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16. Abstract

The Strategic Highway Research Program (SHRP) study, during the development of the Superpave mixture design and analysis process, provided a series of test protocols using the Superpave shear tester (SST) to predict the performance of the hot mix asphalt (HMA) mixtures. Excessive time is required to conduct all of these test procedures on each HMA mixture designed, which can significantly increase the cost of mixture design. Conducting all of the tests can be confusing and even conflicting. The objective of this study is to evaluate four selected Superpave shear test protocols and determine which of the test protocols is most suitable for predicting asphalt pavement performance. The predominant pavement performance of interest herein is rutting. So, the ultimate goal is to identify the "best" SST test protocol that can evaluate the shearing resistance of HMA.

Researchers selected or developed four different mixtures from very poor to excellent quality using materials from Texas and Georgia. The HMA mixtures selected for this study were: Type C limestone, Type D rounded river gravel, granite stone mastic asphalt (SMA), and granite Superpave. Rutting performance of these mixtures was evaluated using four SST protocols: Frequency Sweep at Constant Height (FSCH), Simple Shear at Constant Height (SSCH), Repeated Shear at Constant Height (RSCH), and Repeated Shear at Constant Stress Ratio (RSCSR). Three laboratory-scale accelerated loaded wheel tests were performed on these four mixtures to compare the results with those from the SST. The loaded wheel tests used in this study were: Asphalt Pavement Analyzer, 1/3-Scale Model Mobile Load Simulator, and Hamburg Wheel Tracking Device.

Researchers recommended the FSCH test as the "best" SST protocol. To determine the precision of the FSCH test, compacted specimens from three HMA mixtures were sent to six different laboratories across the US to conduct an interlaboratory test program.

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EVALUATION OF SUPERPAVE SHEAR TEST PROTOCOLS

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CHAPTER 1:
INTRODUCTION

BACKGROUND

Permanent deformation or rutting is a major issue for hot mix asphalt (HMA) pavements throughout the nation. Texas is no exception. Under a recent National Cooperative Highway Research Program (NCHRP) study (*1*), researchers conducted a national survey and found that rutting was overwhelmingly the most important HMA pavement distress. The main contributors to increased rutting observed in the last decade appear to be higher truck tire pressures and axle loads and increased truck traffic (*2*). Truck tire pressure has been increased from 70-80 psi to 120-140 psi in the past 20 years. As a result, the top pavement layer (HMA) is subjected to more and higher stresses and is thus more susceptible to rutting. A 1987 study (*3*) suggests that truck tire pressure will continue to increase. Therefore, the best technology available must be used to design and construct HMA pavements to ensure they are capable of carrying this increasing traffic load.

Approximately 94 percent of paved roads in Texas are asphalt pavements. Each year, rehabilitation of these existing roads and construction of new roads require about 12 million tons of HMA. District pavement engineers, area engineers, and laboratory supervisors are constantly faced with decisions regarding selection of the best asphalt mixture design to use in construction or rehabilitation of particular pavement. Texas Department of Transportation (TxDOT) and contractors need to have a method available that is capable of verifying that the selected mixture design is not likely to exhibit premature distress during service under the anticipated traffic loading, temperature regime, and pavement substrate. Rutting, the major form of premature distress, is caused mainly by insufficient shearing strength of HMA (*4*). Therefore, a laboratory test method that can verify a mixture's shearing strength and, hence, the rutting resistance would be extremely valuable to not only TxDOT but also to all highway specifying agencies and contractors who are required to warranty pavement performance.

PROBLEM STATEMENT

During the early 1990s under the Strategic Highway Research Program (SHRP) study, development of the Superpave mixture design and analysis process provided a series of test protocols with various test conditions. Superpave volumetric mixture design alone does not provide adequate warranty against pavement distresses like permanent deformation and or fatigue cracking. To ensure better resistance to different kinds of distresses, SHRP introduced the intermediate (Level II) and complete (Level III) mixture design and analysis system, which depend on traffic level. The Superpave Shear Tester (SST) was introduced as a component of the Superpave mixture design and analysis system to perform all load-related performance tests. This testing device is capable of using both static and dynamic loading in confined and unconfined conditions. Initially, SHRP researchers proposed six different SST test protocols to characterize HMA. The six different tests were as follows:

- Volumetric Test,
- Uniaxial Test,
- Frequency Sweep at Constant Height (FSCH) Test,
- Simple Shear at Constant Height (SSCH) Test,
- Repeated Shear at Constant Height (RSCH) Test, and
- Repeated Shear at Constant Stress Ratio (RSCSR) Test.

These six test procedures measure several engineering properties. Due to lack of time, SHRP researchers could not determine which of the test protocol/protocols were best suited for predicting the rutting performance of HMA. Excessive time is required to conduct all of these test procedures on each asphalt mixture designed. American Association of State Highway and Transportation Officials (AASHTO) Standard TP 7-94 (*5*) calculates the time required for level III and Level II test protocols as 111 and 58 hours, respectively. Level III and Level II require 21 and 12 HMA specimens, respectively, to conduct the tests with the SST. Excessive time required to perform these tests can significantly increase the cost of mixture design. Conducting all of the tests can be confusing and even conflicting.

It is desirable to evaluate these Superpave shear test protocols in a systematic experimental program to determine which engineering property is most suitable to for predicting the shearing strength or rutting susceptibility of HMA.

A recent survey under NCHRP Project 9-19 shows that HMA industries prefer a relatively simple and low-cost equipment to characterize the HMA (*1*). When this study began, test requirements for the SST were complex and expensive. The most readily apparent methods for simplifying the SST equipment appear to involve elimination of the confining pressure. If the SST protocols requiring confining pressure (volumetric test and uniaxial test) prove to correlate more poorly with pavement performance than the other SST protocols, then the large compressor and air seals could be eliminated and the strength requirements of the chamber could be greatly reduced. This is one way to simplify and lower the cost of the equipment. After this study began, AASHTO eliminated the volumetric and uniaxial tests from the SST protocols.

OBJECTIVE

The objective of this study is to evaluate the four remaining Superpave shear test protocols developed by SHRP researchers to determine which of the test protocols is most suitable for predicting asphalt pavement performance. The predominant pavement performance of interest is rutting. So, the ultimate goal will be to identify the "best" suited SST test protocol that can evaluate the shearing resistance of hot mix asphalt.

The secondary objective is to simplify the SST equipment. In fact, if the identified "best" test protocol does not involve confining pressure, the exclusion of the confining chamber will automatically simplify and reduce the cost of the equipment. Other objectives include developing acceptance criteria and a precision statement for the "best" SST protocol.

SCOPE OF REPORT

This report is divided into six chapters. Chapter 1 serves as an introduction, stating the nature of the problem to be addressed, objectives of the research, and scope of work accomplished.

Chapter 2 summarizes an overview of HMA mixture evaluation techniques with an emphasis on the permanent deformation. It covers the different test methods used in last decades to identify the rutting resistance of HMA. This literature review describes mechanistic, empirical, and accelerated pavement testing. This chapter provides information on HMA shearing strength evaluation procedures and their relationship to pavement performance.

Chapter 3 is a description of the experimental program. The work plan includes the following tasks: planning of study, materials selection and acquisition, testing to characterize asphalt cement and aggregates, mixture design and/or calibration, and testing to evaluate asphalt concrete mixtures. HMA evaluation tests include tests with the SST, Asphalt Pavement Analyzer (APA), 1/3-Scale Model Mobile Load Simulator (MMLS3), and Hamburg Wheel Tracking Device (HWTD).

Chapter 4 covers analysis of the results from different tests that have been conducted to evaluate the shearing resistance of different HMA mixtures. This chapter includes the results and analysis of SST tests and other laboratory-scale rutting tests. This chapter includes comparative analysis of results from SST and other laboratory-scale rutting tests.

Chapter 5 presents results of the interlaboratory study of the Frequency Sweep at Constant Height test, which was found to be the "best" SST test protocol. FSCH tests were conducted at four regional Superpave Centers, Asphalt Institute (AI), and Federal Highway Administration (FHWA). This chapter presents the precision statement for the "best" test protocol determined on the basis of the interlaboratory test study.

Chapter 6 presents conclusions and recommendations that arose from the study. Detailed results of some tests are discussed in the appendix.

CHAPTER 2:
LITERATURE REVIEW

GENERAL

HMA material characterization was introduced in the early twentieth century. Many test methods were developed during last century. Some of them are empirical test methods and correlated poorly with field performance. Many of the test methods have become obsolete. In this study, researchers concentrated on the test methods used for characterizing the shearing properties of HMA. Currently, the Superpave volumetric mixture design procedure lacks a basic design criterion to evaluate fundamental engineering properties of the asphalt mixture that directly affects performance (*1*).

Hveem Stability Testing

In late 1920s, Francis Hveem of the then California Highway Department developed the Hveem stabilometer. The purpose of this testing was to measure the stability of highway materials under various states of confinement (*6*). This brilliant test was developed as an empirical measure of internal friction within a mixture (*7*). Later, this stability test for asphalt mixture was standardized in American Society for Testing and Materials (ASTM) D 1560 and AASHTO T 246.

The Hveem stability tester applies a vertical axial load on a 4-inch (102 mm) diameter by 2.5-inch (64 mm) high HMA specimen in a confined stress condition using a rubber membrane. It measures the resulting horizontal pressure and displacement at an applied vertical pressure of 400 psi (2760 kPa) at 140°F (60°C). This temperature is designed to simulate the most critical yet typical filed condition. Hveem stability of asphalt concrete is calculated using the following equation:

$$S = \frac{22.2}{\dfrac{P_{\mathrm{h}} D}{P_{\mathrm{v}} - P_{\mathrm{h}}} + 0.222},$$

where, S = Hveem stability number of asphalt concrete,

D = displacement of the specimen,

P_v = vertical pressure applied (400 psi), and

P_h = horizontal pressure gauge reading in psi.

Different agencies slightly modified the testing procedure and the equation. The stability value of fluid will be near zero as $P_v \cong P_h$. Whereas, the stability value of steel will be near 100 as P_h approaches zero.

Researchers also developed a companion test using the cohesiometer to measure cohesion or the tensile characteristics of HMA. The cohesiometer was rarely used, as the testing error is very high and there is little correlation between the test result and actual performance. Although widely used for many years, Hveem stability testing itself is not highly correlated with the field performance.

Marshall Stability and Flow Testing

Bruce Marshall developed the Marshall asphalt concrete mixture design while working for the Mississippi Highway Department in the late 1930s. Later, the U.S. Army Corps of Engineers modified the Marshall mixture design system. As part of the mixture design system, strength of the asphalt mixture is determined using the Marshall stability and flow tester to determine optimum asphalt content. It has also been used for quality control. This test, to determine the relative potential of a mixture to exhibit instability, was later standardized as ASTM D 1559 and AASHTO T 245.

The Marshall test apparatus applies a vertical compressive load on the cylindrical surface of a specimen using two semicircular testing heads. The specimen is 4 inches (102 mm) in diameter by 2.5 inches (63.5 mm) high and is compacted in a specified manner. The test is conducted at 140°F (60°C) and rate of loading is 2 inches (50 mm) per minute. This temperature is a critical yet practical field condition. This apparatus provides two materials indicators: stability and flow.

Marshall stability is the maximum load sustained by the specimen before failure. The Marshall flow value is the total vertical deformation (in 0.01 inches [0.025 mm]) of the specimen at maximum load. Higher stability values indicate stronger mixtures. Sometimes another property called Marshall stiffness index (ratio between Marshall stability and Marshall flow number) is used to characterize mixtures.

It is believed that a mixture with higher a stability number or higher Marshall stiffness index will be resistant to permanent deformation or rutting. There is very little correlation data between Marshall stability number and actual field performance of HMA mixtures (*7*).

Superpave Shear Tester

To improve the performance, durability, and safety of United States roads, Congress established SHRP in 1987 as a 5-year research program. Fifty million dollars of the one hundred and fifty million dollars of the SHRP research funds were used for the development of asphalt specifications to directly relate laboratory analysis with field performance. Superpave™ was the final product of the SHRP research effort. Superpave is a complete mixture design and analysis system with three major components: asphalt binder specification, mixture design methodology, and analysis system.

Under the mixture design and analysis part of the SHRP research, researchers developed two devices to quantify the performance of an HMA mixture. They are the Superpave Shear Tester and the Indirect Tensile Tester. Initially, researchers proposed six test protocols using the SST to characterize the permanent deformation and fatigue resistance of HMA mixtures. Permanent deformation and fatigue are both load-related distresses. In this study, researchers will concentrate on the permanent deformation characterization only.

During the SHRP study, several universities were included in a team formed to select test methods to characterize the permanent deformation property of HMA mixtures (*8*). This team developed the SST machine. This machine measures some basic material properties responsible for permanent deformation: nonlinear elastic property, Vermeer plastic property, viscoelastic property, and tertiary creep property (*9*). The SST can provide material constitutive relations necessary for mechanistic road response models (*6*). The original intent of many of these tests was that they would be used as input into performance models developed during SHRP (*10*). Despite successful application in the research field, most of the tests, material properties, and theoretical models are not currently in common use by the industry. The six SST test protocols to measure the permanent deformation characteristics of HMA mixtures are presented below. Chapter 3 discusses further details of these test protocols.

Uniaxial, Volumetric, Simple Shear Test

These three test methods measure the nonlinear elastic property and Vermeer plastic property. Only one load cycle (static) is applied in these tests. Basically, they are load-unload tests. All three tests involve loading a test specimen at a specified controlled stress magnitude, holding the load for a specified time, and unloading the test specimen at a specified rate (*9*). Uniaxial and volumetric tests use confining pressure, while the simple shear test does not. The loadings of the specimens are designed to imitate field conditions.

During the loading and unloading process, the specimens are subjected to elastic and plastic strain. From the elastic part of the stress-strain graph, two major material properties are calculated: elastic modulus and Poisson's ratio. The plastic part provides a volumetric constant, peak angle of friction, factor related to the cohesive shear strength, and friction angle at constant volume. All of these materials properties are predicators of rutting of HMA mixtures.

Frequency Sweep Test

Frequency sweep is a strain-controlled repeated test that is used to measure the viscoelastic behavior of asphalt mixtures. A small magnitude of sinusoidal shearing strain is applied on the specimen at 10 different frequencies, and the stress response is measured. Due to the viscoelastic behavior of an HMA mixture, the specimen's stress response is not in the same phase as the applied strain. The stress is always lagging behind the applied strain. The ratio between the stress response and the applied strain is used to compute the complex shear modulus. The measured time delay between the strain and stress response is used to compute shear phase angle.

Higher complex modulus indicates a stiffer mix that is more resistant to rutting. Lower shear phase angle indicates more elastic behavior that is more resistant to rutting.

Repeated Shear Test

There are two types of repeated shear tests: constant height and constant stress. The objective of developing these tests was to find a check for an HMA mixture's susceptibility to tertiary creep. Tertiary creep is a severe form of rutting where a small number of load repetition can cause a large amount of plastic deformation. Tertiary creep indicates gross instability of the mixture.

A large number of repeated loads is applied in both cases, and the shearing deformation is measured. In a constant stress test, repeated synchronized haversine shear and axial load pulses are applied to the specimen. Each load pulse is followed by a rest period. The ratio of haversine axial load to shear load is maintained at a constant ratio within the range of 1.2 to 1.5. The repeated shear load shear at constant stress ratio was included in the Superpave method as a screening test to identify mixtures that exhibit tertiary plastic flow, indicating instability and leading to premature rutting (*11*).

In the constant height test, a haversine shear load of a specified magnitude is applied on the specimen, and a variable axial load is applied to keep the specimen height constant. Each load pulse is followed by a rest period.

Evaluation of Superpave Shear Tester

Romero and Mogawer (*12*) conducted a research study to determine if the results from the SST could be used to differentiate the properties of five laboratory-prepared asphalt mixtures without the need for any models. They analyzed the properties obtained from the SST and compared them with the performance of the respective mixtures tested at the Accelerated Loading Facility (ALF) at FHWA. They examined the FSCH, SSCH, and RSCH tests. Some of their conclusions were: the trends observed in most SST tests were consistent with the ALF performance, the complex shear modulus at 104°F (40°C) obtained from the FSCH test was able to discern good and bad mixtures, RSCH test results were extremely variable, shear modulus ranking matched with ranking from the ALF, and elastic strain from the SSCH test was not able to discern among the mixtures.

Tayebali et al. (*13*) conducted a study to evaluate the performance of the RSCH test. They examined three field mixtures and tested using the RSCH test, Georgia Loaded Wheel Tester (GWLT), and French Rut Tester (FRT). When comparing with field rutting performance (after several years of service), the researchers found that the RSCH test can clearly identify the well-performing versus poor-performing mixtures.

Stuart and Izzo (*14*) compared the $G^*/\sin\delta$ of five binders (note: not mixtures) tested using the dynamic shear rheometer (DSR) with the results from HMA mixtures tested using the GLWT, HWTD, and FRT. They observed that $G^*/\sin\delta$ correlates very well with results from the GLWT and reasonably correlates with the results from the HWTD and FRT.

Anderson et al. (*10*) documented the SST performance of various asphalt mixtures tested at Asphalt Institute since 1994. The objective of that study was to create a database to provide guidance to the users indicating how a project's asphalt mixture compares to other mixtures with performance history.

Anderson et al. (*15*) presented a case history on Specific Pavement Study No. 9 (SPS-9). To showcase the Superpave system, four SPS-9 pilot projects were built in the summer of 1992. Reviewing the field performance of those sections after 7 years of service and comparing the laboratory test results on those mixtures, the authors concluded that two SST tests, SSCH and FSCH, correctly ranked the observed pavement rutting performance.

Shenoy and Romero (*16*) suggested a procedure to unify the sets of curves generated from the FSCH tests performed at various temperatures on different mixtures. The procedure involves the use of a normalizing frequency parameter. They also proposed that the temperature at which the normalizing parameter becomes equal to 1.0 could be considered as a specification parameter for assessing mixture performance. Researchers determined the specification parameter for various mixtures of known performance and found it to follow the performance rankings in all studied cases.

Unconfined Compressive Strength Test

The unconfined compressive strength test procedure to evaluate the strength of asphalt concrete has been widely used by the pavement industry (*6*). This test is a standardized test in ASTM D 1074 and AASHTO T 167. In this test, a fixed or monotonic axial load is applied on a cylindrical specimen at a steady rate until the specimen fails. The peak load divided by the cross-sectional area is referred to as the unconfined compressive strength of the mixture. The specimen size is 4 inches (100 mm) in diameter and 8 inches (200 mm) in height. There is little evidence that the field rutting performance correlates with the unconfined compressive strength.

Uniaxial Creep Test

The uniaxial creep test has been used for many years to estimate the rutting potential of HMA mixtures. The creep test may involve either static or repeated haversine load. In both cases, an axial load is applied on the cylindrical specimen and the resulting permanent deformation with time is measured. The total plastic strain induced during a creep test is recorded and plotted as function of the logarithm time or load applications. Typical creep behavior of asphalt concrete is composed of (1) primary densification, (2) steady-state creep, and (3) a tertiary creep phase. Using the creep test data in the VESYS software, one can predict the rut depth progression of a pavement by assuming permanent vertical strain is directly proportional to the resilient strain (*6*).

Uniaxial creep test data can be used to evaluate the permanent deformation potential of asphalt concrete mixtures when the laboratory creep testing is performed in such a manner as to simulate realistic field stress conditions (*17*).

Resilient Modulus Test

Resilient modulus is one of the most common methods of measuring HMA mixture stiffness. In this test procedure, a repeated haversine axial load is applied on the cylindrical surface of a specimen (similar to the indirect tension test). The specimen is not loaded to failure; rather, it is loaded to a stress level between 5 and 20 percent of normal strength. The standard test procedure can be obtained from ASTM D 4123. There is no good correlation between modulus of resilience and rutting (*7*). This is no surprise, since by definition, the test does not produce shear strain in the specimen. Results typically correlate strongly with binder characteristics but not with aggregate characteristics.

Dynamic Complex Modulus Test

Dynamic complex modulus of an HMA mixture is determined by applying sinusoidal compressive stress along the axis of a cylindrical specimen. The standardized test procedure is presented in ASTM D 3497. Researchers developed this test during the 1960s. According to the ASTM procedure, the height-to-diameter ratio should be 2:1 to sufficiently minimize the effect of friction and resulting shearing stress at the top and bottom of the specimen. The most common specimen size is 4 inches (100 mm) × 8 inches (200 mm). The applied load usually ranges up to 35 psi (241.5 kPa). Tests are usually conducted usually at three different temperatures and three different frequencies. Typical temperatures are 41, 77, and 104°F (5, 25, and 40°C), and the loading frequencies are 1, 4, and 16 Hz.

The dynamic complex modulus is calculated by dividing the repeated vertical stress by the resulting repeated axial strain. The primary purpose of measuring dynamic modulus was to determine the stress-strain relationship. This test has limited use because of long testing time, complexity and cost of equipment, and large specimen size (*7*).

NCHRP Project 9-19

During the late 1990s, FHWA established NCHRP Project 9-19. The objectives of the NCHRP project are to "(1) develop simple performance tests for permanent deformation and fatigue cracking for incorporation in the Superpave volumetric mix design method, and (2) develop and validate an advanced material characterization model and the associated calibration and testing procedures for hot mix asphalt used in highway pavements" (*18*). Under this project, Dr. Matt Witczak and his co-workers proposed "some simple performance tests" to evaluate the permanent deformation of HMA mixtures. They are as follows.

Dynamic Modulus

The dynamic modulus test procedure is similar to ASTM D 3497 with some modifications. In the proposed test method, dynamic modulus (E*) and phase angle (N) are measured from the sinusoidal axial load application on a cylindrical Superpave Gyratory Compactor (SGC) specimen at a single temperature (T_{eff} = 77 to 140°F [25 to 40°C]) and design loading frequency (0.1 to 10 Hz). Here, the complex modulus is calculated by dividing the stress by the axial strain. The phase angle is the angle lagging by the axial strain from the axial stress. The concept is similar to the FSCH test by SST equipment. The specimen used for this test is 4 inches (100 mm) in diameter and 6 inches (150 mm) in height. Witczak, et al. (*1*) reported excellent correlations between $|E^*|/\sin\phi$ and rutting performance from certain test tracks and the FHWA-ALF.

Flow Number

Flow number is the number of load repetitions at which shear deformation begins under constant volume. In this test protocol, the SGC compacted cylindrical specimen is subjected to repetitive axial load in a triaxial environment at a single temperature (T_{eff} = 77 to 140°F [25 to 40°C]). The load is applied for a duration of 0.1 s, followed by a rest period of 0.9 s. Usually, a 10-30 psi (69-207 kPa) stress is applied and the cumulative permanent axial and radial strains are recorded throughout the test. The specimen dimension is the same as that of the dynamic modulus test. Witczak et al. (*1*) reported from good to excellent correlations between flow number and rutting performance from certain test tracks and the FHWA-ALF.

Flow Time

Flow time is defined as the postulated time when shear deformation starts under constant volume. In this test protocol, the SGC compacted cylindrical specimen is subjected to static axial load in a triaxial environment at a single temperature (T_{eff} = 77 to 140°F [25 to 40°C]). The applied stress and the resulting permanent axial and radial strains are recorded throughout the test to calculate the flow time. Witczak et al. (*1*) reported from good to excellent correlations between flow time and rutting performance from certain test tracks and the FHWA-ALF.

Full-Scale Test Track

Several different test tracks have been constructed since the 1960s to evaluate a wide variety of pavement parameters. Some test tracks have been built to serve specific purposes. Some of the test tracks constructed and tested so far include:

- AASHO Road Test
- University of Illinois Test Track
- MnRoad
- WesTrack
- NCAT Test Track

These test tracks are also capable of mixture material characterization. Usually, this type of test pavement is constructed with several different test sections. Full-scale loaded trucks with/without drivers are operated continuously on the test pavements, and the distresses are measured at regular intervals. A huge number of load repetitions is required to simulate field conditions. This type of mixture characterization is probably the best method to simulate field conditions. The two major problems with this method are, of course, cost and time. That is why test tracks are limited only for major research purposes.

LABORATORY-SCALE ACCELERATED TESTS

For the last two decades, the use of laboratory-scale wheel testers to estimate the rutting potential of HMA mixture has become more popular. Most of the wheel testers estimate rutting susceptibility of asphalt mixtures by applying repeated wheel passes in a comparatively short period and usually employ an elevated temperature to accelerate the damage. Many transportation agencies and pavement industrial firms have begun using loaded wheel testers (LWT) to supplement their mixture design procedure (*19*). Several studies mention the use of loaded-wheel testers (*19*, *20*, *21*, *22*, *23*, *24*, *25*).

The LWTs provide an accelerated evaluation of rutting potential in their HMA mixtures. LWTs enable asphalt mixtures and pavement structures to be evaluated in a fraction of time required for normal trafficking. The accelerated pavement tester (APT) can be full-scale or scaled-down to some degree. According to Metcalf (*26*), the full-scale APT should have controlled application of a prototype wheel loading, at or above the appropriate legal load limit, to a prototype or actual layered, structural pavement system to determine pavement response and performance under a controlled, accelerated accumulation of damage in a compressed time period. This acceleration of damage can be achieved by means of increased repetitions, modified loading condition, and imposed climatic conditions, or a combination of those factors. The overall idea is to simulate, as closely as possible, a real-life situation.

