT Strength:				RSCH Max. Perm. Shear Strain, (%)
	70 × 140-mm Cylinders Cored from Gyratory Specimens		150 or 100 × 150 mm Gyratory Specimens		
	Comp., Unconfined, (MPa)	Comp., Confined, (MPa)	Comp., Unconfined, (MPa)	IDT, (kPa)	
ID-2, SB	3.61	4.00	3.37	462	1.05
ID-3, SB	3.41	4.14	3.15	483	.65
ID-2, AC-20	3.23	3.78	2.66	386	1.72
ID-3, AC-20	2.96	3.36	2.36	393	1.13
ID-3, AC-20 MF+	2.91	3.43	2.44	400	.98
ID-3, AC-20 MF++	2.97	3.32	—	—	1.14
NY-316, N_{design} 76	1.79	2.45	1.72	200	2.84
NY-12, N_{design} 96	2.38	3.10	2.03	262	2.37
I-81, N_{design} 109	2.00	2.34	2.29	255	2.89
I-81, N_{design} 126	2.22	2.80	2.41	290	2.20

TABLE 5 Values for *c* and ϕ

Mixture	Intercept (kPa)	Slope (degrees)	*c* (kPa)	ϕ (degrees)
From Standard Triaxial Testing				
ID2/AC20	721	0.553	865	33.5
ID3/AC20	859	0.390	933	23.0
ID2/SB	735	0.581	903	35.5
ID3/SB	707	0.582	869	35.6
ID3/AC20/MF+	802	0.449	898	26.7
ID3/AC20/MF++	806	0.426	891	25.2
NY76	374	0.576	457	35.2
NY96	476	0.596	592	36.6
NY109	652	0.348	695	20.4
NY126	503	0.551	603	33.4
From Abbreviated Protocol				
ID2/AC20	538	0.597	670	36.7
ID3/AC20	590	0.517	690	31.1
ID2/SB	627	0.635	812	39.5
ID3/SB	699	0.558	842	33.9
ID3/AC20/MF+	582	0.508	675	30.5
ID3/AC20/MF++	—	—	—	—
NY76	256	0.704	360	44.8
NY96	352	0.655	465	40.9
NY109	324	0.717	465	45.8
NY126	384	0.682	526	43.0
Using All Data				
ID2/AC20	510	0.640	664	39.8
ID3/AC20	568	0.561	685	34.1
ID2/SB	630	0.633	814	39.3
ID3/SB	676	0.592	838	36.3
ID3/AC20/MF+	533	0.599	665	36.8
NY76	277	0.662	369	41.4
NY96	354	0.675	480	42.5
NY109	377	0.598	471	36.7
NY126	421	0.617	534	38.1

The parameters found using the abbreviated protocol also appear to make better sense intuitively; the SB-modified binder, for example, shows improved cohesion values for the abbreviated protocol but not for the standard method. The abbreviated protocol appears viable and produces relatively accurate estimates of cohesion and internal friction, perhaps even more reliable than those found using standard triaxial data.

As discussed previously, field data on rutting exists only for the PA-11 mixtures. Rut depths at a traffic level of 1 million ESALs were estimated for each of the four primary PA-11 mixtures included in this study (see Table 1). Because of these limited data, elaborate analyses of strength parameters and observed rut resistance could not be performed. The approach used here involved calculation of R^2 values for simple linear relationships between observed rutting and primary strength parameters, such as cohesion c and angle of internal friction ϕ. Values determined from the standard tests, the abbreviated protocol, and the combined data set were all included in the analysis. Also included were the values for MPSS from the repeated shear at constant height data. Only four parameters showed R^2 values greater than 80%: MPSS (R^2 = 81%); cohesion, as determined using the abbreviated protocol and using the combined data (R^2 = 93% and 95%, respectively); and IDT strength (R^2 = 96%). As an example of these relationships, Figure 1 shows estimated rut depth at 1 million ESALs plotted as a function of cohesion as determined using the abbreviated protocol.

Regression analyses were performed using triaxial strength parameters as predictors for MPSS as determined from the RSCH. The best overall predictor of MPSS was found to be mixture cohesion, with R^2 values of 76%, 62%, and 74% for the standard, abbreviated protocol, and full data set, respectively. Unconfined compressive strength was also a fair predictor of MPSS, with R^2 values of 81%, 41%, and 67% for the standard, abbreviated protocol, and full data set, respectively. As seen in Figure 2, the relationship between MPSS and IDT strength was quite good, with an R^2 value of 80%.

These strong relationships between RSCH data and mixture cohesion, and especially IDT strength, may at first appear confusing and potentially spurious. However, this relationship has a simple and reasonable explanation. IDT strength is an excellent measure of mixture cohesion; because most asphalt concrete mixtures being placed today contain good quality, angular aggregates, their rut resistance becomes largely a function of cohesion. Therefore, IDT strength not only provides a good measure of cohesion but also relates very well to rut resistance for most mixtures.

VERIFICATION STUDY

Gokhale's study largely confirmed the findings of the original research (*9*). A strong correlation was observed between MPSS and IDT strength at high temperature (R^2 = 88%). As with the original study, an excellent correlation (R^2 = 99%) was found between mixture cohesion and IDT strength, verifying the usefulness of the IDT test in measuring this engineering property (*9*).

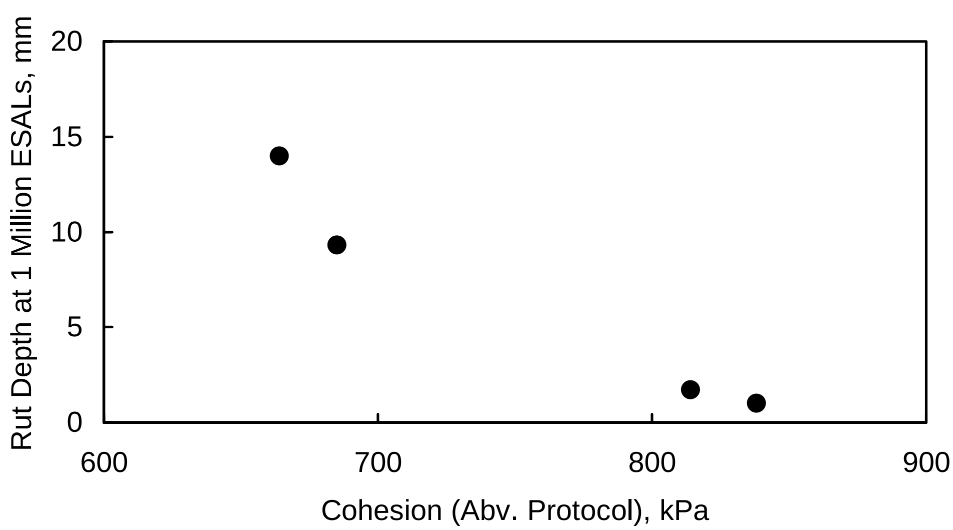

FIGURE 1 Plot of rut depth at 1 million ESALs versus cohesion for PA-11 mixtures.

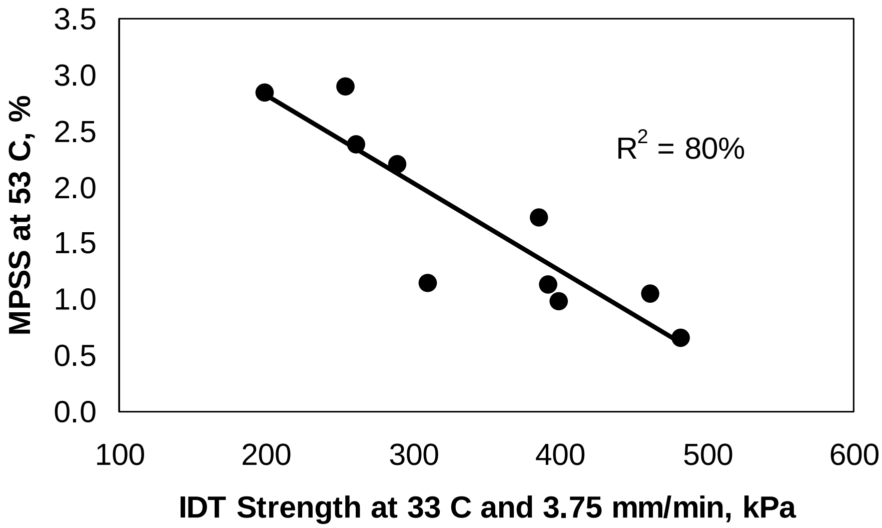

FIGURE 2 Relationship between RSCH maximum permanent shear strain and IDT strength.

Perhaps most significantly, an excellent correlation (R^2 = 94%) also was observed between the rutting observed in the ALF study and IDT strength, as shown in Figure 3. This plot shows the number of ALF wheel passes to rut depths of 10, 15, and 20 mm for each of the five ALF mixtures, corrected to 4% air voids; the relationship is stronger than that reported by Gokhale, who did not correct IDT strengths for differences in air voids (*9*). Gokhale's data support the relationship found in the original study between field rutting and IDT strength and further demonstrate the potential effectiveness of this test for quality control and forensics (*9*).

CONCLUSIONS AND RECOMMENDATIONS

The high-temperature IDT strength test is a simple, inexpensive, and effective test for evaluating the rut resistance of Superpave mixtures. Care must be taken in selecting appropriate temperatures and loading rates for the high-temperature IDT tests; in this study, the IDT strength test was performed at 3.75 mm/min at a temperature 20°C below the critical pavement temperature for permanent deformation. On the basis of the Asphalt Institute guidelines for interpreting maximum permanent shear strains from the RSCH test (*15*) and the relationship observed in this study between MPSS and IDT strength, preliminary guidelines can be generated for evaluating rut resistance on the basis of IDT strength tests:

- IDT strength > 440 kPa: excellent rut resistance
- 320 kPa < IDT strength ≤ 440 kPa: good rut resistance
- 200 kPa< IDT strength ≤ 320 kPa: fair rut resistance
- IDT strength ≤ 200 kPa: poor rut resistance

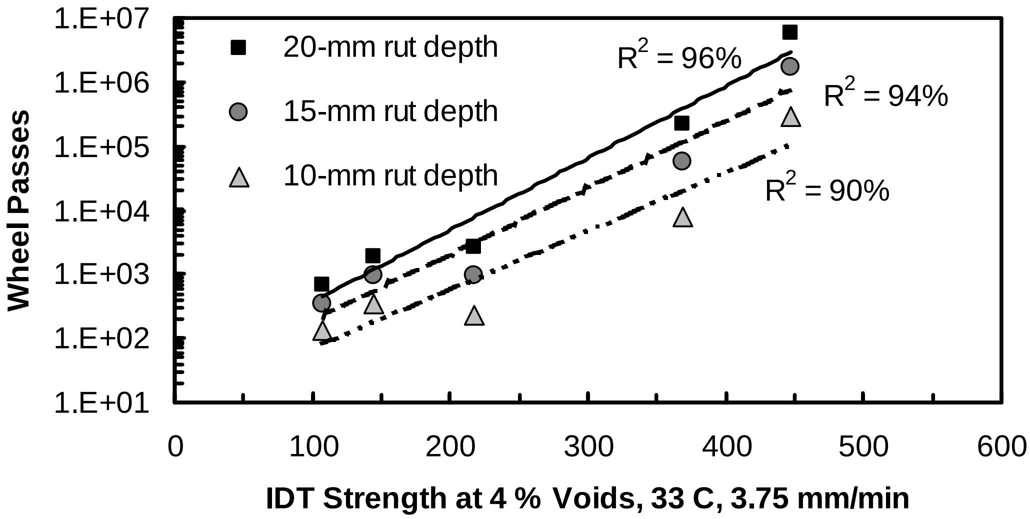

FIGURE 3 FHWA ALF mixtures, wheel passes to different rut depths as a function of high temperature IDT strength; lines from top to bottom represent exponential fits to data for 20-mm, 15-mm, and 10-mm rut depths, respectively (*9*).

Additional research using the IDT as a simple performance test is needed in a number of areas. Some limited testing has shown that similar test results might be produced by testing at a temperature 10°C below the average 7-day maximum high pavement temperature at a loading rate of 50 mm/min. Under these conditions, failure is quite rapid, and the test can easily be completed within 30 s. Using this approach, the high-temperature IDT strength test could be performed on standard Marshall loading systems with no additional equipment other than an IDT loading head. This modification to the procedure should be evaluated because it would greatly simplify implementation of the test. Additional data should be gathered relating IDT strength to field performance. Before implementation, carefully designed testing programs to evaluate the robustness and precision of the final test procedure should be performed.

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AUTHORS' NOTE

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of Pennsylvania at the time of publication. This report does not constitute a standard, specification, or regulation.

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Improved Testing Procedure for Quality Control of Asphalt Concrete Mixtures Using the Field Shear Test

DONALD W. CHRISTENSEN
RAMON F. BONAQUIST
Advanced Asphalt Technologies, LLC

The field shear test is an improved version of a device originally developed during NCHRP Project 9–7. The field shear test was redesigned and evaluated as part of NCHRP Project 9–18. Although the device was in many ways improved, the precision of the modulus data it generated appeared to be too poor for quality control testing. However, an improved testing protocol was suggested at the close of the project. This procedure involves averaging four determinations on each specimen tested with the field shear test and rotating or flipping the specimen between each determination. The research presented here was intended to evaluate this new testing protocol. It was found that modulus data generated using the new protocol are substantially more precise than those generated using the original protocol and are, as a result, more sensitive to changes in mixture composition. The precision and sensitivity of complex shear modulus data gathered using the field shear test with the new protocol are probably adequate for quality control testing in the field. Further research should be done to evaluate the field shear test in conjunction with actual construction projects following typical quality control testing plans.

This paper presents the results of an evaluation of the field shear test (FST) using a recently proposed improved protocol. The FST device was originally developed during NCHRP Project 9–7 as a rugged, simple device for performing quality control (QC) testing on asphalt concrete specimens in the field (*1, 2*). The FST was designed to perform many of the same tests performed by the Superpave® shear test (SST), a large and expensive device used to evaluate various performance-related properties, such as complex shear modulus (|G*|) and resistance to permanent deformation (*3*). The original FST was promising—it was small, rugged, and relatively easy to use. However, there were a number of problems with the device. During NCHRP Project 9–18, the FST was improved and reevaluated. Modulus measurements made with this device appeared to be well suited for QC use, except that the variability appeared to be too high—a problem seen in virtually every other modulus measurement technique currently used on asphalt concrete (*4, 5*). Variability in modulus testing of asphalt concrete results not only from variability in the test procedure but also from variability among specimens and even within a single specimen because of variations in air content and distribution of aggregate particles.

The NCHRP Project 9–18 Final Report recommended evaluating a new testing protocol for the FST. This procedure involves averaging four modulus determinations for each specimen tested and rotating or flipping the specimen between each determination so that the specimen is sheared in a different sense and in a slightly different location each time. This procedure helps reduce variability caused by nonhomogeneity and slight differences in test setup (*4, 5*). The purpose of the testing summarized in this paper was to evaluate the sensitivity and precision of the FST using this improved protocol.

The test program was straightforward, and in all important aspects, it followed the same sensitivity test plan used during the initial evaluation performed as part of NCHRP Project 9–18. The same set of specimens was used, although their modulus was somewhat higher because of gradual hardening of the mixtures over time. Four different aggregates were used in the test program, and the composition of each of the resulting four primary mixtures was varied in a consistent way, with changes in coarse aggregate, mineral filler, and asphalt binder content. Four replicate specimens were tested for each mixture variation, ensuring adequate degrees of freedom for statistical analyses. To date, only one FST prototype exists, so the results of this study should be considered preliminary. Additional evaluations of the FST under field conditions are needed, preferably using two or three devices and a number of different operators.

BACKGROUND

This paper presents results of a continuation of an engineering and research effort begun in 1994 on the QC of Superpave asphalt concrete mixtures. This effort was begun with NCHRP Project 9–7 "Field Procedures and Equipment to Implement SHRP Asphalt Specifications," which was one of the earliest efforts to investigate QC and acceptance procedures for mixtures produced using the Superpave system (*1*). As part of NCHRP Project 9–7, the original field shear test was developed as a simple and rapid procedure for evaluating several performance-related properties of asphalt concrete mixtures for use in QC and acceptance testing. The FST was designed to perform the same basic battery of tests performed using the Superpave shear tester (SST), including the frequency sweep test, which determines dynamic complex modulus ($|G^*|$) and phase angle, and the repeated shear at constant height (RSCH) test, which measures permanent deformation under repeated loading at high temperatures (*1, 2*). The RSCH test is most often used to measure the maximum permanent shear strain (MPSS), which increases with decreasing rut resistance (*3*). Complex modulus has not been used as frequently in measuring rut resistance, but it is attractive for use in QC testing because it is a rational property that potentially reflects mixture composition, aggregate gradation, and asphalt binder grade.

The SST is a relatively complicated and expensive piece of equipment. In addition, preparing specimens for testing using the SST involves compacting asphalt concrete mixture with the Superpave gyratory compactor to a 150-mm diameter specimen, sawing a 50-mm high disc from the specimen, and gluing this disk to two aluminum or stainless steel platens. Three transducers are mounted to this specimen for measuring horizontal and vertical deformations. A sketch of a typical SST specimen is shown in Figure 1. Because of the time and expense involved in performing these tests, the SST is clearly unsuitable for use as a QC and acceptance test for asphalt concrete mixtures. The original FST was designed to shear gyratory specimens across their diameter, as shown in Figure 2. Specimens could be tested using this device without any sawing or gluing, and the device itself was smaller, simpler to operate, and less expensive than the SST. Therefore, it was potentially appropriate for QC and acceptance testing. However, initial evaluations of the FST during NCHRP Project 9–7 revealed several problems. Modulus values determined with the FST did not always compare well with those found using the SST and exhibited relatively poor precision (*1, 3*). The device did not include a temperature control chamber, which probably explained some of the problems with the test data. Furthermore, the extent of testing was not adequate for evaluating the sensitivity of the device to changes in mixture composition. It was decided that although the FST was a promising test for QC and acceptance testing, further evaluation and refinement of the device and procedure were needed.

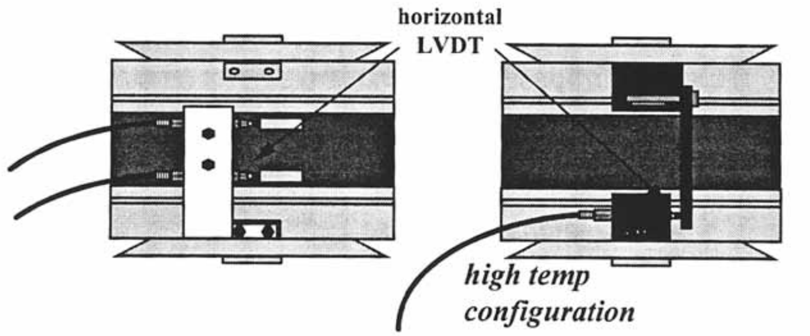


FIGURE 1 SST specimens showing two different approaches for mounting linear variable differential transformers (LVDTs) for deflection measurements (*8*).

