TECHNICAL REPORT STANDARD PAGE

1. Report No. **FHWA/LA.09/453**	**2. Government Accession No.**	**3. Recipient's Catalog No.**
4. Title and Subtitle **Development of a Design Methodology for Asphalt Treated Mixtures**	**5. Report Date** **December 2013** **6. Performing Organization Code** **LTRC Project Number: 04-4B** **State Project Number: 736-99-1449**	
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9. Performing Organization Name and Address **Louisiana Department of Transportation and Development**	**10. Work Unit No.** **11. Contract or Grant No.**	
12. Sponsoring Agency Name and Address **Louisiana Department of Transportation and Development** **P.O. Box 94245** **Baton Rouge, LA 70804-9245**	**13. Type of Report and Period Covered** **Final Report (01/07-06/09)** **14. Sponsoring Agency Code**	

15. Supplementary Notes

Conducted in Cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract

This report summarizes the results of a study that was conducted to develop a simplified design methodology for asphalt treated mixtures that are durable, stable, constructible, and cost effective through the examination of the performance of mixtures that have different aggregate gradation from typically available sources. The study was conducted in two parallel parts, Part I and Part II. Part I consisted of developing a design methodology for asphalt treated mixtures and conducting a laboratory testing program to characterize the behavior of the designed mixtures. Eight aggregate sources and two types of asphalt binders were considered in this part. Part I of this study also included conducting static as well as repeated load triaxial tests to characterize the performance of three unbound granular base materials. Furthermore, a parametric analysis was conducted using MEPGD software to evaluate the benefits of incorporating the asphalt treated mixtures in the design of a pavement structure. In Part II four overlay rehabilitation projects were selected in Louisiana to evaluate the constructability and in-situ proprieties of the asphalt treated mixtures designed in Part I. The results of Part I showed that the asphalt treated mixtures containing limestone aggregates, LS I and LS II, had the best laboratory performance among all other mixtures designed in this study. Furthermore, their performance was similar to conventional base course HMA ones at high and intermediate temperatures. The results of the laboratory tests conducted in Part I also showed that the asphalt treated base mixtures have made significant improvements over unbound granular base materials in terms of stiffness and permanent deformation resistance. In addition, the MEPDG analysis showed asphalt treated mixtures can be used to extend the service life and/or reduce the design thickness of a pavement structure. The results of Part II of this study demonstrated that the asphalt treated mixtures can be successively produced in conventional HMA plants and constructed in the field. In addition, the in-situ test results showed that asphalt treated mixtures exhibited similar moduli to those of conventional HMA base course mixtures.

17. Key Words Asphalt treated mixtures, MEPDG, field testing		**18. Distribution Statement** **Unrestricted. This document is available through the National Technical Information Service, Springfield, VA 21161.**	
19. Security Classif. (of this report)	**20. Security Classif. (of this page)**	21. No. of Pages 91	22. Price

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Each research project will have an advisory committee appointed by the LTRC Director. The Project Review Committee is responsible for assisting the LTRC Administrator or Manager in the development of acceptable research problem statements, requests for proposals, review of research proposals, oversight of approved research projects, and implementation of findings.

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Development of a Design Methodology for Asphalt Treated Mixtures

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State Project No. 736-99-1449

conducted for
Louisiana Department of Transportation and Development
Louisiana Transportation Research Center

December 2013

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This report summarizes the results of a study that was conducted to develop a simplified design methodology for asphalt treated mixtures that are durable, stable, constructible, and cost effective through the examination of the performance of mixtures that have different aggregate gradation from typically available sources. The study was conducted in two parallel parts, Part I and Part II. Part I consisted of developing a design methodology for asphalt treated mixtures and conducting a laboratory testing program to characterize the behavior of the designed mixtures. Eight aggregate sources and two types of asphalt binders were considered in this part. Part I of this study also included conducting static as well as repeated load triaxial tests to characterize the performance of three unbound granular base materials. Furthermore, a parametric analysis was conducted using Mechanistic Empirical Pavement Design Guide (MEPDG) software to evaluate the benefits of incorporating the asphalt treated mixtures in the design of a pavement structure. In Part II, four overlay rehabilitation projects were selected in Louisiana to evaluate the constructability and in-situ proprieties of the asphalt treated mixtures designed in Part I. The results of Part I showed that the asphalt treated mixtures containing limestone aggregates, LS I and LS II, had the best laboratory performance among all other mixtures designed in this study. Furthermore, their performance was similar to conventional base course hot mix asphalt (HMA) ones at high and intermediate temperatures. The results of the laboratory tests conducted in Part I also showed that the asphalt treated base mixtures have made significant improvements over unbound granular base materials in terms of stiffness and permanent deformation resistance. In addition, the MEPDG analysis showed asphalt treated mixtures can be used to extend the service life and/or reduce the design thickness of a pavement structure. The results of Part II of this study demonstrated that asphalt treated mixtures can be successively produced in conventional HMA plants and constructed in the field. In addition, the in-situ test results showed that asphalt treated mixtures exhibited similar moduli to those of conventional HMA base course mixtures.