Contrary to full-scale APT, model or laboratory APT does not attempt to model real-world conditions but rather manipulates these conditions directly and/or artificially to evaluate the critical performance parameters of materials and structures in an accelerated time frame. Controlling variables such as pavement temperatures, base stiffness, aging influence, moisture condition, loading conditions, and fundamental failure mechanisms may be induced in a fraction of time and cost compared to full-scale testing under real-world conditions. Following is a short description of several loaded-wheel testers used in the USA.

Georgia Loaded-Wheel Tester

Georgia Department of Transportation and Georgia Institute of Technology jointly developed the GLWT device in the mid-1980s (*19*, *27*). This machine could be mentioned as a pioneer of laboratory-scale loaded wheel testers in the USA. Rut testing of HMA specimens is accomplished by applying a 100-lb (445 N) aluminum wheel load onto a pneumatic hose pressurized to 100 psi (690 kPa). This pressurized hose applies a load directly on the specimens. Usually 8000 cycles of repeated (forward and backward) wheel loads are applied to the specimens. The device will accept either cylindrical or beam specimens. Rolling wheel compactors, vibratory compactors, or the Superpave gyratory compactor can accomplish compaction of specimens. Field cores or slab specimens can also be used. Typically, the air void contents of the test specimens are 4 percent or 7 percent. Test temperature of the GLWT ranges from 95 to 140°F (35 to 60°C).

Several studies showed the GLWT is capable of ranking the mixtures similar to the field rutting performance (*19*, *28*, *29*).

Asphalt Pavement Analyzer

The APA is basically a modified and improved version of the GLWT. Operation of the APA is similar to that of the GLWT. By far, the APA is the most popular and commonly used loaded wheel tester in the USA. Pavement Technology, Inc. started manufacturing this equipment from the mid-1990s. The APA is capable of evaluating rutting, fatigue, and moisture resistance of HMA mixtures. The fatigue test is performed on beam specimens supported on the two ends. Rutting and moisture-induced damage evaluations can be performed on either cylindrical or beam specimens. This machine is capable of testing in both dry and wet conditions.

Oscillating beveled aluminum wheels apply a repetitive load through high-pressure hoses to generate the desired contact pressure. The loaded wheel oscillates back and forth over the hose. While the wheel moves in the forward and backward directions, the linear variable differential transducers (LVDTs) connected to the wheels measure the depression at regularly specified intervals. Usually, three replicates of specimens are tested in this machine. Rut evaluation is typically performed by applying 8000 load cycles. The wheel load is usually 100 lb (445 N), and the hose pressure is 100 psi (690 kPa). Some researchers have successfully used this device with higher wheel load and contact pressure (*30*). APA testing can be performed using chamber temperatures ranging 41 to 160°F (5 to 71°C) (*31*).

Several research projects have been conducted to evaluate performance of the APA. Choubane et al. (*24*) indicated that the APA might be an effective tool to rank asphalt mixtures in terms of their respective rut performance. Kandhal et al. (*25*) reported that the APA has the ability to predict relative rutting potential of HMA mixtures. They also mentioned that the APA is sensitive to asphalt binder and aggregate gradation. Uzarowski and Emery (*32*) found good correlations between the rutting resistance predicted by the APA and actual field performance of asphalt concrete pavements.

Hamburg Wheel Tracking Device

The Hamburg wheel tracking device (HWTD) is an accelerated wheel tester. Helmut-Wind, Inc. in Hamburg, Germany, originally developed this device (*20*). It has been used as a specification requirement for some of the most traveled roadways in Germany to evaluate rutting and stripping (*19*). Use of this device in the USA began during the 1990s. Several agencies undertook research efforts to evaluate the performance of the HWTD. The Colorado Department of Transportation (CDOT), FHWA, National Center for Asphalt Technology (NCAT), and TxDOT are among them.

Since the adoption of the original HWTD, significant changes have been made to this equipment. A U.S. manufacturer now builds a slightly different device. The basic idea is to operate a steel wheel on a submerged, compacted HMA slab or cylindrical specimen. The original HWTD uses a slab with dimensions of 12.6 inches × 10.2 inches × 1.6 inches (320 mm × 260 mm × 40 mm). The slab is usually compacted at 7 ± 1 percent air voids using a linear kneading compactor. The test is conducted under water at constant temperature ranging from 77 to 158°F (25 to 70°C). Testing at122°F (50°C) is the most common practice (*19*). The sample is loaded with a reciprocating motion of the 1.85-inch (47 mm) wide steel wheel using a 158-lb force (705 N). Usually, the test is conducted at 20,000 cycles or up to a specified amount of rut depth. Rut depth is measured at several locations including the center of the wheel travel path, where usually it reaches the maximum value. One forward and backward motion comprises two cycles.

Precision Metal Works, a Kansas-based company, now manufactures a HWTD. Their device is slightly different and improved from the original version. This device is capable of testing with both slab and cylindrical specimens. The HWTD measures rut depth, creep slope, stripping inflection point, and stripping slope (*19*). The creep slope is the inverse of the deformation rate within the linear range of the deformation curve after densification and prior to stripping (if stripping occurs). The stripping slope is the inverse of the deformation rate within the linear region of the deformation curve after the stripping takes place. The creep slope relates primarily to rutting from plastic flow, and the stripping slope indicates accumulation of rutting primarily from the moisture damage (*22*). The stripping inflection point is the number of wheel passes corresponding to the intersection of creep slope and stripping slope.

Tim Aschenbrener (*20*) found an excellent correlation between the HWTD and pavements with known field performance. He mentioned that this device is sensitive to the quality of aggregate, asphalt cement stiffness, length of short-term aging, refining process or crude oil source of the asphalt cement, liquid and hydrated lime anti-stripping agent, and compaction temperature.

Izzo and Tahmoressi (*22*) conducted a repeatability study of the HWTD. Seven different agencies took part in that study. They experimented with several different versions of the HWTD. They used both slab and Superpave gyratory compacted specimens. Some of their conclusions were the device yielded repeatable results for mixtures produced with different aggregates and with test specimens fabricated by different compacting devices, and cylindrical specimens compacted with the SGC are acceptable for moisture susceptibility evaluation of different mixtures.

Model Mobile Load Simulator – 1/3 Scale

The MMLS3 was introduced as a scaled-down accelerated pavement testing device for use in a controlled environment (*33*). Researchers developed the MMLS3 by scaling down from the full-scale Texas Mobile Load Simulator (*19*, *34*). It is used for testing scale-model pavement sections or test pads. This machine has been used successfully on actual roadways. The MMLS3 has four single tires in series, linked together to form an endless chain. Each wheel is attached to a 1-foot diameter pneumatic tire. The wheels move around a set of looped rails in the vertical plane on a fixed frame and apply loads to a short section of pavement. Some of the advantages of this type of APT device are that the load is always moving in one direction, many repetitions are possible in a short period of time, and a relatively high trafficking speed is possible.

The wheels can be laterally displaced across a 6-inch (150 mm) wide path in a normal distribution about the centerline to simulate traffic wandering. The tires may be inflated up to a pressure of 120 psi (800 kPa). Axle loads varying between 470 and 600 lb (2100 and 2700 N) are possible. The axle loads are automatically kept constant at a predetermined value by the special suspension system. Nominal wheel speed is 8.2 ft/s (2.5 m/s), applying about 7200 load cycles per hour. A single variable-speed motor drives the chain of four wheels. Typical number of cycles used for the testing is between 100,000 and 200,000.

The MMLS3 can be used to evaluate the asphalt pavement sections in wet and dry conditions. An environmental chamber surrounding the machine is used for elevating the temperature. The test temperature can be increased up to 140°F (60°C). Performance monitoring during MMLS3 testing includes measuring the rut depth from transverse profiles at multiple locations of the wheelpath. Currently, there is no standard procedure for this test method.

At Stellenbosch University, South Africa, researchers are examining the ability of the MMLS3 to predict the fatigue behavior of HMA mixtures. They are using Superpave gyratory compacted specimens for this purpose. Research studies by the TxDOT used the MMLS3 to determine the relative performance of two rehabilitation processes and establish the predictive capability of this device (*19*). Comparison of pavement responses under full-scale Texas Mobile Load Simulator and the scaled-down MMLS3 showed good correlation when researchers considered actual loading and environmental conditions.

French Wheel Tracker

The Laboratoire Central des Ponts et Chausees developed the French Wheel Tracker, also known as the FRT, during the 1970s and 1980s (*35*). Recently, the Colorado Department of transportation and FHWA, at their Turner Fairbank Highway Research Center, conducted a research program to evaluate the performance of the FRT (*19*).

The test specimen is an asphalt slab with typical dimensions of 7.1 inches × 19.7 inches (180 mm × 500 mm). The FRT can apply wheel loads simultaneously on two test slabs. Loading is accomplished by applying a 1125-lb (5000 N) wheel load using a smooth pneumatic tire pressurized at 87 psi (600 kPa). The pneumatic tire passes over the slab center at the rate of 120 times per minute. For rut susceptibility evaluation, the test is conducted at a higher temperature range, typically 122 to 140°F (50 to 60°C).

FRT rut depth is defined by the deformation expressed as a percentage of the original slab thickness. The rut depth is measured across the width of the specimen. The typical number of cycles used with the FRT is 6000 (*34*).

Purdue University Laboratory Wheel Tracking Device

This device, also known as "PURWheel," was developed at Purdue University. The device was designed as a flexible general-purpose tester (*36*). It is capable of evaluating rutting potential and/or moisture sensitivity of HMA (*19*). In this device, load is applied on compacted test slab through a pneumatic tire.

The slab specimens are usually 11.4 inches (290 mm) wide and 12.2 inches (310 mm) long. The thickness varies from 1.5 to 3 inches (38 to 75 mm), depending on the type of mixture. The linear kneading compactor developed at Purdue University accomplishes compaction of laboratory specimens. A typical range of specimen air voids is 6 to 8 percent. The wheel load and tire contact pressure are 385 lb (1713 N) and 90 psi (620 kPa), respectively. The test environment can be hot/wet or hot/dry. Test temperature can range from room temperature to 149°F (65°C).

During testing, rutting is measured across the wheelpath. The PURWheel is typically operated for 20,000 wheel passes or until 0.8 inch (20 mm) of rutting has occurred. Moisture sensitivity of HMA mixtures is defined as the ratio of the number of cycles required for 0.5-inch (12.7 mm) rut depth in a wet condition to the number of cycles required for 0.5-inch (12.7 mm) rut depth in a dry condition.

CHAPTER 3:
EXPERIMENTAL DESIGN

PLAN OF STUDY

All six of the SST protocols are intended to characterize the load-related behavior of HMA. During the inception of this research study, all of the six SST protocols were under consideration. Later in the study, TxDOT and the researchers were informed of AASHTO's decision to discontinue three of the SST protocols. In the AASHTO provisional standard, Interim Guide for April 2001, only three tests were recommended; they are Simple Shear at Constant Height, Frequency Sweep at Constant Height, and Repeated Shear at Constant Height. The logic behind discontinuing the volumetric and uniaxial tests was due to the complexity of the test procedures, complexity of the test setups, and inconsistency of the test results. AASHTO also stated that the repeated shear at constant stress ratio test does not provide any new property that repeated shear at constant height test cannot provide. Researchers of this study and TxDOT readily accepted elimination of the volumetric test and uniaxial test. But the researchers wanted to examine all four of the other test protocols. The test plan is divided into the four following steps:

- Materials selection and acquisition: This step includes identification of four HMA mixtures with different rutting properties and collection of the aggregate, asphalt, and other components to produce those mixtures.
- Asphalt cement and aggregate characterization: The individual HMA mixture components were tested to determine if they meet Superpave requirements.
- Mixture design and verification: One new mixture was designed and three other mixture designs borrowed from other agencies were verified in the laboratory.
- Asphalt concrete mixture evaluation: Performance tests to establish rut resistance of the HMA mixtures were performed. Performance tests of HMA included the Superpave Shear Tester and Asphalt Pavement Analyzer, 1/3-Scale MMLS, and Hamburg Wheel Tracking Device.

MATERIALS SELECTION AND ACQUISITION

Researchers and the former project director, Mr. Tahmoressi, in the project kick-off meeting, identified four HMA mixtures with known or predictable field performance. Ideally, these four mixtures should exhibit field performance (particularly related to rutting) from excellent to poor. The reason for setting such criteria for the candidate mixtures was to examine the relative sensitivity of the SST and other HMA-characterizing test methods. The HMA mixtures selected for this study were: Type C limestone, Type D rounded river gravel, granite stone mastic asphalt (SMA), and granite Superpave. These mixtures were developed in Texas and Georgia. Once TTI researchers received the mixture design and mixture constituents, specimens were compacted using the respective design methods to verify the optimum asphalt content. In some cases, minor modifications (changes in asphalt content) were necessary to achieve the desired air void content. Table 2 summarizes the four mixture designs used in this study. The following paragraphs provide a brief description of these mixtures.

Type C Limestone Mixture

This HMA mixture was originally designed at Colorado Materials Company located in San Marcos, Texas. Colorado Materials Company supplied this mixture to several districts for numerous projects. The districts primarily using this mixture are San Antonio, Yoakum, Austin, Corpus Christi, and Bryan. The overall subjective rating of field performance of this mixture is good. The aggregates used for this mixture are Colorado Type C, Colorado Type D, Colorado Type F, Colorado manufactured sand, and Colorado field sand. All aggregates were collected from Colorado Materials except the field sand. The field sand was collected from Bryan, Texas. The asphalt used in the research study was PG 64-22, supplied by Koch Materials, Inc. Since the materials source was slightly different from the original source, the mixture design was checked in the laboratory and the optimum asphalt content was found to be 4.4 percent instead of 4.6 percent. Table 1 and Figure 1 present the mixture design gradation. From now on, this Type C limestone mixture will be referred to as the limestone mixture.

Type D River Gravel Mixture

This HMA mixture was designed at the TTI laboratory to obtain a rut-susceptible mixture. The aggregates were collected from the local Brazos River valley. This mixture uses mostly uncrushed and rounded river gravel and field sand. PG 64-22 asphalt was used in this mixture. It does not have any field performance history. Aggregate gradation is depicted in Table 1 and Figure 1. From now on this mixture will be referred to as the river gravel mixture.

Table 1. Gradations of Four Mixtures.

Sieve size, inch (mm)	Percent Passing			
	Limestone	River Gravel	Granite SMA	Granite Superpave
1.00 (25.0)	100.0	100.0	100.0	100.0
0.75 (19.0)	100.0	100.0	100.0	99.0
0.50 (12.5)	96.4	100.0	89.0	60.0
0.37 (9.5)	81.3	96.0	63.0	50.0
0.18 (4.75)	54.5	70.4	25.0	36.0
0.09 (2.36)	37.7	50.6	19.0	27.0
0.07 (1.8)	28.8	41.3	15.0	22.0
0.023 (0.6)	22.6	33.1	14.0	17.0
0.0117 (0.3)	14.7	21.8	13.0	12.0
0.0058 (0.15)	6.9	9.8	12.0	8.0
0.00293 (0.075)	4.4	3.7	10.0	4.1

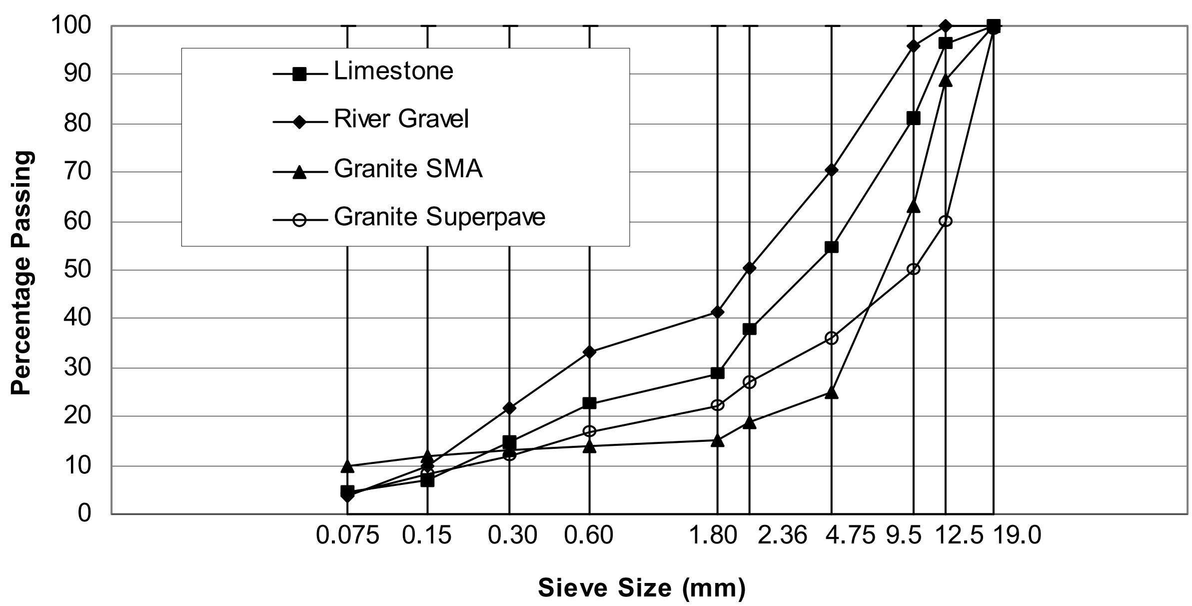

Figure 1. Gradations of Mixtures Used in Study.

Granite SMA Mixture

This SMA mixture is usually considered to be a very rut-resistant mixture. Georgia Department of Transportation (DOT) provided this mixture design. They use this mixture on their heavy-duty pavements. The mixture was designed using 50 blows of the Marshall hammer. The granite aggregate used for the mix design was obtained from a Vulcan Materials quarry in Lithia Springs, Georgia. PG 76-22 asphalt, supplied by Koch Materials, Inc., was used in this mixture. In addition to granite aggregate, 0.4 percent mineral fiber, 1.0 percent lime, and 9.0 percent fly ash were used in the mixture. Boral Materials Technology provided the fly ash. To produce the same gradation as the original design, researchers at TTI fractionated all aggregates into different ASTM standard sieve sizes and recombined them in accordance with mixture design. The purpose of using the mineral fiber is to prevent drain down of liquid asphalt mastic from the mixture.

During verification of the mix design, the optimum asphalt content was reduced to 5.9 percent from 6.1 percent (original optimum asphalt content). Table 1 and Figure 1 illustrate the design aggregate gradation.

Table 2. Mixture Design Information.

Mixture Type	Binder Content (%)	Binder Type	Rice Density (gm/cc)	Design Method
Limestone	4.4	PG 64-22	2.428	Texas Gyratory
River Gravel	5.5	PG 64-22	2.416	Texas Gyratory
Granite SMA	5.9	PG 76-22	2.396	Marshall 50-blow
Granite Superpave	4.0	PG 64-22	2.481	Superpave

Granite Superpave Mixture

Georgia DOT also provided this 19.0-mm Superpave mixture design. The idea behind selecting this mixture was to obtain a mixture with very good rut resistance. In Georgia, this mixture is known as "Level B." The aggregate for this mixture has the same source as the granite SMA mixture. The filler includes 1.0 percent hydrated lime. During the mix design process, the number of SGC gyrations used are 7, 86, and 134, as $N_{initial}$, N_{design}, and $N_{maximum}$, respectively. PG 64-22 asphalt was used for this mixture. In fact, the same asphalt was used in the limestone, river gravel, and granite Superpave mixtures. During the mixture verification process at the TTI laboratory, the optimum asphalt content was increased to 4.0 percent instead of the original 3.9 percent. The aggregate for this mixture was sieved to different size fractions and recombined to obtain an accurate gradation. Aggregate gradation is shown in Table 1 and Figure 1.

Tests for Asphalt Cement Characterization

One of the three major components of the Superpave mixture design process is the asphalt binder performance grading specification (AASHTO MP1). Asphalt binder is tested under conditions that simulate its critical stages during service in a pavement, such as:

- during transportation, storage, and handling - original binder is tested;
- during mix production and construction - simulated by short-term aging the original binder in a rolling thin film oven (RTFO); and
- after 5 to 10 years of service - simulated by long-term aging the binder in the rolling thin film oven test plus the pressure aging vessel (PAV).

The appendix includes results of the binder tests.

Dynamic Shear Rheometer

Researchers used the DSR to characterize viscous and elastic behavior of asphalt binders at high and intermediate service temperatures. The DSR measures the complex shear modulus (G^*) and phase angle (δ) of asphalt binders at the desired temperature and frequency of loading. Complex modulus is a measure of the total resistance of a material to deformation when repeatedly sheared. It consists of two components:

- storage modulus (G') or the elastic (recoverable) part, and
- loss modulus (G'') or the viscous (nonrecoverable) part.

The lag time between the applied peak stress and resulting peak strain is the phase angle (δ). For perfectly elastic materials, the phase angle is 0 degrees, and, for perfectly viscous fluid materials, it is 90 degrees. Asphalt binders behave like elastic solids at very low temperatures and like viscous fluids at high temperatures. However, at typical pavement service temperatures, asphalt behaves like a viscoelastic material; therefore, δ will be greater than zero but smaller than 90 degrees (*7*).

The DSR is used to determine the rutting parameter of the asphalt binder at high temperatures for unaged binders and short-term aged binders. For rutting resistance, a high complex shear modulus (G^*) value and low phase angle (δ) are both desirable. Higher G^* values indicate stiffer binders that are more resistant to rutting. Lower δ values indicate more elastic asphalts that are more resistant to rutting. Therefore, a larger $G^*/\sin\delta$ signifies more resistance to permanent deformation by the asphalt binder.

The DSR is also used to determine the fatigue resistance of the asphalt binder at intermediate temperatures for long-term aged binders. For fatigue resistance, a low complex modulus value and a low phase angle are both desirable. Therefore, smaller values of $G^*\sin\delta$ should indicate more resistance to fatigue cracking.

Bending Beam Rheometer (BBR)

The BBR measures a binder's resistance to thermal cracking. Thermal cracking may occur in asphalt pavements when the temperature drops rapidly at low temperatures. The BBR uses a transient creep-bending load on the center of an asphalt cement beam specimen held at a constant low temperature. This test is performed on asphalt binder that has been subjected to long-term aging. From this test, two parameters are obtained:

- creep stiffness - a measure of how the asphalt binder resists the constant creep loading, and
- m-value - a measure of the rate at which the creep stiffness changes with time of loading.

If creep stiffness increases, the thermal stresses developed in the pavement due to thermal shrinking also increase and thermal cracking becomes more likely. If m-value decreases (the curve flattens), the ability of the asphalt binder to relieve thermal stresses decreases and the propensity for thermal cracking increases.

Direct Tension Tester (DTT)

The DTT measures the low-temperature ultimate tensile strain of the binder. This test is performed using binder that has been subjected to long-term aging. The DTT is used only when the asphalt creep stiffness obtained from the BBR is greater than 300 MPa but smaller than 600 MPa. The DTT is used because there are some asphalt binders that may have high creep stiffness but do not crack because they can stretch farther before breaking. Larger failure strain indicates more ductile binders and, therefore, more resistant to cracking.

Rotational Viscometer

The rotational viscometer (often referred as the Brookfield viscometer) was adopted in Superpave for determining the viscosity of asphalt binder at high temperatures, primarily to ensure that it is sufficiently fluid for pumping or mixing. Rotational viscosity is determined by measuring the torque required to maintain a constant rotational speed of a cylindrical spindle. The rotational viscometer determines optimum HMA mixing and compaction temperatures.

Mixing and Compaction Temperature

Superpave HMA mixtures are mixed and compacted under equiviscous temperature conditions corresponding to 0.17 Pa•s and 0.28 Pa•s, respectively (*4*). Viscosity of the asphalt was tested using the Brookfield rotational viscometer at 275°F (135°C) and 347°F (175°C). Plotting the result on a viscosity versus temperature graph (log-normal), the mixing and compaction temperatures were determined for PG 64-22 asphalt. But applying the same procedure unusually yielded excessive mixing and compaction temperatures for PG 76-22 asphalt. Therefore, the mixing and compaction temperatures used for the PG 76-22 were those suggested by the supplier. Table 3 describes the mixing and compaction temperatures for both asphalts.

Details of binder testing and the results and determination of the mixing and compaction temperatures are described in the Appendix.

Table 3. Mixing and Compaction Temperatures.

Asphalt Grade	Mixing		Compaction	
	Temp Range (°F)	Selected Temp, (°F)	Temp Range, (°F)	Selected Temp, (°F)
PG 64-22	317 - 330	320	294 - 306	295
PG 76-22	315 - 335	330	280 - 310	310

Tests for Aggregate Characterization

Superpave specifications contain two categories of aggregate properties: consensus properties and source properties (*4*). Consensus properties are those aggregate characteristics that are critical to well-performing asphalt mixtures. These properties include:

- coarse aggregate angularity,
- fine aggregate angularity,
- flat and elongated particles, and
- clay content.

The specific criteria for these consensus aggregate properties are based on traffic level and position of the layer within the pavement structure.

Source properties are those aggregate properties that, although important for the asphalt mixture performance, they are not considered critical, and no critical values for those properties were defined by Superpave (*4*). Criteria for the aggregate source properties are left to the local agencies. Those properties include:

- toughness,
- soundness, and
- deleterious materials.