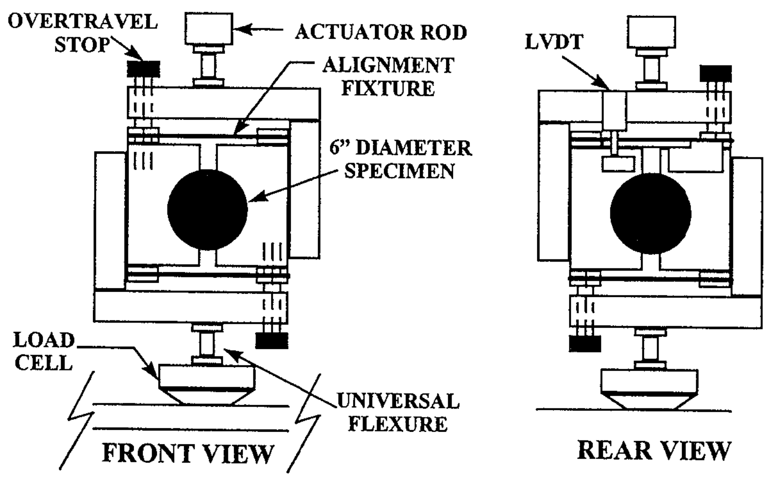


FIGURE 2 Sketch of original field shear test developed during NCHRP Project 9–7 (*4*).

NCHRP Project 9–18 was initiated in 1999 with the objective of evaluating and refining the FST to produce a device better suited for QC and acceptance testing of Superpave asphalt concrete mixtures in the field (*4, 5*). After a review of the design of the original FST, which included surveys of engineers and technicians and stress analyses of the FST test geometry and possible alternatives, the 9–18 research team decided that a different test geometry was needed, and the FST was redesigned. The new FST developed during NCHRP Project 9–18 shears a gyratory specimen in the same plane and sense as the SST. However, as in the original FST, the test is performed directly on a standard gyratory specimen without sawing or gluing. The redesigned FST uses hydraulic clamps to hold the gyratory specimen around each end and a servopneumatic system to apply the sinusoidal load to the specimen. It is compact and easy to use, and initial results showed improvement over the original FST in terms of accuracy and repeatability. A sketch of the new FST geometry is shown in Figure 3; a photograph of the actual device is shown in Figure 4. RSCH test data produced with the new FST were not of good quality; better results were obtained in measuring |G*| values. Although modulus data determined using the new FST were somewhat sensitive to changes in mixtures composition, the precision was found to be inadequate for QC and acceptance testing (*4, 5*). However, toward the conclusion of NCHRP Project 9–18 researchers realized that because the frequency sweep test could be performed so quickly using the new FST, multiple determinations could be made with this device, and an average value could be determined and reported. Between each determination, the specimen could be rotated or flipped so that the variations because of uneven air void distribution and other nonhomogeneities could be greatly reduced. One of the recommendations of NCHRP Project 9–18 was to perform follow-up testing to evaluate this new protocol to determine whether it would significantly improve the precision and sensitivity of modulus measurements made using the new FST.

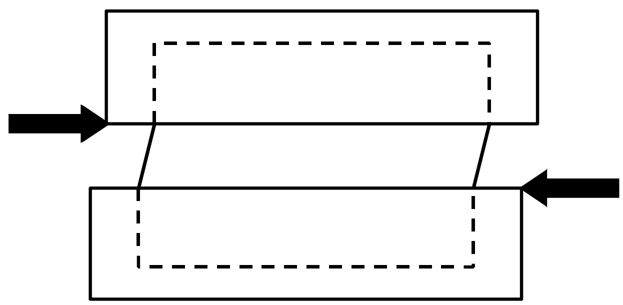


FIGURE 3 Sketch geometry for new field shear test.

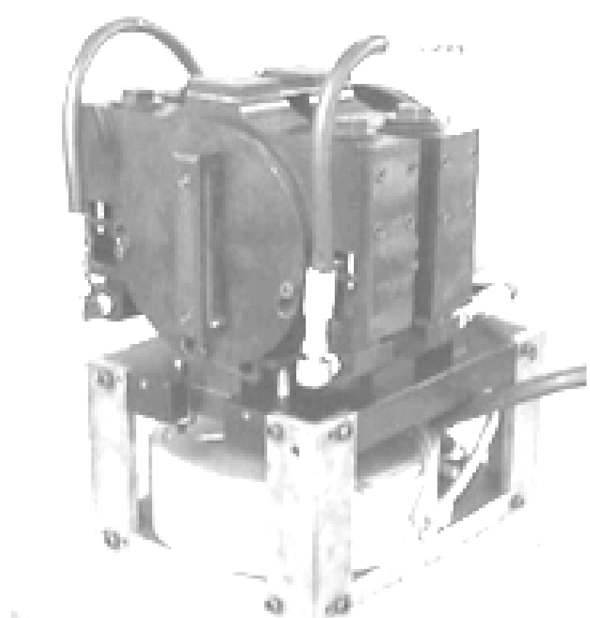


FIGURE 4 Photograph of new FST.

To evaluate the effectiveness of the FST using this new protocol, a short project was initiated in which the original set of specimens used in NCHRP Project 9–18 were to be retested to determine the dynamic complex modulus in shear (|G*|); this paper summarizes the results of this test program. The experiment design and analysis used were identical to that used in the original sensitivity testing performed as part of NCHRP Project 9–18. Details of the test program to evaluate the precision and sensitivity of the FST using the improved protocol are given in the following sections.

MATERIALS, METHODS, AND EXPERIMENT DESIGN

The sensitivity test program involved testing four different materials: 9.5-mm, 12.5-mm, 19-mm, and 25-mm Superpave mixtures. The aggregate sources were different for each of these materials. The volumetric factors and gradations for the mixtures are summarized in Table 1. For each of the four primary mixtures, the design aggregate gradation and binder contents were systematically varied to produce a total of eight mixture types or variants. The factors that were varied were coarse aggregate content (± 6% on the 2.36-mm sieve), mineral filler content (± 2% on the 0.075-mm sieve), and binder content (± 0.5). The improved protocol was used to perform the frequency sweep test at a temperature of 40°C; the RSCH test was not performed.

Four replicate specimens were prepared using an Interlaken gyratory compactor following procedures as outlined in AASHTO TP4 except that the mass of the batches was adjusted to obtain specimens with a nominal height of 115 mm. After mixing, the material was short-term oven aged in accordance with AASHTO PP2 at a temperature of 135°C for 4 h. The mixtures required different levels of compaction effort to produce specimens within the specified air void tolerance of 4.0 ± 0.5%.

Before testing, the specimens were conditioned at 40°C in an environmental chamber for a minimum of 2 h. The specimens were then placed in the FST, and the hydraulic clamps were closed on the specimen with a pressure of 10.3 MPa (this is the pressure of the hydraulic fluid, not of the clamps on the specimen). The door to the environmental chamber holding the FST was then closed, and the specimen was allowed to equilibrate for approximately 5 min. A frequency sweep test was then performed. The hydraulic clamps were loosened, the specimen was rotated 90°, and the clamps were retightened. The door to the chamber was closed, the specimen equilibrated again for 5 min, and the frequency sweep repeated. A third frequency sweep was performed after flipping the specimen front to back, and a fourth frequency sweep was performed after again rotating the specimen 90°. In this way, four separate frequency sweep determinations were made on each specimen. The final frequency sweep values were the averages of these four determinations.

RESULTS

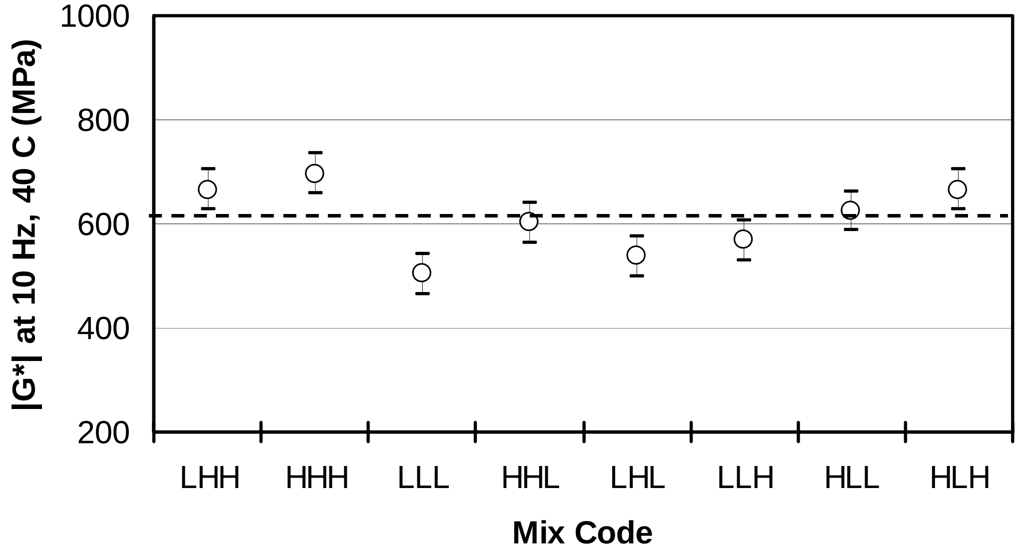
The results of the sensitivity testing were first analyzed graphically, as shown in Figure 5. This plot shows average |*G**| values at 40°C and 10 Hz for the eight variations of each mixture. Included on the plot are error bars representing ± 2 s confidence intervals for the mean (n = 4 replicates) and the overall average for the mixture (dashed line). Because the eight mixture types vary substantially in composition, good sensitivity is indicated when the error bars do not overlap and when the error bars for the eight variations tend not to include the overall average.

TABLE 1 Volumetric Properties of Design Mixtures

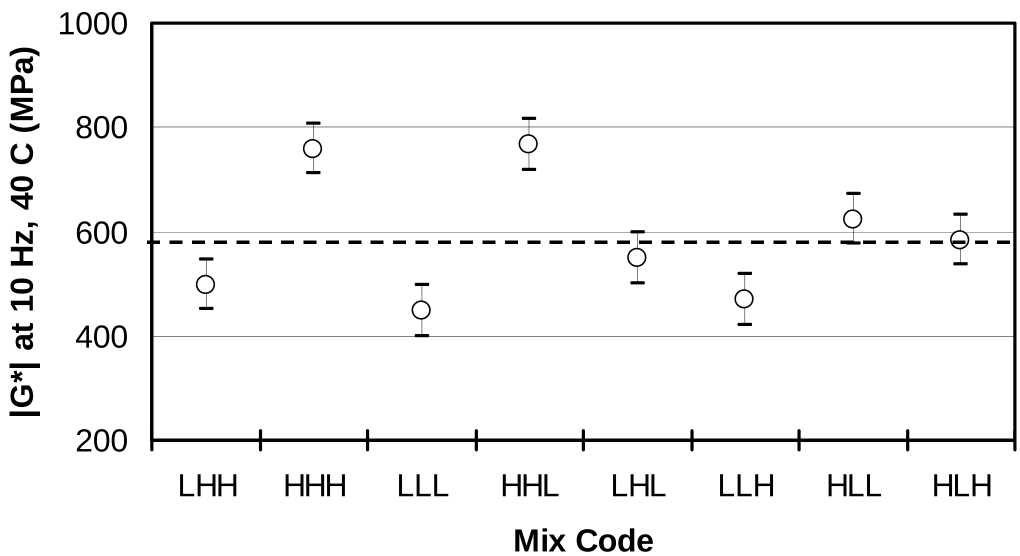
Property	9.5 mm	12.5 mm	19.0 mm	25.0 mm
N_{design}	65	75	96	100
Binder grade	PG 64–22	PG 76–22	PG 64–22	PG 64–22
Coarse aggregate angularity. (one face/two face)	100/100	100/100	100/100	95/80
Fine aggregate angularity	45.0	47.2	52.1	45.0
Flat and elongated, % (Ratio 5:1)	1.6	3.0	1.9	1.4
Sand equivalent, %	83	55	80	74
Binder content, %	6.2%	4.75%	4.4%	4.4%
Compaction, % G_{mm}				
$N_{initial}$	85.2%	86.4%	85.9%	85.4%
N_{design}	96.0%	96.0%	95.8%	96.0%
$N_{maximum}$	97.8%	97.3%	97.2%	97.0%
Voids in mineral aggregate (VMA), %	17.2	14.6	14.5	12.8
Voids in total mixture (VTM), %	4.0	4.0	4.2	4.0
Voids filled with asphalt (VFA), %	76.7	72.6	71.0	68.8
Fines to effective binder ratio (F/A)	1.2	1.2	1.1	0.8
Sieve Size, mm	Gradation, % Passing			
37.5	100	100	100	100
25	100	100	100	97
19	100	100	94	86
12.5	100	97	73	63
9.5	97	75	52	46
4.75	62	39	33	33
2.36	42	30	24	26
1.18	27	24	17	16
0.6	18	18	14	10
0.3	11	11	10	7
0.15	8	7	6	4
0.075	6.8	5.3	3.6	3.0

NOTE. N_{design}, the design number of gyrations for Superpave asphalt concrete (AC) compaction; G_{mm}, the maximum theoretical specific gravity; $N_{initial}$, the initial number of gyrations for Superpave AC compaction; $N_{maximum}$, the maximum number of gyrations for Superpave AC compaction.

Large error bars that overlap for most mixtures indicate poor sensitivity. For three of the mixtures, confidence intervals for six of the eight mixture variations do not include the overall mean, although for the fourth (the 25-mm mixture), confidence intervals for five of the eight variations exclude the mean. Thus, the |G*| value measured with the improved protocol differentiated 23 of 32 mixture variations. This indicates a good degree of sensitivity for all mixtures, in that changes in mixture composition in general produced changes in modulus that were statistically significant. For comparison, in the first round of sensitivity testing, as performed during NCHRP Project 9–18, differentiation was observed in only 17 of 32 mixtures. The new protocol for |G*| measurements produced data that in general were significantly more sensitive to changes in mixture composition.



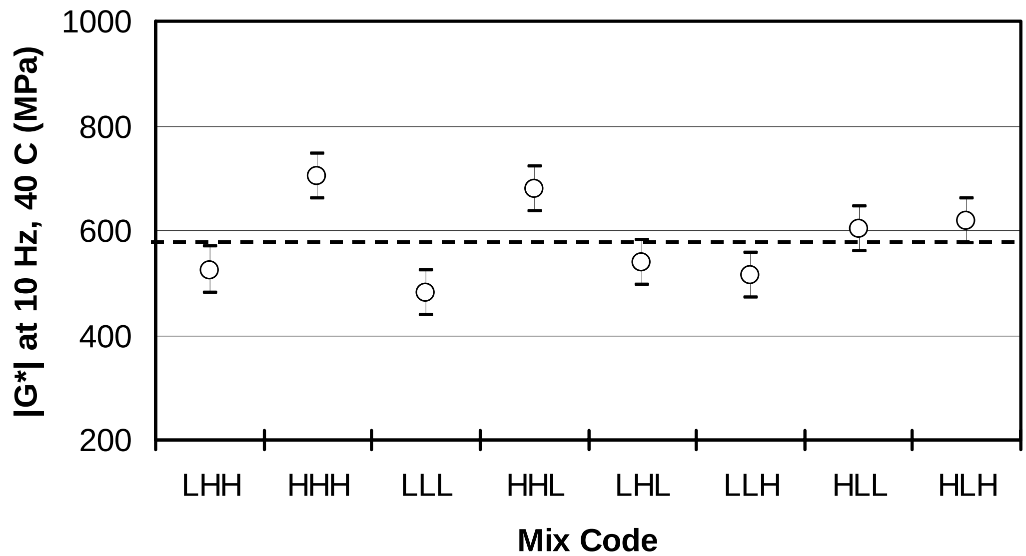
(a)



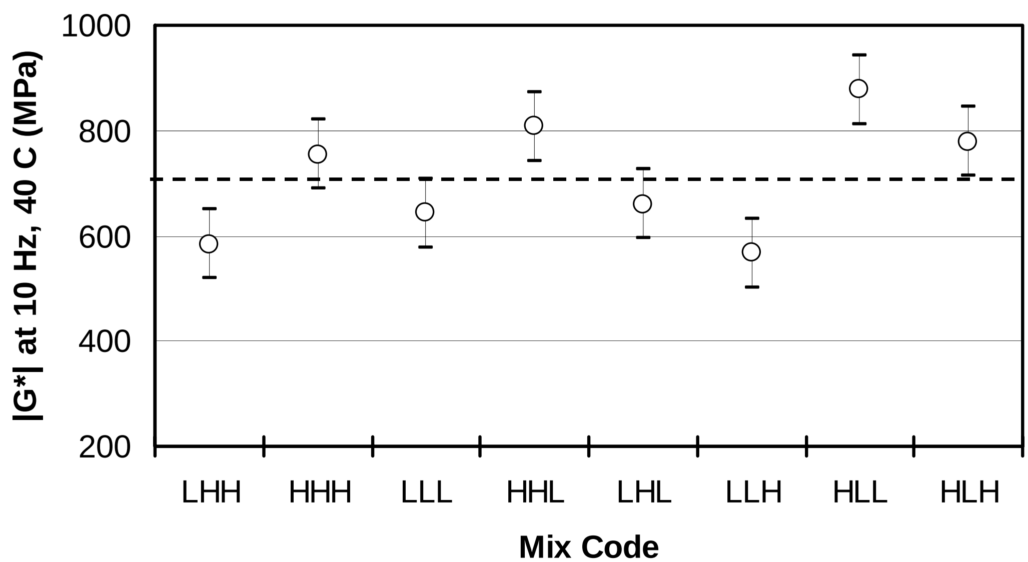
(b)

FIGURE 5 Average complex modulus at 10 Hz and 40°C with 2-s error bars. Mixture code represents high (H) or low (L) values for coarse aggregate, mineral filler, or binder content, respectively: (*a*) 9.5-mm mixture, and (*b*) 25-mm mixture. Dashed line represents mean value for mixture.

(*continued*)



(c)



(d)

FIGURE 5 (*continued*) **Average complex modulus at 10 Hz and 40°C with 2-s error bars. Mixture code represents high (H) or low (L) values for coarse aggregate, mineral filler, or binder content, respectively: (*c*) 19-mm mixture, and (*d*) 12.5-mm mixture. Dashed line represents mean value for mixture.**

For the original test protocol, the coefficient of variation (CV) values for |G*| measurements made with the FST ranged from 7.9% to 16.9%, with an overall value (based on the pooled standard deviation and grand average) of 14.5%. Using the new protocol, the CV values ranged from 6.3% to 9.2%, with an overall value of 8.0%. Therefore, the new protocol appears to have significantly improved the precision of complex modulus measurements made using the FST. For use in QC, where a lot average is typically based on n = 5 tests, the 2 standard deviation error values (based on the standard deviation of the mean) would be 13% using the original protocol; using the new procedure, this value is reduced to 7.2%. The improved sensitivity with the new protocol is a result of the reduced variability in the data. The precision of modulus measurements made with the FST using the new protocol appears to be very good compared with other techniques and is probably adequate for QC purposes.