ACKNOWLEDGMENTS

The U.S. Department of Transportation, Federal Highway Administration (FHWA), Louisiana Department of Transportation and Development (LADOTD), and Louisiana Transportation Research Center (LTRC) financially supported this research project. The assistance of the asphalt laboratory staff at LTRC is greatly appreciated. The guidance of the Project Review Committee to this research project is also greatly appreciated.

OBJECTIVE

The primary objective of this research was to develop a simplified design methodology for asphalt treated mixtures that are durable, stable, constructible, and cost effective through the examination of the performance of mixtures that have a different aggregate gradation from typically available sources.

A secondary objective of this research was to compare the performance of the asphalt treated mixtures designed in this study to unbound granular base materials currently used in the construction of base layers in Louisiana.

IMPLEMENTATION STATEMENT

Based on laboratory and field results of this experiment and with the direction of the Project Review Committee, the use of asphalt treated mixtures shall be included as an allowable material for future roadway projects. The minimum thickness of the asphalt treated mixture layer shall be 3 inches. This mixture shall be allowed in the construction of the wearing course layer of roadway shoulders, base course layers in flexible and rigid pavements, and in pavement widening and patching.

For pavement design properties, it is recommended that a structural layer coefficient of 0.30 be used, which is equal to the one used for asphalt base coarse mixtures.

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INTRODUCTION

The increase in energy costs has led to a significant rise in the cost of mixtures containing asphalt cement. This resulted in a need to search for alternatives that reduce the cost of those mixtures without compromising performance. One such alternative is the use of asphalt treated mixtures. Asphalt treated mixtures are hot mix asphalt (HMA) mixtures consisting of crushed rock or natural gravel mixed with low percentages (2.5 - 4.5) of paving grade asphalt cement. Those mixtures cost less than typical HMA mixtures because they can be produced with less expensive aggregates and lower percentages of asphalt cement binder. Asphalt treated mixtures can be used in construction of base course layers as well as shoulders of a pavement structure.

Limited studies were conducted in the past decades to evaluate the use of asphalt treated mixtures as base course layers. The results of those studies showed that asphalt treated mixtures had several advantages over untreated granular base material. Rostron et al. indicated that the strength coefficient of the base layer was tripled when using asphalt treated base material as compared to conventional untreated granular base material *[1]*. Furthermore, they reported that the strength coefficient of an untreated granular base layer decreased with increasing the thickness of the layer, while the base course layer constructed using asphalt treated mixtures showed an increase in strength as the thickness increased, suggesting that such a layer provided a much better structural support especially when the underlying layer is a weak subgrade soil.

Benkelman et al. studied the performance of flexible pavement sections with four different types of base materials at the American Association of State Highway and Transportation Officials (AASHTO) road test facility *[2]*. The four base materials included: crushed stone, well-graded uncrushed gravel, a cement-treated material, and an asphalt treated material. The results of their study suggested that the sections with an untreated granular base did not perform as well as those with the asphalt and cement treated bases. In addition, the bases with asphalt treated mixtures offered considerably more resistance to consolidation and displacement at low temperatures.

Currently, state agencies do not have a formal method to design an asphalt treated mixture. However, specifications for asphalt treated mixtures are similar to those required for binder and wearing hot mixture asphalt concrete layers. This has resulted in limiting the use of such mixtures in pavement construction. This project evaluates the mechanical and physical properties of asphalt treated mixtures using fundamental engineering tests. A simplified design methodology for asphalt treated mixtures that are structurally stable, durable, and cost effective is developed and recommended.