Only the consensus aggregate properties were considered in this study because they can be related to permanent deformation in HMA mixtures. The source aggregate properties were not examined since they do not correlate particularly well with pavement deformation (*37*, *38*).

Coarse Aggregate Angularity (CAA)

CAA is the percent by weight of aggregates larger than #4 sieve (4.75 mm) with one or more fractured faces. Higher CAA enhances coarse aggregate internal friction and thus HMA rutting resistance (*4*). CAA was measured following ASTM D 5821-95. A fractured face is defined as an angular, rough, or broken surface of an aggregate particle created by crushing, other artificial means, or nature. A face will be counted as fractured only if it has a projected area at least as large as one-quarter of the maximum projected area (maximum cross-sectional area) of the particle and the face has sharp and well-defined edges (*39*).

Superpave requires the minimum value for CAA to be a function of traffic level and position within the pavement. The selected depth from the surface was less than 4 inches (100 mm) primarily because the study is focused on plastic deformation in the asphalt layers, and this type of rutting occurs mainly in the uppermost asphalt layers.

Table 4 shows the coarse aggregate angularity test results. Except for the river gravel, all other aggregates exhibited 100 percent crushed faces. River gravel is well below the Superpave CAA requirement. In fact, rounded uncrushed river gravel was selected intentionally to design a rut-susceptible mixture.

Table 4. Coarse Aggregate Angularity Test Results.

Mixture Type	One Fractured Face (%)	Two Fractured Face (%)
Limestone	100	100
River Gravel	30	19
Granite SMA	100	100
Granite Superpave	100	100

Fine Aggregate Angularity (FAA)

FAA is the percent air voids present in loosely compacted aggregates of a specified gradation smaller than the #8 sieve (2.36 mm). Higher void contents generally mean more fractured faces. This criterion is designed to ensure a high degree of fine aggregate internal friction and thus rutting resistance (*4*). The test procedure followed was ASTM C 1252, Method A. Superpave has a required minimum value for fine aggregate angularity as a function of traffic level and layer position within the pavement. Table 5 shows test results.

Only river gravel did not meet the Superpave FAA criterion for a heavy traffic surface mixture (FAA 45), and limestone marginally met the criteria. Chowdhury et al. (*40*) demonstrated that FAA values for aggregate containing 100 percent crushed but cubical particles were sometimes lower than those for aggregates containing rounded particles. Crushed sedimentary rocks such as limestone often fall in this category even though they may have an excellent performance history.

Table 5. Fine Aggregate Angularity Test Results.

Mixture Type	Fine Aggregate Angularity (%)
Limestone	45.2
River Gravel	40.2
Granite SMA	47.1
Granite Superpave	47.1

Flat and Elongated Particles (F&E)

According to Superpave, F&E is the percentage by mass of coarse aggregate particles larger than a #4 sieve (4.75 mm) that have a maximum to minimum dimension ratio greater than five. This criterion is an attempt to avoid flat or slender particles with a tendency to break during construction and under traffic. The test procedure followed was ASTM D 4791 (Table 6).

Superpave specifies a maximum value for F&E coarse aggregate particles as a function of traffic level.

Table 6. Flat and Elongated Aggregate Test Results.

Mixture Type	Flat & Elongated (%)
Limestone	4
River Gravel	6
Granite SMA	5
Granite Superpave	5

Clay Content

Clay content is the percentage of clay material (by volume) contained in the aggregate fraction finer than a #4 sieve (4.75 mm). Superpave has a required minimum value for clay content of fine aggregate particles as a function of traffic level. This property ensures that the relative proportion of clay-like or plastic fines in granular soils and fine aggregates is not excessive. The test procedure followed was ASTM D 2419-95 (Table 7).

Table 7. Clay Content Test Results.

Mixture Type	Clay Content (%)
Limestone	75
River Gravel	90
Granite SMA	80
Granite Superpave	80

SUPERPAVE SHEAR TESTER

During the SHRP study, two mechanical devices were developed to characterize the HMA mixture. They are the Superpave Shear Tester and the Indirect Tensile Tester (IDT). The original Superpave mixture analysis procedures used the test results from these two devices to determine the extent of several distresses like permanent deformation, fatigue cracking, and low-temperature cracking. Presently, an intermediate analysis is used for pavements with traffic levels up to 10 million equivalent single axle loads (ESAL), and complete analysis is used for heavily trafficked pavements, i.e., those exceeding 10 million ESALs.

Since this study focuses on the permanent deformation characteristics of HMA mixtures, only the Superpave Shear Tester will be discussed. The same testing performed by SST can be used to evaluate both permanent deformation and fatigue characteristics of HMA mixtures. The SST is capable of performing the following tests:

- Uniaxial Test,
- Volumetric Test,
- Frequency Sweep at Constant Height Test,
- Simple Shear at Constant Height Test,
- Repeated Shear at Constant Height Test, and
- Repeated Shear at Constant Stress Ratio Test.

Description of Equipment

Currently, there are only two SST manufacturers. They are Cox & Sons, Inc. and Interlaken Technology Corporation. With some differences, these two machines are similar in construction and operation. The SST is a closed-loop feedback, servo-hydraulic system that was designed to characterize the load-related behavior of asphalt mixtures. This closed-loop feedback system can be maintained by either stress control or strain control. The SST system is composed of four main components: testing apparatus, control unit, environmental control unit, and hydraulic system.

Testing Apparatus

The main component of the SST testing apparatus is the loading device. The loading device is composed of a reaction frame and shear table. This loading device can apply simultaneously vertical, horizontal, and confining loads to the test specimen. These loads may be applied statically, ramped up or ramped down, or repetitively in various wave shapes. The testing apparatus also accommodates other components, such as temperature and pressure control, hydraulic actuator, and input and output transducers.

Figure 2. Cox Superpave Shear Tester.

The shear table holds the HMA specimen during testing, the horizontal actuator imparts shearing load to the specimen, and the vertical actuator applies an axial load to the specimen. Loads are transferred to the specimens through the loading platens glued on both sides of the specimens. Linear variable differential transducers (LVDTs) are attached to specimens to measure response of the specimens due to the application of loads. The system functions as a closed-loop feedback system using the signal sent by the LVDTs. These LVDTs can be axial, shear, or radial, depending the type of test.

Control and Data Acquisition System

The control unit consists of hardware and software systems. The hardware interfaces with the testing apparatus through input and output transducers. The control unit consists of controllers, signal conditioners, and a computer and its peripherals. The software consists of the algorithms required to control the test and acquire data during the test. The control and data acquisition system is used to automatically control and record the required measurement parameters during the test. It records the load cycle, time, vertical and horizontal loads, specimen deformation in all directions (vertical, horizontal, and radial), and test chamber temperature.

Environmental Control Unit

The purpose of this unit is to maintain a constant temperature and pressure inside the testing chamber. The environmental unit is capable of providing temperatures within a range of 34 to 176 °F (1 to 80 °C) with an accuracy of ±1°F (±0.5°C). This unit precisely controls air pressure inside the testing chamber. Air pressure is normally applied at a rate of 10 psi (69 kPa) per second up to a maximum value of 122 psi (840 kPa). The air pressure is usually applied using compressed air from a separate storage tank. Uniaxial and volumetric tests require a confining pressure on the specimens.

Hydraulic System

The hydraulic system provides the necessary force to apply loads on the specimens at different testing conditions. A hydraulic motor powers two actuators. The capacity of each actuator is approximately 7190 lb (32 kN) with a resolution of 9 lb (2 N). The vertical actuator applies an axial force to the specimen, and the horizontal actuator drives the shear table that imparts shear loads to the specimen. The hydraulic system is also capable of creating a confining pressure of 145 psi (1000 kPa).

Specimen Preparation and Instrumentation

Specimens for SST testing are typically compacted using the AASHTO Standard TP4. The specimen diameter is always 6 inches (150 mm), but the specimen height is 2 inches (50 mm) for mixtures with 0.75-inch (19-mm) nominal maximum size of aggregate and 1.5 inches (38 mm) for mixtures with smaller nominal maximum size. Researchers believe that all specimens should be 2 inches, regardless of the nominal maximum size. This 2-inch height is the final height of the specimen after saw cutting on both sides.

FSCH and SSCH test specimens are compacted at 7 percent air voids, and specimens for two other tests are compacted at 3 percent air voids. The tolerance adopted for compaction was 0.5 percentage points for air voids for all SST tests, following the AASHTO Standard TP 7-01 (AASHTO Standard, Interim, April 2001). Researchers sawed both ends of all test specimens. These saw cuts were perpendicular to the axis of the specimens such that the height of the specimens was 2 ± 0.1 inches (50 ± 2.5 mm). Both ends have to be smooth and mutually parallel within 0.08 inch (2 mm).

For the four tests that do not require confining pressure, the specimens were glued between two loading platens made of aluminum. A platen-specimen assembly device was used to glue the specimens to platens. This Superpave gluing device compresses the specimen between the platens with a 5 psi (34.5 kPa) load for 30 minutes while the glue sets up. This gluing device facilitates bonding between the specimen and the loading platens and maintains the top and bottom platens in a relatively parallel position during the gluing operation. The platens must be parallel to eliminate undesirable stress concentration. An epoxy-type glue with a minimum hardness stiffness modulus of 290 ksi (2000 MPa) is required for bonding between the specimen and the platens. Devcon Plastic Steel epoxy cement performs satisfactorily for this purpose.

Depending on the test procedure, LVDTs are mounted on the specimen-platen assembly to measure the load response or deformation in the axial (vertical), shear (horizontal), and radial (circumferential) directions. Only uniaxial tests and volumetric tests require radial LVDTs. The axial and shear LVDTs can be mounted on the surface of specimens directly or on the platens. Researchers prefer LVDTs to be mounted directly on the specimens. That way, measured deformation depends solely on the specimen and the glue and platen properties do not affect measured deformation.

After marking their locations with a template, mounting screws were attached to the sides of the specimen with a cyanoacrylate glue with an accelerator and, once it set up, the horizontal LVDT holders were attached and the LVDTs were installed. The difference in horizontal displacement was measured between the two LVDTs with a gauge length of 1.5 inches (38.1 mm). Researchers conducted tests using the Cox & Sons 7000 SHRP Superpave Shear Tester.

Figure 3. Superpave Gyratory Compactor.

DESCRIPTION OF TESTS

Originally, Superpave adopted five SST tests to characterize HMA mixtures and a sixth test (RSCH) was optional for evaluating the tertiary rutting of a mixture. At the beginning of this study, researchers planned to evaluate all six SST tests. Later, when AASHTO dropped some of the tests, this project task was modified. Since uniaxial and volumetric tests were not performed in this study, only little about those test procedures will be discussed.

Volumetric Test

This volumetric strain test requires confining pressure. In this test method, a confining pressure is applied at a steadily increasing rate up to certain level (depending on test temperature), it remains steady for a while, and then is slowly decreased. The volumetric strain test is usually performed at three different temperatures: 39, 68, and 104°F (4, 20, and 40°C). During the test, circumferential and axial deformations are measured. Sometimes this test is referred to as the 'hydrostatic' test.

Uniaxial Test

The uniaxial test uses static loading. In this test procedure, an axial load is applied on the specimen in a prescribed way. The axial load is increased at a constant rate up to a certain level and remains steady for a while before decreasing at constant rate. During the application of the axial load, a variable confining pressure is applied to maintain a constant circumferential strain. Axial deformation is the primary output of this test, which is designed to measure the elastic and plastic characteristics of an asphalt mixture. The uniaxial test is conducted at three different temperatures.

Frequency Sweep Test at Constant Height

The frequency sweep at constant height is a strain-controlled test where the cylindrical specimen is subjected to dynamic loading over a wide range of frequencies. The horizontal actuator applies the dynamic shear load. The horizontal actuator is controlled by the closed-loop feedback measurements from the shear LVDT to keep the shearing strain at a specified maximum value (Figure 4). The maximum shearing strain is 0.005 percent. When the test specimen is sheared, it attempts to dilate, which tends to increase its height. The vertical actuator controlled by closed-loop feedback measurements from axial LVDTs applies a variable axial load to keep the specimen at a constant height. The frequency sweep test is performed at frequencies of 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. The test is started from higher frequency to lower frequency. Details of the test procedure are described in AASHTO Standard TP 7-01, Procedure A (*41*).

The specimens tested with the FSCH test are compacted to 7 ± 0.5 percent air voids. The specimen dimension is 6 inches (150 mm) in diameter and 2 inches (50 mm) in height.

For temperature monitoring throughout the test, a dummy specimen with thermocouples drilled into its core was placed inside the test chamber. Researchers used a similar dummy specimen for other SST tests. The SST machine records axial deformation, shear deformation, axial load, shear load, and temperature for each of the 10 frequencies in the FSCH test. Data of all applied cycles were not recorded. The number of cycles sampled depends on the type of SST machine (Cox or Interlaken). In this study at TTI, researchers used the SST manufactured by Cox and Sons. The number of cycles applied and sampled for each level of frequencies by the

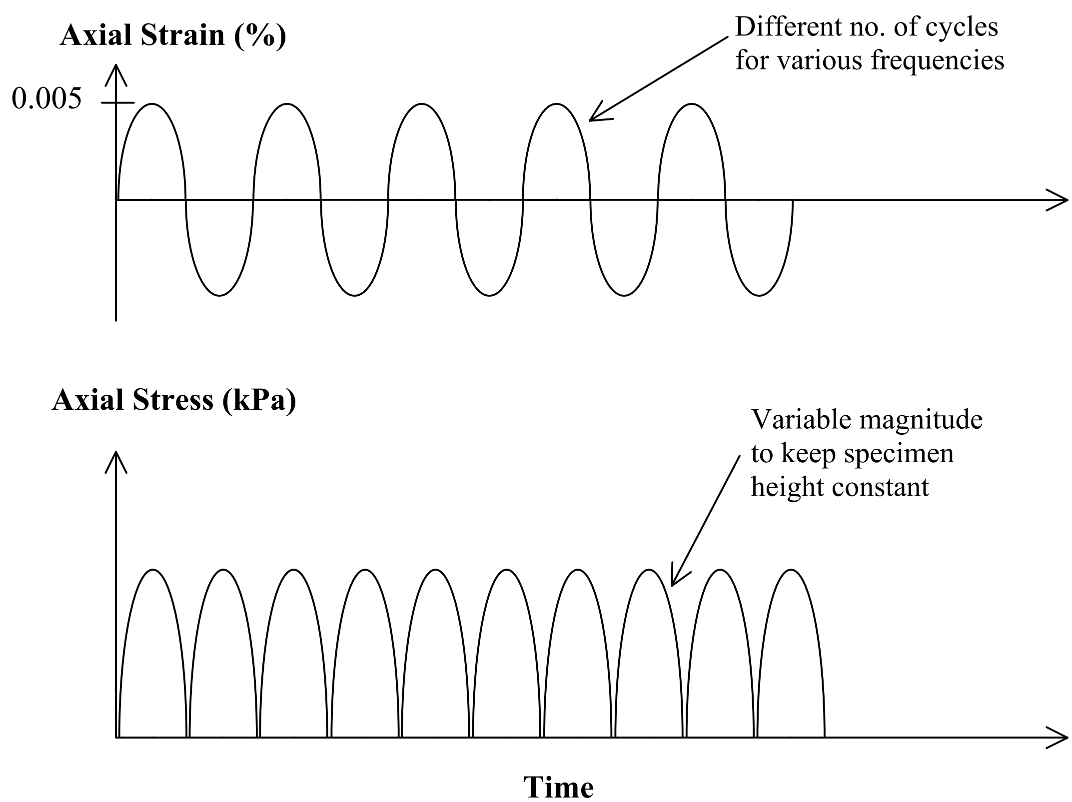

Figure 4. Shear Strain and Axial Stress Pulses in the FSCH Test.

Cox SST machine is given in Table 8. The data collected in this test were used to calculate the complex shear modulus and shear phase angle at each level of frequency. Calculations of complex shear modulus and shear phase angle were performed with help of ATS software. The average of three replicate specimens is reported as the result.

Table 8. Frequencies, Number of Cycles Applied, and Data Points per Cycle.

Frequency (Hz)	Total Cycles (Number)	Cycles Sampled (Number)	Data Points Per Cycle (Number)
10	50	10	60
5	50	10	60
2	20	10	60
1	20	10	60
0.5	10	10	60
0.2	10	10	60
0.1	10	10	60
0.05	5	1	60
0.02	5	1	60
0.01	5	1	60

Simple Shear at Constant Height Test

The SSCH test was developed to measure the elastic and plastic properties of HMA mixtures. This test is used for both intermediate and complete analysis of Superpave mixtures. This test uses static loading. The SSCH test is performed on the same specimens tested by the FSCH test. Stresses are applied on the specimen as shown in Figure 5. Shear stress is applied at a rate of 10.15 ± 0.7 psi (70 ± 5 kPa) per second up to the certain stress level, depending on the test temperature. The stress level is maintained for 10 s, and afterwards, it is reduced to 0 stress at a rate of 3.62 psi (25 kPa) per s. The test continues for an additional 10 s at a zero stress level. When the specimen is subjected to a controlled shearing stress, it attempts to dilate, which causes the height to increase. Using the feedback from axial LVDT, the vertical actuator applies an axial load of variable magnitude to keep the specimen height constant.

Typical data recorded in this test include axial deformation, axial load, shear load, shear deformation, and temperature. Shear load and shear deformation data were used to calculate the maximum deformation, permanent deformation, and elastic recovery. Researchers averaged results from three replicate specimens.

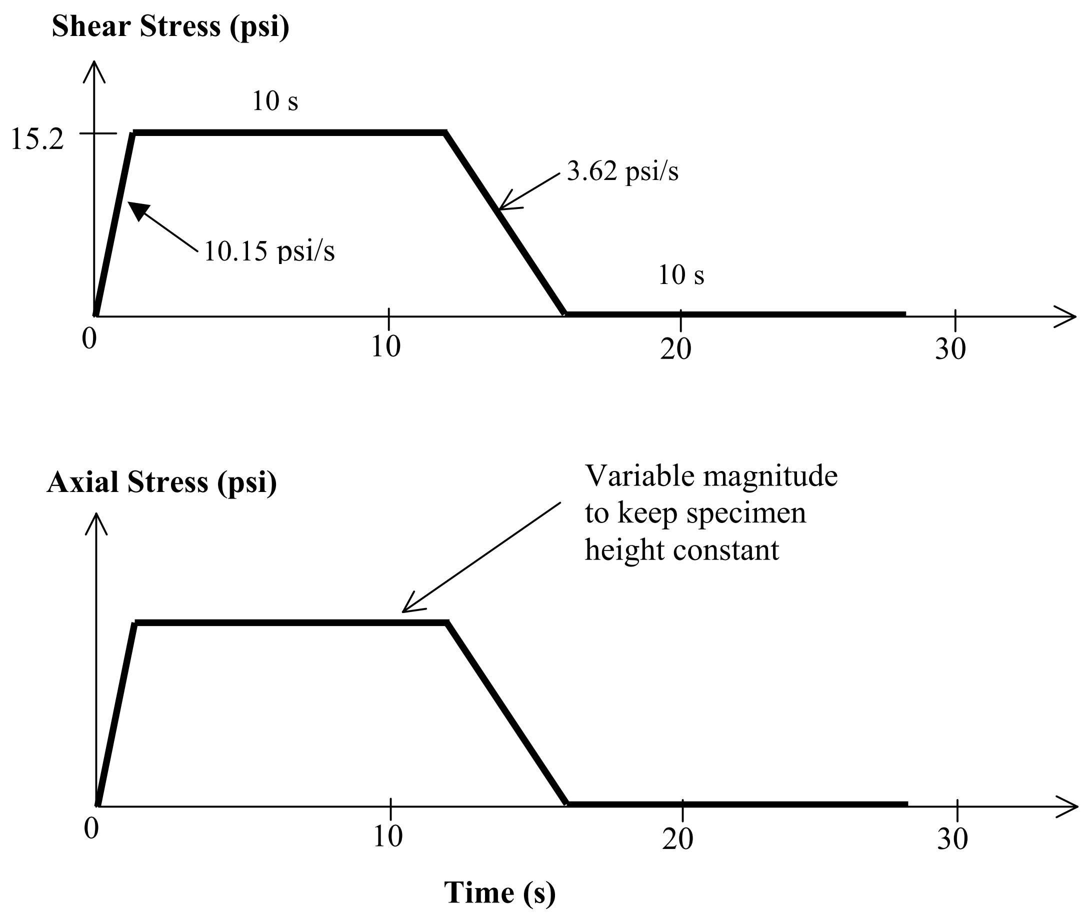

Figure 5. Typical Stress Application for the SSCH Test.

Repeated Shear at Constant Height Test

The RSCH test was an optional test for Superpave mixture design (intermediate and complete analysis). AASHTO recently endorsed this test procedure in TP 7-01. In this test, a haversine shear load is applied on the specimen to achieve a controlled shear stress level of 10 psi (69 kPa). During the application of this repeated shear load, a variable axial load is applied to keep the specimen height constant. A schematic of the load application is depicted in Figure 6. A 0.7-s load cycle consists of 0.1-s of shear loading followed by a 0.6-s rest period.

The test temperature can be determined in many ways, but it is most commonly calculated as the 7-day maximum pavement temperature (at a depth of 2 inches [50 mm]) for the project location (*41*). AASHTO suggests that if the mixture in question is a surface course and

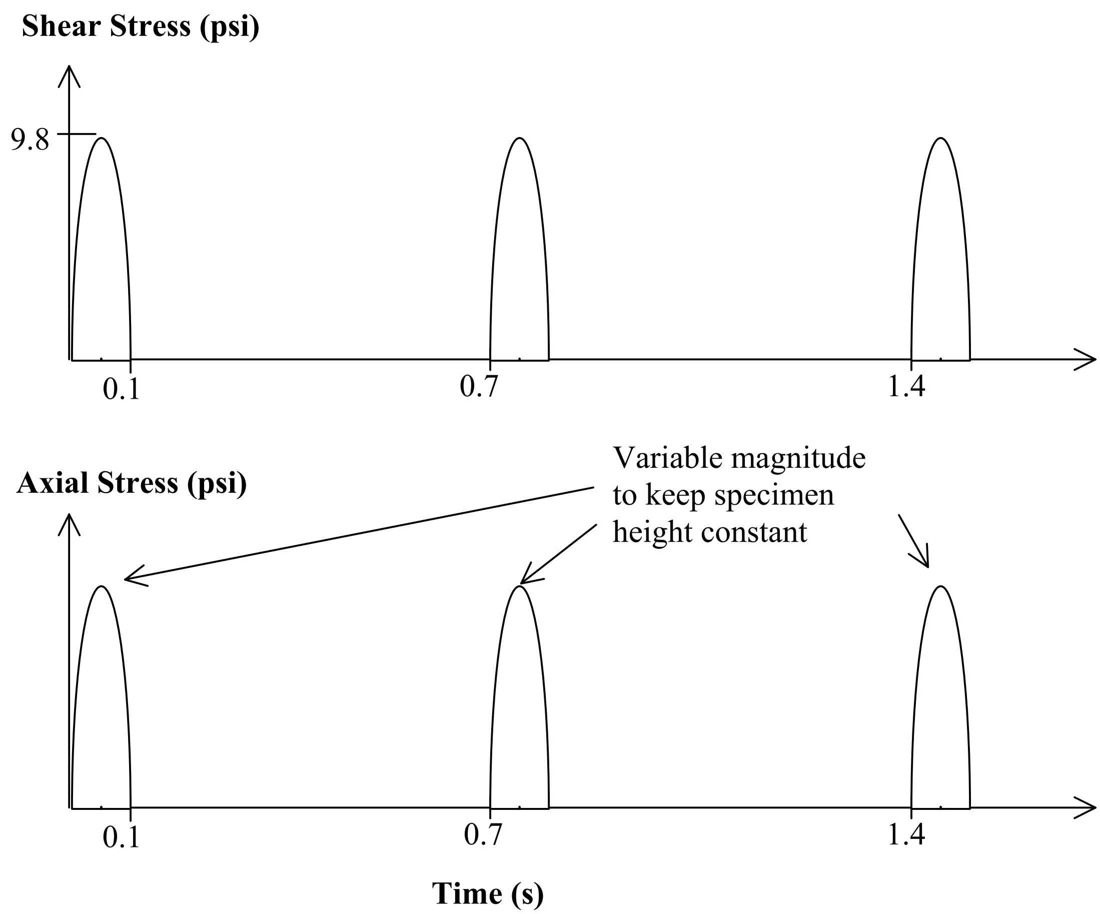

Figure 6. Stress Pulses in the RSCH Test.

the thickness of the layer is less than 2 inches (50 mm), then actual layer thickness may be used as the depth for calculating the test temperature. In this project, the project location is considered to be College Station, Texas, and the surface mixture thickness is considered to be 2 inches (50 mm) or more. Pavement temperature at the project location was calculated using SHRP Manual A-648A (*42*).

Using the SHRP A-648A manual, the 7-day maximum pavement temperature for College Station, Texas, is found to be 140°F (60°C) with 50 percent reliability. Maximum pavement temperature is calculated at depth of 0.8 inches (20 mm). Using the method presented in SHRP Manual A-648-A, the temperature at a depth of 2 inches (50 mm) is calculated as 131°F (55°C) for the design location. Researchers used the same test temperature for all four mixtures.