ANALYSIS AND DISCUSSION

A statistical analysis of the sensitivity test data was performed using multiple regression techniques. Each of the four aggregate types was analyzed separately because it was believed that the sensitivity of the FST might vary considerably depending on aggregate type. Coarse aggregate content, mineral filler content, and asphalt binder content were used as predictors of modulus, along with all possible interactions of these factors. Preliminary analysis indicated that time or order of compaction in some cases also had a significant effect on |G*|; therefore, these also were included as predictors in the statistical analysis. The objective of this analysis was to determine whether any of the parameters related to mixture composition, or the four interaction terms involving these factors, had statistically significant effects on |G*| measured using the FST. Statistically significant interaction terms implied that the effect of one factor depends on the level of another. For example, for all of the aggregates except the 12.5-mm size, the coarse aggregate × mineral filler interaction term was significant, indicating that the effect of changes in coarse aggregate content on |G*| depends on the mineral filler content (and vice versa). The greater the number of statistically significant factors in the model and the higher the R^2 value for the model, the greater the sensitivity of the FST for that particular aggregate.

The results of this analysis are summarized in Table 2, which shows statistically significant factors for all four models. In this case, statistical significance is defined as $\alpha \leq 0.05$, meaning that the chance of incorrectly concluding that a factor is significant is 5% or less. Primary factors were deemed significant if interaction terms involving those factors were significant, regardless of the significance level of the primary factor. For the 9.5-mm mix, the modulus values determined with the FST were sensitive to changes in coarse aggregate content, mineral filler content, and asphalt content, along with two interaction terms. Modulus values for the 25- and 19-mm mixtures were sensitive to changes in coarse aggregate content and mineral filler content; the interaction term for these factors was also significant for both of these aggregates. The 12.5-mm mixture was sensitive only to changes in coarse aggregate and asphalt binder content. Modulus values determined using the new protocol with the FST were fairly sensitive to changes in mixture composition, with R^2 values ranging from 72% to 88%. In the first round of testing, R^2 values ranged from 55% to 91%. For comparison purposes, analyses of air void and VMA data using the same approach resulted in R^2 values ranging from 88% to 99%. Although the sensitivity of the modulus measurements is still not as great as that of air voids or VMA, it provides important information on an engineering property with a reasonable degree

TABLE 2 Statistically Significant Factors in Explaining Variability in FST Modulus Data

Parameter	9.5 mm	25 mm	19 mm	9.5 mm
Coarse aggregate content	X	X	X	X
Mineral filler content	X	X	X	—
Asphalt binder content	X	—	—	X
Coarse aggregate × mineral filler	X	X	X	—
Coarse aggregate × asphalt binder	—	—	—	—
Mineral filler × asphalt binder	X	—	—	—
Coarse aggregate × mineral filler × asphalt binder	—	—	—	—
Compaction time or order	X	X	X	—
r^2, %, improved protocol	83.4	87.5	84.2	72.4
r^2, %, original protocol	90.5	91.4	55.3	60.6

of precision. Modulus measurements made with the FST in general appear to be most sensitive to changes in coarse aggregate content and mineral filler content and are not always sensitive to changes in asphalt content.

CONCLUSIONS AND RECOMMENDATIONS

The data and analysis presented in this paper led to the following conclusions and recommendations:

- A new protocol has been developed for measuring complex modulus (|G*|) with the FST. It involves averaging four determinations for each specimen tested and rotating or flipping the specimen between each determination. Because of the ease of using the FST, the testing can be completed within approximately 10 min even when taking four different readings.
- The new protocol produces |G*| data that are significantly more precise than those produced using the earlier protocol, which involved taking only one measurement per specimen.
- Complex modulus measurements made using the new FST with the new protocol are sensitive to changes in mixture composition, especially changes in coarse aggregate content and mineral filler content, and, to a lesser degree, asphalt binder content.
- The overall CV for |G*| values at 10 Hz and 40°C, determined using the FST and the new protocol was found to be 8.0%, which is quite good for modulus measurements on asphalt concrete specimens.
- Using the new protocol, |G*| measurements made using the FST appear to be suitable for QC purposes.
- Additional research should be performed to evaluate the effectiveness of |G*| measurements made using the FST in field projects designed to simulate QC or acceptance testing. The purpose of this research should be to further evaluate the reliability and precision of the FST and to compare |G*| data generated with the FST to other QC data.

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Donald W. Christensen served as Principal Investigator for NCRHP Project 9–18, and managed the short testing program summarized in this paper. Ramon Bonaquist provided substantial technical input during NCHRP Project 9–18 and the subsequent testing to evaluate the new protocol. Laboratory testing was performed or supervised by Kevin Knechtel and Don Jack, both of Advanced Asphalt Technologies, LLC (AAT). The new FST device, as described in this paper, was designed and built by EnduraTec, Inc., under the supervision of Kent Vilendrer, and was lent to AAT for use in NCHRP Project 9–18 and related research. The authors gratefully acknowledge the assistance of EnduraTec and various material suppliers in completing this research.

AUTHORS' NOTE

This work was sponsored by NCHRP. The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of NCHRP or the Transportation Research Board. This report does not constitute a standard, specification, or regulation.

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Potential Applications of the Hollow Cylinder Tensile Tester as a Simple Performance Test

WILLIAM G. BUTTLAR
University of Illinois at Urbana–Champaign

GHAZI G. AL-KHATEEB
Turner–Fairbank Highway Research Center, Federal Highway Administration

DANIEL S. SHERMAN
University of Illinois at Urbana–Champaign

A hollow cylinder tensile tester (HCT) was developed that can be used to obtain fundamental properties of asphaltic paving mixtures, such as creep compliance, tensile strength, and dynamic modulus, at low and intermediate temperatures. The device was originally developed to be a compact, portable, and operationally simple surrogate test to obtain properties similar to the Superpave® Indirect Tensile Tester (IDT) (e.g., creep compliance and tensile strength). However, a recent study has shown that the HCT also can be used to obtain the dynamic complex modulus (E*) of hot-mix asphalt in tension. By applying pressure to the inner cylinder wall of the specimen, a tensile or "hoop" response is induced. The load system also can be used to measure specimen deformation, which makes specimen preparation and device operation free of mounted sensors and, hence, simple and rapid. A production version of this device would resemble a portable gyratory compactor in size, simplicity, and portability. The possibility of using the HCT as a simple performance test in the asphalt industry was explored. The results of several recent studies are summarized; these indicate that the HCT produces accurate measures of creep compliance and dynamic complex modulus of HMA compared with the IDT. Very reasonable values of tensile strength also were obtained with the HCT, because strength variations followed logical trends with changes in aggregate type and polymer modification level. It appears that the HCT is compatible with the requirements of the NCHRP 1–37A software models (used in the AASHTO Mechanistic-Empirical Design Guide) for thermal cracking performance prediction (e.g., a suitable surrogate test for the IDT). Because it also can measure E* at low to intermediate temperatures, the HCT appears to be able to collect the necessary inputs for running the fatigue cracking performance prediction models used in NCHRP 1–37A software. The HCT device is currently configured to apply tensile loads to asphalt mixture specimens at low and intermediate temperatures and is therefore not currently applicable to the study of permanent deformation (rutting). In addition, testing to date has been limited to mixtures with nominal maximum aggregate size no greater than 19 mm.

Significant advances in pavement modeling and monitoring of field sections have occurred over the past decade. These advances have helped the industry move closer toward implementing performance-based design of flexible pavements and paving materials. However, the benefits of these advanced design tools cannot be fully realized unless state highway agencies and other parties involved in the design of bituminous mixtures have widespread access to reliable, repeatable, and operationally simple test equipment that provide the necessary inputs to these new design tools. For reliable design of durable asphalt concrete paving mixtures, it is generally accepted that volumetric-based mixture design methods should be supplemented with mechanical testing of the mixture. In the past, both the Marshall and Hveem mix design methods have used mechanical testing as a supplement to volumetric-based procedures. During the Strategic Highway Research Program (SHRP), significant strides were made to improve volumetric-based mix design procedures, including the development of a gyratory compactor with better compaction characteristics and larger mold size than the widely accepted Marshall hammer. An initial attempt was made to supplement the new volumetric mix design procedure with fundamental mixture tests and performance prediction models. However, the mixture tests developed under SHRP research were originally designed to serve as research tools. As a result, they lacked the simplicity, ruggedness, portability, and cost-effectiveness to be used for routine design, quality control, and quality assurance.

Thus, early implementations of the Superpave mixture design method involved conducting only volumetric-based procedures. Recognizing this deficiency, some agencies have temporarily supplemented these procedures with one of a variety of mechanical tests. These tests were often empirical in nature; that is, the measured quantity from the test procedure yielded an index parameter rather than a fundamental (or "engineering") property. This approach has similar limitations as the Marshall stability and flow test. Index values, like stability and flow, can be related empirically only to field performance. Fundamental properties, however, can be used directly in pavement structural and distress (or "performance prediction") models. These models are a critical element of mechanistic-empirical pavement design procedures, such as those used in the AASHTO Mechanistic-Empirical Design Guide.

On the basis of these considerations, there is clearly a need to develop a more universal mechanical test procedure to supplement Superpave volumetric mix design. In fact, one of the objectives the National Cooperative Highway Research Program (NCHRP) Project 9–19, "Superpave Support and Performance Models Management," was "to develop simple performance tests for permanent deformation and fatigue cracking for incorporation in the Superpave volumetric mix design method." Because the hollow cylinder tensile test (HCT) was still under development, it was not possible to consider this device under NCHRP 9–19. However, the HCT device has a number of characteristics that will appeal to some agencies that wish to consider alternatives to the NCHRP 9–19 recommended procedures. These characteristics can be summarized as follows:

- The HCT device was designed to obtain fundamental mixture properties at low and intermediate temperatures and can, therefore, be used to design against thermal cracking, fatigue cracking, and possibly top-down cracking of pavements. Because thermal cracking was not addressed in NCHRP 9–19, the HCT may be particularly desirable for agencies concerned with various forms of pavement cracking.
- The HCT device test is capable of modulus, creep, and tensile strength testing. Devices that measure only modulus, or modulus and creep, do not permit direct evaluation of the strength of the material. Many other industries rely on strength measurements for design and control of materials because material defects cannot be readily detected at lower strain levels.
- The HCT device can be used as a stand-alone simple performance test or used in conjunction with the AASHTO Mechanistic-Empirical Design Guide software for performance-based mixture design and performance-related specifications. Application to the AASHTO Mechanistic-Empirical Design Guide would be limited to thermal cracking and fatigue

predictions (i.e., it could not be used for rutting predictions) and may require adjustment of model calibration factors.

- The HCT device is compact, portable, and relatively simple in terms of sample preparation and testing.

The remainder of this paper will give the background information used to develop these concepts.

DEVELOPMENT OF THE HCT

The HCT was recently developed (Figures 1 and 2) as a surrogate test device to measure creep compliance and tensile strength of asphalt concrete at low temperatures (*1*–*3*). The main reason for developing a surrogate test for the Superpave Indirect Tensile Test IDT, which was developed under the Strategic Highway Research Program (*4, 5*), was to address the lack of portability, the complexity of operation, and the relatively high cost of IDT equipment. Later, it was found that the HCT testing mode had other inherent benefits, as described in a later section. Previous studies have investigated the use of hollow cylinders to obtain fundamental properties of asphalt concrete in torsional shear and compression (*6*–*8*). However, the test method presented here differs from these approaches in that the primary focus is to obtain tensile properties by applying cavity pressure and measuring cylinder expansion.

The basic principle of the HCT is to apply internal pressure to the inner cavity of a hollow cylinder specimen, which results in circumferential (hoop) tension. Applied stress is linearly related to applied pressure. The resulting strain is linearly related to cavity volume change or can be directly measured by using strain gages or measuring cavity volume change, which is linearly related to circumferential strain. The primary advantage of the HCT device is its reduced size and operational simplicity relative to the IDT. Another advantage of HCT testing

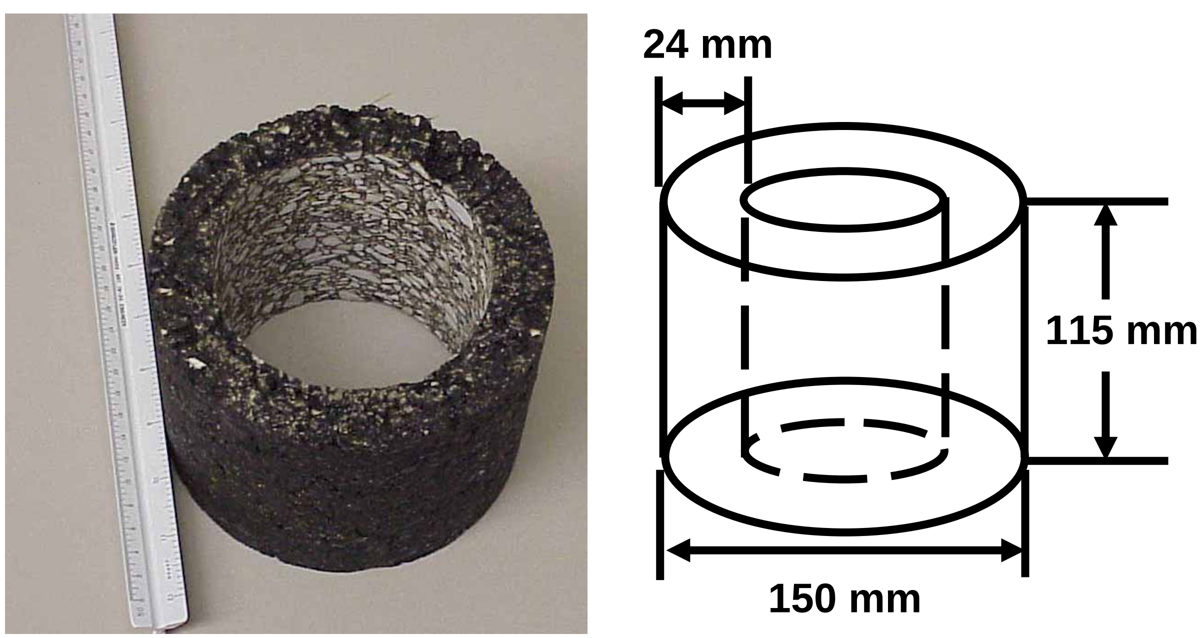

FIGURE 1 Hollow cylinder specimen and typical dimensions.

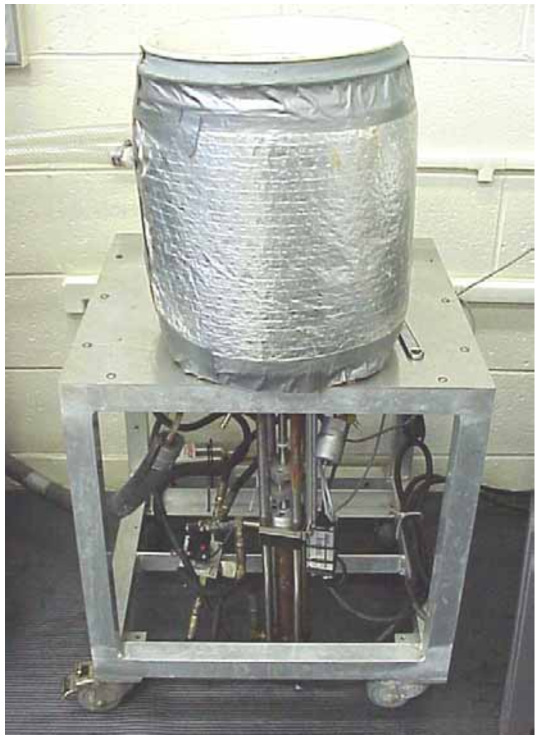

FIGURE 2 Portable test assembly: servohydraulic actuator (below) and temperature bath (above).

is the nature of the stress and strain fields, which are relatively uniform, predominated by tension, and free of significant stress concentrations. This is particularly advantageous for strength testing, because indirect tension and direct tensile tests are sometimes influenced by stress intensities present near the loading platens. Hollow cylinders are easily produced by coring standard gyratory-compacted asphalt mixture specimens (Figure 3).

Thick-walled hollow cylinder formulas can be used to interpret HCT test results, and minor correction factors can be applied, if desired, to enhance measurement accuracy, which account for effects of eccentric coring of test specimens and percentage of loaded area of the inner wall (*1*). Because a sealing system (Figure 4) is required to keep the inflatable membrane from escaping out the specimen ends, the internal pressure in the hollow cylinder is not perfectly uniform, and correction factors can be applied to closed-form solutions to accurately describe the stress and strain fields. However, the correction factors are typically small (less than 5%). An improved sealing system is currently under development for this device, which will result in nearly uniform internal pressure and further reduce the magnitude of correction factors.
The HCT arrangement has several inherent benefits:

1. The ability to apply tension with minimal end effects;
2. A unique ability to accurately measure average specimen strain through cavity volume measurements;
3. The use of a pressure intensifier rather than a load frame, in which the intensifier requires only 5% of the force required in indirect tensile testing to produce a given specimen stress level;

FIGURE 3 Coring fixture at Advanced Transportation Research and Engineering Laboratory to produce 100-mm cavity in standard gyratory specimen.