SCOPE

This research study was conducted in two parallel parts, Part I and Part II. Part I consisted of designing asphalt treated mixtures with different aggregate sources and conducting a laboratory testing program to characterize the behavior of the designed mixtures. The considered aggregate sources included four limestones, a sandstone, granite, Novaculite, and Rhyolite. A SBS polymer modified asphalt cement meeting Louisiana specifications for PG 70-22 was used for all mixtures in this study. The laboratory testing program was performed in two phases, Phase I and Phase II. Phase I evaluated the physical and strength properties of the aggregates sources. Furthermore, it examined high and intermediate temperature properties of the asphalt treated mixture using load wheel tracking and indirect tensile strength tests, respectively. In Phase II, a suite of mechanistic tests were performed to further study the stability and durability of asphalt treated mixtures that passed Phase I. This included: permeability, modified Lottman, dynamic modulus, flow number, semi circular bend, and dissipated creep strain energy tests. Part I of this study also included conducting static as well as repeated load triaxial tests to characterize the performance of three unbound granular materials used in this study. The results of the laboratory testing program were used to conduct parametric analysis using MEPDG software to evaluate the benefits of incorporating the asphalt treated mixtures in the design of a pavement structure.

In Part II of this study, four overlay rehabilitation projects were selected in Louisiana to evaluate the constructability of asphalt treated mixtures designed in Part I. In each of the four selected projects, a one-mile test section of the roadway lane shoulder was constructed using one of the asphalt treated mixtures designed and evaluated in Part I. Cores were obtained at 15 test points at each test section. In addition, Falling Weight Deflectometer (FWD), Light Falling Weight Deflectometer (LFWD), and Portable Seismic Pavement Analyzer (PSPA) were performed at the test points to characterize in-situ properties of asphalt treated mixtures. Various laboratory tests were also performed to examine the physical and mechanical properties of the asphalt treated mixtures that were used in the construction of the field test sections. Laboratory tests performed in this part included: permeability, Indirect Tensile Strength (ITS), Loaded Wheel Tester (LWT), Lottman, Semi-Circular Bend (SCB), Dissipated Creep Strain Energy (DCSE), Dynamic Modulus (|E*|), and Flow Number (FN) tests.

METHODOLOGY

This research study was conducted in two parallel parts, Part I and Part II (Figure 1). Part I examined the behavior and performance of asphalt treated mixtures as well as unbound granular base materials. Whereas Part II evaluated asphalt treated mixtures from ongoing field projects. This section provides detailed information on the materials considered and their properties. It also describes the laboratory and field testing programs performed in this study.

Part I: Laboratory Study

Design of Asphalt Treated Mixtures

Materials-Aggregate Materials. Aggregates from eight different sources representing five types of materials were selected in this study. The selected aggregates included: four limestones, sandstone (SS), granite (GR), Novaculite (NV), and Rhyolite (RY). The four limestone materials considered in this study included two porous limestones, PLS I and PLS II, and two conventional limestones, LS I and LS II.

Materials-Asphalt Cement Binders. An SBS polymer modified asphalt cement meeting Louisiana specifications for PG 70-22 was used for all mixtures in this study. Verification of the asphalt binder grade was performed according to AASHTO R29-02 test method *[3]*. It is noted that the selection of the PG 70-22 was based on the results of the partial testing factorial that evaluated the influence of the binder type on the behavior of asphalt treated mixtures. Two asphalt cement binders, PG 70-22M and neat PG 64-22, were considered in the partial testing factorial.

Mixture Design

Conventional HMA mixtures are designed with consideration to volumetric and densification criteria. Furthermore, aggregates are generally combined in typical percentages that are developed from years of experience. For the proposed asphalt treated mixtures, the aggregate structure was composed of 75 percent of -1.5 inch sieve crusher run materials from each of the selected aggregate sources and 25 percent of coarse sand (CS). This structure was selected based on the observed field performance. Figure 2 graphically represents the aggregate gradations used in this study. It is noted that aggregate structures for all mixtures considered were on the fine side of the maximum density line.