AASHTO suggests continued loading for 5000 load cycles or until the permanent shear strain reaches 5 percent. Researchers decided to apply 10,000 load cycles or 5 percent strain, whichever comes first. The SST records the axial load, shear load, axial deformation, and shear deformation. At the end of the test, the permanent shear strain is calculated and reported as a percentage. Figure 7 illustrates a shearing deformation of a specimen subjected to repeated loading. The strain curve has three distinct parts: large strain with few load applications due to initial compaction, a linear pattern of steady rise in strain due to plastic deformation, followed by an abrupt rise in strain after few more load applications. The latter part of the graph indicates tertiary rutting, which usually occurs when the air void content falls below some critical value. The presence of tertiary rutting also indicates that the mixture is very unstable or tender (*9*).

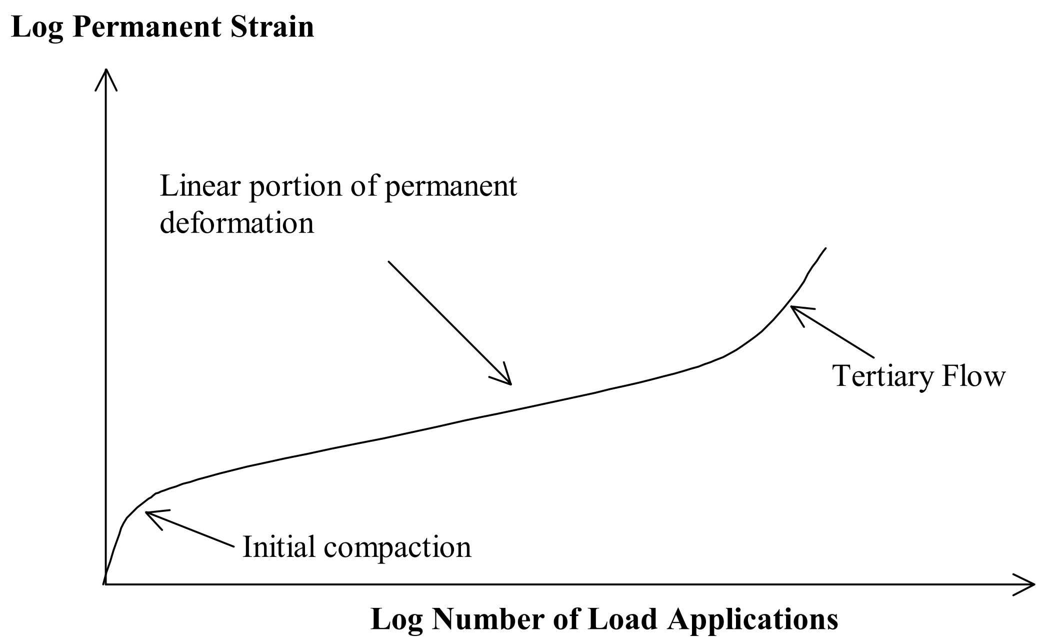

Figure 7. Permananent Deformation versus Repeated Load Applications.

Repeated Shear at Constant Stress Ratio Test

RSCSR is no longer included in the Superpave test protocols. This test was dropped from the interim edition of AASHTO Provisional Standards in April 2001 (*41*). Before then, this test was used for both intermediate and complete analysis of Superpave mixtures. This test was considered as a screening test to delineate an asphalt mixture that is subject to tertiary rutting (*4*). This form of rutting normally occurs at low air void contents and is a result of mixture instability.

In the RSCSR test, repeated synchronized haversine shear and axial load pulses are applied to the specimen as shown in Figure 8. The haversine ([1-cos 2]/2) load pulse approximates the effect of a wheel load on a pavement (*9*). The load cycle requires 0.7 s, wherein a 0.1-s load is followed by a 0.6-s rest period. The ratio of axial load to shear load is maintained during the test at a constant ratio within the range of 1.2 to 1.5.

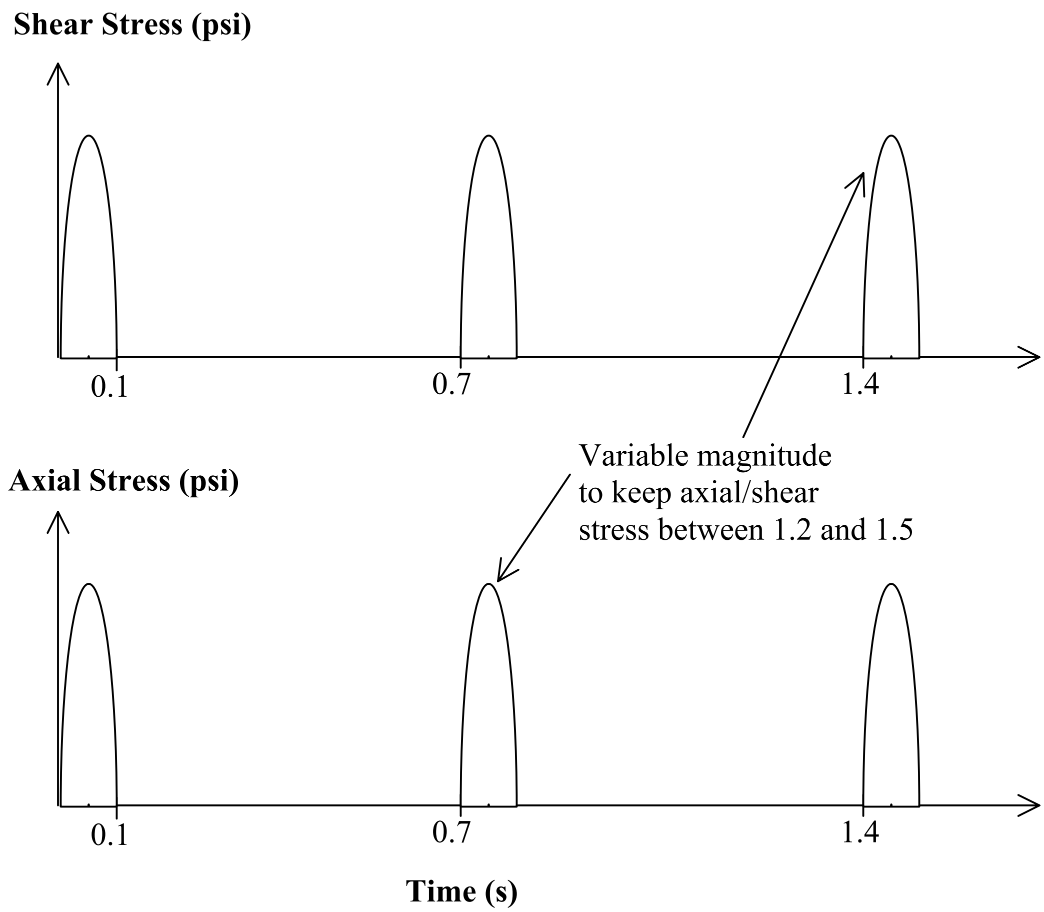

Figure 8. Stress Pulses in the RSCSR Test.

AASHTO suggests performing the test with 5000 cycles or 5 percent accumulated shearing strain, whichever comes first. Based on previous experience, the researchers decided to use 10,000 cycles or 5 percent accumulated shearing strain, whichever comes first because, with only 5000 cycles of loading, most mixtures may not show any tertiary rutting. Researchers decided to perform this test at same temperature (131°F [55°C]) as the RSCH test. The SST records the axial load, shear load, axial deformation, and shear deformation. At the end of the test, the permanent shear strain is calculated and reported in percent.

ACCELERATED WHEEL TESTING

Researchers planned to conduct certain laboratory-scale rutting tests on the selected HMA mixtures. The objectives of these tests were to compare the results from SST tests with the results from laboratory-scale rut tests. Initially, researchers planned to conduct two laboratory-scale tests: Asphalt Pavement Analyzer and 1/3-Scale Model Mobile Load Simulator. Later, when two of the SST test protocols were dropped from the test plan due to AASHTO's recommendation, the Hamburg Wheel Tracking Device was included in the plan.

Asphalt Pavement Analyzer

The APA is a multifunctional loaded wheel tester used for evaluating permanent deformation (Figure 9). Oscillating beveled aluminum wheels apply a repetitive load through high-pressure hoses to generate the desired contact pressure. Rutting susceptibility of HMA can be assessed by the APA using beam or cylindrical specimens under repetitive wheel loads and measuring the amount of permanent deformation under the wheelpath.

In this study, six cylindrical specimens for each mixture were prepared using the Superpave gyratory compactor. Specimen size was 6 inches (150 mm) in diameter and 3 inches (75 mm) in height. The APA manufacturer recommends using three pairs of specimens to test each mixture. Specimens were prepared with 4 percent air voids, and rutting tests were performed at 147°F (64°C) for all the mixtures.

Each set of specimens was subjected to 8000 load cycles (*31*). One load cycle consists of one forward and one backward movement of the wheel. The wheel load and hose pressure were 100 lb (445 N) and 100 psi (690 kPa), respectively. The vertical LVDT attached to the wheel measures the rut depth at four different points on each set of specimens. Two specimens in one mold form a set of specimens. Figure 10 shows the specimens set up in the APA machine. The average of four readings is calculated as the rut depth of one set of specimens. The average of three rut depths measured on three sets of specimens is reported as mixture rut depth.

Figure 9. Asphalt Pavement Analyzer.

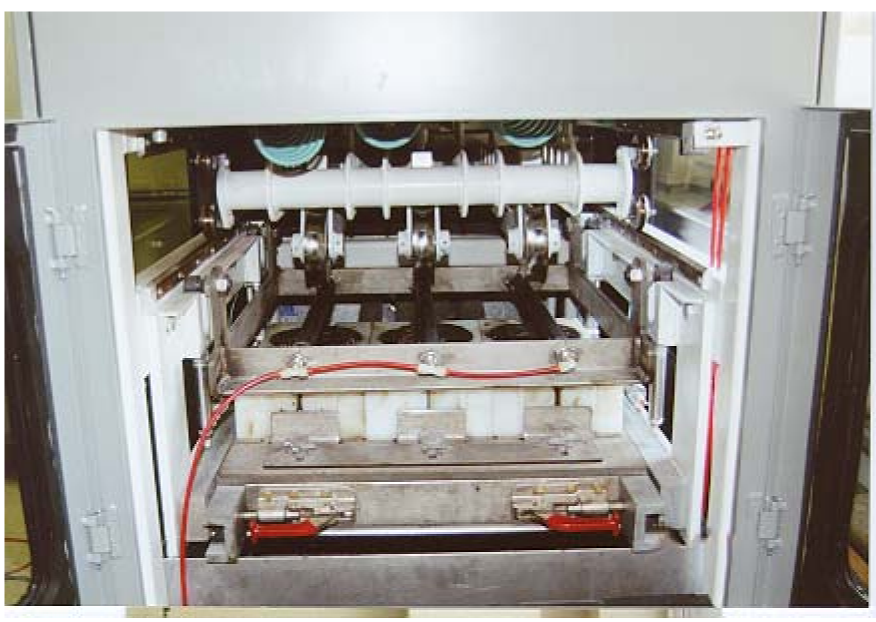

Figure 10. APA Test Setup.

1/3-Scale – Model Mobile Load Simulator

The 1/3-Scale Model Mobile Load Simulator (MMLS3) was introduced as a scaled-down APT device for use in a controlled environment (*33*). The advantages of this type of APT device are:

- The load is always moving in one direction.
- Many repetitions are possible in a short period.
- A relatively high trafficking speed is possible.

Figure 11 shows a schematic of the MMLS3. It consists of four recirculating axles, each with a single 12-inch (300 mm) diameter wheel. The wheels can be laterally displaced across 6 inches (150 mm) in a normal distribution about the centerline to simulate traffic wandering. The tires may be inflated up to a pressure of 120 psi (800 kPa). Axle loads varying between 470 to 600 lb (2100 to 2700 N) are possible. The axle loads are automatically kept constant at a predetermined value by. Nominal wheel speed is 8.2 ft/s (2.5 m/s), applying about 7200 load cycles per hour. A single variable speed motor drives the chain of four wheels.

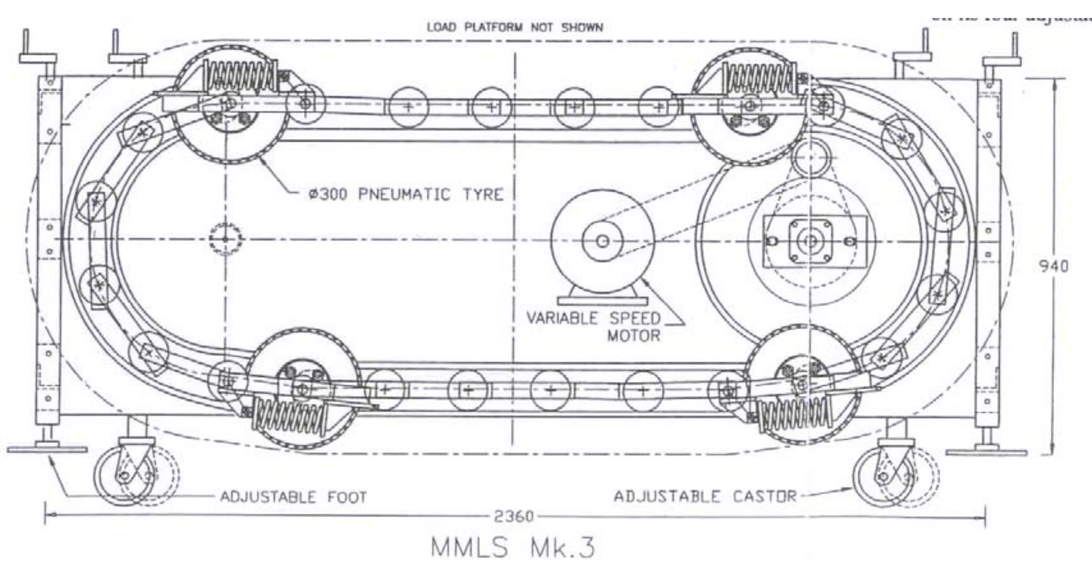

Figure 11. Schematic of the MMLS3.

MMLS-3 Test Specification

The following specifications were used for the MMLS3 tests described in this report:

Wheel load	:	470 lb (2100 N)
Tire pressure	:	100 psi (690 kPa)
Rate of loading	:	6000 axles per hour
Test temperature	:	122°F (50°C)

No lateral wandering of the wheel was applied during testing. MMLS3 testing commenced after heating the compacted slabs for at least 12 hours. Stoppages during MMLS3 testing were limited to 25 minutes between profilometer measurements.

TxDOT MMLS3 Testing Facility

The materials laboratory at TxDOT in Austin was used for this test. Researchers prepared the mixtures at TTI and hauled them to Austin for compaction and testing.

The MMLS3 tests were conducted in a temperature-controlled (environmental) chamber specifically built for the device. The chamber allows testing to be performed at temperatures ranging from 32 to 140°F (0 to 60°C). The chamber contains an overhead hoist for positioning the MMLS3 and a laboratory compaction roller.

The compaction roller consists of guide rails, the mould, and a roller assembly. The mould consists of two stackable metal frames with inside dimensions of 36 inches wide × 72 inches long (920 mm × 2800 mm). The bottom section has a thick metal floor. The mould fits inside and is clamped to the guide rail assembly. The test pavement is constructed inside the mould. The roller assembly contains an 18-inch diameter × 36-inch wide (450 mm diameter x 900 mm wide) steel drum. The drum is mounted onto a framework with four steel wheels that run on the underside of the guide rails to provide a downward reaction force. The vertical position of the drum is adjustable by means of a hand crank. A separate hand crank through a chain and sprocket reduction system drives the drum.

The testing pad contained a semipermanent compacted HMA base, on which all four were placed and compacted. The thickness of the existing base layer was about 3 inches. Researchers raised the temperature of the environmental chamber to 50°C (122°F) and waited for 12 hours for temperature stabilization before starting the test.

Compaction of Test Pad

TTI prepared mixes for the MMLS3 tests in College Station and brought them to the TxDOT materials laboratory in Austin (a 2-hour drive) in sealed drums placed within an insulated "hotbox" for heat retention. The temperature loss of the mixtures in the drums was typically on the order of 68°F (20°C). The sealed drums were immediately placed in a large forced-draft oven, and the mixes were reheated to the appropriate compaction temperatures.

The same compaction procedure was adopted for each of the slabs. What follows is an outline of this procedure. The surface of the existing base within the mould was covered with roofing felt. The purpose of using the roofing felt was to protect the existing base layer during removal of the surface layer after testing. After removal of the surface layer, no visual distress was observed on the base layer. The roofing felt was painted with diluted emulsion tack coat on top surface only. After the HMA material for the test slabs had reached compaction temperature, it was placed within the MMLS3 test slab mould on top of the tacked roofing felt. A hoe and shovels were used to spread the asphalt evenly within the mould. One pass with the compaction roller was used to screed excess material and level the surface.

Breakdown compaction of the test slabs was performed manually using a gasoline-powered vibratory plate compactor (tamper). Density of the slabs was monitored intermittently during compaction using a nuclear density gauge that had been calibrated for each particular mix prior to compaction. After the target density had been achieved using the plate compactor, 5 to 10 passes with the roller compactor were applied to smooth the pavement surface. A fine water spray was used to prevent material from clinging to the drum. The final thickness of the compacted layer was 70 mm.

In some cases, researchers placed a neoprene sheet between the drum and the pavement surface to prevent transverse surface shear cracks during compaction when adequate density had not been reached using the vibratory plate compactor.

MMLS3 Test Setup

Figure 12 shows a schematic of the MMLS test setup. The asphalt slabs were compacted to a thickness of approximately 2.5 inches (62 mm) within the mould assembly. The MMLS3 was centered on top of the asphalt slab. The MMLS3 was not directly attached to the mould assembly to prevent the transmission of vibrations and external forces onto the test slab. As will be discussed later, the loading of the MMLS3 is uniform on a 3.28-ft (1 m) linear section centered on the test slab. Test measurements were therefore confined to the shaded test area indicated in Figure 12, although the MMLS tire track extends beyond this area.

Transverse profilometer measurements were taken at three locations centered 10 inches (250 mm) apart along the test area, as shown on Figure 12. These transverse profiles were designated A, B, and C in the direction of travel.

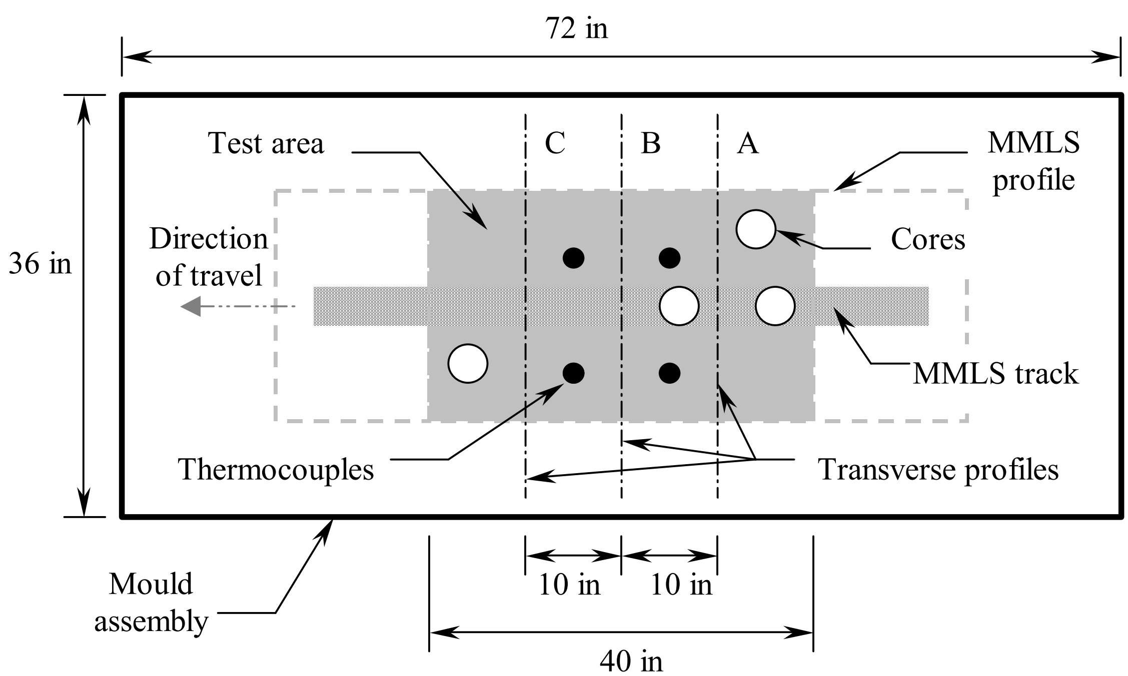

Figure 12. Schematic of MMLS3 Test Setup.

Figure 12 indicates the relative positions of thermocouples installed in the slab as well as possible positions of cores within and outside of the wheel track that were taken for density measurements after MMLS3 testing. The wheel track without application of the wandering function is about 3.15 inches (80 mm) wide.

Temperature Control

Each of the MMLS tests was performed within a temperature-controlled environmental chamber with the asphalt slabs heated to 122°F (50°C).

Researchers installed four bead probe K-type thermocouples within the asphalt slab to monitor temperature during the tests, two on the surface, one at a depth of 1 inch (25 mm), and another at a depth of 2 inches (50 mm) beneath the surface of the asphalt. These were glued to the slab surface using quick-drying epoxy. The temperature of the asphalt slab was monitored on a regular basis with adjustments made, if necessary.

Typically, the asphalt slabs required about 5 hours of heating before the temperature stabilized to 122°F. After initial profilometer measurements, if necessary, MMLS3 trafficking was delayed shortly to allow the pavement surface temperature to return to 122°F.

It should be noted that heating of the environmental chamber alone allowed the full depth of the asphalt slab to be heated to 122°F. The use of the surface plenum ducts was not necessary.

Profilometer Measurements and Rutting Definition

The profilometer rutting measurement system consists of a 3.28-ft (1 m) long sliding plate frame to which a vertical reader unit is attached. The frame fits into two aluminum extrusion tracks (guide rails) that are mounted on both sides of the mould configuration and that serve as the reference datum for the vertical rut measurements. The vertical reader is attached to a Mitutoyo KM/KC counter display unit, which in turn is connected to a laptop computer via the RS-232 port.

The vertical reader is free to move horizontally and incorporates an arm, which is free to move vertically, connected to a 2-inch (50 mm) diameter wheel that travels along the surface profile being measured. This allows profilometer measurements to be captured as a two-dimensional array (x, y) on the computer.

During an MMLS3 test, profilometer measurements were taken prior to MMLS3 trafficking and thereafter at specific intervals during trafficking to allow an accurate definition of the cumulative rutting curve. Since the maximum number of axles to be applied for this test series was limited to 100,000 per test, researchers decided to take readings after the following number of axles: 0, 1,000, 10,000, 50,000, and 100,000. Three transverse profilometer measurements were taken after each interval at locations described previously and indicated in Figure 12.

Rutting profiles or surface elevations with trafficking are developed relative to the transverse profiles taken before MMLS3 trafficking (i.e., 0-axles reading). The maximum rut depths given in this report were determined by applying an imaginary straight edge over the maximum surface elevations and calculating the vertical distance to the lowest surface elevation, as shown in Figure 13.

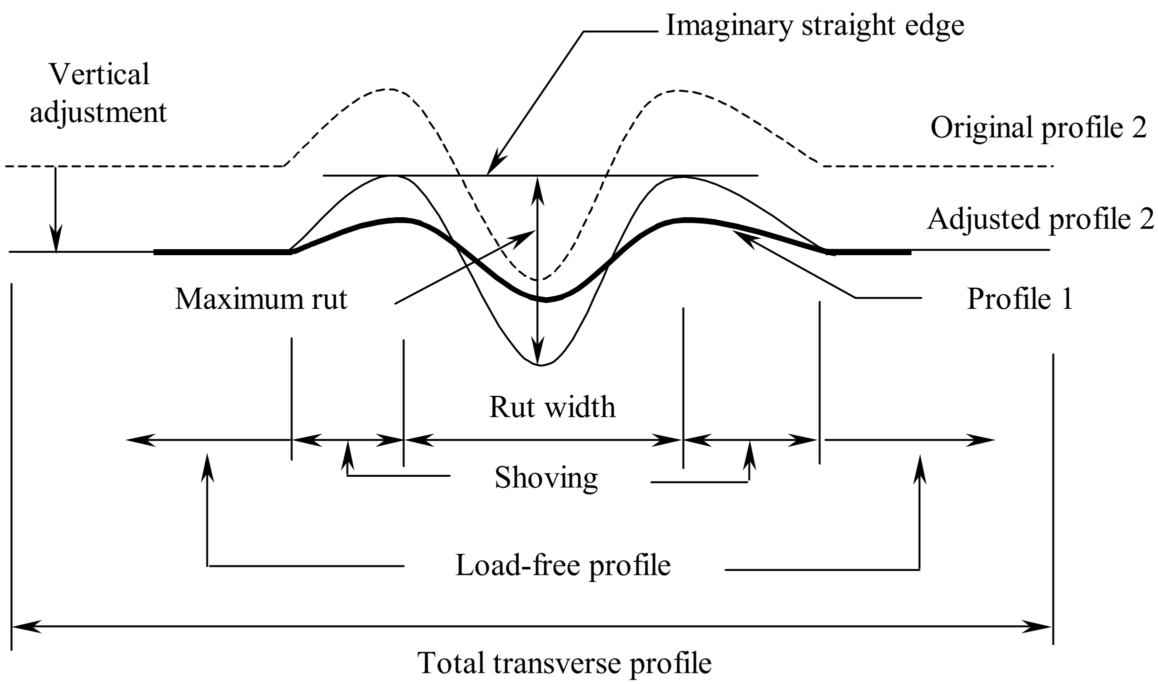

Figure 13. Adjustments to Vertical Transverse Surface Profiles.