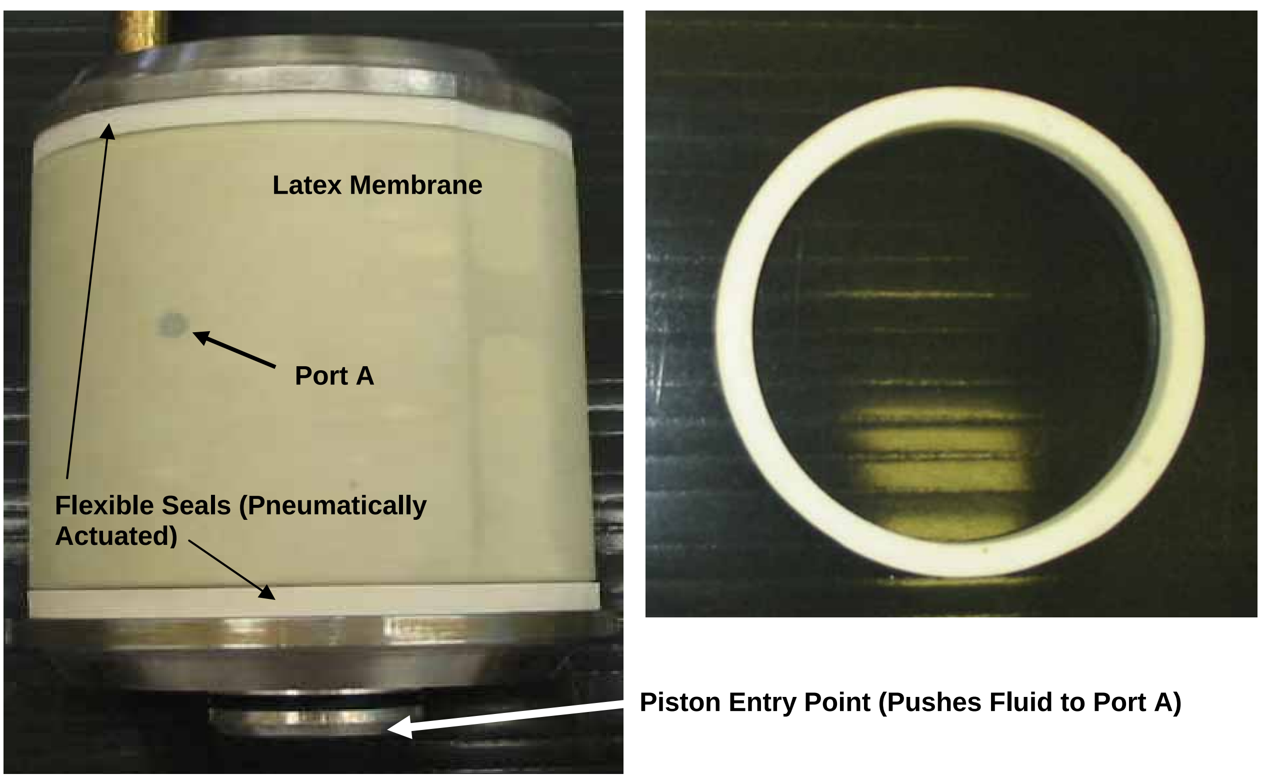

FIGURE 4 Pressurizing cylinder with inflatable latex membrane (left); flexible seal shown on right.

4. Minimal machine compliance and damping effects, and;
5. Test simplicity and portability (Figure 2).

VALIDATION OF FUNDAMENTAL PROPERTIES OBTAINED WITH THE HCT

The following section provides a summary of testing conducted to verify that fundamental properties of asphalt mixtures can be obtained with the HCT. The properties considered were creep compliance, mixture tensile strength, and complex (or "dynamic") modulus.

Creep Compliance

A testing program was established to compare the creep compliance of hot-mix asphalt as measured in the HCT with those obtained using the Superpave IDT. Four different asphalt concrete mixtures were produced, with varying nominal maximum aggregate sizes (summarized in Table 1). These mixtures allowed for investigation of potential factors that influence HCT and IDT results, including nonhomogeneity of gyratory-compacted specimens, spalling at specimen ends, and particle-size-to-gage-length ratio.

Several Superpave performance-graded (PG) binders—including PG 58–22; PG 64–22; PG 64–28 (styrene-butadiene-styrene [SBS] polymer-modified), and PG 70–34 (SBS polymer-modified)—were used in the preparation of asphalt mixtures. A Brovold gyratory compactor was used to make 150-mm diameter × 115-mm tall cylinders, which were cored to produce hollow-cylinder specimens with cavities of 102-mm diameter. Indirect tension specimens were produced by cutting gyratory specimens with a water-cooled masonry saw to produce 50-mm thick × 150-mm diameter specimens. Bulk specific gravity testing was conducted along with maximum theoretical specific gravity testing to estimate air voids on the original gyratory specimens and the HCT and IDT specimens. By trial and error, the number of gyrations needed to produce indirect tension and hollow cylinder specimens at 4% air voids was determined. A slightly higher number of gyrations was required for HCT specimens because they comprise the outer portion of the gyratory specimen, which is generally less dense than the inner core.

TABLE 1 Asphalt Mixtures Used in Compliance Study

Mixture Type	Project Name (If Applicable)	Aggregate Type	Binder Type
Dense-Graded Little-Rock Binder Course Mix (LR19 PG 64–22)	NA	19-mm max aggregate size limestone	PG 64–22 (AC-20)
Polymer-Modified Sand-Asphalt Mixture (SANDAC PG 70–34)	Peoria Airport Interlayer Mix	4.75-mm max aggregate size sand	PG 70–34 (SBS-Modified Binder)
Rantoul Traditional Overlay Mix (RNAC PG 58–22)	Rantoul NAC Demonstration Project	9.5-mm max aggregate size limestone	PG 58–22 (AC-10)
Rantoul Polymer-Modified Mix (RNAC PG 64–28)	Rantoul NAC Demonstration Project	9.5-mm max aggregate size limestone	PG 64–28 (SBS Polymer-Modified Binder)

NOTE: NA, not applicable; NAC, National Aviation Center; SBS, styrene-butadiene-styrene. PG denotes binder grade designation from the Superpave Performance-Graded binder specification (AASHTO MP-1)

Creep compliance tests using the HCT device and the IDT equipment were conducted at three test temperatures (–20°C, –10°C, and 0°C), with the exception of the LR19 PG 64–22 mixture, which was tested at –10°C only. Creep tests were conducted for 100 s following Superpave IDT test protocols outlined in AASHTO TP-9 specifications. In the HCT creep tests, a constant internal pressure was applied to the inner wall of the hollow cylinder using a control pressure mode, and the tensile strain was monitored throughout the 100-s test using 2-in. strain gages applied at the mid-height of the inner wall of the specimen. A system to measure specimen strain by measuring the volume change of the internal cavity was not completed at the time of this study. Such a system, however, has been developed recently, as described by Buttlar et al. (*9*). In the IDT test, surface-mounted sensors are attached to the center of the flat faces of the specimen in an attempt to reduce end effects caused by loading platens (*4*). Vertical and horizontal displacement transducers with 37.5-mm gage length were used. Data analysis procedures used can be found in Buttlar et al. (*9*).

Typical creep compliance versus time curves at three temperatures (0°C, –10°C, and –20°C) obtained in HCT and IDT testing are presented in Figures 5 and 6. Figure 7 presents HCT creep compliance data at 100 s loading time plotted against IDT results at the same loading time and test temperature. In general, creep compliances obtained with the two devices were found to be in good agreement. Given the considerable difference in testing modes (indirect tension versus hollow cylinder tension), the similarity in measured compliance suggests that each test is capable of capturing fundamental properties of asphalt concrete at low temperatures, in this case, creep compliance.

A discrepancy that appears to follow a repeatable trend, however, can be observed in Figures 5 through 7. It appears that the measured creep compliance values obtained with the two devices diverge slightly at the warmest test temperature (0°C) and at longer loading times. In Figure 7, the points at the bottom of the unity line (lower creep compliances) fall directly on the unity line. At higher creep compliances, HCT values tend to be larger than those obtained in the IDT. It is hypothesized that as the mixture becomes more compliant, end effects in the IDT test become more significant. As the loading heads penetrate into the specimen, localized damage can lead to stress redistribution, possibly reducing horizontal deflections (perpendicular to the direction of load) measured on the IDT specimen. Lower horizontal deflections result in lower estimated creep compliance (*5*).

First Failure and Ultimate Tensile Strength

The HCT and the IDT test methods were both used for tensile strength testing (*3*). Although tensile strength is not a fundamental property, it is nevertheless a very common and useful quantity in the field of engineering. Tensile strength tests were conducted at a temperature of –10°C. A factorial of three replicates, two test methods, one temperature, and nine mixtures resulted in 54 strength tests (27 HCT and 27 IDT tests).

In the HCT strength tests, a constant rate of ram displacement was applied until failure of the specimen was reached. By examining a database of more than 20 mixtures tested in the Superpave Indirect Tensile Test, it was determined that the 12.5-mm per minute ram compressive displacement loading rate used, on average, resulted in a tensile strain rate across the failure plane of approximately 100×10^{-6} mm/mm. It was determined that a ram displacement of 12.5-mm/s, somewhat coincidently, produced a tensile strain rate at the mid-height of the inner wall of 100×10^{-6} mm/mm. Of course, this rate depends on specimen geometry—including inner and outer wall diameter and pressurizing piston cross-sectional

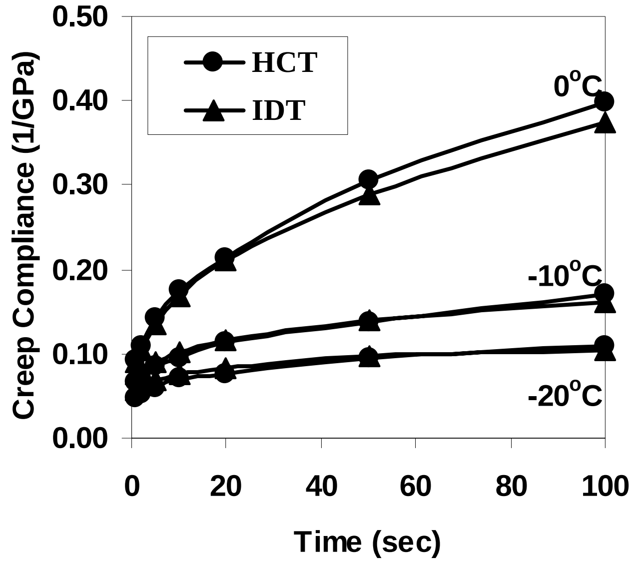

FIGURE 5 Creep compliance comparison for Rantoul National Aviation Center PG 64–28 mixture, average of three tests (*10*).

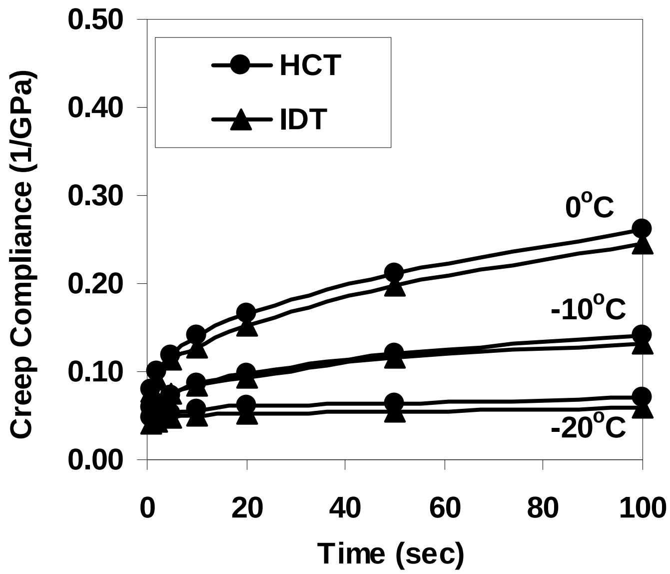

FIGURE 6 Creep compliance comparison for Rantoul National Aviation Center PG 58–22 mixture, average of three tests (*10*).

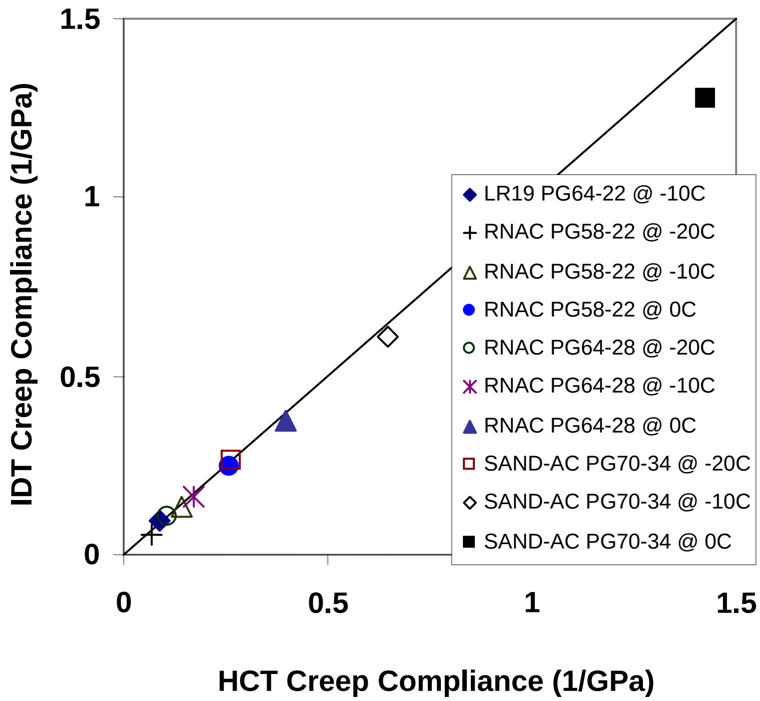

FIGURE 7 Creep compliance comparison for all mixtures at 100-s loading time (*10*).

area—and is influenced by fluid and membrane compressibility and specimen stiffness. In thick-walled cylinders loaded with cavity pressure, tensile stresses on the inside wall are higher than those on the outside wall of the specimen (a ratio of 0.63 outer/inner tensile stress for HCT specimens used herein); therefore, failure should presumably initiate on the inside wall and propagate through the thickness to the outside wall of the specimen. However, it was not initially known if any nonuniformities in gyratory specimens, such as void gradients, preferential aggregate orientation, and aggregate breakdown during compaction would alter this expected trend.

The pressure of the cavity-pressurizing fluid was monitored throughout the test with a pressure transducer. A system of tracking crack propagation was developed that involved the use of ultralightweight wire crack detection gages. The wires were attached circumferentially at as many as three locations on both the inside and outside of HCT specimens. The onset and propagation of fracture was monitored using this system, allowing detection of first failure and giving insight into crack propagation tendencies.

Unlike fundamental measures of modulus or compliance, which, for asphalt concrete at low and intermediate test temperatures are only slightly affected by test mode (if at all), measures of ultimate tensile strength are generally strongly dependent on specimen geometry, size, and stress states. For instance, the ratio of direct tensile strength of portland cement concrete (PCC) to flexural strength ranges from 0.30 to 0.77, and the ratio of the direct tensile strength to splitting strength can be as low as 0.41 for PCC (*11*). In the IDT, a distinction was made between ultimate tensile strength and “true” tensile strength (first-failure tensile strength).

The surface-mounted sensors on IDT specimens are used to determine the time of "first failure" by identifying the time and corresponding load at which the maximum difference between vertical and horizontal deformations on each side of the specimen was reached (*12*). True tensile strength for the IDT was found to be, on average, approximately 80% of the strength based on ultimate load but dependent on probably a number of factors, including Poisson's ratio and specimen geometry.

Figure 8 compares first failure tensile strengths as determined by the HCT and IDT. Three key factors considered in the analysis of first failure strength results included maximum aggregate size, aggregate type, and level of binder polymer modification. Although only one comparison can be made at this time, with regard to the aggregate type, the first failure tensile strength obtained from the HCT mode appears to produce the expected ranking of mixtures. This finding is illustrated in Figure 8. The dense-graded 19-mm limestone binder course asphalt mixture (LR19 PG 64–22) was compared with the dense-graded 19-mm basalt binder course asphalt mixture (BAS19 PG 64–22). The HCT first-failure tensile strengths for these mixtures were found to be 3.30 and 4.19 MPa, respectively. Conversely, there was no significant difference between the first-failure tensile strengths of these two mixtures obtained from the IDT mode; they were found to be 3.06 and 2.82 MPa, respectively.

Figure 8 also shows that the two mixes with the highest first failure strength as measured by the HCT were the furthest from the unity line as a result of the relatively low IDT first-failure tensile strength values. The higher strengths as measured in the HCT appear plausible because the PG 70–34 sand-asphalt mixture with approximately 5% SBS binder modification is expected to possess high tensile strength because of the toughness and excellent adhesive characteristics of this highly modified binder. Likewise, the basalt aggregate in the BAS19 mixture along with PG 64–22 binder would be expected to yield a high tensile strength mixture at –10°C. It is hypothesized that the lower first-failure strengths in the IDT for these two mixtures were caused by damage and large deformations in the vicinity of the loading heads (because of high vertical compressive and shear stresses), which caused a reduced "net" loading rate across the failure plane. Under linear elastic conditions, the strain and stress fields across the vertical, diametral failure plane in the IDT are linearly related to the applied ram displacement. However, damage under loading heads under displacement control results in a slower net loading rate on the IDT specimen.

Slower loading rates lead to lower tensile strength values. Conversely, in the HCT the pressurizing bladder provides steady cavity expansion until failure and hence a nearly linear strain increase throughout the strength test. Currently, IDT and HCT testing is being conducted on 12 additional mixtures having modified binders, which will yield additional insight toward the first failure behavior of modified mixtures.

Ultimate Tensile Strength Results

The effects of the three major factors considered in the study also were investigated with respect to ultimate tensile strength. It was found that highly modified mixtures showed very high HCT ultimate tensile strength values (Figure 9) and that a high correlation of strength-to-modification level existed. A high level of correlation between the level of polymer modification and the fracture energy of asphalt mixtures at low temperatures has also been reported by Hesp et al. (*13*). Conversely, the correlation between IDT ultimate tensile strength and level of polymer modification was found to be very weak. Four different levels of SBS modification (0%, 2%,

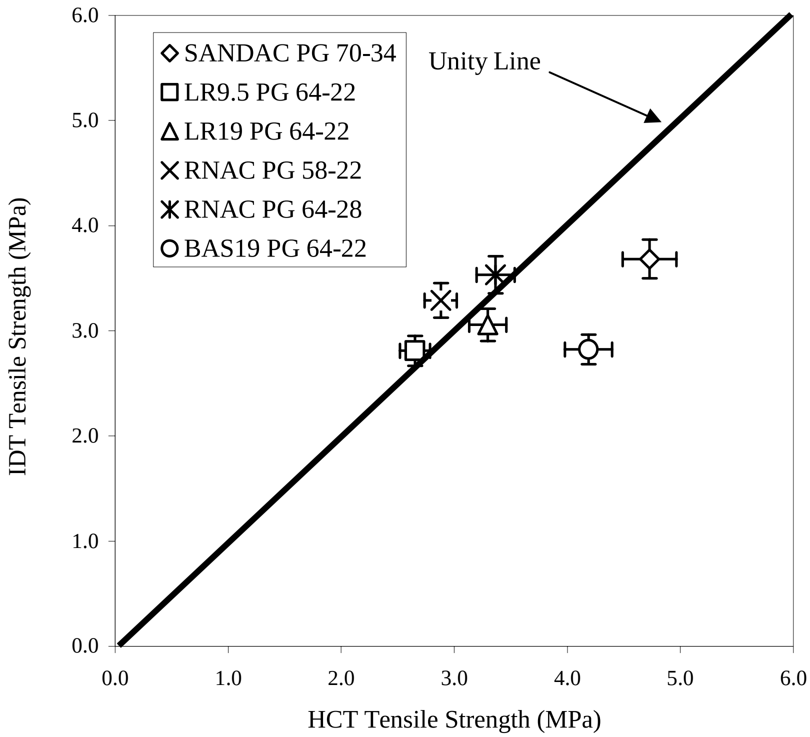

FIGURE 8 First failure tensile strength for all mixtures at –10°C (*3*).