A limited test factorial was conducted to select the asphalt content to be used. Three asphalt cement contents, namely, 2.0, 3.0, and 4.0 percent, and limestone aggregate were considered in this factorial. The indirect tensile strength (ITS) test was used to evaluate the response of asphalt treated mixtures. Table shows the results of ITS test. Three percent asphalt cement

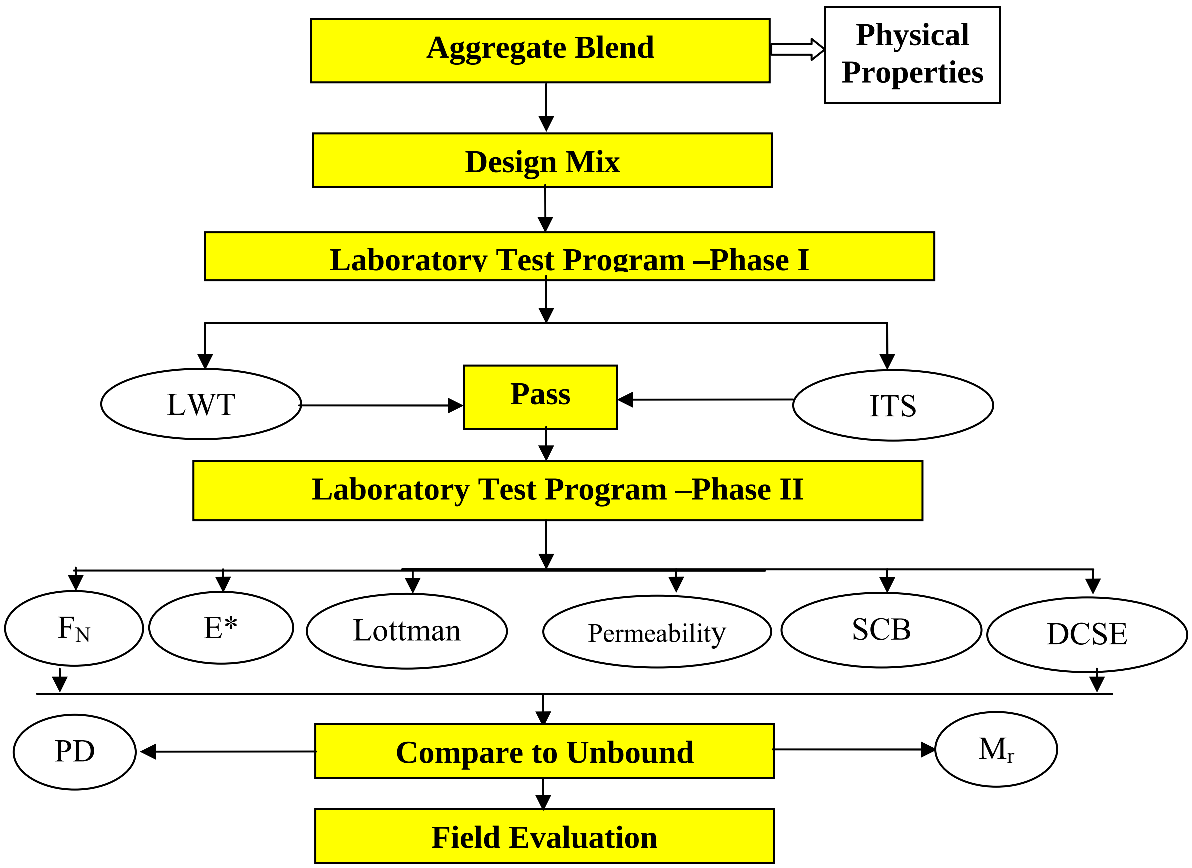

Figure 1
Flowchart followed in this study

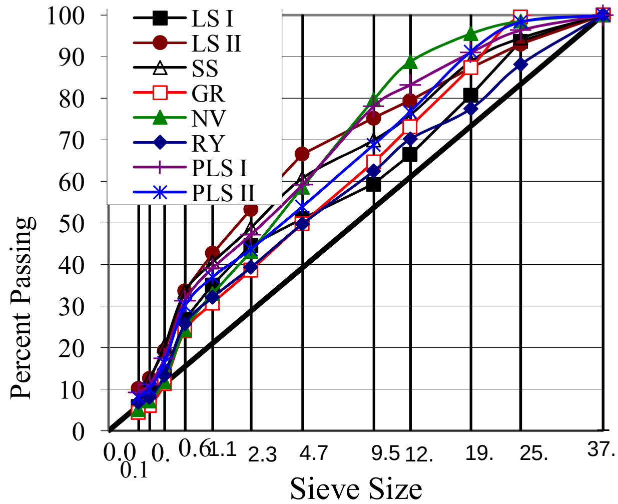

Figure 2
Aggregate gradation curves

content was found to provide an acceptable performance level for the selected aggregate structure. Therefore, it was used for the mixtures evaluated this study.