When necessary, researchers made minor adjustments to the rutting profiles to account for errors resulting from temperature effects and misalignment. The profilometer measures a transverse surface profile across a total width of about 25 inches (750 mm). The rutting profile is centered across 4 to 5 inches (80 to 100 mm) on this total profile. Shoving is usually apparent at the edges of the rut extending about 6-8 inches (150-200 mm), as shown in Figure 13. The profile beyond the shoving range is not influenced by loading and may be used as a reference for adjusting vertical deviations. The difference in the sum of squares of two load-free profiles is minimized to adjust vertical profiles.

The profilometer (Figure 14) used for the rutting measurements is accurate to 0.008 inches (0.2 mm) in the horizontal and vertical directions.

Figure 14. Profilometer Used to Measure MMLS3 Rut Depth.

Hamburg Wheel Tracking Device

Unlike the other laboratory-scale rut testers, the HWTD test is always conducted in submerged condition. Besides many other properties, this device provides the moisture susceptibility information of the HMA mixture. Researchers tested all four mixtures using two different HWTDs following TxDOT specification Tex-242-F.

Researchers conducted HWTD tests at the TxDOT laboratory at Austin (Figure 15). Accordingly, specimens for each mixture were compacted and saw cut at the TTI laboratory and tested at the TxDOT facility. These HWTD tests were repeated near the end of the project at the TTI laboratory when the new TTI machine was installed. Precision Metal Works (PMW), Salina, Kansas, manufactured both machines.

Four specimens of each of the four mixtures were compacted using the Superpave gyratory compactor. Cylindrical specimens were compacted following the AASHTO TP4 standard. The specimens are 6 inches (150 mm) in diameter and 2.5 inches (63 mm) high. One cylindrical side of each specimen was trimmed slightly so that two specimens together form a set and fit into the mold (Figure 15). The sample setup is shown in Figure 16. The measured air void content of each specimen was within 7 ± 1 percent. A circulating water pump and a water heater keeps the water temperature constant at 122 ± 2°F (50 ± 1°C) throughout the test. The HWTD is programmed to start 30 minutes after the water temperature reaches the desired level. This 30-minute time period is designed to precondition the submerged specimens. The machine

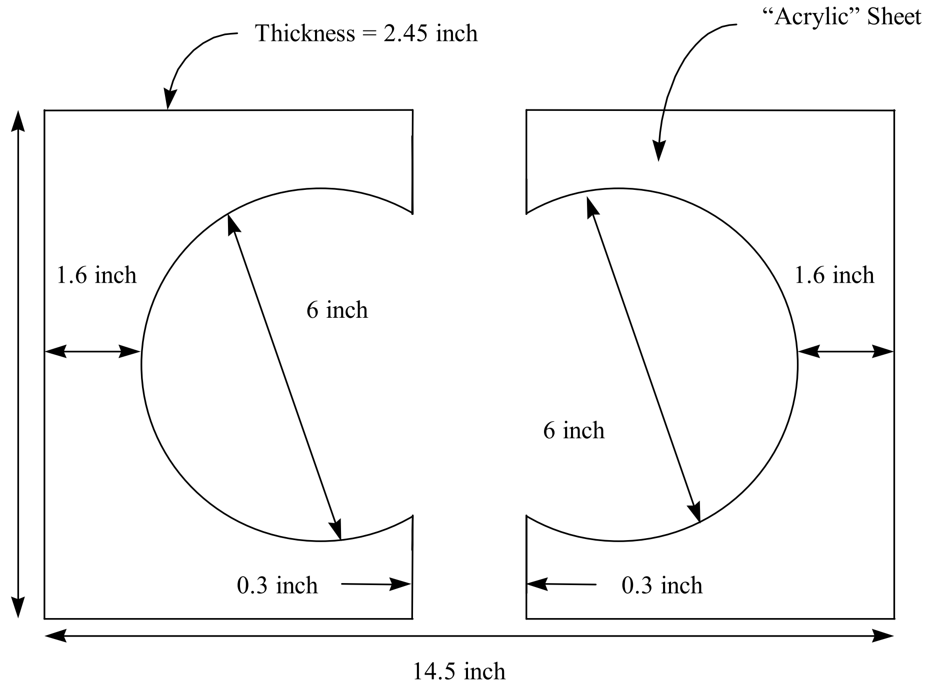

Figure 15. Hamburg Wheel Tracking Device Mold.

Figure 16. HWTD Loaded with Specimens.

was set to terminate the test at 20,000 cycles or 0.5 inches (12.5 mm) of rut depth, whichever occurs first. It is noteworthy that one forward and one backward pass of wheel comprise two cycles. All the tests were conducted at the rate of 52 cycles/minute.

Table 9 summarizes the test conditions and specimen description for all mixtures.

Table 9. Test Condition and Specimen Description for the Different Mixtures.

Mixture Type	Test Identification	Asphalt Content	Number of Specimens Tested	Air Voids (%)	Test Temperature (°F)
Limestone Type-C	FSCH	Design	3	7	39, 68, and 104
	SSCH	Design	3	7	39, 68, and 104
	RSCH	Design	3	3	131
	RSCSR	Design	3	3	131
	APA	Design	6	4	147
	Hamburg	Design	4	7	122
	MMLS-3	Design	One Slab*	7	122
River Gravel	FSCH	Design	3	7	39, 68, and 104
	SSCH	Design	3	7	39, 68, and 104
	RSCH	Design	3	3	131
	RSCSR	Design	3	3	131
	APA	Design	6	4	147
	Hamburg	Design	4	7	122
	MMLS-3	Design	One Slab*	7	122
Granite SMA	FSCH	Design	3	7	39, 68, and 104
	SSCH	Design	3	7	39, 68, and 104
	RSCH	Design	3	3	131
	RSCSR	Design	3	3	131
	APA	Design	6	4	147
	Hamburg	Design	4	7	122
	MMLS-3	Design	One Slab*	7	122
Granite Superpave	FSCH	Design	3	7	39, 68, and 104
	SSCH	Design	3	7	39, 68, and 104
	RSCH	Design	3	3	131
	RSCSR	Design	3	3	131
	APA	Design	6	4	147
	Hamburg	Design	4	7	122
	MMLS-3	Design	One Slab*	7	122

* Readings were taken at three different locations

CHAPTER 4: RESULTS AND DISCUSSIONS

GENERAL

Researchers tested four different asphalt mixtures using four different types of SST testing and three different types of accelerated loading wheel tests to evaluate their permanent deformation resistance characteristics.

While conducting the tests, researchers were careful to follow the appropriate test standards. Although tests were performed in different time periods of the project, researchers always tested freshly made specimens to avoid the effects of aging. The results of each test are described separately.

SUPERPAVE SHEAR TESTER

The four selected HMA mixtures were tested using the SST machine following four different test protocols. Details of the test procedures are presented in Chapter 3. The following paragraphs describe the SST results.

Frequency Sweep at Constant Height

Researchers conducted the FSCH test at three different temperatures and on three specimens from each mixture. Specimens were tested at the lower temperature and then at the higher temperature to minimize the damage caused by shearing strain. The frequency sweep test was performed using 10 different frequencies, starting at the higher frequency toward the lower. The test applies a repeated sinusoidal horizontal shear strain with peak amplitude of approximately ±0.005 percent and a variable axial stress to maintain constant specimen height. Shear strain is applied at different frequencies, including 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. The specified strain level was selected during the SHRP program to ensure that the viscoelastic response of the asphalt mixture is within the linear range. This means that the ratio of stress to strain is a function of loading time and not of the stress magnitude. In some cases within this range of frequencies, it has been observed that at the high and low frequencies the behavior becomes nonlinear. Huber (*42*) shows that the dynamic shear modulus (ratio of stress to strain) of asphalt cement is approximately linear between the frequency range of 0.01 to 10 Hz.

Before testing, the specimens were preconditioned by applying a controlled sinusoidal shear strain at a frequency of 10 Hz for 100 cycles and peak-to-peak amplitude of 0.0001 mm/mm. A detailed description of this test method is given in AASHTO TP7, Procedure A (*41*) and Superpave Asphalt Mixture Analysis: Lab Notes (*43*).

Axial deformation, shear deformation, axial load, shear load, and temperature at each of the 10 different frequencies were recorded. The data obtained from the FSCH test were used to calculate two properties: complex shear modulus (with its real and imaginary parts) and phase angle.

Complex Shear Modulus

Raw data recorded from SST machine were fed into ATS software to calculate the complex shear modulus (CSM) and shear phase angle (SPA) for each frequency. Figures 17, 18, and 19 show the mean complex shear modulus of all mixtures plotted against the logarithm of frequency and tested at 39, 68, and 104°F, respectively.

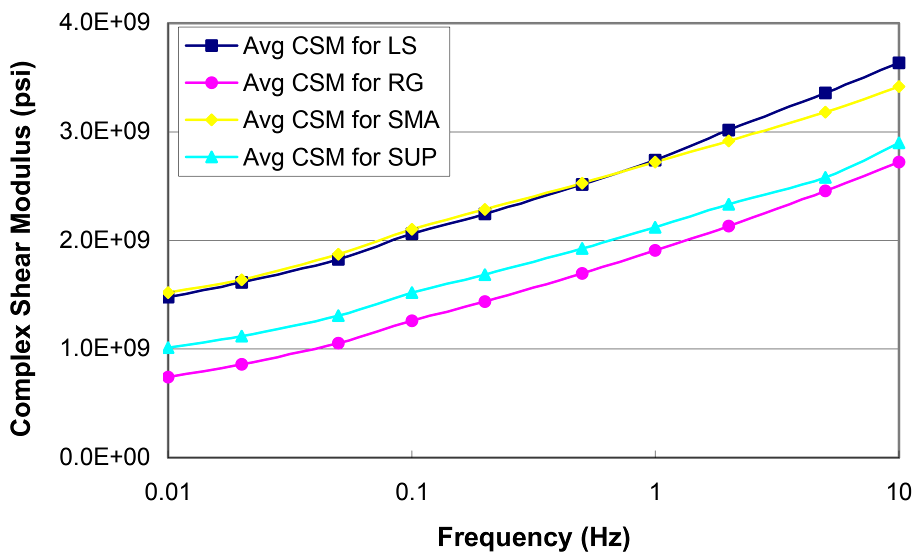

Figure 17. Complex Shear Modulus at 39°F.

Figure 17 illustrates that CSM increases linearly with the increasing frequency. The CSMs for the limestone and SMA mixtures remained very close throughout the frequency range. The river gravel mixture consistently yielded the lowest CSM. At this low temperature (39°F), the difference between the CSMs for the different mixtures appears the same, regardless of frequency.

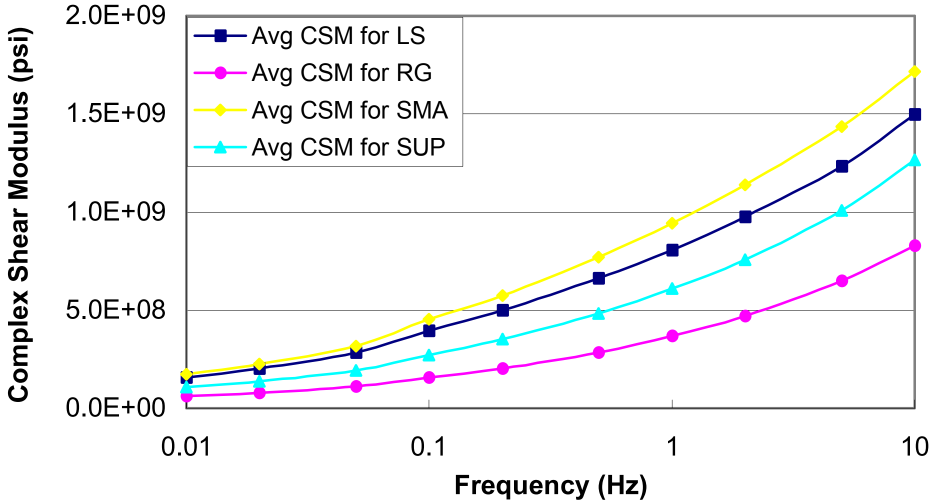

Figure 18. Complex Shear Modulus at 68°F.

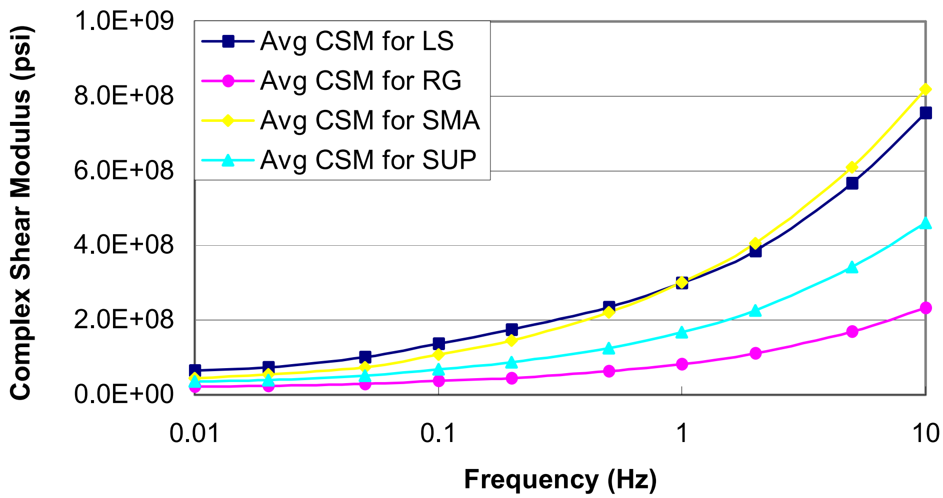

Figure 19. Complex Shear Modulus at 104°F.

In Figure 18, the CSM values increase significantly with frequency. The complex shear moduli of all mixtures are similar at lower frequencies, and they diverge exponentially at higher frequencies. The granite SMA mixture shows the highest CSM for all frequency ranges, followed by limestone mixture. In Figure 19, the granite SMA mixture shows a marginally higher CSM than limestone mixture at frequencies above 1 Hz. The position of granite Superpave and river gravel mixtures are distinct in both Figures 18 and 19.

Shear Phase Angle

Shear phase angle is defined as the lag time between the application of a stress and the corresponding strain. SPA was calculated using the ATS software (*44*). Figures 20, 21, and 22

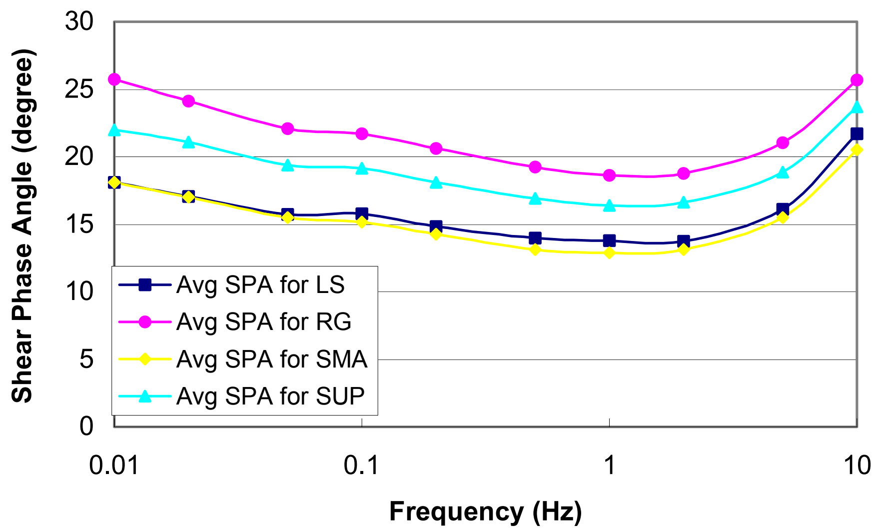

Figure 20. Shear Phase Angle versus Frequency at 39°F.

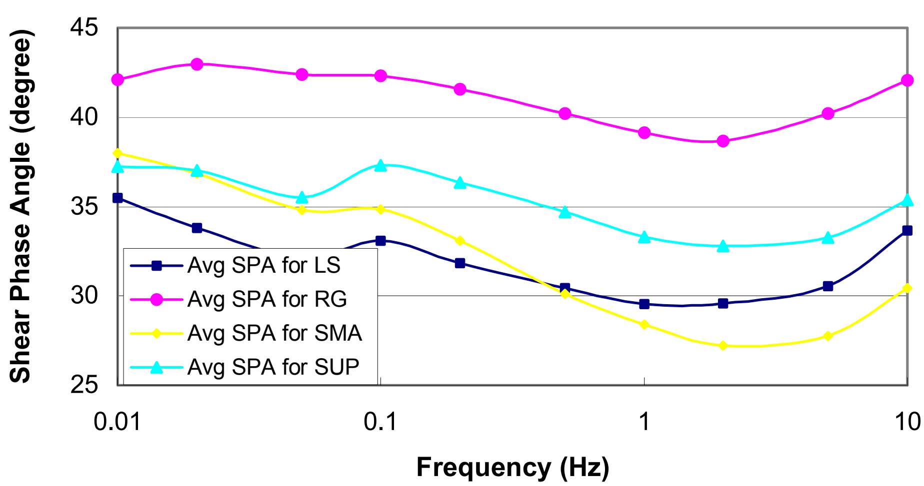

Figure 21. Shear Phase Angle versus Frequency at 68°F.

present the mean SPA from tests at 39, 68, and 104°F, respectively. At 39°F, the SPA of each mixture follows a similar trend. In Figure 20, the limestone and granite SMA mixtures show lowest SPA. River gravel always yields highest SPA, and granite Superpave is near the center of the plots.

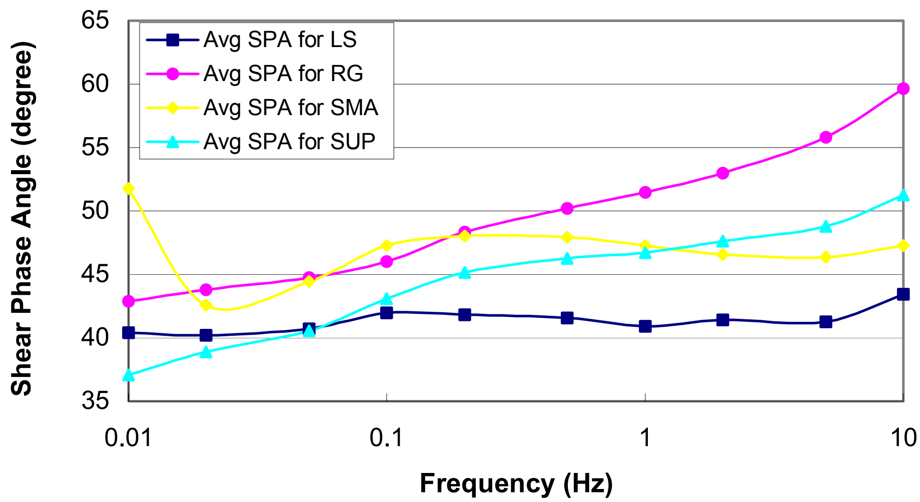

Figure 22. Shear Phase Angle versus Frequency at 104°F.

In Figures 21 and 22, the trends are not consistent but the river gravel mixture generally exhibits the highest angle, while the limestone and granite SMA yield the lower range of SPA.

Sometimes, the G*/sin δ term is used to estimate the rutting susceptibility of HMA mixtures. Since SPA (δ) was not very consistent, researchers did not pursue plotting G*/sin δ.

Simple Shear at Constant Height

The SSCH test was performed at three different stress levels and test temperatures (Table 10). Shear stress was applied at a rate of 10.15 ± 0.72 psi (70 ± 5 kPa) per s up to the stress level indicated in Table 10. The stress level was maintained for 10 s and then reduced to zero at a rate of 3.62 psi/s (25 kPa/s). As the specimen is sheared, it tries to dilate (increase in height). A variable (controlled by axial LVDT feedback) axial load was applied to maintain a constant specimen height (±0.0005 inch).

All specimens were preconditioned for 100 cycles with a shear stress having a peak magnitude of approximately 1.0 psi (6.9 kPa). Each cycle has duration of 0.7 s, consisting of a 0.1-s loading period followed by a 0.6-s rest period in a haversine wave form.

Table 10. Stress Level Applied in the SSCH Test.

Test Temperature, °F (°C)	Shear Stress, psi (kPa)
39 (4)	50 (345)
68 (20)	15.2 (105)
104 (40)	5.1 (35)

The SSCH was performed after the FSCH test using the same specimens. The tests at the lowest temperatures were performed first. A detailed description of this test method is provided in AASHTO TP7, Procedure B (*41*) and Superpave Asphalt Mixture Analysis: Lab Notes (*43*).

Researchers recorded the axial deformation, shear deformation, axial load, shear load, and temperature throughout the test. Material properties calculated from the recorded data were maximum shear strain, permanent shear strain, and elastic recovery. Data were manually analyzed using a spreadsheet.

Maximum Shear Strain

Maximum shear strain usually occurs at the end of a steady applied load before it starts decreasing. This strain measurement indicates the total deformation susceptibility of the mixture. It is calculated using the following formula:

$$\gamma_{\max} = \frac{\delta_{\text{shear,maximum}} - \delta_{\text{shear,initial}}}{\text{h}},$$

where: $(_{max}$ = maximum shear strain,

$*_{shear, maximum}$ = maximum deformation recorded by the LVDT,

$*_{shear, initial}$ = initial shear deformation at the start of the test, and

h = gauge length, 1.5 inches (38 mm).

Permanent Shear Strain

Permanent shear strain is the deformation measured at the end of the test. It indicates the plastic deformation characteristics of the mixture. Permanent shear strain is calculated by subtracting the initial shear deformation from the final shear deformation and dividing that number by the gauge length of shear LVDT. SSCH test results are shown in Table 11.

Table 11. SSCH Test Result.

Measured Property	Test Temperature, °F (°C)		
	39 (4)	68 (20)	104 (40)
Limestone Mixture			
Maximum Strain (in/in)	0.00025267	0.00106227	0.00067080
Permanent Strain (in/in)	0.00004533	0.00052080	0.00036080
Elastic Recovery (%)	82.31	50.55	46.07
River Gravel Mixture			
Maximum Strain (in/in)	0.00068613	0.00248120	0.00403800
Permanent Strain (in/in)	0.00026560	0.00146107	0.00279660
Elastic Recovery (%)	62.40	41.16	30.40
Granite SMA			
Maximum Strain (in/in)	0.00023040	0.00066187	0.00104267
Permanent Strain (in/in)	0.00004360	0.00035933	0.00062200
Elastic Recovery (%)	81.32	51.87	40.73
Granite Superpave			
Maximum Strain (in/in)	0.00041227	0.00160720	0.00153347
Permanent Strain (in/in)	0.00013133	0.00102280	0.00097600
Elastic Recovery (%)	68.91	38.20	36.57

Elastic Recovery

When the load imposed on the specimen is withdrawn, the specimen partially rebounds to its original shape. This phenomenon is due to the elastic properties of the mixture. More elastic recovery indicates better resistance to permanent deformation. Elastic recovery is calculated using the following equation.

$$\text{Recovery} = \frac{\delta_{\text{shear,maximum}} - \delta_{\text{shear,final}}}{\delta_{\text{shear,final}}} \times 100,$$

Where: Recovery = percentage of shear deformation recovered after the removal of load,

$*_{\text{shear,maximum}}$ = maximum recorded deformation by the shear LVDT, and

$*_{\text{shear,final}}$ = final recorded shear deformation at the end of the test.

Figures 23, 24, and 25 present the maximum shear strain, permanent shear strain, and elastic recovery, respectively. In these figures, the mixture designation is abbreviated. For example, SM104 means granite SMA mixture tested at 104°F. Clearly, the river gravel mixture demonstrates the highest maximum shear strain and permanent shear strain at all temperatures.

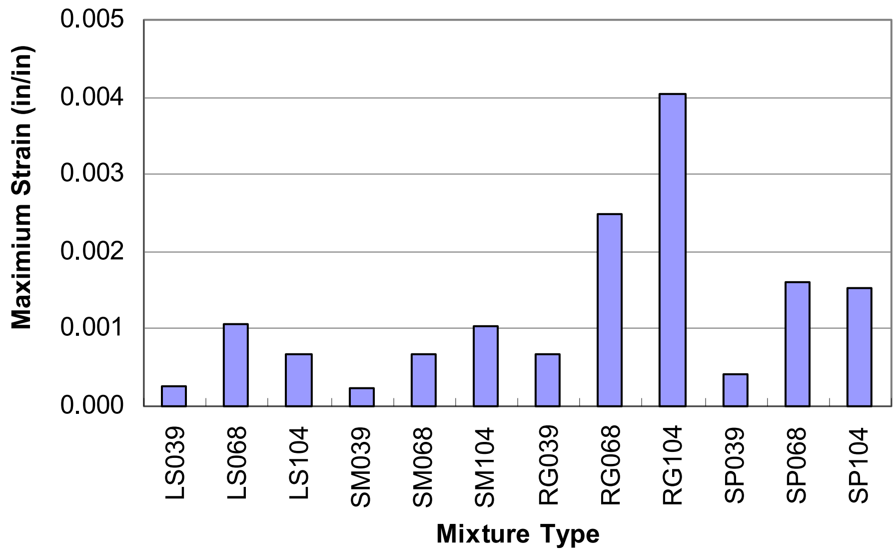

Figure 23. Maximum Shear Strain for Different Mixtures.