4%, and 6%) were used to produce four 9.5-mm dense-graded asphalt modified mixtures. The HCT ultimate tensile strengths were found to be 4.67, 5.21, 5.79, and 6.43 MPa for 0, 2, 4, and 6% SBS, respectively. The IDT ultimate tensile strengths for the same percentages of SBS were found to be 3.32, 3.58, 3.16, and 3.93 MPa, respectively. Again, the ability of the HCT to provide a constant strain rate up to failure might be the key to the higher, more consistent strength measurements relative to the indirect testing mode. The affect of maximum aggregate size on the sensitivity of ultimate tensile strength as obtained from the HCT and the IDT modes also was investigated. No significant aggregate size effect was detected (*3*) for mixtures up to 19-mm nominal maximum aggregate size.

Complex (“Dynamic”) Modulus

To determine whether the HCT is capable of measuring the complex modulus (E*) for the thermal cracking and fatigue models in the AASHTO Mechanistic-Empirical Design Guide, a testing program was established to compare the complex modulus in tension as measured by the HCT with that obtained in the more traditional uniaxial compression arrangement (*14*). In the asphalt industry, complex modulus is often referred to as “dynamic” modulus (ASTM D3497). Because the scope of the study was to assess the feasibility of characterizing E* with the HCT device, a limited testing program was conducted. Complex modulus was measured using both the HCT device (tensile testing) and the more traditional uniaxial compression test arrangement.

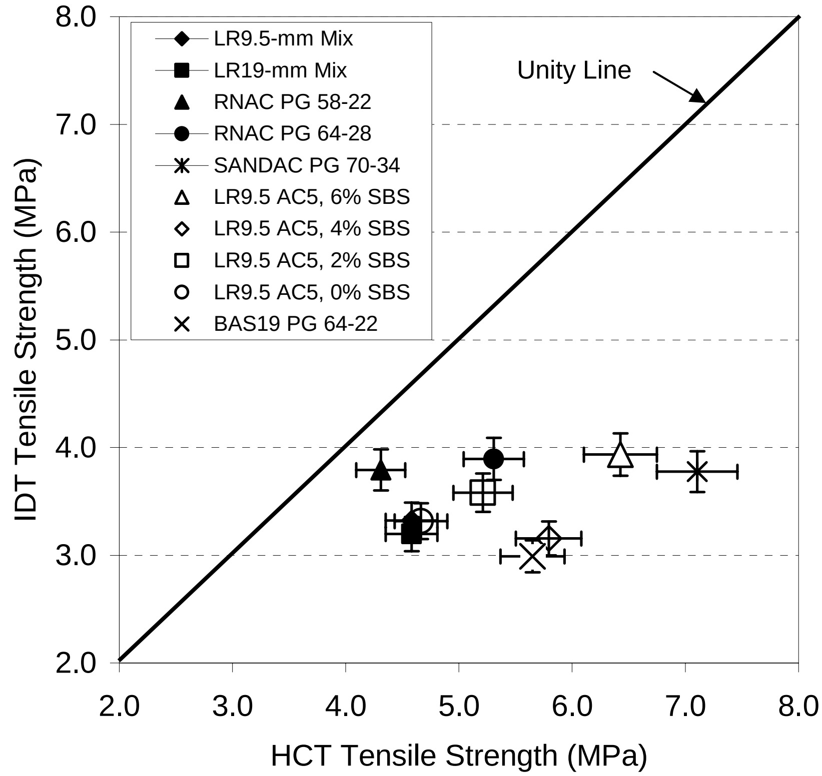

FIGURE 9 Ultimate tensile strength for all mixtures at –10°C (*3*).

Overall, the findings from the E* comparative study show that the HCT produces reasonable estimates of asphalt mixture dynamic modulus for the range of test temperatures and test frequencies covered in this study. The measured E* values in tension and compression were in good agreement with the Witczak dynamic modulus predictive equation (*15*). Figure 10 is a plot of the test results for the general aviation mixture tested during this study. The results for the interstate mixture showed similar trends to Figure 10 (*14*), and were therefore omitted for brevity. The tension and compression test results followed the expected trends with respect to frequency, temperature, and mixture changes; for example, increased E* values were noted for increased test frequency, decreased test temperature, and stiffer binder grade.

Also, as expected, the effect of frequency on E* was greatly diminished at 0°C, because the binder master stiffness curve becomes relatively flat at lower temperatures and higher test frequencies. It is interesting to note that tension and compression testing yielded similar results for both 0°C and 20°C. A divergence in test results would be expected for E* testing at higher temperatures (*16*). This, combined with the fact that compression or shear testing is more desirable for rutting evaluations, makes it reasonable to conclude that the HCT device described herein would have little or no application in the direct evaluation of rutting resistance of asphalt mixtures. However, as will be described in the next section, the inner core removed when producing hollow cylinder specimens could be used for rutting evaluation.

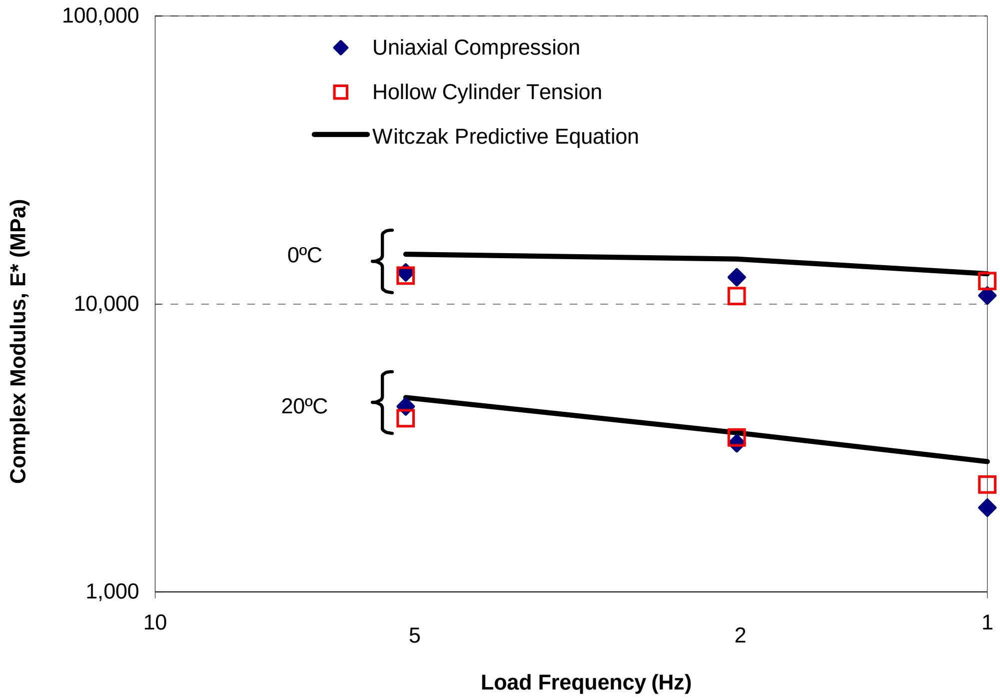

FIGURE 10 Complex modulus comparison for 9.5-mm nominal maximum aggregate size mixture with PG 58–22 binder (*14*).

HCT AS A SIMPLE PERFORMANCE TEST

In terms of using the HCT as a stand-alone simple performance tester, it is not yet clear what role this device might play in this regard. On one hand, the HCT is arguably the simplest, smallest, and yet most resolute and repeatable fundamental test device available, which are all certainly desirable characteristics for a test device to be used in the asphalt industry. On the other hand, the paradigm created under Marshall and Hveem mix design methods involved the use of proof testers at higher temperatures to assess mixture stability, an indicator of rut-resistance. Thus, although the HCT device clearly shows great potential for simplifying testing equipment and testing rigors associated with mixture testing under the AASHTO Mechanistic-Empirical Design Guide, its role as a stand-alone simple performance tester may be most desirable in situations when thermal cracking and fatigue cracking are the predominant distress modes to be addressed through performance testing. This would be the case in cooler climates or when agencies have sufficient confidence in developing rut-resistant mixtures through other means, such as material selection, careful design of aggregate structure, torture testing, and so forth.

Because the fabrication of hollow cylinder specimens involves coring of a standard 150-mm diameter gyratory specimens (Figure 3), it may be possible to use the “left over” inner core specimen for performance testing at higher temperatures for rutting considerations. A second testing device would be required to carry out rutting-related characterization (uniaxial or triaxial compression) because rutting is related to compression and shear properties at high in-service temperatures. Nevertheless, such a testing suite would have the advantage of halving the required material quantities, oven space, and preparation time for complete performance testing.

Additional work is needed to fully evaluate the HCT across a broader range of materials. In particular, the accuracy and repeatability of the HCT for mixtures with larger aggregate sizes has not yet been established, even though the data presented in this paper suggested that good results can be obtained for mixtures up to 19-mm nominal maximum aggregate size. Furthermore, specimen nonhomogeneity caused by taking the outer portion of the gyratory specimen, which is in contact with the mold during compaction does not appear to significantly affect test results, but merits additional investigation (*17*). Finally, the volume-based strain measuring system should prove to be a highly desirable alternative to strain gages because it eliminates the time and cost associated with strain gages (*9*). Another advantage of the volume-based strain measuring system is the ability to assess average strain across the entire inner surface of the hollow cylinder, which will tend to increase test repeatability.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

This paper explored the possibility of using the HCT as a simple performance test in the asphalt industry. There are many benefits to testing in the HCT mode, including test simplicity, test portability, simple stress states and minimal end effects, and simplicity of specimen preparation. The results of several recent studies were summarized, indicating that the HCT produces fundamental measures of creep compliance and complex modulus of HMA. Furthermore, tensile strength results, although a test-mode-dependent and specimen-size-dependent property, appeared reasonable when compared with the Superpave IDT.

It appears that the HCT is compatible with the requirements of the NCHRP 1–37A software models (used in the AASHTO Mechanistic-Empirical Design Guide) for thermal cracking performance prediction, for example, a suitable surrogate test for the IDT. Because it also can measure E* at low-to-intermediate temperatures, it appears possible to use the HCT to collect the necessary inputs for running the fatigue cracking performance prediction models used in NCHRP 1–37A software. As opposed to most E* test devices, the HCT is capable of measuring E* in tension, which is desirable because fatigue damage in asphalt pavements is predominantly a tension-induced phenomenon. It is anticipated that minor changes to the AASHTO Mechanistic-Empirical Design Guide distress model calibration constants will be needed when using HCT data for asphalt material property inputs.

Because the HCT measures properties in tension, it is not anticipated that the device can be used in rutting evaluations. However, because the fabrication of hollow cylinder specimens involves coring of a standard 150-mm diameter gyratory specimens, it may be possible to use the leftover inner core specimen for performance testing at higher temperatures. A second testing device would be required to carry out rutting-related characterization (uniaxial or triaxial compression), because rutting is related to compression and shear properties at high in-service temperatures.

Ongoing research efforts will lead to a more rigorous assessment of the accuracy and repeatability of the HCT across a broad range of mixtures in the very near future. A prototype volume-based strain measurement system was developed recently, and it has produced very promising results. This system will make the HCT a very rapid, simple, and cost-effective test method, which are desirable characteristics for a simple performance test to supplement volumetric mixture design procedures.

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AUTHORS' NOTE

Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the authors and do not necessarily reflect the views of Test Quip, Inc., or FHWA.

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Performance Testing for Hot-Mix Asphalt

E. RAY BROWN
PRITHVI S. KANDHAL
JINGNA ZHANG
National Center for Asphalt Technology

There has been much emphasis on the development and acceptance of performance test(s) for hot-mix asphalt. Although there has been much work in recent years to evaluate a number of performance tests, none have been adopted nationally. A summary of the literature on performance tests is presented. A list of tests is included, with the advantages and disadvantages of each test discussed. The emphasis was on examining a wide range of tests and identifying performance tests for permanent deformation that would be easiest to adopt. This information should be helpful to anyone considering the adoption of a performance test for hot-mix asphalt. Much research is under way to examine existing and new performance tests. The information provided here was the best information available at the time of preparation. As newer tests are developed, they should be evaluated, compared with the tests discussed in this report, and adopted if they show significant advantages.

The Superpave® Mixture Design and Analysis System was developed in the early 1990s under the Strategic Highway Research Program (SHRP). The Superpave design method for hot-mix asphalt (HMA) mixtures consists of three proposed phases: (*a*) materials selection, (*b*) aggregate blending, and (*c*) volumetric analysis on specimens compacted using the Superpave Gyratory Compactor (SGC) (*1*). The method was intended to have a fourth step that would provide a way to analyze the mixture properties and to determine performance potential; however, this fourth step is not yet available for adoption. Most highway agencies in the United States have now adopted the volumetric mixture design method. However, no performance test is available to complement the Superpave volumetric mixture design method. The traditional Marshall and Hveem mixture design methods had associated strength tests. Although the Marshall and Hveem stability tests were empirical, they did provide some measure of the mix quality. A significant amount of work is going on to develop a strength test (for example, NCHRP 9–19); however, one was not finalized for adoption by the time this report was prepared, and it will likely be several months to years before one is recommended nationally. Considering that approximately 2 million tons of HMA is placed in the United States during a typical construction day, contractors and state agencies must have some means as soon as practical to better evaluate the performance potential of HMA. These test methods do not need to be perfect, but they should be available in the immediate future to ensure good mix performance.

Research from WesTrack, NCHRP 9–7 (Field Procedures and Equipment to Implement SHRP Asphalt Specifications) and other experimental construction projects has shown that the Superpave volumetric mixture design method alone is not sufficient to ensure reliable mixture performance over a wide range of materials, traffic, and climatic conditions. The HMA industry needs a simple performance test to help ensure that a quality product is produced.

There are five areas of distress for which guidance is needed: fatigue cracking, rutting, thermal cracking, friction, and moisture susceptibility. All of these distresses can result in a loss of performance, but rutting is the one distress that is most likely to be a sudden failure as a result of unsatisfactory HMA. Other distresses are typically long-term failures that show up after a few years of traffic.

Because of the immediate need for some method to evaluate performance potential, the National Center for Asphalt Technology (NCAT) Board of Directors requested that NCAT provide guidance that could improve mixture analysis procedures. It is anticipated that this guidance can be adopted until something better is developed in the future through projects such as NCHRP 9–19 and others. However, partly as a result of warranty work, the best technology currently available needs to be identified and adopted. This report provides a first step in identifying appropriate tests. It is anticipated that the findings in this report will be reviewed on a regular basis to determine whether improved guidance is available and needs to be implemented.

The purpose of this project was to evaluate available information on permanent deformation, fatigue cracking, low-temperature cracking, moisture susceptibility, and friction properties and, as appropriate, to recommend performance test(s) that could be adopted immediately to ensure improved performance. Emphasis was placed on permanent deformation.

DESCRIPTIONS OF DISTRESS MECHANISMS

There are many reports that provide much detail on the failure mechanisms for the various HMA distresses. A very brief description of the failure mechanism for each distress mechanism is provided in this section.

Permanent Deformation

Rutting (or permanent deformation) results from the accumulation of small amounts of unrecoverable strain as a result of repeated loads applied to the pavement. Rutting can occur as a result of problems with the subgrade, unbound base course, or HMA. The focus of this effort is permanent deformation caused by HMA mix problems. Permanent deformation in HMA is caused by consolidation or lateral movement of the HMA under traffic. Shear failure (lateral movement) of the HMA courses generally occurs in the top 100 mm of the pavement surface (*2*); however, it can occur deeper if satisfactory materials are not used. Rutting in pavement usually develops gradually with increasing numbers of load applications, typically appearing as longitudinal depressions in the wheel paths sometimes accompanied by small upheavals to the sides. It is typically caused by a combination of densification (decrease in volume and, hence, increase in density) and shear deformation and can occur in any one or more of the HMA layers as well as in the unbound materials underneath the HMA. Eisenmann and Hilmer (*3*) found that rutting was mainly caused by deformation flow rather than volume change.

Fatigue Cracking

Fatigue cracking often is called alligator cracking because its closely spaced crack pattern is similar to the pattern on an alligator's back. This type of failure generally occurs when the pavement has been stressed to the limit of its fatigue life by repetitive axle load applications. Fatigue cracking is often associated with loads that are too heavy for the pavement structure or more repetitions of a given load than provided for in design. The problem is often made worse by inadequate pavement drainage, which contributes to this distress by allowing the pavement layers to become saturated and to lose strength. The HMA layers experience high strains when the underlying layers are weakened by excess moisture and fail prematurely in fatigue. Fatigue cracking also can be caused by repetitive passes with overweight trucks or inadequate pavement thickness because of poor quality control during construction (*4, 5*).

Fatigue cracking can lead to the development of potholes when the individual pieces of HMA physically separate from the adjacent material and are dislodged from the pavement surface by the action of traffic. Potholes generally occur when fatigue cracking is in the advanced stages and when relatively thin layers of HMA have been used.

In the past, fatigue cracking was thought to initiate from the bottom and to migrate toward the surface. These cracks began because of the high tensile strain at the bottom of the HMA. Recently, fatigue cracks have been observed starting at the surface and migrating downward. The surface cracking starts because of tensile strains in the surface of the HMA. Generally speaking, it is believed that, for thin pavements, the fatigue cracking typically starts at the bottom of the HMA and, for thick pavements, the fatigue cracking typically starts at the HMA surface. Typically, fatigue cracking is caused by a lack of adequate pavement structure and is not typically caused by a lack of control of HMA properties; however, these properties can certainly have a secondary effect.

Low-Temperature Cracking

Low-temperature cracking of asphalt pavements is attributed to tensile strain induced in HMA as the temperature drops to some critically low level. As its name indicates, low-temperature cracking is a distress type caused by low pavement temperatures rather than by applied traffic loads even though traffic loads likely do play a role. Thermal cracking is characterized by intermittent transverse cracks (i.e., perpendicular to the direction of traffic) that may occur at a surprisingly consistent spacing (*5*). Low-temperature cracks form when an asphalt pavement layer shrinks in cold weather. As the pavement shrinks, tensile strains build within the layer. At some point along the pavement, the tensile stress exceeds the tensile strength and the asphalt layer cracks. Thus, low-temperature cracks often occur from a single event of low temperature. Low-temperature cracking also can be a fatigue phenomenon that results from the cumulative effect of many cycles of cold weather. The magnitude and frequency of low temperatures and stiffness of the asphalt mixture on the surface are major factors in the occurrence and intensity of low-temperature transverse cracking. The crack starts at the surface and works its way downward. The mixture stiffness, which is primarily related to the properties of the asphalt binder, is probably the greatest contributor to low-temperature cracking.

Moisture Susceptibility

Environmental factors such as temperature and moisture can have a profound effect on the durability of HMA pavements. When critical environmental conditions are coupled with poor materials and traffic, premature failure may result from stripping of the asphalt binder from the aggregate particles.