The design number of gyrations required to produce a sample with the same density as expected in the field was determined based on the Superpave gyratory compactor (SGC) locking point. This is the number of gyrations after the rate of change in height of the asphalt mixture sample is equal to or less than 0.05 mm for three consecutive gyrations. Figure 2 shows the rate of change in height with the number of gyrations curve obtained for different asphalt treated mixtures evaluated in this study. It is noted that the curves for the different mixtures reached an asymptotic value at about 28 gyrations. Based on that result, the design number of gyration was selected to be 30. The job mix formula (JMF) of all mixtures considered in this study are summarized in Table 1.

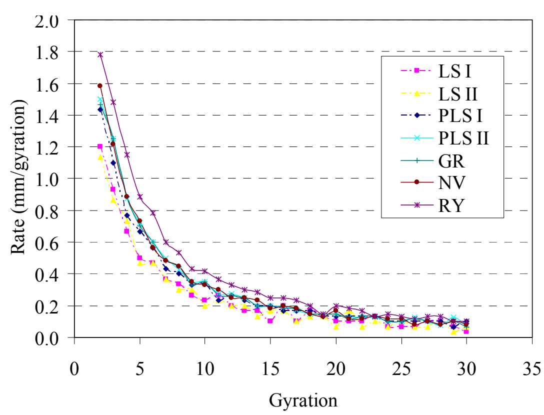

Figure 3
Rate of change in height versus number of gyration

Laboratory Characterization of Asphalt Treated Mixtures

A laboratory testing program was conducted to characterize the design asphalt treated mixtures. The testing program consisted of two phases, Phase I and Phase II (Figure 1). Phase I was a screening phase in which different physical and strength properties including: coarse aggregate angularity, fine aggregate angularity, gradation analysis, specific gravity, sand equivalent, absorption, and Micro-Deval were examined for the eight aggregate sources considered in this study. Furthermore, high and intermediate temperature properties of each asphalt treated mixture were evaluated in Phase I using the load wheel tracking (LWT) test and indirect tensile strength (ITS) test, respectively.

Table 1
Job mix formula for asphalt treated mixtures

Mixture Designation	LS I-70	LS II-70	PLS I-70	PLS II-70	SS-70	GR-70	NV-70	RY -70
Mix type	25 mm	25 mm	19 mm	19 mm	25 mm	25 mm	19 mm	37.5 mm
Aggregate blend	75% LS I 25% CS	75% LS II 25% CS	75% PLS 25% CS	75% SS 25% CS	75% SS 25% CS	75% GR 25% CS	75% NV 25% CS	75% RY 25% CS
Binder content, %	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Air void at N=30, %	7.4	7.8	12.6	11.0	11.0	10.3	11.9	9.9
Metric (U.S.) Sieve	Blend Gradation							
37.5 mm (1½ in)	100	100	100	100	100	100	100	100
25 mm (1 in)	94	93	96	98	94	99	98.5	88
19 mm (¾ in)	81	87	91	91	89	87	96	77
12.5 mm (½ in)	66	79	83	77	76	73	89	70
4.75 mm (No.4)	51	67	59	54	61	50	59	50
2.36 mm (No.8)	45	53	47	39	49	39	43	39
1.18 mm (No.16)	35	43	39	30	41	31	33	32
0.3 mm (No.50)	13	19	17	16	20	11	12	13
0.075 mm (No.200)	6	10	9	8	7	4	5	7

Based on the results of the Phase I, only those mixtures that met the screening criterion selected in this study were further investigated in Phase II. The screening criterion used in this study was a maximum rut depth of 0.47 inch at 20,000 load cycles in the LWT test and a minimum ITS of 150 psi. The values selected for the screening criterion were derived from test results for well performing HMA mixtures in the field *[4]*.

In Phase II, permeability, semi circular bend (SCB), dissipated creep strain energy (DCSE), dynamic modulus |E*|, and flow number (F_N) tests were conducted. Triplicate specimens were tested for each asphalt treated mixture considered in this study. Table 2 outlines a summary of the laboratory tests that were performed in Phase II to measure the performance of the asphalt treated mixtures contained in this study.