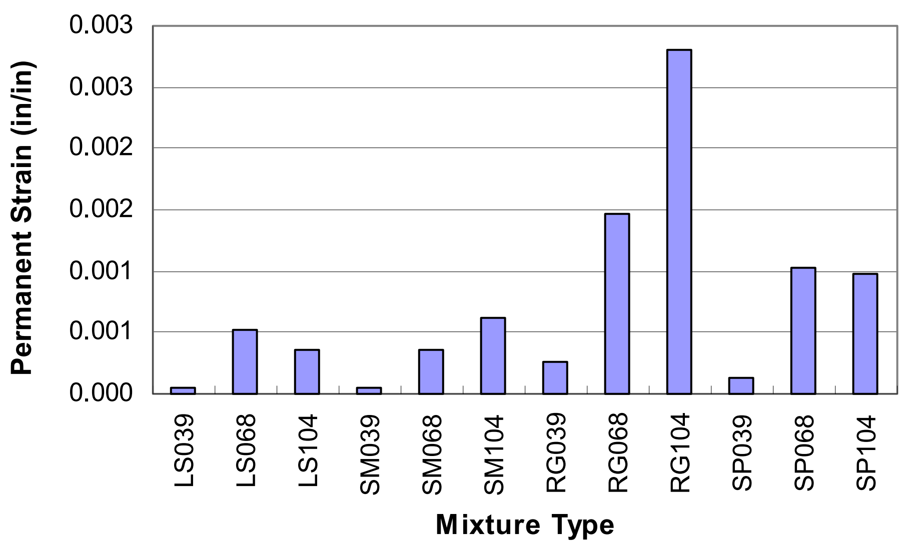

Figure 24. Permanenet Shear Strain for Different Mixtures.

Elastic recoveries at the lower test temperature are higher than those at the higher temperature for every mixture. Elastic recovery for the different mixtures tested at the same temperature does not vary appreciably.

Considering results at the same temperature, the limestone mixture and granite SMA mixture are very similar with respect to the three properties measured by SSCH test. The granite Superpave mixture yielded higher maximum strain and permanent strain than the limestone and granite SMA mixtures.

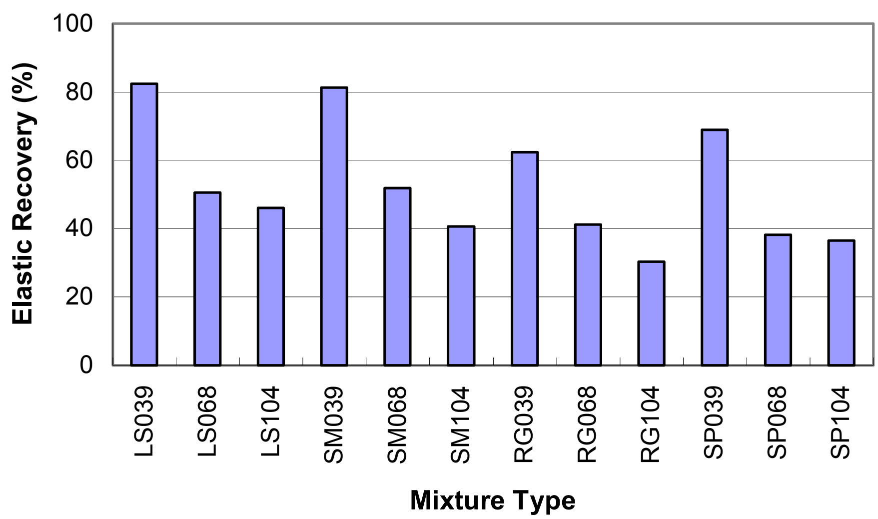

Figure 25. Elastic Recovery of Different Mixtures.

Repeated Shear at Constant Height

In the RSCH test, repeated haversine shear load pulses (10 psi [69 kPa]) were applied to the specimen. When the shear load is applied, the test specimen tends to dilate. To prevent vertical dilation, a controlled axial load is applied to keep the specimen at a constant height. The loading cycle requires 0.7 s, wherein a 0.1-s load is followed by 0.6-s rest period. This test was performed at the design asphalt content. Specimens containing 3 percent air voids were used to increase their sensitivity to tertiary rutting. Air voids of 3 percent were achieved by applying more gyrations during the compaction process.

Before testing, the specimens were preconditioned by applying 100 cycles of a haversine shear load with a peak magnitude of 1 psi (6.9 kPa). After preconditioning, the specimens were subjected to 10,000 load cycles at a temperature of 131°F (55°C) in accordance with the ATS Manual, Version 3.1 (*44*) and AASHTO TP7, Procedure C (*41*). Chapter 3 describes determination of test temperature.

The permanent shearing strain after 10,000 cycles of each specimen and their average is depicted in Figure 26. Permanent shearing strain was calculated from the recorded data using the following equation.

$$\gamma_p = \frac{\delta_{shear,final} - \delta_{shear,initial}}{h},$$

where, γ_p = permanent shear strain,

$\delta_{shear,final}$ = final deformation recorded by the shear LVDT at the end of the test (10,000 cycles),

$\delta_{shear,initial}$ = initial deformation at the start of the test (nominally zero), and

h = the gauge length of shear LVDT, 1.5 inch (38 mm).

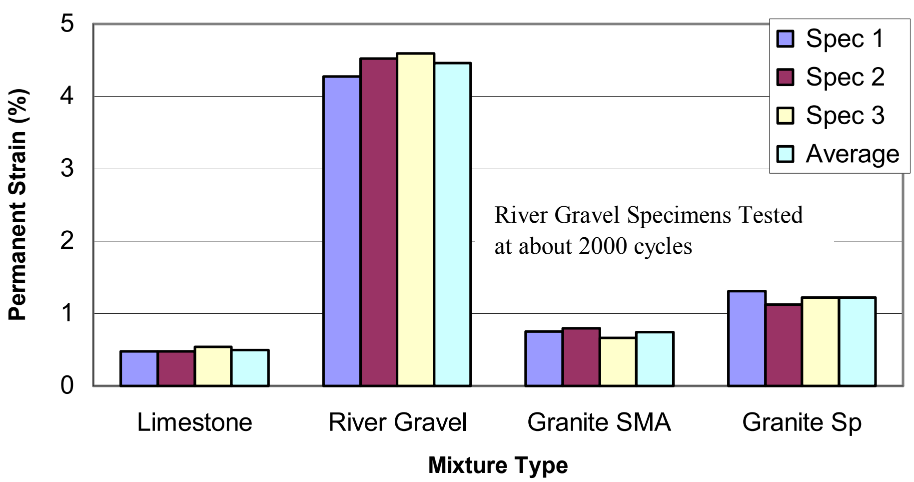

Figure 26. Permanent Shear Strain from RSCH Test.

The river gravel mixture experienced substantial accumulated shear strain. The test was set up for 10,000 cycles or 5 percent accumulated strain, whichever comes first. Except for the river gravel mixture, each specimen from all other mixtures completed 10,000 load cycles. River gravel specimens survived only about 2000 cycles before failure. The bar chart (Figure 26) shows that the limestone mixture yielded lowest strain. The granite SMA yielded higher strains than limestone, and granite Superpave yielded higher strains than granite SMA.

Repeated Shear at Constant Stress Ratio

The shear stress and axial stress selected correspond to a strong base condition and was 14.2 psi (98 kPa) and 21.45 psi (148 kPa), respectively (*9*, *43*). This test was performed at the design asphalt content, but at 3 percent air voids to enhance tertiary rutting

All specimens were preconditioned by applying 100 cycles of shear load pulses with a peak magnitude of 1 psi (6.9 kPa) and corresponding axial loads. After preconditioning the specimens, the repeated shear test was initiated. A detailed description of this test method is given in AASHTO TP7, Procedure C (*5*). The test was conducted at 131°F (55°C) and was set to apply a maximum of 10,000 load cycles or 5 percent accumulated permanent shear strain, which ever comes first. Permanent shear strain was calculated using the following equation.

$$\gamma_{\mathrm{p}} = \frac{\delta_{\mathrm{shear,final}} - \delta_{\mathrm{shear,initial}}}{\mathrm{h}},$$

where, $(_{\mathrm{p}}$ = permanent shear strain,

$*_{\mathrm{shear,final}}$ = final deformation recorded by the shear LVDT at the end of the test (10,000 cycles),

$*_{\mathrm{shear,initial}}$ = initial deformation at the start of the test (nominally zero), and

h = gauge length of shear LVDT, 1.5 inch (38 mm).

Figure 27 depicts the accumulated permanent shear strain at the end of the test. Like the RSCH test, the river gravel mixture did not survive 10,000 load cycles. The specimens failed between 1200 and 2200 cycles. In this test, the granite SMA mixture yielded the lowest permanent strain. The limestone mixture yielded marginally higher strain than the granite Superpave mixture.

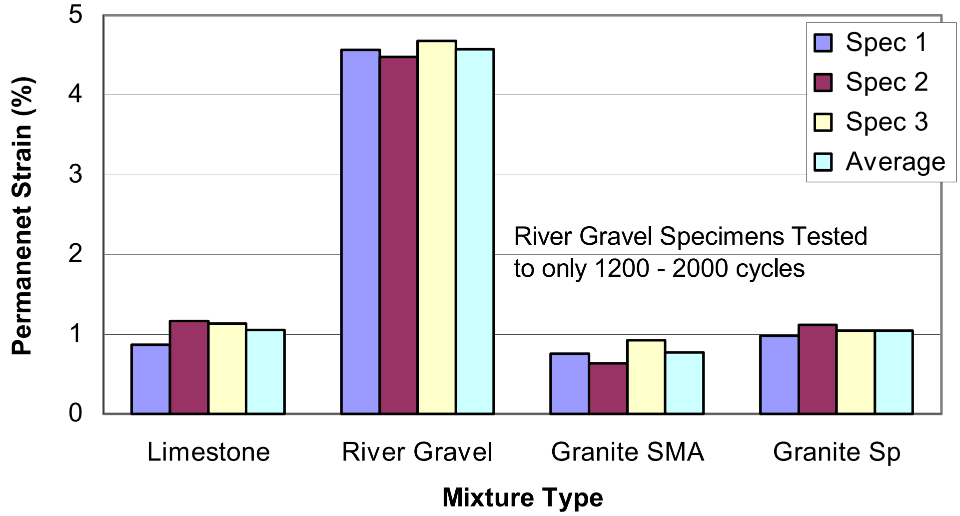

Figure 27. Permanent Shear Strain from RSCSR Test.

Mixture Rankings by SST Protocols

All SST test results were used to rank the mixtures regarding their resistance to permanent deformation. The FSCH test provides two material properties: complex shear modulus and shear phase angle. Higher complex modulus indicates higher rut resistance. Lower phase angle indicates more elastic behavior of the mixture and thus higher rut resistance. On the basis of the CSM and SPA values, researchers ranked the mixtures. A ranking of 1 means highest rut resistance and higher numbers indicate higher rut susceptibility. Sometimes, the values were so close that the same rank was given to more than one mixture. Table 12 exhibits the mixture rankings prepared from the FSCH test results. CSM and SPA were determined for each of the

Table 12. Mixture Rankings by FSCH Test.

Mixture Type	Test at 39°F			Test at 68°F			Test at 104°F		
	G* at 10 Hz	Overall[1] δ	Overall[1] G*	G* at 10 Hz	Overall[1] δ	Overall[1] G*	G* at 10 Hz	Overall[1] δ	Overall[1] G*
Limestone	1	1	1	2	1	2	2		1
River Gravel	4	4	4	4	4	4	4	4	4
Granite SMA	2	1	1	1	1	1	1	2	1
Granite Superpave	3	3	3	3	3	3	3	2	3

[1] Overall ranking was derived from the position of the plot.

10 frequencies and at three different temperatures. The CSMs were used in two ways to determine the rankings: CSM at 10 Hz and the CSM versus frequency graphs. The reason for using CSM at 10 Hz was that this frequency resembles that of highway traffic. Since the shear phase angle did not follow a consistent trend, the overall SPA (SPA versus frequency graph) was used for ranking.

Rankings of the river gravel mixture were very consistent, as shown in Table 12. The granite SMA appears to be the most rut resistant mixture, followed by the limestone and granite Superpave mixtures. Generally, the rankings from the FSCH test were quite consistent.

Table 13 provides mixture rankings prepared from the SSCH test results. Three material responses used in this ranking are: maximum shear strain (MS), permanent shear strain (PS), and elastic recovery (ER). Higher maximum shear strain and higher permanent shear strain both indicate of more rut susceptibility. On the other hand, higher elastic recovery indicates lower rut susceptibility. Ranking for SSCH at 39 and 68°F were quite consistent but changed a little at 104°F.

Table 13. Mixture Rankings by SSCH Test.

Mixture Type	Test at 39°F			Test at 68°F			Test at 104°F		
	MS	PS	ER	MS	PS	ER	MS	PS	ER
Limestone	2	2	1	2	2	1	1	1	1
River Gravel	4	4	4	4	4	2	4	4	2
Granite SMA	1	1	2	1	1	1	2	2	2
Granite Superpave	3	3	3	3	3	4	3	3	4

MS - Maximum Strain PS - Permanent Strain ER - Elastic Recovery

Table 14 presents mixture rankings from RSCH and RSCSR test results. In both tests, accumulated permanent shearing strain at the end of the test was used to rank the mixtures. The limestone mixture performed best in the RSCH test, and granite SMA performed best in the RSCSR test. Performance of the limestone and granite Superpave mixtures in the RSCSR test was very similar. The performances of the river gravel mixture in both tests were the worst, as expected.

Table 14. Mixture Rankings by Repeated Shear Tests.

Type of Mixture	Ranking	
	RSCH	RSCSR
Limestone	1	2
River Gravel	4	4
Granite SMA	2	1
Granite Superpave	3	2

RESULTS FROM LOADED WHEEL TESTERS

Researchers conducted three different types of laboratory-scale accelerated loaded wheel tests on the four mixtures studied. The purpose of this testing was to compare the results with those from the SST. Although the wheel testers do not provide basic material properties, they have the ability to rank rutting susceptibility of HMA mixtures, as indicated in literature (*19*, *20*, *25*).

Asphalt Pavement Analyzer

The final rut depths measured by the APA for the four mixtures are given in Table 15 and Figure 28. Only the river gravel mixture could not withstand the complete 8000 cycles. The average rut depth experienced by the river gravel mixture was 0.71 inches at 6000 cycles. The test was stopped at that point because at that level of rutting, the wheel can no longer apply a full load due to excessive sagging of the pressure hose.

Table 15. Final Rut Depth Measured by APA.

Mixture Type	Rut Depth at 8000 Cycles (inch)			Average Rut Depth (inch)
	Left Set	Middle Set	Right Set	
Limestone	0.211	0.188	0.174	0.191
River Gravel *	0.783	0.673	0.666	0.707
Granite SMA	0.136	0.113	0.105	0.118
Granite Superpave	0.150	0.150	0.179	0.159

*at about 6000 cycles

The granite SMA mixture performed the best in the APA test, followed by granite Superpave and limestone mixtures. Figure 29 exhibits the mean cumulative rut depth of all mixtures.

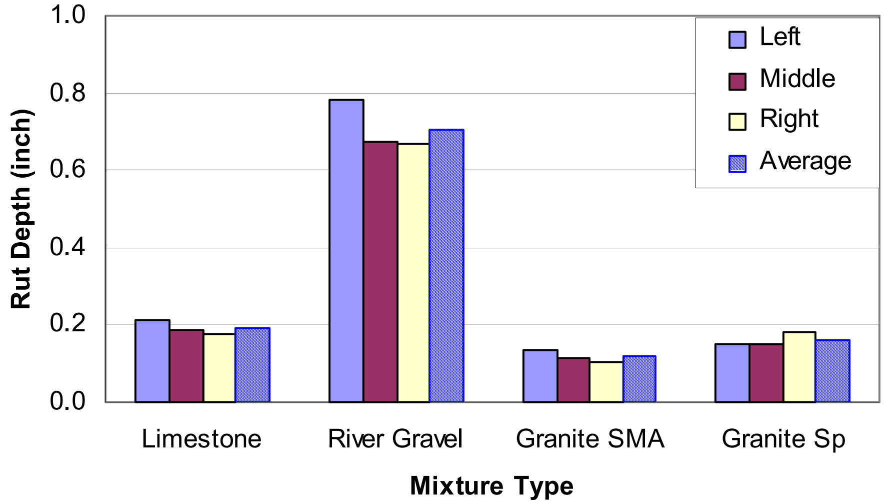

Figure 28. Rut Depth Measured by APA.

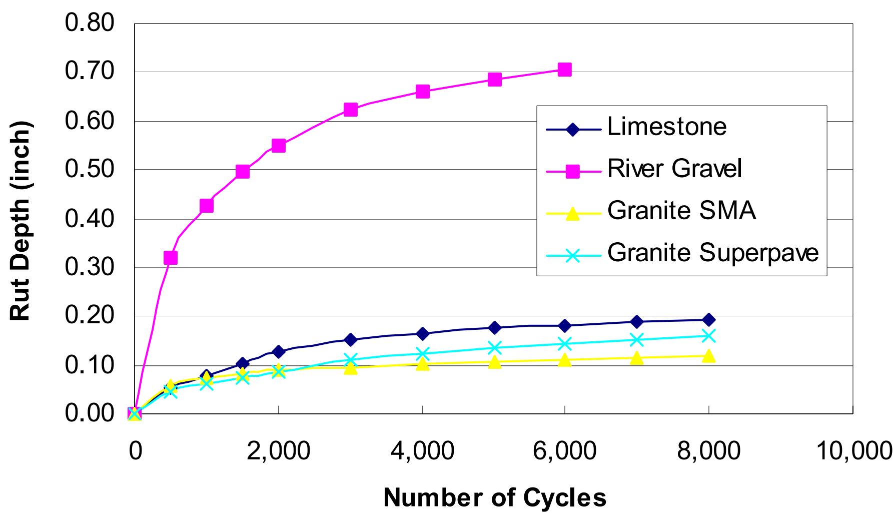

Figure 29. Comparison of Mean Cummulative Rutting (APA).

Model Mobile Load Simulator – 1/3 Scale

Unlike the other loaded wheel testers, researchers measured rut depth from the MMLS3 manually. Details of the MMLS3 are presented in Chapter 3. Bulging of the HMA above the surface datum alongside the wheelpath is clearly apparent in the rutting profiles measured for each of the MMLS3 tests. The degree of bulging varies from about 0.02 inches (0.5 mm) for the granite SMA mixture to about 0.2 inches (5 mm) for the river gravel mixture. This indicates that some of the deformation occurred without a change in volume.

Figure 30 shows details of a MMLS3 rut measurement from the granite SMA. The rise or peak within the wheelpath is about 0.2 inches (0.5 mm) high and is the outline of an aggregate particle jutting out. It can be seen that initially (and even after 1000 axle loads), this particle was embedded but with additional trafficking, the mortar and material alongside the particle was displaced. Definition of maximum rut depth is clearly influenced by this irregular rut profile. For the purpose of this report, maximum rut depths were measured to the deepest trough within the rut path.

The intent was to terminate MMLS3 trafficking after the application of 100,000 axles or whenever a mix developed a 0.4 inches (0.10 mm) rut, whichever occurred first. Tests on the river gravel mixture were terminated prematurely due to excessive rutting, and 120,000 axles were applied to the granite SMA mixture, the test overrunning slightly.

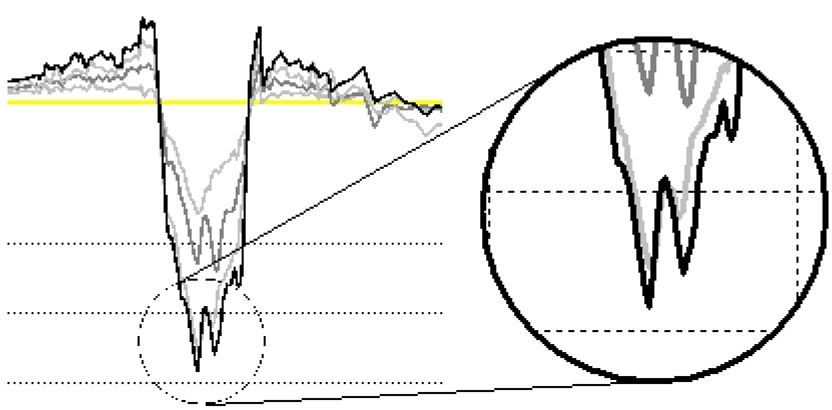

Figure 30. Zoom of a Typical MMLS3 Wheelpath Rut Measurement.

Limestone

Figure 31 shows cumulative rutting measured across the three transverse positions for the limestone mixture. Mean rut depth after the application of 100,000 MMLS3 axles was about 0.25 inches (6.4 mm). Deviation of the rutting across the three transverse positions increased with trafficking. Most of the rutting occurred in first 10,000 axles.

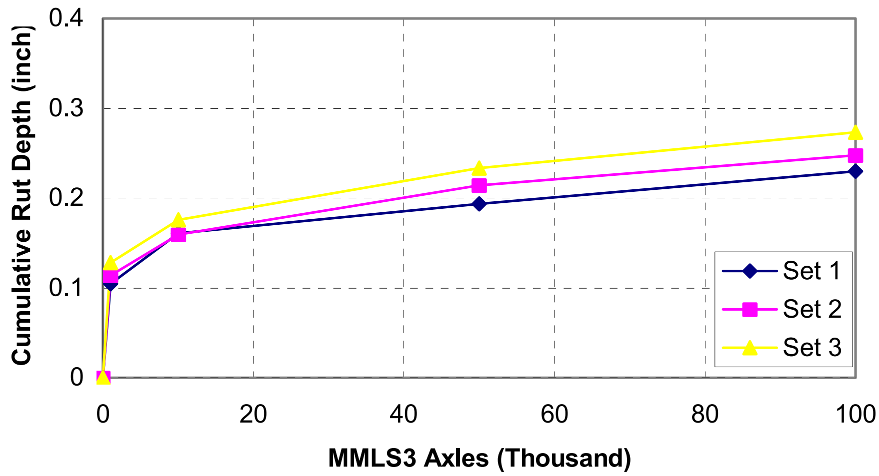

Figure 31. Cumulative Rutting on Limestone Mixture.

River Gravel

Figure 32 shows cumulative rutting measured across the three transverse test positions for the river gravel mixture. This test ended prematurely, with the material rutting in excess of 0.6 inches (15 mm) after only 5000 MMLS3 load applications. Anticipating the premature failure of this tender mixture, researchers recorded a set of readings at 100 axles.

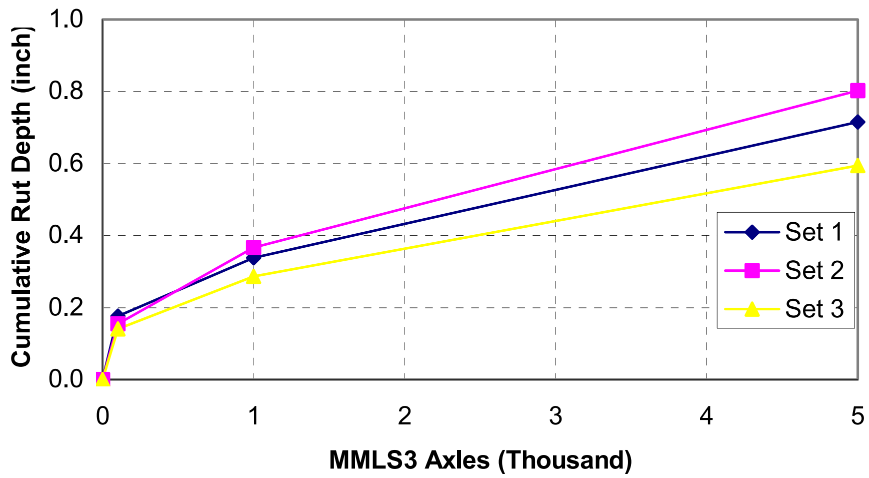

Figure 32. Cumulative Rutting on River Gravel Mixture.

Granite SMA

Figure 33 shows cumulative rutting measured across the three transverse positions for the granite SMA mixture. Mean rutting after the application of 120,000 MMLS3 axles was about 0.17 inches (4.2 mm). This mixture was tested 20,000 more axles due to a timing problem. The deviation of the rutting across the three transverse positions increased with trafficking.