Moisture can degrade the integrity of an HMA matrix by three mechanisms (*6*):

1. Loss of cohesion (strength) of the asphalt film, which may be caused by several mechanisms;
2. Failure of the adhesion (bond) between the aggregate and asphalt; and
3. Degradation or fracture of individual aggregate particles when subjected to freezing.

When the aggregate tends to have a stronger preference for water than asphalt, the asphalt often is “stripped” away. Stripping leads to loss in quality of mixture and ultimately leads to failure of the pavement because of raveling, rutting, or cracking.

Friction Properties

Friction during wet conditions continues to be a major concern for most highway agencies around the world. Recognizing the importance of providing safe pavements for travel during wet weather, most highway agencies have established programs to provide adequate pavement friction or skid resistance (*7*).

Friction is defined as the relationship between the vertical force and the horizontal force developed as a tire slides along the pavement surface (*4*). The friction of a pavement surface is a function of the surface texture, which is divided into two components (*8*): microtexture and macrotexture. The microtexture provides a gritty surface to penetrate thin water films and to produce good frictional resistance between the tire and the roadway. The macrotexture provides drainage channels for water expulsion between the tire and the roadway thus allowing better tire contact with the pavement to improve frictional resistance and to prevent hydroplaning.

To the vehicle operator, friction is a measure of how quickly a vehicle can be stopped. To the design engineer, friction is an important safety-related property of the pavement surface that must be accounted for through proper selection of materials, design, and construction. In terms of pavement management, friction is a measure of serviceability. The decrease of friction below a minimum acceptable (safe) level prevents the pavement from serving its desired function. In a life-cycle cost analysis of pavement performance, the pavement designer and the owner agency may need to consider restoring friction at some point.

Friction characteristics that are desirable in a good pavement surface are as follows (*9*):

1. High friction (Ideally the friction when wet should be acceptable.);
2. Little or no decrease of the friction with increasing speed (The friction of dry pavement is nearly independent of speed, but this is not the case for wet pavement.);
3. No reduction in friction with time from polishing or other causes; and
4. Resistance to wear by abrasion of aggregate, attrition of binder or mortar, or loss of particles.

Many states have methods they have found to be successful to ensure good friction with local materials. Work is needed to develop a national standard to test and evaluate friction properties of HMA in the laboratory.

COMPARISON OF METHODS TO EVALUATE PERMANENT DEFORMATION

This report focuses on permanent deformation. It identifies methods that have been used to evaluate permanent deformation. The tests that appeared to have some potential for predicting rutting performance were selected for further evaluation with four mixes with known relative performance. Detailed test results are available in the NCAT Report 2001-05 (*10*). A summary of the advantages and disadvantages of each of the tests considered for permanent deformation is provided in Table 1.

The tests evaluated in this study can be classified as one of six types:

1. Diametral tests,
2. Uniaxial tests,
3. Triaxial tests,
4. Shear tests,
5. Empirical tests, and
6. Simulative tests.

TABLE 1 Comparative Assessment of Test Methods

<table>
<tr><th colspan="2">Test Method</th><th>Sample Dimension</th><th>Advantages</th><th>Disadvantages</th></tr>
<tr><td rowspan="4">Fundamental: Diametral Tests</td><td>Diametral static (creep)</td><td>4 in. diameter × 2.5 in. height</td><td>• Test is easy to perform.
• Equipment generally available in most laboratories.
• Specimen is easy to fabricate.</td><td rowspan="4">• State of stress is nonuniform and strongly dependent on the shape of the specimen.
• Test may be inappropriate for estimating permanent deformation
• High-temperature (load) changes in the specimen shape affect the state of stress and the test measurement significantly.
• Tests were found to overestimate rutting.
• For the dynamic test, the equipment is complex.
• Test relates poorly to permanent deformation.</td></tr>
<tr><td>Diametral repeated load</td><td>4 in. diameter × 2.5 in. height</td><td>• Test easy to perform.
• Specimen is easy to fabricate.</td></tr>
<tr><td>Diametral dynamic modulus</td><td>4 in. diameter × 2.5 in. height</td><td>• Specimen is easy to fabricate.
• Test is nondestructive.</td></tr>
<tr><td>Diametral strength test</td><td>4 in. diameter × 2.5 in. height</td><td>• Test is easy to perform.
• Equipment is generally available in most laboratories.
• Specimen is easy to fabricate.
• Minimum test time is required.</td></tr>
<tr><td rowspan="4">Fundamental: Uniaxial Tests</td><td>Uniaxial static (creep)</td><td>4 in. diameter × 8 in. height and others</td><td>• Test is easy to perform.
• Test equipment is simple and generally available.
• Test is wide spread, well known.
• More technical information is available.</td><td>• Ability to predict rutting is questionable.
• Restricted test temperature and load levels do not simulate field conditions.
• Test does not simulate field dynamic phenomena.
• Difficult to obtain 2:1 ratio specimens in laboratory.</td></tr>
<tr><td>Uniaxial repeated load</td><td>4 in. diameter × 8 in. height and others</td><td>• Test better simulates traffic conditions.</td><td>• Equipment is more complex.
• Restricted test temperature and load levels do not simulate field conditions.
• Difficult to obtain 2:1 ratio specimens in laboratory.</td></tr>
<tr><td>Uniaxial dynamic modulus</td><td>4 in. diameter × 8 in. height and others</td><td>• Test is nondestructive.</td><td>• Equipment is more complex.
• Difficult to obtain 2:1 ratio specimens in laboratory.</td></tr>
<tr><td>Uniaxial strength test</td><td>4 in. diameter × 8 in. height and others</td><td>• Test is easy to perform.
• Test equipment is simple.
• Minimum test time is required.</td><td>• Test has questionable ability to predict permanent deformation.</td></tr>
</table>

(*continued on next page*)

TABLE 1 (*continued*) **Comparative Assessment of Test Methods**

	Test Method	Sample Dimension	Advantages	Disadvantages
Fundamental: Triaxial Tests	Triaxial static (creep confined)	4 in. diameter × 8 in. height and others	• Test and equipment are relatively simple. • Test temperature and load levels better simulate field conditions than unconfined tests. • Test is potentially inexpensive.	• Test requires a triaxial chamber. • Confinement increases complexity of the test.
	Triaxial repeated load	4 in. diameter × 8 in. height and others	• Test temperature and load levels better simulate field conditions than unconfined test. • Test better expresses traffic conditions. • Test can accommodate varied specimen sizes. • Criteria are available.	• Equipment is relatively complex and expensive. • Test requires a triaxial chamber.
	Triaxial dynamic modulus	4 in. diameter × 8 in. height and others	• Provides necessary input for structural analysis. • Test is nondestructive.	• At high temperature, it is a complex test system (small deformation measurement sensitivity is needed at high temperature). • Some possible minor problems occur because of stud, LVDT arrangement. • Equipment is more complex and expensive. • Test requires a triaxial chamber.
	Triaxial strength	4 or 6 in. diameter × 8 in. height and others	• Test and equipment are relatively simple. • Minimum test time is required.	• Test's ability to predict permanent deformation is questionable. • Test requires a triaxial chamber.
Fundamental: Shear Tests	SST frequency sweep test–shear dynamic modulus	6 in. diameter × 2 in. height	• The applied shear strain simulates the effect of road traffic. • AASHTO standardized procedure is available. • Specimen is prepared with SGC samples. • Master curve could be drawn from different temperatures and frequencies. • Test is nondestructive.	• Equipment is extremely expensive and rarely available. • Test is complex and difficult to Run; usually need special training. • SGC samples need to be cut and glued before testing.
	SST repeated shear at constant height	6 in. diameter × 2 in. height	• The applied shear strains simulate the effect of road traffic. • AASHTO procedure is available. • Specimen available from SGC samples.	• Equipment is extremely expensive and rarely available. • Test is complex and difficult to run, usually need special training. • SGC samples need to be cut and glued before testing. • Test results have a high COV. • More than three replicates are needed.

NOTE: APA, asphalt pavement analyzer; COV, coefficent of variation; FRT, French rutting tester; GLWT, Georgia loaded-wheel tester; GTM, gyratory testing machine; HWTD, Hamburg wheel-tracking device; LVDT, linear variable differential transformer; LWT, loaded wheel tester; RLWT, rotary loaded wheel tester; SGC, Superpave gyratory compactor; SST, Superpave shear test.

(*continued on next page*)

TABLE 1 (*continued*) **Comparative Assessment of Test Methods**

Test Method		Sample Dimension	Advantages	Disadvantages
Fundamental: Shear Tests	Triaxial shear strength test	6 in. diameter × 2 in. height	• Test time is short.	• Test is used much less. • Confined specimen requirements add complexity.
Empirical Tests	Marshall test	4 in. diameter × 2.5 in. height or 6 in. diameter × 3.75 in. height	• Test is wide spread, well known, and standardized for mix design. • Test procedure is standardized. • Test is easiest to implement and short test time. • Equipment available in all laboratories.	• Test cannot correctly rank mixes for permanent deformation. • Little data indicate test is related to performance.
	Hveem test	4 in. diameter × 2.5 in. height	• Test was developed with a good basic philosophy. • Test time is short. • Triaxial load is applied.	• Test was not used as widely as Marshall in the past. • California kneading compacter is needed. • Test cannot correctly rank mixes for permanent deformation.
	GTM	Loose HMA	• Test simulates the action of rollers during construction. • Parameters are generated during compaction. • Criteria are available.	• Equipment is not widely available. • Test cannot correctly rank mixes for permanent deformation.
	Lateral pressure indicator	Loose HMA	• Test occurs during compaction.	• Problems occur interpreting test results. • Few data are available.
Simulative Tests	APA	Cylindrical 6 in. × 3.5 or 4.5 in. or beam	• Test simulates field traffic and temperature conditions. •Test was modified and improved from GLWT. • Test is simple to perform. • 3–6 samples can be tested at the same time. • This is the most widely used LWT in the United States. • Guidelines (criteria) are available. • Cylindrical specimens use SGC.	• Test is relatively expensive except for the new table top version.
	HWTD	10.2 in. × 12.6 in. × 1.6 in.	• Test is widely used in Germany. • Test can evaluate moisture-induced damage. • 2 samples are tested at the same time.	• Test has less potential to be accepted widely in the United States
	FRT	7.1 in. × 19.7 in. × 0.8 to 3.9 in.	• Successfully used in France. • Two HMA slabs can be tested at one time.	• Not widely available in the United States

(*continued on next page*)

TABLE 1 (*continued*) **Comparative Assessment of Test Methods**

Test Method		Sample Dimension	Advantages	Disadvantages
Simulative Tests	PURWheel	11.4 in. × 12.2 in.× 1.3, 2, 3 in.	• Specimen can be taken from field as well as laboratory prepared.	• Linear compactor is needed. • Test is not widely available.
	Model mobile load simulator	47 in. × 9.5 in.× thickness	• Specimen is scaled to full-scaled load simulator.	• Extra materials are needed. • Test is not suitable for routine use. • A standard for laboratory specimen fabrication needs to be developed. • Test is not widely available in United States.
	RLWT	6 in. diameter × 4.5 in. height	• Test uses SGC sample. • There is some relationship with the APA rut depth	• Test is not widely used in United States. • Very few data are available.
	Wessex device	6 in. diameter × 4.5 in. height	• Two specimens can be tested at one time. • Test uses SGC samples.	• Test is not widely used or well known. • Very few data are available.

NOTE: APA, asphalt pavement analyzer; RLWT, rotary loaded wheel tester; SGC, Superpave gyratory compactor.

The results of the analysis and discussion on all of these tests are provided in the following sections.

Diametral Tests

The diametral tests involved creep, repeated load permanent deformation, dynamic modulus, and tensile strength test. The diametral test does not appear to be a suitable test for evaluating permanent deformation. This is a tensile-type test that is likely to be affected more by changes in binder properties than would be expected to be seen in the field. Because this is a tensile test, it is not reasonable that it would be a good predictor of rutting. The cost of equipment to conduct the diametral tests is relatively low when repeated loading is not required. If repeated loading is required, the cost is considerably higher and the difficulty of the testing is increased. Few performance data are available to show that any diametral tests are useful in predicting rutting. Data are available to indicate that there is a trend between this type of test and performance, but other test methods are more suitable. Tests results (*10*) have shown that these tests do not measure up to the reasonableness test. Laboratory tests show that the indirect tensile strength test results and the repeated load tests do not match the expected performance. Although these tests may have some applicability in indicating performance, other tests are more likely to be successful. These tests should not be considered for immediate adoption.

Uniaxial Tests

A second type of test that potentially can be used to predict performance is the uniaxial test. The four types of test considered were creep, repeated load permanent deformation, dynamic modulus, and compressive strength. One of the biggest problems with this type of test is its questionable ability to predict performance because of the amount of load and temperature that can be used for testing. It is believed that the temperature and stress applied in the laboratory should be similar to hat the mixes will actually be subjected to in the field. The load or temperature must be decreased significantly from that expected in the field, otherwise these tests cannot be conducted without premature failure of the samples. The test is simple and inexpensive to conduct when using static loads; however, the complexity and cost increase considerably when dynamic loads are required. Little information is available for these tests that correlate test results to performance. Some work has been accomplished that shows trends with performance (*11–13*). Because of the lack of a good correlation with performance and lack of specific criteria, none of these tests are recommended for immediate adoption to predict permanent deformation; however, some of these tests are being studied in NCHRP 9–19 and may prove to be acceptable when this study is completed.

Triaxial Tests

A third type of test that was considered is the triaxial test. The difference between this series of tests and the uniaxial tests discussed is that the triaxial tests include confining pressure. Applying a confining pressure allows one to more closely duplicate the in-place pressure and temperature without prematurely failing the test sample. Some rutting information is available for the confined creep and repeated load tests (*12*, *13*). Less information is available for the dynamic modulus and strength tests. These triaxial tests are complicated somewhat by the requirement for a triaxial cell, but this does not preclude the use of this test. The confined creep and repeated load tests have been used and do have some potential in predicting rutting. Both of these tests are being studied in NCHRP 9–19 and may be considered for use in the future. The confined creep test is simple and easy, but the correlation with rutting is not very good. It has been recognized widely that the confined repeated load deformation test is better correlated with performance but more difficult to conduct. At this time, these tests are not recommended for immediate adoption primarily because of the lack of specific criteria and specific test procedure in some cases. At the conclusion of NCHRP 9–19, sufficient data will be available to adopt one or more of these tests, if appropriate, and to provide details concerning test procedures.

Shear Tests

A fourth type of test considered was the shear test, including the Superpave shear test (SST). The SST test is very complicated and expensive, and it currently does not have an acceptable model to predict performance. This test is not reasonable for quality control testing. At this time, none of the SST tests are finalized sufficiently for immediate adoption.

Empirical Tests

A fifth series of tests considered were empirical tests, including Marshall stability and flow, Hveem stability, gyratory testing machine (GTM), and lateral pressure indicator. Marshall and Hveem tests were used for years with very limited success. The GTM has had limited use for many years. It does have some potential but sufficient information is not available for immediate adoption. The lateral pressure indicator (LPI) is a new test that does show some promise, but more research is needed to show that it is related to performance. The LPI requires very little additional effort and very little cost. None of these tests should be selected for use at this time.

Simulative Tests

The final series of tests involves simulative tests, which primarily include wheel-tracking tests. The asphalt pavement analyzer (APA), Hamburg wheel-tracking device (HWTD), and French rutting tester (FRT) appear to provide reasonable results and do have some data that correlate with performance that can be used to determine specific criteria. Although the wheel-tracking tests are not mechanistic, they do tend to simulate what happens in the field. Mechanistic tests are being studied by others (NCHRP 9–19) and may be available for adoption in the near future.

Most tests evaluated for their ability to predict performance have actually been compared with the results from one of these wheel-tracking devices because they do simulate rutting in the laboratory. Based on all available information, it is recommended that the APA, HWTD, and FRT be considered for use in mix design and quality control/quality assurance (QA/QC). Sufficient data are available to set criteria, and these are provided later in the recommendations. The simulative tests (wheel-tracking tests) appear to be the only type of test ready for immediate adoption. These tests are not the final answer, but they can serve the industry until a better answer is available.

Discussions on all the test methods in Table 1 can be found in NCAT Report 2001-05 (*10*). Only the recommended procedures (APA, HWTD, and FRT) are illustrated in this paper.

TESTS READY FOR ADOPTION

The stress conditions in a pavement as a loaded wheel passes over it are extremely complex and cannot be precisely calculated or replicated in a laboratory test on a sample of HMA. Hence, it is very difficult to accurately predict performance using a mechanistic approach. This mechanistic approach is much closer to being realized now than in the past, but much work is needed still. Simulative tests, in which the actual traffic loads are modeled, have been used to compare the performance of a wide range of materials, including HMA. In this situation, one does not have to calculate the stresses because stresses similar to that on the roadway are applied to provide a stress state similar to that in the field and the performance monitored. It is very difficult to closely simulate the stress conditions observed in the field, but these tests attempt to do that. Several simulative test methods have been used in the past and are currently being used to evaluate rutting performance. The APA, HWTD, and FRT are discussed in the following sections.

Asphalt Pavement Analyzer

The APA (Figure 1) is a modification of the Georgia loaded-wheel tester (GLWT). It was first manufactured in 1996 by Pavement Technology, Inc. The APA has been used in an attempt to evaluate the rutting, fatigue, and moisture resistance of HMA mixtures.

The GLWT (Figure 2) was developed during the mid-1980s through a cooperative research study between the Georgia Department of Transportation and the Georgia Institute of Technology. Much data have been generated by the Georgia Department of Transportation and others to support the potential use of this device or the APA, which has superseded the GLWT. Testing of samples within the GLWT generally consists of applying a 445-N load onto a pneumatic linear hose pressurized to 690 kPa (100 psi). The load is applied through an aluminum wheel onto the linear hose, which resides on the samples. Test specimens are tracked back and forth under the applied stationary loading. Testing is typically accomplished for 8,000 loading cycles (one cycle is defined as the backward and forward movement over samples by the wheel). However, some researchers have suggested fewer loading cycles may suffice (*14*).

Because the APA is the second generation of the GLWT, it follows a very similar rut testing procedure. A loaded wheel is placed on a pressurized linear hose, which sits on the test specimens and then is tracked back and forth to induce rutting. Similar to the GLWT, most testing in the APA is carried out to 8,000 cycles. Unlike the GLWT, samples also can be tested dry or while submerged in water.