A partial testing factorial was also conducted in Phase II to investigate the effect of the asphalt binder type on high and intermediate temperature properties of asphalt treated mixtures. Three aggregate sources, LS I, LS II, and SS, and two asphalt binder types, PG-64-22, and PG 70-22m, were considered in this factorial. In addition, LWT, FN, SCB, and DCSE tests were used for this investigation.

Sample Preparation. The asphalt treated mixture samples were prepared in this study in accordance with AASHTO T 312-04 procedure *[5]*. Four sample sizes were fabricated for the fundamental engineering property tests in this study. These include a 4-in. diameter and about 2.5-in. high, 5.91-in. diameter x 6.69-in. high, 5.91-in. x 2.24-in. high cylindrical specimens, and 3.2-in. x 10.2-in. x 12.6-in. beam specimens. The cylindrical specimens were compacted with the Superpave gyratory compactor (SGC), while the beam samples were compacted using a kneading compactor. It is noted that all samples were compacted to an air void level that corresponds to 30 SGC gyrations.

Table 2
Asphalt treated mixture performance test conditions

Laboratory Test	Performance Indication	Test Temperature	Test Protocol
Permeability	Durability	25ºC	ASTM PS129-01 *[6]*
Modified Lottman	Moisture susceptibility	25ºC	AASHTO MP 2 *[7]*
SCB	Resistance to crack propagation	25ºC	Mull et al. *[8]*
DCSE	Fracture resistance	10ºC	Roque et al. *[9]*
Dynamic Modulus	Elastic properties of rutting analysis	Various Temperatures	AASHTO TP 62-03 *[10]*
Flow Number	Resistance to permanent deformation	54.4ºC	AASHTO TP 62-03 *[11]*

The 4-in. x 2.5-in. high cylindrical specimens and 3.2-in. x 10.2-in. x 12.6-in. beams were employed in ITS and LWT tests, respectively without any fabrication. For the SCB test, semi-circular shaped specimens were prepared by slicing the 5.91-in. x 2.25-in. high cylindrical specimens along their central axes into two equal semi-circular samples. A vertical notch was then introduced along the symmetrical axis of each semi-circular specimen in order to study the fracture properties of asphalt mixtures with regard to the crack propagation. Three nominal notch depths of 1-in., 1.25-in., and 1.5-in. were introduced using a special 0.12-in. saw blade, where each sample contained a single vertical notch along its symmetrical axis.

For the DCSE test, SGC compacted 5.91-in. x 2.25-in. high cylindrical samples that were trimmed down to 1.96-in. to create a smooth surface to attach the deflection-measuring studs properly. Finally, the cylindrical samples for the |E*| and FN tests were fabricated by coring and sawing 3.94-in. diameter x 5.91-in. high test specimens from the middle of the 5.91-in. diameter x 6.69-in. high SGC compacted cylindrical specimens.

Loaded Wheel Tracking (LWT) Test. This test was conducted according to AASHTO T 324, *Standard Method of Test for Hamburg Wheel-Track Testing of Compacted HMA,* to determine rutting characteristics of HMA mixtures *[12]*. The Hamburg type LWT device (Figure 4) used in this study can test two slabs at a time using two reciprocating solid-steel wheels of 8-in. in diameter and 1.85-in. in width. The compacted samples were conditioned at 50°C for 90 minutes prior to the start of the test and were submerged under hot water (50°C) throughout the duration of the test. A fixed load of 158 lb. with a rolling speed of 0.68 mi/h, the rate of 56 passes/min, was applied. The test continued for 20,000 cycles or 0.78-in. deformation, whichever was reached first. The rut dept at 20,000 cycles was measured and used in the analysis.

Indirect Tensile Strength (ITS) Test. This test was conducted at 25°C according to AASHTO T245 *[13]*. A cylindrical sample was loaded to failure at a deformation rate of 2 inch/min. Triplicate specimens were tested for each mixture type considered in this study.