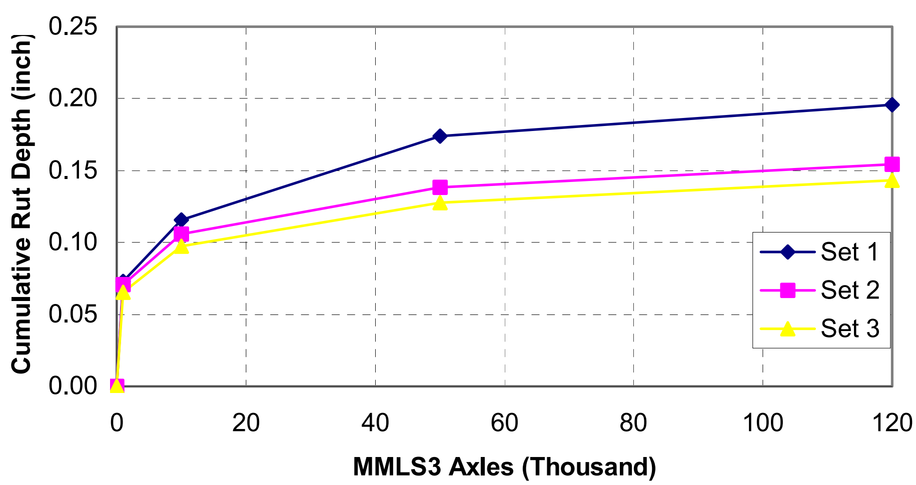

Figure 33. Cumulative Rutting on Granite SMA Mixture.

Granite Superpave

Figure 34 shows cumulative rutting measured across the three transverse positions for the granite Superpave mixture. Mean rutting after the application of 100,000 MMLS3 axles was on the order of 0.32 inches (8.1 mm). Like the other mixtures, the deviation for the rutting across the three transverse positions increased with trafficking.

Figure 35 compares the mean cumulative rutting for the four mixtures tested. In each of the tests, the rutting developed quickly, with more than 0.1 inches (2.5 mm) of rutting within the first 10,000 load applications. Based on Figure 34, the mixtures rank from best to worst as granite SMA, limestone, granite Superpave, and river gravel mixture.

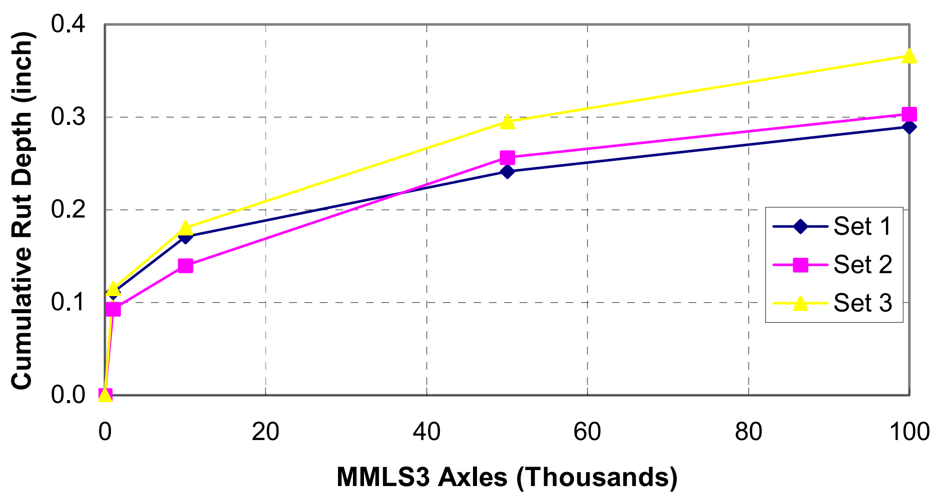

Figure 34. Cumulative Rutting on Granite Superpave Mixture.

Slab Density

The intention was to compact each of the slabs for the MMLS tests to the same density (93 percent of theoretical maximum density). But in practice, this was not possible due to the small size of the test slab and the compaction equipment available. Slab densities were monitored during compaction using a nuclear density gauge. After completion of MMLS rut testing, sections of each slab were removed and cores were cut from these sections (inside and outside of the trafficked wheel paths) to verify actual slab compaction and to evaluate the change in density (in the MMLS wheelpaths) with trafficking. The results are shown in Table 16.

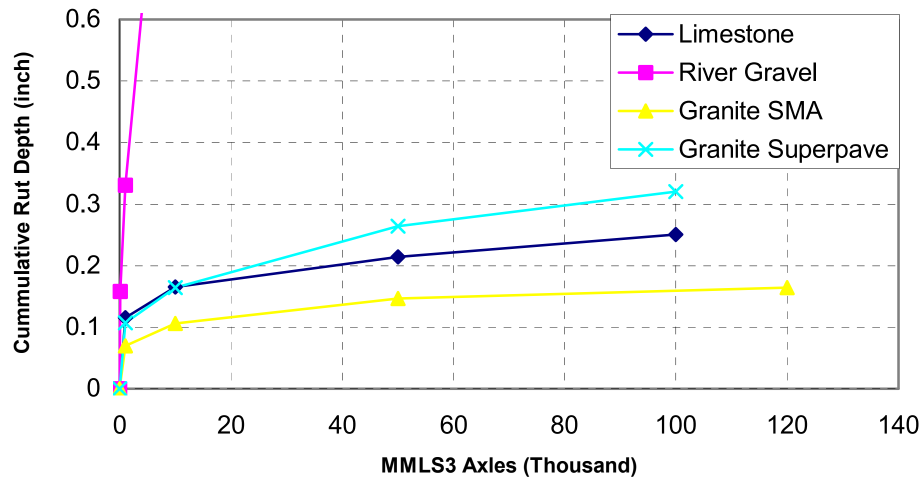

Figure 35. Comparison of Mean Cumulative Rutting for All Mixtures (MMLS3).

Table 16. MMLS3 Slab Density.

Mixture Type	Voids in the Mix					
	Outside Wheelpath			Inside Wheelpath		
	1	2	Avg.	1	2	Avg.
Limestone	8.11	7.50	7.81	5.64	5.11	5.38
River gravel*	3.55	3.68	3.62	2.85	2.89	2.87
Granite SMA	6.0	6.1	6.05	6.1	6.0	6.05
Granite Superpave	6.05	5.93	5.99	4.03	4.55	4.29

* Tested to only 5000 MMLS3 axles

There were significant differences in the initial compaction densities of the slabs before MMLS3 testing (Table 16). Air voids in the granite (SMA and Superpave) and limestone mixes were reasonably similar (7 ± 1 percent) but the river gravel mixture was over compacted. It is noteworthy that no change in the density of the SMA mix was apparent with trafficking. This is because the SMA design yields stone to stone contact of the angular coarse aggregate.

Hamburg Wheel Tracking Device

Due to late delivery of TTI-owned HWTD, researchers performed the test at the TxDOT laboratory in Austin. Accordingly, specimens for each mixture were compacted and saw cut at the TTI laboratory and tested at the TxDOT facility. Near the end of project, HWTD testing was repeated at the TTI laboratory. Precision Metal Works, Salina, Kansas manufactured both machines. This report focuses on the tests conducted at the TxDOT laboratory.

Four cylindrical specimens were prepared for each of the four mixtures following the AASHTO TP4 standard. The cylindrical side of each specimen was trimmed slightly so that two specimens together can form a set. The air void of each specimen was within 7 ± 1 percent. The HWTD water temperature was kept constant at 122 ± 2°F (50 ± 1°C) throughout the test. The machine was set to terminate the test at 20,000 cycles or 0.5 inches (12.5 mm) of rut depth, whichever occurred first. All tests were conducted at the rate of 52 cycles/minute.

Tables 17 and 18 present the HWTD test results from the TxDOT and TTI laboratories, respectively. Results from both labs are similar. Like all other tests, the river gravel mixture performed worst among the four mixtures. Granite SMA performed the best, followed by the granite Superpave. Figure 36 presents the rut depths for the four specimens as measured or calculated at 10,000 load cycles.

From Figure 37, it is evident that the premature failure of the limestone mixture is due to moisture damage. Test results on the limestone mixture tested at both labs suggest that significant stripping began at about 10,000 load cycles. Figure 37 does not suggest any stripping of river gravel mixture. The river gravel mixture demonstrated a linear rutting accumulation as function of load cycles. Aggregate from the river gravel specimen was not washed out. This mixture apparently failed early due to its low shear strength before any stripping occurred. The two granite mixtures showed no sign of stripping.

Table 17. Final Rut Depth Produced by HWTD at TxDOT.

Mixture Type	Left Wheel Path		Right Wheel Path		Average Rut Depth (inch)
	Number of Cycle	Rut Depth (inch)	Number of Cycles	Rut Depth (inch)	
Limestone	13,600*	0.500	12,650*	0.500	> 0.500
River Gravel	3,960*	0.500	3,510*	0.500	> 0.500
Granite SMA	20,000	0.167	20,000	0.124	0.146
Granite Superpave	20,000	0.318	20,000	0.335	0.327

* Stopped test early due to excessive deformation.

Table 18. Final Rut Depth Produced by HWTD at TTI.

Mixture Type	Left Wheel Path		Right Wheel Path		Average Rut Depth (inch)
	Number of Cycle	Rut Depth (inch)	Number of Cycles	Rut Depth (inch)	
Limestone	13,101	0.500	13,490	0.500	N/A
River Gravel*	2,219	0.500	2,320	0.500	N/A
Granite SMA	20,000	0.140	20,000	0.208	0.174
Granite Superpave	20,000	0.484	20,000	0.343	0.414

* Stopped test early due to excessive deformation.

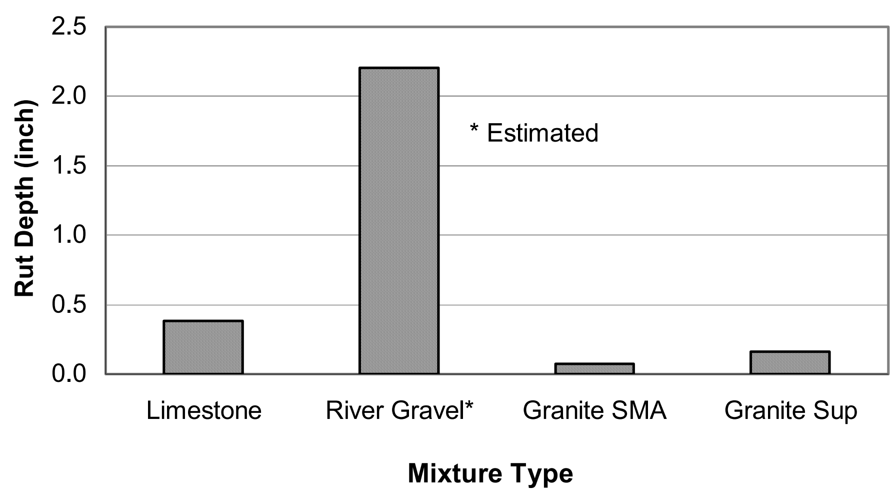

Figure 36. Rut Depth Produced by HWTD at 10,000 Cycles (TxDOT).

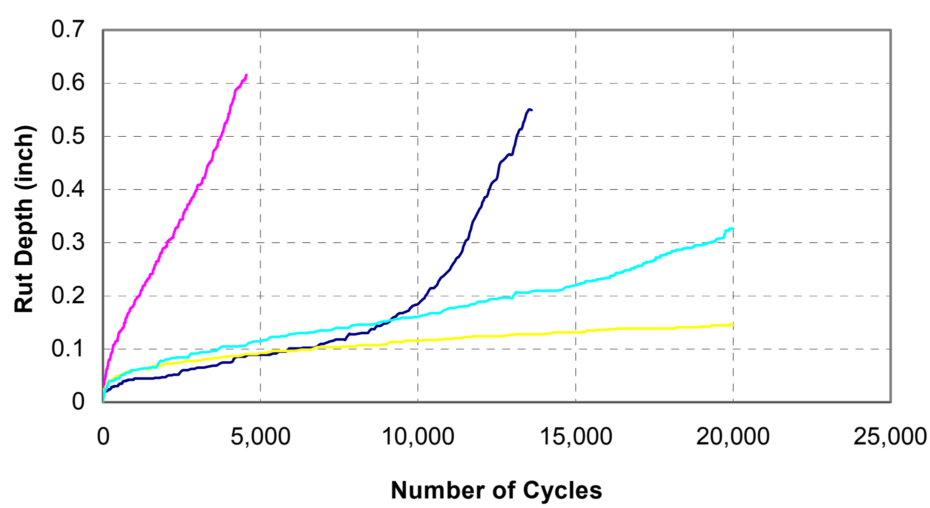

Figure 37. Comparison of Mean Cumulative Rutting (HWTD-TxDOT).

Mixture Rankings by Loaded Wheel Testers

Mixture rankings by the loaded wheel testers were necessary to compare with the rankings from the SST tests. Table 19 summarizes the mixture rankings from the three loaded wheel testers. The loading conditions, test environments, and test specimens utilized in the different LWTs are quite different from each other. Only the HWTD can discriminate the moisture susceptibility of the mixtures. The researchers believe that the HWTD produces the most severe test conditions. It therefore appears that the rutting performance measured by the HWTD is not directly comparable with the rutting from the other two tests. Although none of these wheel testers reproduce a real pavement situation, researchers believe that the MMLS3 probably best simulates actual pavement loading and response.

Table 19. Mixture Ranking by Accelerated Wheel Testers.

Type of Mixture	Ranking		
	APA	MMLS3	HWTD
Limestone	3	2	3
River Gravel	4	4	4
Granite SMA	1	1	1
Granite Superpave	2	3	2

SELECTION OF "BEST" SST PROTOCOL

The main objective of this research study was to identify which of the SST test protocols is best suited to predict the permanent deformation characteristics or shearing resistance of HMA paving mixtures. The "best" protocol will likely depend on the particular pavement structure; traffic load, intensity, and speed; climate; and characteristics of the mix itself. Therefore, it is particularly difficult to identify the single best protocol. Researchers had only subjective evaluations of the field performance of the four mixtures tested. Since all the four mixtures were not exposed to same field conditions (e.g., traffic, environment, construction variations, base condition), it was not possible to rank on the basis of field performance. Researchers heavily relied on results from the loaded wheel testers and other relevant factors. The overall mixture rankings made using different SST protocols were fairly consistent.

The ranking resulted from three loaded wheel testers were reasonably consistent. All three loaded wheel testers ranked the granite SMA and Type D river gravel mixtures as best and worst, respectively. The limestone mixture ranked third on the APA and HWTD tests and second on MMLS3 test, whereas the granite Superpave mixture ranked third on the MMLS3 test and second on the APA and HWTD tests. Among the loaded wheel testers, researchers consider the MMLS3 test most closely related to the actual field condition. There are several advantages of using the MMLS3 over other loaded wheel testers (APA and HWTD) used in this study. They are:

- applies load with loaded pneumatic tire (though smaller than vehicle tire),
- specimen shape, size, and compaction procedures are more similar to the roadway pavement,
- the elastic base underneath the slab more closely simulates actual pavement,
- wheel wandering is possible, and
- tire movement is in one direction.

Some of the drawbacks of MMLS3 are that, in this study, tests were performed only on dry surfaces, and rut depth reading was not continuous. It is possible to run the MMLS3 test in moist condition. In fact, recently, the authors were informed that researchers at the University of Stellenbosch in South Africa are conducting the MMLS3 tests on submerged specimens.

HWTD tests were performed only submerged under water using steel wheels. HWTD considers the moisture effect, but the constant presence and scouring effect of hot water questions the credibility of this device to rank only the rutting susceptibility of HMA. Both the APA and the HWTD apply bidirectional loads. The APA applies load through a pressurized hose and the HWTD applies the load with steel wheels. The width of the initial load applied by the APA is about the same as the diameter of the largest aggregate in a typical HMA mixture.

Reseachers calculated coefficients of variation (CV) of each parameter to determine the repeatability of different test results. Tables 20, 21, 22, and 23 present the coefficients of variation calculated from different SST and loaded wheel test results. Table 20 shows that shear phase angle yields relatively low CV when compared to complex modulus. The CV of complex shear modulus at high temperatures is lower than those at lower temperature. The CV of the

SSCH test results was very high (Table 21) when compared to the CV of the FSCH (Table 20), RSCH, and RSCSR (Table 22). The RSCH test yielded a relatively low CV with a fairly low range of values (Table 22). By comparison, the CV values from the RSCSR test were somewhat inconsistent in that they exhibited a notably higher range of values.

Table 20. CV of Mixture Properties Detrermined by the FSCH Test at 10 Hz Cycle.

Mixture Type	Coefficient of Variation					
	Test Temp, 39°F		Test Temp, 68°F		Test Temp, 104°F	
	CSM	SPA	CSM	SPA	CSM	SPA
Limestone	26.3	10.7	13.2	8.8	18.0	4.7
River Gravel	22.0	4.0	12.9	6.0	5.4	6.9
Granite SMA	11.0	5.5	8.4	2.9	0.4	0.4
Granite Superpave	15.5	2.6	12.1	3.5	13.1	3.7

CSM – Complex shear modulus, SPA- Shear phase angle

Table 21. CV of Mixture Properties Determined by the SSCH Test.

Mixture Type	Coefficient of Variation								
	Test Temp, 39°F			Test Temp, 68°F			Test Temp, 104°F		
	MS	PS	ER	MS	PS	ER	MS	PS	ER
Limestone	23.1	37.5	5.0	27.6	23.9	7.9	10.7	7.1	4.5
River Gravel	40.4	48.6	11.2	14.6	16.0	4.7	16.2	10.1	14.0
Granite SMA	18.0	31.8	3.5	72.0	97.0	27.7	18.0	23.0	10.4
Granite Superpave	29.0	42.2	6.1	42.9	50.2	16.9	17.0	20.0	5.1

MS – Maximum strain, PS – Permanent strain, ER – Elastic recovery

Table 22. CV of Mixture Properties Determined by the Two Repeated Shear Tests.

Mixture Type	Coefficient of Variation	
	RSCH – Permanent Strain	RSCSR – Permanent Strain
Limestone	7.3	15.7
River Gravel	3.7	2.2
Granite SMA	9.1	18.8
Granite Superpave	7.6	6.6

Table 23 presents the coefficients of variation of the test results from the loaded wheel testers. On the average, there was little difference in the CV from these three tests. However, the CV of the HWTD tests exhibited more inconsistency between the different materials. This is particularly due to the fact that the HWTD requires only two replicate tests, whereas, three measurements were used in the APA and MMLS3 tests.

Table 23. CV of Mixture Properties Determined by the Loaded Wheel Testers.

Mixture Type	Coefficient of Variation		
	APA	MMLS3	HWTD
Limestone	9.8	8.7	16.7
River Gravel	9.4	14.9	8.4
Granite SMA	13.4	16.9	20.8
Granite Superpave	10.3	12.8	3.7

CV determined based on the final rut depth.

To test the sensitivity of the test methods, Duncan's multiple range pair tests were conducted on all test results. The statistical analysis was performed at a 95 percent confidence level. Tables 24, 25, 26, and 27 present the Duncan groupings for the different SST and loaded wheel tests results. In these tables, test values at a given temperature with the same letters are not significantly different. Conversely, different letters indicate they are significantly different. In some cases, when the same mixture fell into more than one group, more than one letter (A, B or B, C) are used.

Table 24. Duncan Grouping of the FSCH Results.

Mixture Type	Duncan Grouping					
	Test at 39°F		Test at 68°F		Test at 104°F	
	CSM (G*)	SPA (δ)	CSM (G*)	SPA (δ)	CSM (G*)	SPA (δ)
Limestone	A	A, B	A, B	A, B	A	A
River Gravel	A	C	B	C	C	C
Granite SMA	A	A	A	A	A	A, B
Granite Superpave	A	B	B	B	B	B

Table 25. Duncan Grouping of the SSCH Results.

Mixture Type	Duncan Grouping								
	Test at 39°F			Test at 68°F			Test at 104°F		
	MS	PS	E R	M S	P S	E R	MS	PS	ER
Limestone	A	A	A	A, B	A, B	A	A	A	A
River Gravel	B	B	B	C	C	A	C	C	C
Granite SMA	A	A	A	A	A	A	A, B	A	A, B
Granite Superpave	A, B	A	B	B, C	B, C	A	B	B	B, C

MS – Maximum strain, PS – Permanent strain, ER – Elastic recovery

Table 26. Duncan Grouping of the Two Repeated Shear Test Results.

Mixture Type	Duncan Grouping	
	RSCH – Permanent Strain	RSCSR – Permanent Strain
Limestone	A	B
River Gravel	D	C
Granite SMA	B	A
Granite Superpave	C	B

Table 27. Duncan Grouping of the Loaded Wheel Tests Results.

Mixture Type	Duncan Grouping		
	APA	MMLS3	HWTD
Limestone	B	B	B
River Gravel	C	D	C
Granite SMA	A	A	A
Granite Superpave	A, B	C	A

Researchers selected the FSCH as the best suited SST protocol because:

- Mixture ranking by the FSCH test generally conforms to the ranking by the loaded wheel testers, particularly with that of the MMLS3.
- Both the FSCH and RSCH tests were good candidates for the "best" SST protocol.
- The FSCH test measures two fundamental material properties (complex shear modulus and shear phase angle), which can be tied with the pavement performance predictive model, whereas, the RSCH test measures permanent shear strain, which is not a fundamental material property.
- Material properties determined by the FSCH test can be utilized in both the rutting and fatigue predictive models.
- The FSCH test was strain controlled and thus provided better control during the test than the RSCH. That is, the measured mixture properties are not affected by the test parameters. Controlling strain to a low level minimizes the specimen damage more than stress control.
- Among the different parameters determined by the four SST tests, shear phase angle yields the lowest coefficient of variation.
- Mike Anderson from Asphalt Institute stated in the summary report of AASHTO TP7 revisions that a similar type of permanent strain is developed in repeated shear tests whether the test is conducted at a constant height (RSCH) or constant stress ratio (RSCSR). Results of this study support this basic idea even though the mixture rankings by the RSCH and RSCSR tests are slightly different. This suggests that conducting both of these repeated shear tests are redundant.

CHAPTER 5:
INTERLABORATORY TESTING

GENERAL

One of the tasks of this research study was to develop acceptance criteria for the “best” SST test protocol. The objective of this task was to develop acceptance values for the test property(s) of the best test. A part of this task was to conduct an interlaboratory experiment to determine the precision of the engineering test/property identified most suitable for predicting pavement performance. The main idea is to recommend to TxDOT a test protocol using the SST along with acceptance criteria for HMA mixtures that are suitable for use in a guide or specification. In the AASHTO Provisional Standard for Materials Testing, Interim Edition, April 2001, AASHTO suggested some major changes in the SST test protocols. However, they did not provide any precision or bias for any of the test values. Precision and bias are commonly obtained by interlaboratory testing program. This is a major task.

Researchers evaluated the permanent deformation susceptibility of four HMA mixtures using four SST test protocols and three laboratory-scale rutting tests. On the basis of those test results, Frequency Sweep at Constant Height was selected as the “best” SST test protocol. This is consistent with the findings from other agencies. The most important HMA property provided by the FSCH test is the complex shear modulus. The FSCH test also provides the shear phase angle. Soon after deciding the FSCH was the most suitable test, researchers initiated an interlaboratory testing program among several laboratories throughout the nation.

Other than TTI, there is no institution in Texas capable of conducting SST tests. Researchers contacted most agencies in the U.S. known to operate SST, including the Superpave centers, the Asphalt Institute, and FHWA. Among them, five organizations agreed to perform the FSCH test without any cost to the project. These organizations are:

- FHWA,
- the Asphalt Institute,
- North Central Superpave Center at Purdue University,
- Western Superpave Center at University of Nevada at Reno,
- Southeastern Superpave Center at Auburn University, and

- South Central Superpave Center (using Cox and Interlaken machines with different operators).

A few other qualified organizations did not commit due to their time constraints. The target was to involve at least six different laboratories, as recommended by ASTM E691-99, Standard Practice for Conducting an Interlaboratory Study to Determine the Precision of a Test Method. The South Central Superpave Center, located at TTI, performed the FSCH test using the Interlaken SST and the Cox and Sons SST machines. Different technicians performed the tests on the different machines to qualify as separate laboratories.

Due to the time limitation of the participating laboratories, researchers decided to send three specimens from each of three mixtures. The mixtures selected for this part of the study were Type C limestone, Type D river gravel, and granite SMA. Researchers further decided to conduct the FSCH test at only two temperatures: 68°F (20°C) and 104°F (40°C). There are two reasons for eliminating the tests at 39°F (4°C). Test results show that the FSCH test at that low temperature is not sensitive enough to discriminate between shearing strength of different HMA mixtures. Moreover, researchers from the other participating laboratories complained about the time required to stabilize the temperature at 39°F (4°C).

Three specimens from each of the three mixtures were prepared according to AASHTO TP4 and AASHTO TP7-01. Participating laboratories were instructed to conduct the FSCH test following the most recent AASHTO TP7-01.

The findings were summarized in Report FHWA/TX-04/1819-2, "Precision Statistics for Frequency Sweep at Constant Height Test." A precision statement was included for SST FSCH test was included in that report. Practice ASTM E 691-99 was followed during this task.

The FSCH test method has no bias because the values determined are defined only in terms of the test method. In other words, there are no standard values with which the results of this test can be compared, so it is not possible to establish bias.