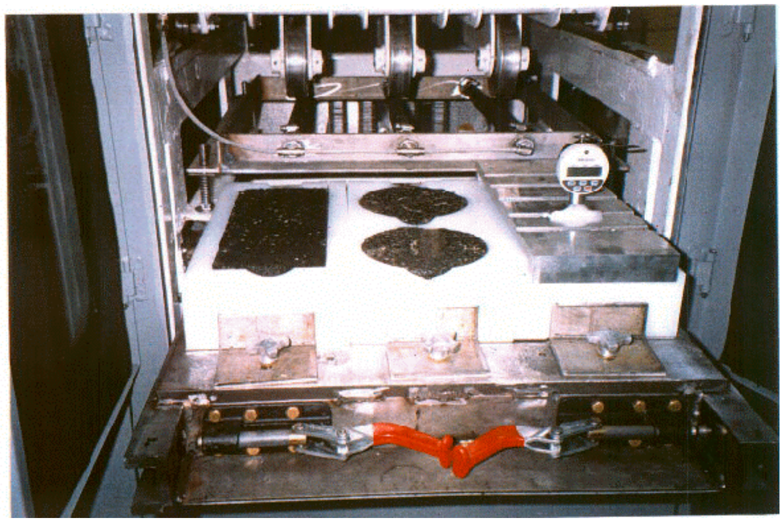

FIGURE 1 The APA.

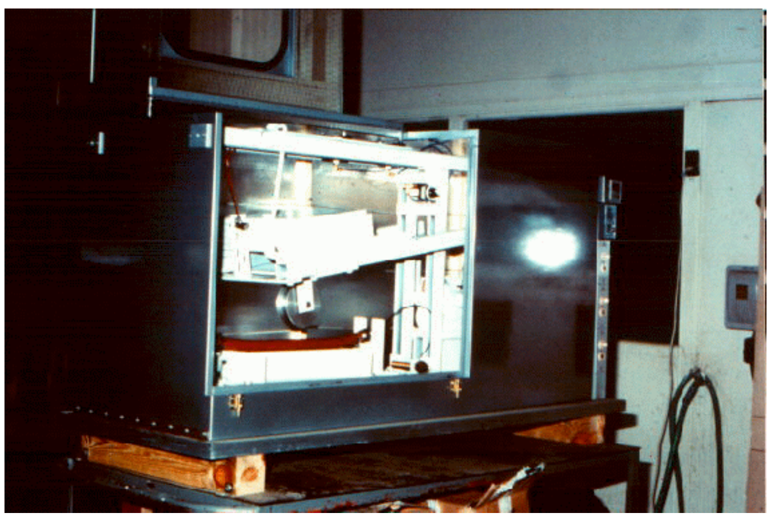

FIGURE 2 The GLWT.

Test specimens for the APA can be either beam or cylindrical. Currently, the most common method of compacting beam specimens is by the asphalt vibratory compactor (*15*). However, some researchers have used a linear kneading compactor for beams (*16*). The most common compactor for cylindrical specimens is the Superpave gyratory compactor (*17*). Beams most often are compacted to 7% air voids; cylindrical samples have been fabricated to both 4% and 7% air voids (*16*). Tests also can be performed on cores or slabs taken from an actual pavement.

Test temperatures for the APA have ranged from 40.6°C to 64°C. The most recent work (*17, 18*) has been conducted at or near expected maximum pavement temperatures, as indicated in the SHRP study.

Wheel load and hose pressure essentially have stayed the same as for the GLWT, 445 N and 690 kPa (100 lb and 100 psi), respectively. One recent study did use a wheel load of 533 N (120 lb) and hose pressure of 830 kPa (120 psi) with good success (*18*). Figure 3 shows a typical

APA rut test result. It indicates that specimens deform rapidly at the beginning of the test and the rate of deformation levels out for mixes that are stable. The amount of permanent deformation per cycle decreases and becomes quite steady after a certain number of load cycles. For unstable mixes, the rate of deformation will actually increase again as more cycles are added.

The WesTrack Forensic Team conducted a study on the performance of some of the test mixes at WesTrack (*19*). Figure 4 presents the results of the actual performance and the predicted performance using the APA. The results are encouraging.

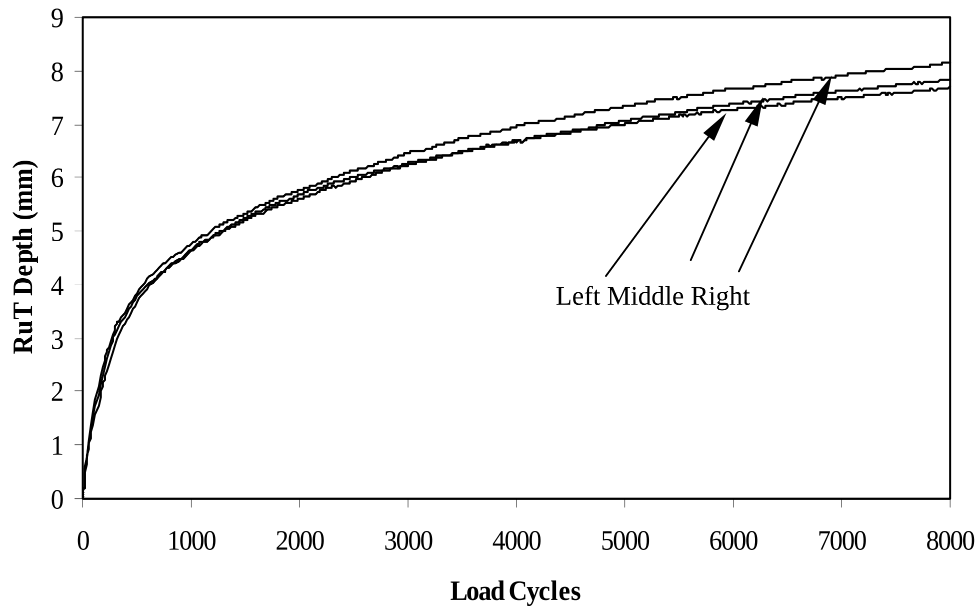

FIGURE 3 Typical APA rut depth versus load cycles.

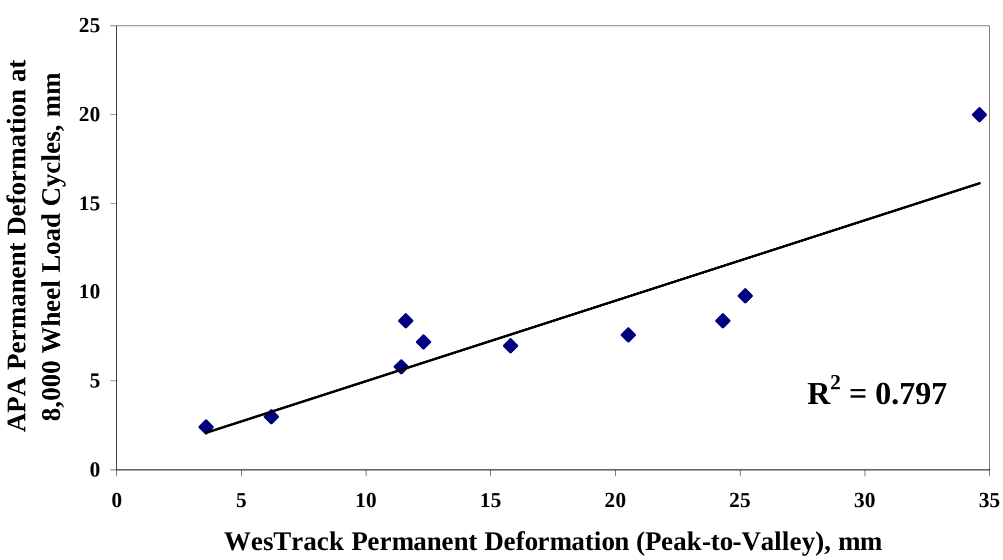

FIGURE 4 APA results versus WesTrack performance (*19*).

The following configurations have been recommended in NCHRP Project 9–17 (*15*) to refine the APA rut test. Test configurations for cylinders include 4% air voids, standard PG temperature, and standard hose. Test configurations for beams include 5% air voids, standard PG temperature, and standard hose. Figures 5 and 6 show the measured rut depths for WesTrack, Minnesota Test Road (MnRoad) test sections versus APA test results for cylinders and beams with 4% and 5% air voids (*15*).

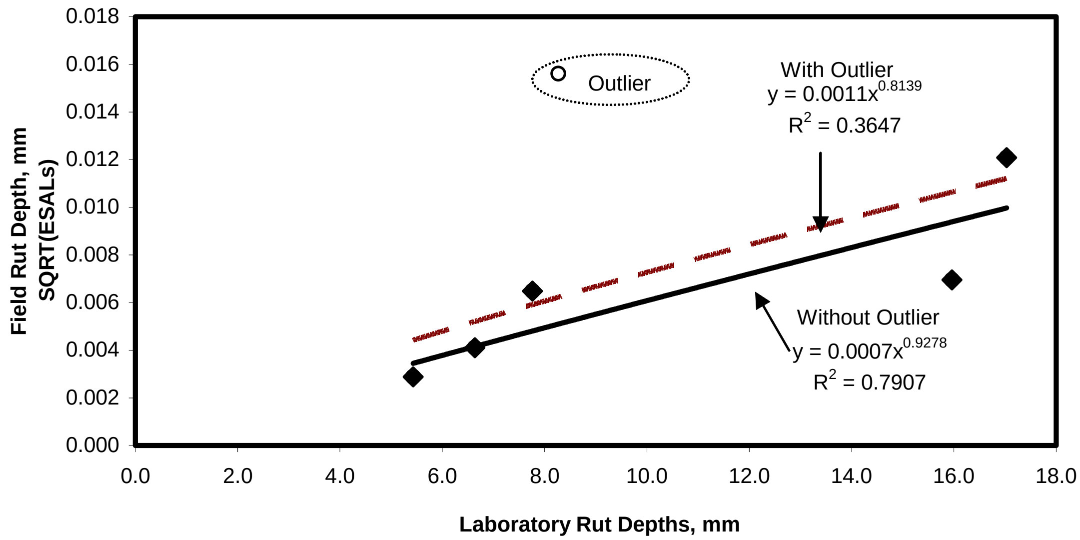

FIGURE 5 Field rut depth versus APA (4% air voids, standard PG temperature, standard hose, and cylinders) test results (after NCHRP 9–17).

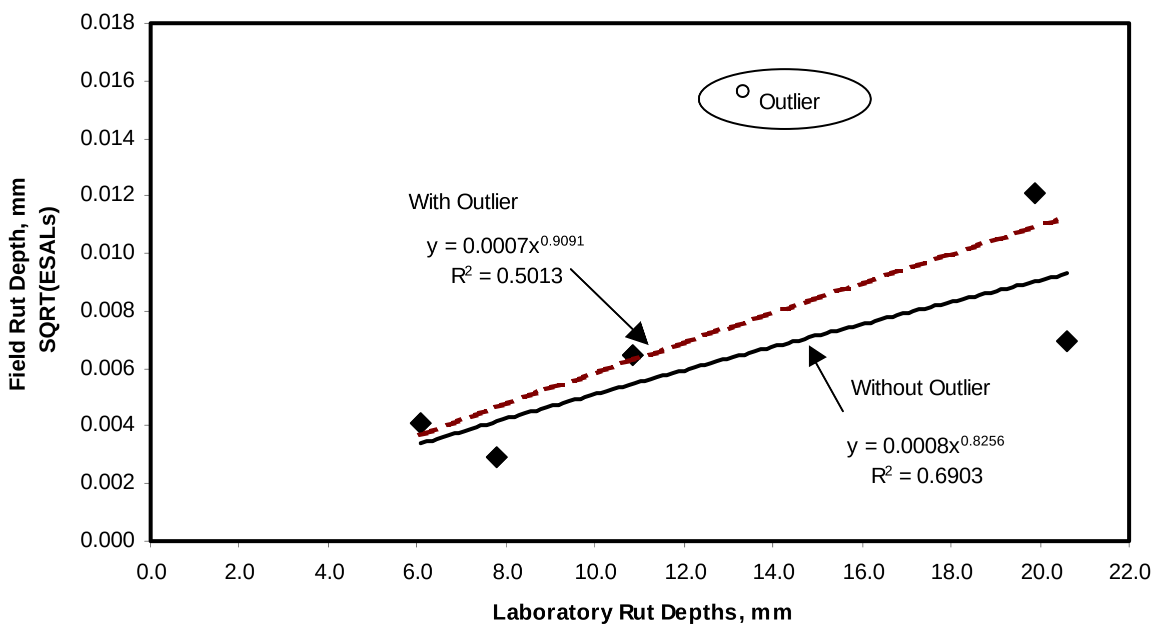

FIGURE 6 Field rut depth versus APA (5% air voids, standard PG temperature, standard hose, and beams) test results (after NCHRP 9–17).

The R^2 for these two plots is low, but there does appear to be one outlier in each of the two figures. If that point is regarded as an outlier, the R^2 for these two plots increases to 0.791 and 0.691 respectively. The R^2 values for combined MnRoad/WesTrack data are very good when the outlier is removed, especially when the different climates, mix types, and traffic loading conditions are considered. Rut depth divided by square root of equivalent single-axle loads (ESALs) was used to normalize the field rut depth. It had been successfully used by NCAT in a national study on rutting (*20*).

Results from the WesTrack Forensic Team study (*19*) and the NCHRP 9–17 project show that use of the APA may help ensure that a satisfactory mix is designed and produced.

Figure 4 indicates that a laboratory rut depth of 6-mm results in a field rut depth of 12.5 mm (0.5 inch). Criteria also have been developed in the past for some other test conditions. Georgia and other states have long specified a maximum rut depth of 5 mm for HMA mixtures as the pass/fail criteria at a temperature of 50°C (*21*). A 2002 study conducted at NCAT (*22*) provided a criterion of 8.2 mm for the APA rut test at standard PG temperature for the location in which the HMA will be used. This higher value for pass/fail criteria is associated with the higher PG temperature used. This test does have the potential to be quickly adopted as a performance test.

Hamburg Wheel-Tracking Device

The HWTD (Figure 7) was developed by Helmut-Wind Incorporated of Hamburg, Germany (*23*). It is used as a specification requirement for some of the most traveled roadways in Germany to evaluate rutting and stripping. Tests within the HWTD are conducted on a slab that is 260 mm wide, 320 mm long, and typically 40 mm thick (10.2 in × 12.6 in × 1.6 in). These slabs are normally compacted to 7 ± 1% air voids using a linear kneading compactor. Testing also has been done using Superpave gyratory compacted samples. Testing in the HWTD is conducted underwater at temperatures ranging from 25°C to 70°C (77°F to 158°F), with 50°C (122°F) being the most common temperature (*24*). Loading of samples in the HWTD is accomplished by applying a 705-N (158-lb) force onto a 47-mm-wide steel wheel. The steel wheel is then tracked back and forth over the slab sample. Test samples are loaded for 20,000 passes or until 20 mm of deformation occurs. The travel speed of the wheel is approximately 340 mm per second (*23*).

As shown in Figure 8, results obtained from the HWTD consist of rut depth, creep slope, stripping inflection point, and stripping slopes.

The creep slope is the inverse of the deformation rate within the linear region of the deformation curve after initial or seating compaction and before stripping (if stripping occurs). The stripping slope is the inverse of the deformation rate within the linear region of the deformation curve, after the onset of stripping. The stripping inflection point is the number of wheel passes corresponding to the intersection of the creep slope and the stripping slope. This value is used to estimate the relative resistance of the HMA sample to moisture-induced damage (*24*).

The WesTrack Forensic Team conducted a study on the performance of coarse-graded mixes at WesTrack sections (*19*). The HWTD was included as one of the four rut testers (APA, HWTD, FRT, and PurWheel tester). Figure 9 presents the results on the actual performance and the laboratory performance using the HWTD. As the figure shows, the HWTD had a correlation coefficient (R^2) of 0.756, which is also very promising.

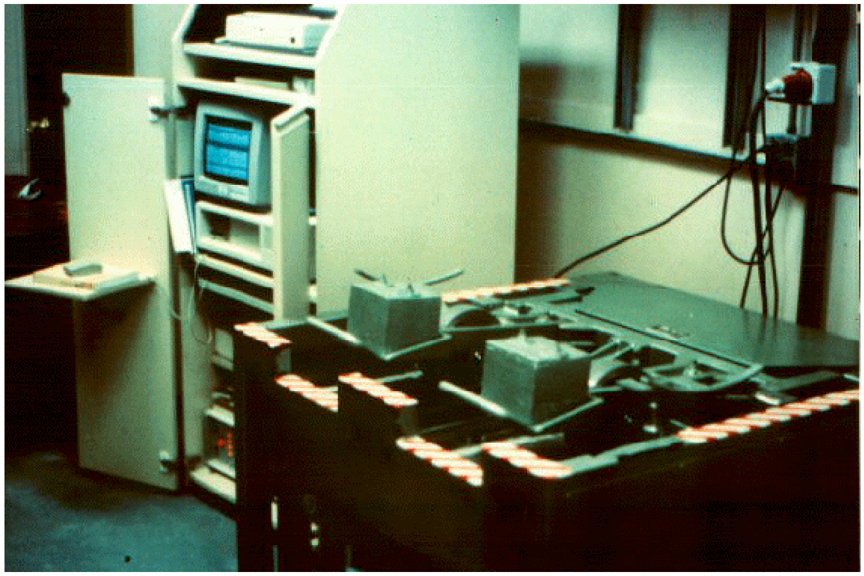

FIGURE 7 The HWTD.

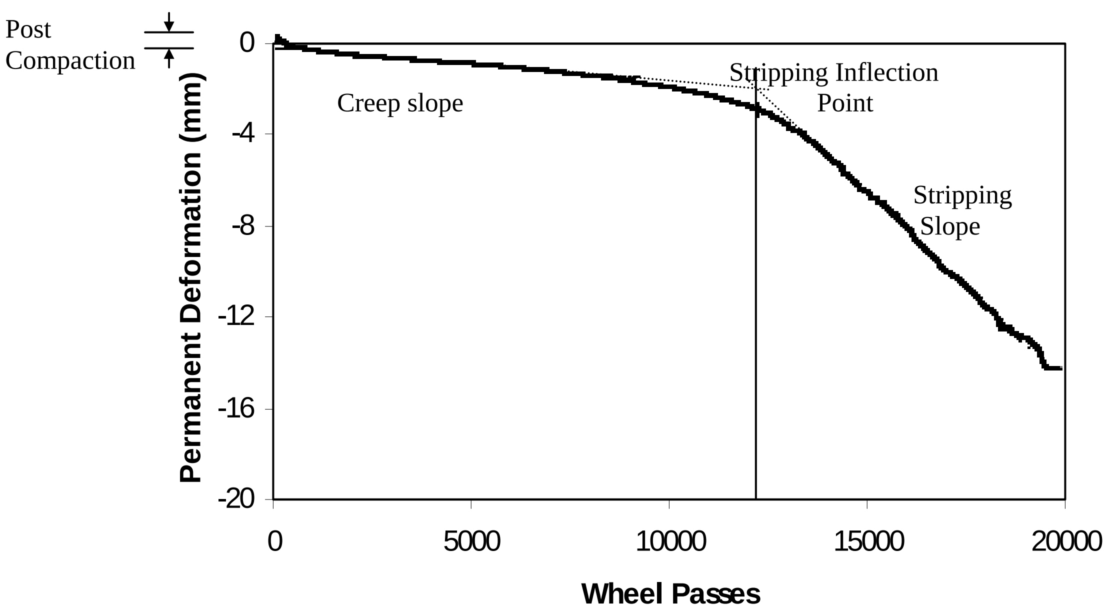

FIGURE 8 Definition of results from the HWTD.

These test results compare very well with the APA results shown in Figure 4. WesTrack Forensic Team members suggested that this test, along with the other three rut testers, should help to ensure good performance. Specific criteria for these tests can be developed when similar materials (aggregates and asphalts) are used. The use of a steel wheel further increases the severity of the test. Because a steel wheel does not deform under the test conditions like a pneumatic tire does, the effective load per unit area is much higher than that occurring during actual field loading. A mixture that survives the HWTD test should be rut resistant in the field;

however, mixtures that do not survive the test also may perform well in the field. Use of this device in mixture pass/fail situations can result in the rejection of acceptable mixtures. However, if the criteria are set correctly, this should be a reasonable test to evaluate rutting or stripping. Potential user agencies need to develop their own evaluation of test results using local conditions (*19*).

Figure 9 shows that a laboratory rut depth of 14 mm would be expected to result in a field rut depth of 12.5 mm (0.5 in.). The city of Hamburg specifies a rut depth of less than 4 mm after 20,000 passes. However, this specification has been determined to be very severe (*23*). A rut depth of less than 10 mm after 20,000 passes has been recommended to be more reasonable (*23*). This test procedure does have potential as a performance tester.

French Rutting Tester (LCPC Wheel Tracker)

The Laboratoire Central des Ponts et Chaussées (LCPC) wheel tracker (also known as the FRT), shown in Figure 10, has been used in France for more than 20 years to successfully prevent significant rutting in HMA pavements (*25*). In recent years, the FRT has been used in the United States, most notably in Colorado and FHWA's Turner–Fairbank Highway Research Center.

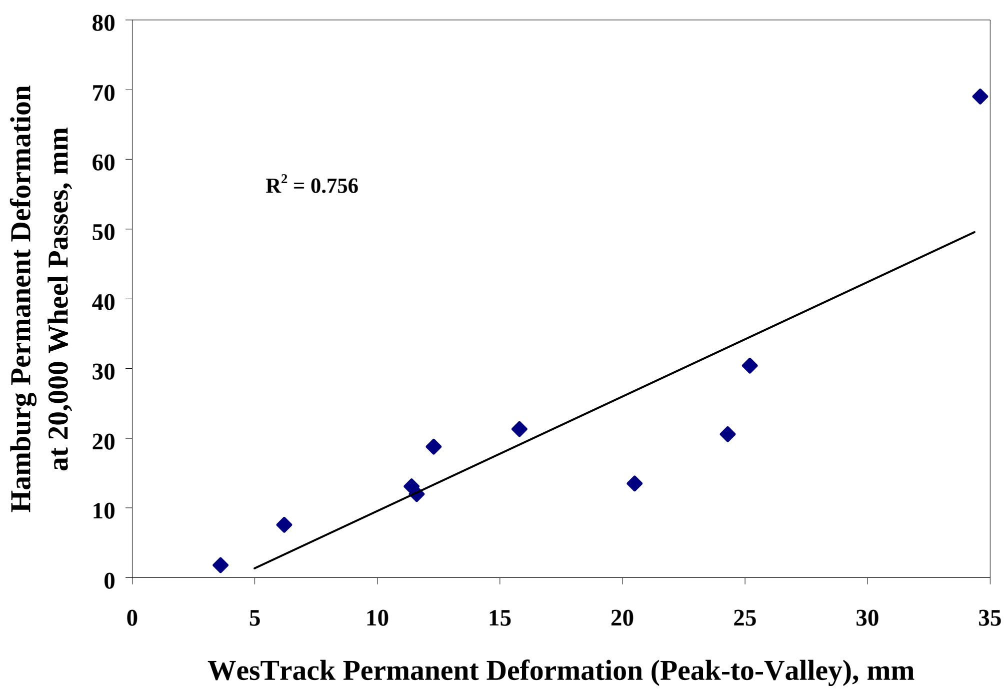

FIGURE 9 The HWTD test results versus WesTrack performance (*19*).

The FRT is capable of simultaneously testing two HMA slabs. Slab dimensions are typically 180 mm wide, 500 mm long, and 20 to 100 mm thick (7.1 in × 19.7 in × 0.8 to 3.9 in) (*26*). Samples are generally compacted with an LCPC laboratory rubber-tired compactor (*27*).

Loading of samples is accomplished by applying a 5,000-N (1,124-lb) load onto a 90-mm-wide pneumatic tire inflated to 600 kPa (87 psi). During testing, the pneumatic tire passes over the center of the sample twice per second (*27*).

Rut depths within the FRT are defined by deformation expressed as a percentage of the original slab thickness. Deformation is defined as the average rut depth from a series of 15 measurements. These measurements consist of three measurements taken across the width of a specimen at five locations along the length of the slab.

The specimen width and the closeness of the confining rigid specimen holder to the location of repeated loading distorts the development of the mixture's shear plane, especially for mixtures containing larger aggregate. As a result, poor mixtures tend to perform better than expected in the FRT (*28*), and discriminating between good and poor performing mixtures becomes difficult.

In France, an acceptable HMA mix typically will have a rutting depth that is less than or equal to 10% of the test slab thickness after 30,000 cycles. The Colorado Department of Transportation and the FHWA's Turner–Fairbank Highway Research Center participated in a study to evaluate the FRT and the actual field performance (*26*). A total of 33 pavements from throughout Colorado that showed a range of rutting performance were used. The research indicted that the French rutting specification (rut depth of less than 10% of slab thickness after 30,000 passes) was severe for some of the pavements in Colorado. No further research was found to adjust the criteria based on this study.

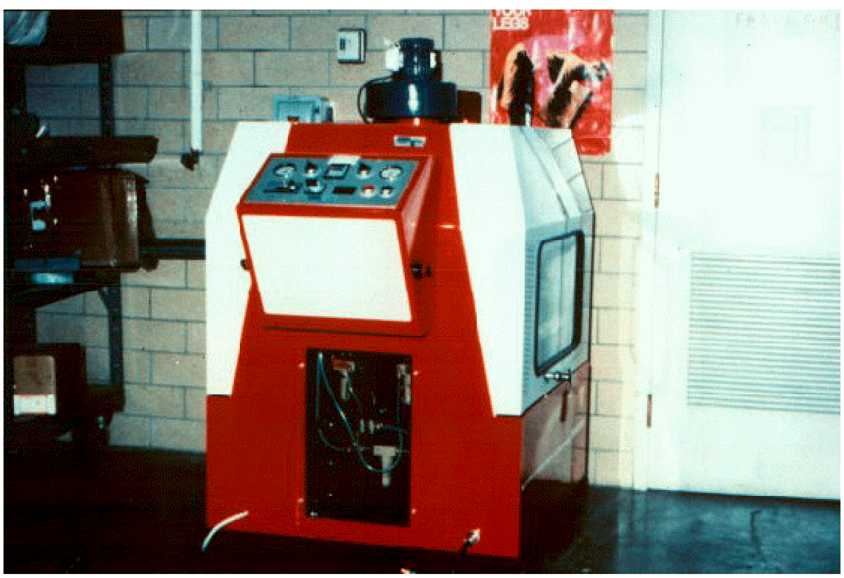

FIGURE 10 The FRT.

Another study by the LCPC compared rut depth from the FRT and field rutting (*29*). Four mixes were tested in the FRT and placed on a full-scale circular test track in the Nantes, France. Results showed that the FRT could be used as a method of determining whether a mixture will have good rutting performance. No criteria were established because of the limits of the data.

Figure 11 shows WesTrack forensic team research results on the actual performance and the predicted performance using the FRT (*19*). As the figure shows, the FRT had a correlation coefficient (R^2) of 0.694. The test results have compared favorably with the APA and the Hamburg testers (Figure 4 and 9).

WesTrack Forensic Team members suggested that the FRT provided useful data when experience is available with similar materials (aggregates and asphalts). Similar to that for the HWTD, potential FRT user agencies should develop their own evaluation of test results using local conditions (*19*). The data indicated that a laboratory rut depth of 10 mm (10% of 100 mm thickness) results in an in-place rut depth of 12.5 mm (0.5 inch). Recall the French specification and study in Colorado, a conservative criterion of 10% of the slab thickness after 30,000 cycles is appropriate for FRT tests.

RECOMMENDED PROCEDURES TO EVALUATE AND OPTIMIZE PERFORMANCE

Predicting performance of HMA is very difficult because of the complexity of HMA, the complexity of the underlying unbound layers, and the varying environmental conditions. Presently, no specific methods are being used nationally to design and control HMA to control rutting, fatigue cracking, low-temperature cracking, and friction properties. Moisture susceptibility tests are being used nationally, but these tests are not very effective. Some additional guidance is needed to minimize the occurrence of these distresses.

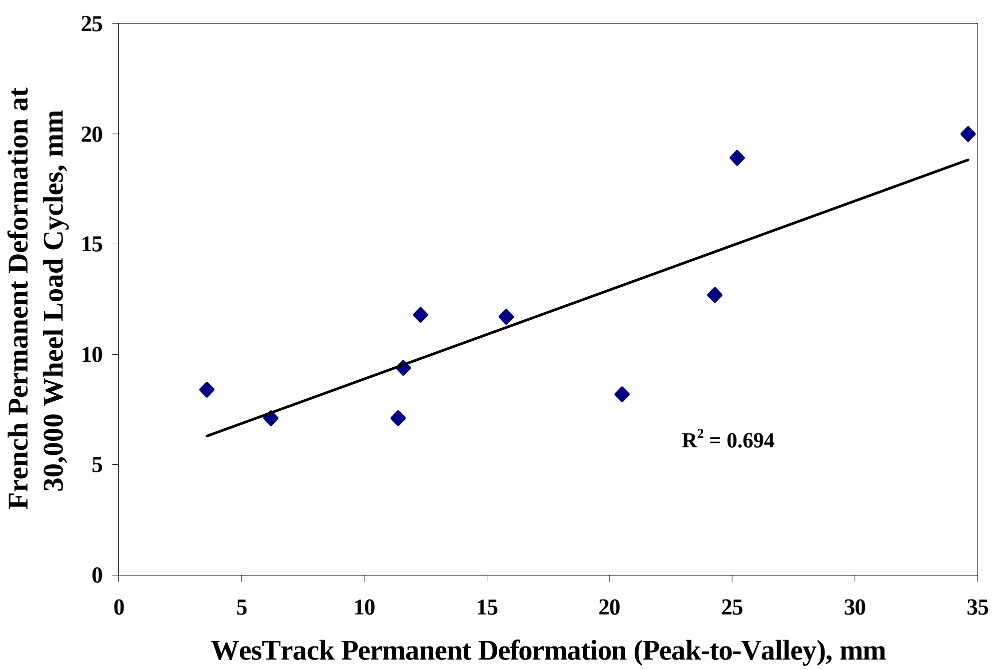

FIGURE 11 The FRT results versus WesTrack performance (*19*).

This report is not meant to be taken as a final document on performance. It is only a starting point. In fact, the recommendations in this report will continue to be evaluated along with new research findings to improve the existing recommendations. Several studies are under way, which should be completed in the near future, to develop additional tests to predict performance. When these improved tests are developed, the guidance provided in this report may be superseded with the new information. However, until better tests and methods of analysis are available, the guidance discussed here is available as a starting point to help provide some indication of performance. Specific guidance is provided only for permanent deformation. The authors believe that this guidance is the best available at this time.

Permanent Deformation

Permanent deformation is probably the most important performance property to be controlled during mix design and QA/QC. Permanent deformation problems usually show up early in the mix life and typically result in the need for major repair, whereas other distresses take much longer to develop. Several tests were considered for measuring rutting potential. Tests that appear ready for immediate adoption include the following three wheel-tracking tests: APA, HWTD, and FRT. Several factors were used to select these tests: availability of equipment, cost, test time, applicability for QC/QA, performance data, criteria, and ease of use. The tests and criteria shown in Table 2 are recommended for immediate use; however, some experience with local materials is recommended before adoption.

The tests in Table 2 are listed in order of priority for recommended use. The test results indicate that all three tests provide similar results. The priority order was selected mostly because of availability of equipment. The information shown in Table 2 is based on limited field results and specific methods of conducting the tests in the laboratory. Any change in test method will likely result in a needed change in criteria. These recommended criteria are developed in general for higher traffic; therefore, they are not necessarily applicable for lower traffic areas.

Before adopting the criteria, tests should be conducted with local materials and mixes to develop an understanding of what type of results to expect. The criteria provided are reasonable based on test results for specific mixes that have been evaluated in the past but that may need to be modified slightly on the basis of local experience. There is more experience with wheel-tracking tests than with any other type of test to predict rutting. Other tests such as creep and repeated load tests have promise, but more work is needed to finalize details before this type of test is used for mix control (research is under way to do this).

TABLE 2 Recommended Tests and Criteria for Permanent Deformation

Performance Tests		Recommended Criteria		Test Temperatures
1st choice	APA	8 mm	@ 8,000 wheel load cycles	High temperature for selecting PG grade
2nd choice	HWTD	10 mm	@ 20,000 wheel passes	50°C
3rd choice	FRT	10 mm	@ 30,000 wheel load cycles	60°C

One recommended approach is to use the APA with cylinders compacted in the Superpave gyratory compactor. Samples compacted for volumetric testing could be tested, thus minimizing overall number of samples required. This will allow QC/QA tests to be conducted quickly without requiring additional compacted specimens. Related information on the recommended performance tests for permanent deformation is provided in appendices A, B, and C of NCAT Report 2001-05 (*10*).

Fatigue Cracking

There has been much research done on the effects of HMA properties on fatigue. Certainly the HMA properties have an effect on fatigue, but the most important factor to help control fatigue is to ensure that the pavement is structurally sound. Because the classical bottom-up fatigue is controlled primarily by the pavement structure, there is no way that a mix test can be used alone to accurately predict fatigue. However steps can be taken to minimize fatigue problems. Some of these steps include the following: (*a*) use as much asphalt in the mix as allowable without rutting problems, (*b*) select the proper grade of asphalt, (*c*) do not overheat the asphalt during construction, (*d*) keep the filler-to-asphalt ratio lower, (*e*) compact the mix to a relatively low void level, and so forth. This is general guidance but this is the approach that is typically used to ensure good fatigue resistance. A more definitive way to control fatigue is needed but is not currently available.

Thermal Cracking

Thermal cracking is a problem in colder climates, and guidance is needed to minimize this problem. Currently, the best guidance to minimize thermal cracking is to select the proper low-temperature grade of the PG asphalt binder for the project location. Other steps during construction can be helpful. For example, do not overheat the asphalt. This will result in stiffening of the binder and will therefore increase the potential for thermal cracking. It is also important to compact the HMA to a relatively low air void level to minimize any future oxidation. At this time, no specific test is recommended for thermal cracking, but in the future, better guidance should be available.

Moisture Susceptibility

Moisture susceptibility is typically a problem that can cause the asphalt binder to strip from the aggregate, leading to raveling and disintegration of the mixture. AASHTO T-283 has been used for several years to help control stripping. This test does not appear to be a very accurate indicator of stripping, but it does help to minimize the problem. The Hamburg test also has been shown to identify mixes that tend to strip.

Some things during the construction process can help to minimize stripping potential. Of course, liquid and lime antistrip agents can be used. Other items include good compaction and complete drying of aggregate. Currently, there is nothing better than AASHTO T-283 to recommend.

Friction Properties

Friction is one of the most important properties of an HMA mixture. There are good methods to measure the in-place friction, but there are not good methods to evaluate mixes in the laboratory for friction. Several state DOTs have methods that they use but these methods have not been adopted nationally. More work is needed to evaluate these local procedures for national adoption.

There are several things that can be done in design and construction to improve friction. The primary concern is friction during wet weather. Use of a mix such as open-graded friction course has been shown to be effective in increasing friction in wet weather. Other methods include using aggregate that does not tend to polish, using mixes that are not over-asphalted, using crushed aggregates, and such. Coarse textured mixes such as stone matrix asphalt have been shown to provide good friction in wet weather. Currently, past experience with local materials is the best information available for providing good friction.

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Author Contact List

Ghazi G. Al-Khateeb
Federal Highway Administration
Turner–Fairbank Highway Research Center
6300 Georgetown Pike, HRDI-11
McLean, VA 22101
Ghazi.Al-Khateeb@fhwa.dot.gov

David A. Anderson
Pennsylvania State University
201 Transportation Research Building
Penn Transportation Institute
University Park, PA 16802-4710
814-863-1912; fax: 814-865-3039
daa@psu.edu

Ramon Bonaquist
Advanced Asphalt Technologies, LLC
108 Powers Court, Suite 100
Sterling, VA 20166
703-444-4200; fax: 703-444-4368
aatt@erols.com

E. Ray Brown
Auburn University
National Center for Asphalt Technology
277 Technology Parkway
Auburn, AL 36832
334-844-6228; fax: 334-844-6248
rbrown@eng.auburn.edu

William G. Buttlar
University of Illinois at Urbana–Champaign
Room 1212
205 North Mathews Avenue MC-250
Urbana, IL 61801-2352
217-333-5966; fax: 217-333-1924
buttlar@uiuc.edu

Donald W. Christensen
Advanced Asphalt Technologies
444 East College Avenue, Suite 510
State College, PA 16801
814-278-1991; fax: 814-278-1991
dwcaat@aol.com

Stacy Glidden
MTE Services, Inc
915 Commercial Court
Onalaska, WI 54650
608-779-6304; fax: 608-781-4694

Salil Gokhale
Applied Research Associates, Inc.
ERES Consultants Division
State Materials Office
Materials Research Park
5007 NE 39th Avenue
Gainesville, FL 32609

Prithvi S. Kandhal
National Center for Asphalt Technology
277 Technology Parkway
Auburn, AL 36832
334-844-6228; fax: 334-844-6248
pkandhal@eng.auburn.edu

Gerald H. Reinke
MTE Services, Inc.
915 Commercial Court
Onalaska, WI 54650
608-779-6304; fax: 608-781-4694
greinke@mathy.com

Daniel S. Sherman
University of Illinois at Urbana–Champaign
205 North Mathews Avenue, MC-250
3214 Newmark Civil Engineering Lab
Urbana, IL 61801
217-766-3794; fax: 217-333-1924
dsherman@uiuc.edu

Jingna Zhang
National Center for Asphalt Technology
277 Technology Parkway
Auburn, AL 36832
334-844-6228; fax: 334-844-6248
zhangj1@eng.auburn.edu

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