CHAPTER 6:
CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Four asphalt mixtures selected at the beginning of the study were tested to measure their shearing resistance. Researchers evaluated the shearing resistance of the mixtures using four different SST protocols. The same mixtures were tested with three laboratory-scale loaded wheel testers to estimate their rutting susceptibility. Results from SST test protocols were compared with results from the loaded wheel testers.

CONCLUSIONS

On the basis of the test results and discussions, several conclusions were drawn. The conclusions are divided into two subcategories.

Superpave Shear Testing

Specimens from each of the four mixtures were tested using four different types SST tests. Test results are discussed in Chapter 4. Researchers made the following conclusions.

- Complex shear modulus (CSM) increases with increasing frequency. The CSM-frequency relationship is more linear in tests conducted at the lowest temperature (39°F).
- At higher temperatures, complex shear moduli of all four mixtures are similar at lower frequencies and they increase exponentially with increasing frequency and diverge significantly.
- The weakest mixture (rounded river gravel) always yielded the highest shear phase angle (SPA).
- The SPA of the four mixtures varies appreciably with frequency in the tests conducted at 68 and 104°F but not at 39°F. This indicates the test results at the higher temperature are much more sensitive to permanent deformation.
- SPAs of different mixtures are quite different for the tests conducted at higher temperature.

- River gravel mixture, the weakest mixture, yields the lowest CSM and highest SPA, and SMA mixture, the best mixture, yields the highest CSM and lowest SPA.
- The river gravel mixture exhibits the highest maximum strain and permanent strain and the lowest elastic recovery.
- Elastic recoveries of all mixtures are very similar for any given temperature.
- Maximum shear strain and permanent shear strain increase from the SSCH tests conducted at 39°F to the test conducted at 68°F but decrease from 68°F to 104°F. This fact suggests that a considerable permanent shear strain occurred at 68°F, thus consolidating the specimen such that it resisted further strain at 104°F.
- Premature failure was observed in the river gravel mixture for both repeated shear at constant height and repeated shear at constant stress ratio test at 131°F.

Accelerated Loaded Wheel Testing

Specimens from the four mixtures were tested using laboratory-scale rutting testers to evaluate the rutting susceptibility of each mixture. Researchers made the following conclusions.

- The laboratory wheel tracking devices ranked the four mixtures in the following manner (best to worst):

APA	HWTD	MMLS3
Granite SMA	Granite SMA	Granite SMA
Granite Superpave	Granite Superpave	Type C Limestone
Type C Limestone	Type C Limestone	Granite Superpave
Type D River Gravel	Type D River Gravel	Type D River Gravel

- Ranking by the APA and HWTD were the same. However, in the APA results, granite Superpave and limestone mixture were not significantly ($\alpha = 0.05$). Whereas in the HWTD test, granite SMA and granite Superpave were not significantly different and the limestone mixture performed very poorly, probably due to stripping.
- HWTD test results depend on both the shearing properties and the moisture susceptibility of the mixture. Test results showed no significance difference between the granite SMA and the granite Superpave mixture. This is probably due to the relatively higher coefficients of variability of the HWTD.

- The MMLS3 appears to be the most sensitive loaded wheel tester in that it was the only one of the three to separate the four mixtures into four significant groups.

RECOMMENDATIONS

- Based on the findings of this study, the researchers recommended the FSCH test as the best SST protocol. The rationale for recommending the FSCH is enumerated near the end of Chapter 4. In summary, the FSCH and the RSCH were leading SST tests. Both ranked the mixtures in the same general order as the three loaded wheel testers, and both showed good sensitivity in their measurements. Although the RSCH test gave slightly more sensitivity than the FSCH test, the FSCH test provided other important advantages. The FSCH test yields two fundamental material properties (complex shear modulus and shear phase angle), which are useful in predictive models for both rutting and fatigue cracking. The RSCH test provides permanent shear strain, which is not a fundamental material property but a test value and is thus dependent on the test parameters. Controlling strain to a low level, as in the FSCH test, minimizes damage to the test specimen more than stress control, as in the RSCH test.
- Researchers believe that there is a need to create a database of the HMA mixtures used statewide. Different types of mixtures with known field performance (Type C, Type D, Type F, Coarse matrix high binder [CMHB], etc.) from different aggregate sources should be tested using the FSCH test. This database could then be used to create a performance model for each mixture.
- Conducting the SSCH test on the same specimen at different temperatures, as specified in AASHTO TP7-01, needs to be reviewed because the maximum strain at higher temperature (104°F) was found to be lower than that at the intermediate temperature (68°F), indicating that the properties of the specimen had been altered by the previous tests.
- Performing the FSCH test at higher temperatures (i.e., higher than 104°F) appears to produce more discriminatory results.

REFERENCES

1. Witczak, M. W., H. V. Quintas, K. Kaloush, T. Pellinen, and M. Elbasyouny. "Simple Performance Test: Test Results and Recommendations," (Task C Interim Report), NCHRP Project 9-19. National Cooperative Highway Research Council, Transportation Research Board, Washington, D.C., 2000.
2. Brown, E. R., and S.A. Cross. A National Study of Rutting in Hot Mix Asphalt (HMA) Pavements. *Journal of the Association of Asphalt Paving Technologists*, Vol. 61, 1992.
3. Tielking, J. T., and F. L. Roberts. Tire Contact Pressure and Its Effect on Pavement Strain. *Journal of Transportation Engineering*, Vol. 113 (1), January 1987.
4. McGennis, R. B., R. M. Anderson, T. W. Kennedy, and M. Solaimanian. *Background of Superpave Asphalt Mixture Design & Analysis*. Report No. FHWA-SA-95-003, Federal Highway Administration, U.S. Department of Transportation, 1995.
5. AASHTO Provisional Standard Specifications for Transportation Materials and Methods of Sampling and Testing, AASHTO, Washington, D.C., June, 1998.
6. Berthelot, C. F. *Mechanistic Modeling of Saskatchewan Specific Pavement Studies-9A Asphalt Concrete Pavements*. Ph.D. Dissertation, Department of Civil Engineering, Texas A&M University, College Station, Texas, May 1999.
7. Roberts, F. L., T. W. Kennedy, P. S. Kandhal, and S. F. Brown. *Hot Mix Asphalt Materials, Mixture, Design, and Construction*. Second Edition, NAPA Education Foundation, Lanham, Maryland, 1996.
8. *Accelerated Performance Related Tests from Asphalt-Aggregate Mixes and Their Use in Mix Design and Analysis Systems*. SHRP Research Report SHRP-A-417. National Research Council, Washington, D.C., 1994.
9. *Superpave Asphalt Mixture Analysis*. Advanced Course Text, National Asphalt Training Center II, FHWA, Office of Technology Applications, Washington, D.C., January 1996.
10. Anderson, R. M., J. R. Bukowski, and P. A. Turner. Using Superpave Performance Tests to Evaluate Asphalt Mixture. *In Transportation Research Record 1681*, TRB, National Research Council, Washington, D.C., 1999.
11. Kennedy, T. W., G. A. Huber, E. T. Harrigan, R. J. Cominsky, C. S. Hughes, H. V. Quintas, and J. S. Moulthrop. *Superior Performing Asphalt Pavements (Superpave): The Product of*

the SHRP Asphalt Research Program. Report No. SHRP-A-410. Strategic Highway Research Program, National Research Council, Washington, D.C., 1994.

12. Romero, P., and W. Mogawer. Evaluation of the Superpave Shear testing Using 19-mm Mixtures from the Federal Highway Administration's Accelerated Loading Facility. *Journal of the Association of Asphalt Paving Technologists*, Vol. 67, 1998.
13. Tayebali, A. A., N. P. Khosla, G. A. Malpass, and H. F. Waller. Evaluation of Superpave Repeated Shear at Constant Height Test to Predict Rutting Potential of Mixes: Performance of Three Pavement Sections in North Carolina. *In Transportation Research Record 1681*, TRB, National Research Council, Washington, D.C., 1999.
14. Stuart, K. D., and R. P. Izzo. Correlation of Superpave G*/sin δ with Rutting Susceptibility from Laboratory Mixture Tests. *In Transportation Research Record 1492*, TRB, National Research Council, Washington, D.C., 1995.
15. Anderson, R. M., G. A. Huber, D. E. Walker, and X. Zhang. Mixture testing, Analysis, and Field Performance of the Pilot Superpave Projects: The 1992 SPS-9 Mixtures. *Journal of the Association of Asphalt Paving Technologists*, Vol. 69, 2000.
16. Shenoy, A., and P. Romero. Specification Parameters from Asphalt Mixtures Using Frequency Sweep Data from the Superpave Shear Tester. *International Journal of Road Materials and Pavement Design*, Vol. 1, 1/2000.
17. Little, D. N., J. W. Button, and H. Youssef. Development of Criteria to Evaluate Uniaxial Creep Data and Asphalt Concrete Permanent Deformation Potential. *In Transportation Research Record 1417*, TRB, National Research Council, Washington, D.C., 1993.
18. http://www4.nationalacademies.org/trb/crp.nsf/All+Projects/NCHRP+9-19 NCHRP Website visited in August 2001.
19. Cooley Jr., L. A., P. S. Kandhal, M. S. Buchanan, F. Fee, and A. Epps. Loaded Wheel Testers in the United States: State of the Practice. Transportation Research E-Circular No. E-C016, Transportation Research Board, National Research Council, Washington, D.C., July 2000.
20. Aschenbrener, T. Evaluation of Hamburg Wheel-Tracking Device to Predict Moisture Damage in Hot-Mix Asphalt. *In Transportation Research Record 1492*, TRB, National Research Council, Washington, D.C., 1995.

21. Chowdhury, A., J. W. Button, and J. D. C. Grau. Effects of Superpave Restricted Zone on Permanent Deformation. ICAR Research Report 201-2F. Aggregates Foundation for Technology, Research and Education, Arlington, Virginia, 2001.
22. Izzo, R. P., and M. Tahmoressi. Testing Repeatability of the Hamburg Wheel-Tracking Device and Replicating Wheel Tracking Devices Among Different Laboratories. *Journal of the Association of Asphalt Paving Technologists*, Vol. 68, AAPT, 1998.
23. Sherwood, J. A., N. L. Thomas, and X. Qi. Correlation of Superpave G*/sin* with ALF Rutting Test Results. Paper Submitted for Publication and Presentation at 77th TRB Annual Meeting, National Research Council, Washington, D.C., 1998.
24. Choubane, B., G. C. Page, and J. A. Musselman. Suitability of Asphalt Pavement Analyzer for Predicting Rutting. *In Transportation Research Record 1723*, TRB, National Research Council, Washington, D.C., 2000.
25. Kandhal, P. S., and R. B. Mallick. Evaluation of Asphalt Pavement Analyzer for HMA Design. NCAT Report No. 99-4. National Center for Asphalt Technology, Auburn University, Alabama, 1999.
26. Metcalf, J. B. *Application of Full-Scale Accelerated Pavement Testing*. NCHRP Synthesis of Highway Practice, 235, Transportation Research Board, National Research Council, Washington, D.C., 1996.
27. Collins, R., D. Watson, and B. Campbell. Development and Use of Georgia Loaded Wheel Tester. *In Transportation Research Record 1492*, TRB, National Research Council, Washington, D.C., 1995.
28. West, R. C., G. C. Page, and K. H. Murphy. *Evaluation of the Loaded Wheel Tester*. Research Report FL/DOT/SMO/91-391, Florida Department of Transportation, 1991.
29. Miller, T., K. Ksaibati, and M. Farrar. Using Georgia Loaded-Wheel Tester to predict Rutting. *In Transportation Research Record 1473*, TRB, National Research Council, Washington, D.C., 1995.
30. Williams, C. R., and B. D. Prowell. Comparison of Laboratory Wheel-Tracking Test Results with WesTrack Performance. *In Transportation Research Record 1681*, TRB, National Research Council, Washington, D.C., 1999.
31. *Asphalt Pavement Analyzer User's Guide*. Pavement Technology, Inc., Covington, Georgia, 1998.

32. Uzarowski, L., and J. Emery. Use of the Asphalt Pavement Analyzer for Asphalt Mix Design and Evaluation. Proceedings of the Canadian Technical Asphalt Association, 45th Annual Conference, 2000.
33. Hugo, F. Texas Mobile Load Simulator Test Plan. Research Report 1978-1, Center for Transportation Research, The University of Texas at Austin, 1996.
34. Van de Ven, M., A. de Fortier Smit, K. Jenkins, and F. Hugo. Scaled Down APT Considerations from Viscoelastic Materials. *Journal of the Association of Asphalt Paving Technologists,* Vol. 67, AAPT, 1998.
35. Prowell, B. D., and R. J. Schreck. Virginia's Use of Laboratory Wheel-Tracking as a Mix Performance Predictor. Paper Presented at the First International Conference, World of Asphalt Pavement, Sydney, Australia, 2000.
36. Galal, K., A. Hand, A. Noureldin, J. Stiady, and T. D. White. Gradation Effects on Hot Mix Asphalt. Paper presented for publication at the 80th Annual Meeting of Transportation Research Board. National Research Council, Washington, D.C., 2001.
37. Amirkhanian, S. N., D. Kaczmarck, and J. L. Buratti. The Effects of Los Angeles Abrasion Test Values on the Strength of Laboratory Marshall Specimens. *In Transportation Research Record 1301*, TRB, National Research Council, Washington, D.C., 1991.
38. Dolar-Mantuani, L. Soundness and Deleterious Substances. In American Society for Testing and Materials, STP 16B, West Conshohocken, Pennsylvania, 1978.
39. American Society for Testing and Materials. ASTM Standards: Road and Pavement Materials, Vol. 04.03, American Society for Testing and Materials, West Conshohocken, Pennsylvania, 1998.
40. Chowdhury, A., J. W. Button, V. Kohale, and D. W. Jahn. Evaluation of Superpave Fine Aggregate Angularity Specification. ICAR Research Report 201-1. Aggregates Foundation for Technology, Research and Education, Arlington, Virginia, 2001.
41. AASHTO Provisional Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Interim Edition, AASHTO, Washington, D.C., April 2001.
42. Huber, G. A. *Weather Database for the Superpave Mix Design System*. Report No SHRP-648A. Strategic Highway Research Program. National Research Council, Washington, D.C., 1994.

43. Federal Highway Administration. Superpave Asphalt Mixture Analysis: Lab Notes. Federal Highway Administration, Office of Technology Applications, Washington, D.C., 1996.
44. Sousa, J. B., and M. Bronstein. ATS Manual, Version 3.1, SHRP Corporation, Walnut Creek, California, 1995.

APPENDIX

MATERIAL CHARACTERIZATION

Researchers tested the asphalt cement used in the asphalt mixtures according to the Superpave asphalt binder specification (AASHTO MP1). The mixing and compaction temperatures and binder complex shear modulus at different frequencies and temperatures were determined. A summary of the results is provided in Tables A1 and A2. These results confirm that the grade of the asphalt cements are PG 64-22 and PG 76-22, respectively.

Table A1. PG 64-22 Test Results and Requirements.

Binder Property	Binder Aging Condition	Test Result	Superpave Requirement
Flash Point (°C)	Unaged	302	>230
Viscosity at 135°C (Pa•s)	Unaged	0.482	<3.00
Dynamic Shear, G*/sin δ at 64°C (kPa)	Unaged	1.045	>1.00
Mass Loss (%)	RTFO aged	0.49	<1.00
Dynamic Shear, G*/sin δ at 64°C (kPa)	RTFO aged	3.688	>2.20
Dynamic Shear, G*sin δ at 25°C (kPa)	PAV aged	3473	<5000
Creep Stiffness, S at –12°C (MPa)	PAV aged	127	<300
m-value at !12°C	PAV aged	0.309	>0.300

Table A2. PG 76-22 Test Results and Requirements

Binder Property	Binder Aging Condition	Test Result	Superpave Requirement
Flash Point (°C)	Unaged	-	>230
Viscosity at 135°C (Pa•s)	Unaged	2.268	<3.00
Dynamic Shear, G*/sin δ at 76°C (kPa)	Unaged	1.801	>1.00
Mass Loss (%)	RTFO aged	0.45	<1.00
Dynamic Shear, G*/sin δ at 76°C (kPa)	RTFO aged	2.952	>2.20
Dynamic Shear, G*sin δ at 31°C (kPa)	PAV aged	2842	<5000
Creep Stiffness, S at -12°C (MPa)	PAV aged	249	<300
m-value at !12°C	PAV aged	0.301	>0.300

The rheological properties of the asphalt cement were determined according to AASHTO TP5. The test apparatus used was a Bohlin controlled stress rheometer. The asphalt cement was aged using the rolling thin film oven test (ASTM D 2872 or AASHTO T 240) and a pressure aging vessel (AASHTO PP1). Stiffness of the asphalt cement at very low temperatures was measured according to AASHTO TP1 using a bending beam rheometer.

The flash point temperature was determined according to ASTM D 92. High-temperature viscosity was measured using ASTM D 4402. Viscosity at 135°C was 410 cP (0.41 Pa•s) (see Figure A1).

To determine the mixing and compaction temperature, both asphalts were tested using the Brookfield viscometer. Figures A1 and A2 illustrate the viscosity-temperature relationship of PG 64-22 and PG 76-22 asphalt, respectively. The PG 64-22 asphalt was tested at 275°F (135°C) and 347°F (275°C). The PG 76-22 asphalt was tested at one additional temperature, 427°F (220°C). The reason for testing at an additional temperature was to intersect the compaction and mixing viscosity ranges with the viscosity-temperature line. According to this method, the PG 76-22 yielded abnormally high mixing and compaction temperatures. As a result, the PG 76-22 was mixed and compacted at temperatures suggested by the asphalt supplier (Koch Materials, Inc.)

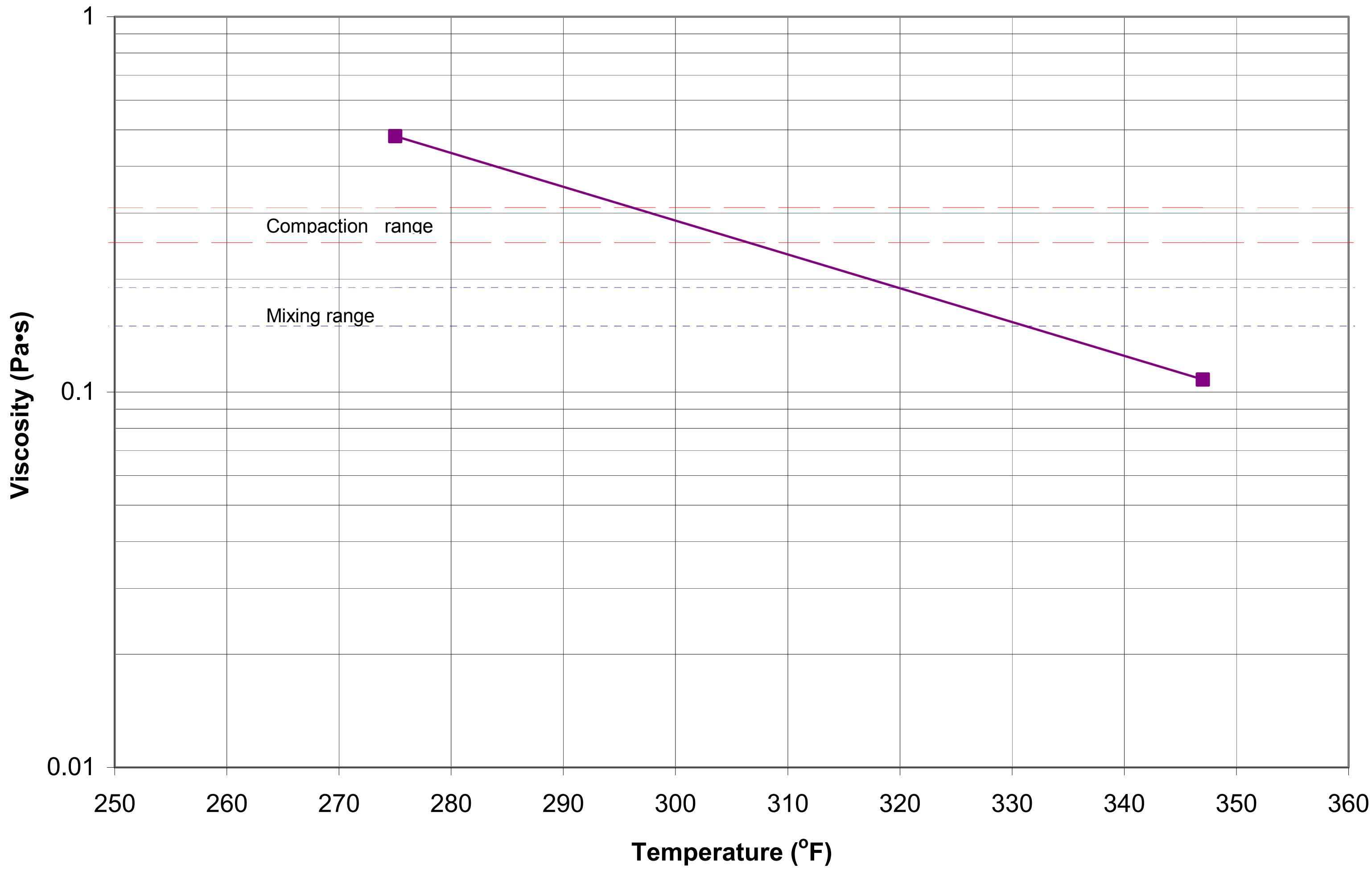

Figure A1. Viscosity-Temperature Relationship of PG 64-22 Asphalt.

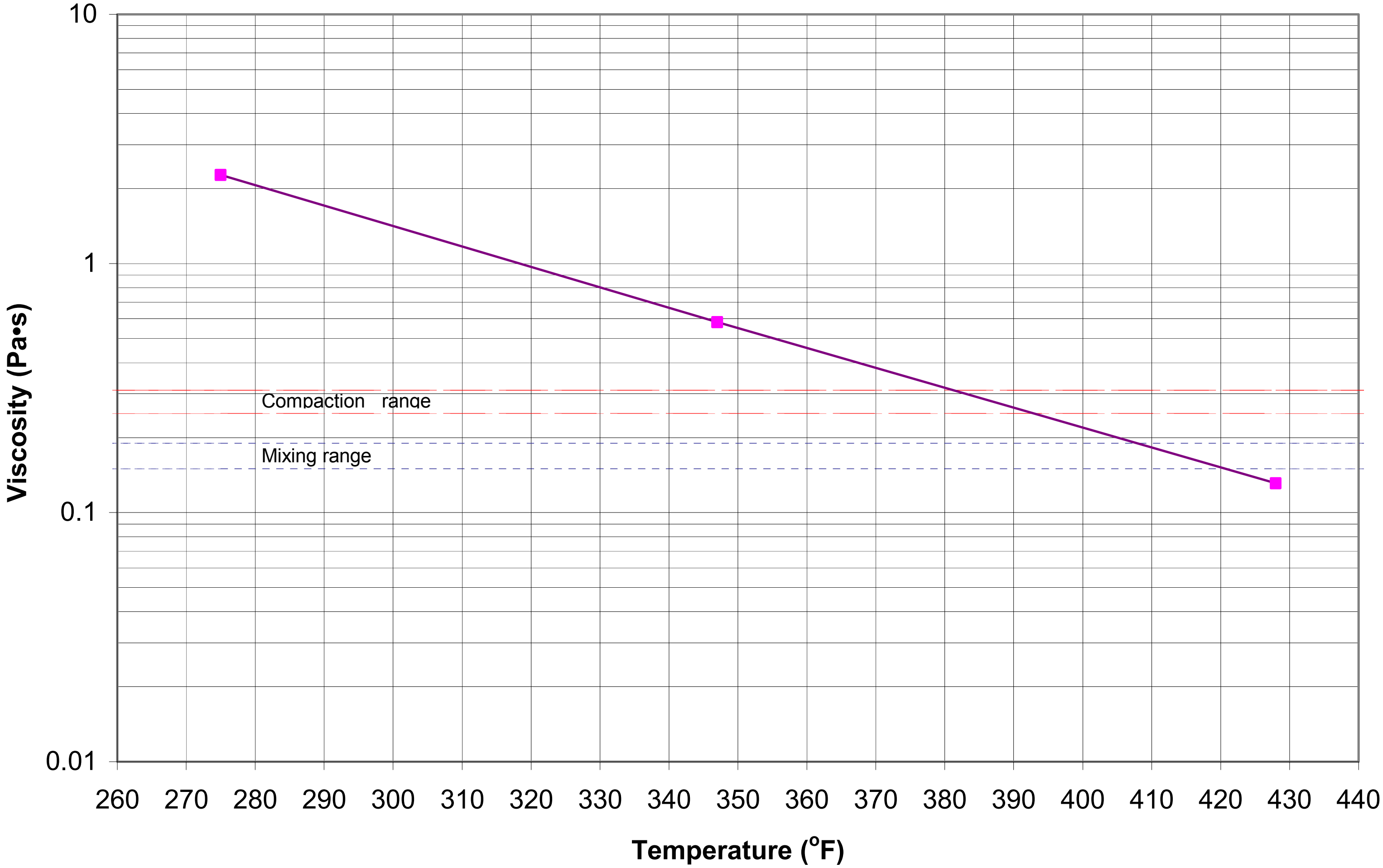

Figure A2. Viscosity-Temperature Relationship of PG 76-22 Asphalt.