Permeability. A falling head permeability test was conducted according to the ASTM PS129-01 to measure the permeability of asphalt mixtures considered in this study *[6]*. In this test, the amount of water head loss through a sample with a 5.91 in. diameter and 2.5 in. height was determined over a given time. The coefficient of permeability in this test is determined using the following equation:

$$K = \left(\frac{L}{T}\right) \times \left(\frac{d^2}{D^2}\right) \times \ln\left(\frac{H_1}{H_2}\right) \tag{1}$$

where, K is the coefficient of permeability (mm/s x 10-4); L is the average specimen thickness; D is the average specimen diameter; d is the graduated cylinder diameter; T is the total time of test, seconds; H_1 is the initial height of water; and H_2 is the final height of water.

Modified Lottman Test. This method evaluates the HMA mixtures' sensitivity to moisture damage, which is necessary to assure its durability. The modified Lottman test basically compares the indirect tensile strength test results of a dry sample and a conditioned sample that is exposed to saturation and freeze-thaw cycles. It is noted that liquid anti-strips were included in all mixtures evaluated in this study. Test results are reported as a tensile strength ratio (TSR), which is defined as the ratio of the original tensile strength that is retained after the moisture and freeze thaw conditioning as shown in equation (2). For laboratory samples, the AASHTO MP 2 (specification for Superpave Volumetric Mix Design) specifies a minimum TSR of 0.80.

$$\text{Tensile Strength ratio (TSR)} = \frac{S_2}{S_1} \tag{2}$$

where, S_1 is the average tensile strength of "unconditioned" specimens, psi; and S_2 is the average tensile strength of "conditioned" specimens, psi.

Semi Circular Bend (SCB) Test. This test was conducted according to the test procedure adopted by Mohammad et al. *[14]*. Triplicate samples were tested for each notch depth, and the test was performed at 25°C. To determine the critical value of J-integral (J_c), semi-circular samples with at least two different notch depths are needed to be tested. In this study, three notch depths of 1 in., 1.25 in., and 1.5 in. were selected based on an a/r_d ratio of between 0.5 and 0.75. During the test, the sample was loaded monotonically to failure at a constant cross-head deformation rate of 0.02 inch/min in a three-point bend load configuration, as shown in Figure 5. The load and deformation were continuously recorded and the critical value of J-integral (J_c) was then determined as follows *[14]*:

$$J_c = -\left(\frac{1}{b}\right)\frac{dU}{da} \tag{3}$$

where, *b* is the sample thickness, *a* is the notch depth, and *U* is the total strain energy to failure, i.e., the area up to fracture under the load-deflection plot.

Figure 4
Hamburg LWT device

Dissipated Creep Strain Energy (DCSE) Test. The dissipated creep strain energy threshold represents the energy that the mixture can tolerate before it fractures. This parameter is determined based on the procedure proposed by Roque et al., using two laboratory tests conducted on the same sample: the indirect resilient modulus test and the indirect tensile strength test *[9]*. Both tests were conducted at 10°C on 5.91-in. diameter and 1.96-in. thick specimens. DCSE is defined as the fracture energy (FE) minus the elastic energy (EE), Figure 6. The fracture energy is defined as the area under the stress-strain curve up to the point where the sample begins to fracture. The elastic energy is the energy recovered after unloading the specimen. The failure strain (ε_f), tensile strength (S_t), and fracture energy are determined from the IT strength test. From the resilient modulus test, the resilient modulus (M_R) is obtained. The calculation of the DCSE is shown in the following:

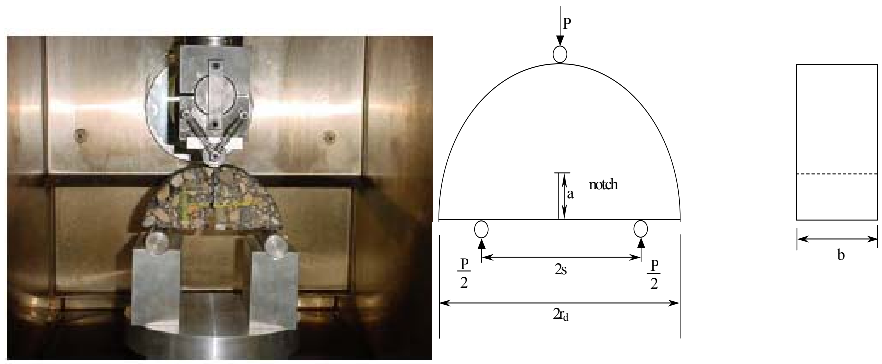

Figure 5
Semi circular bend test setup

$$\varepsilon_0 = \frac{(M_R \times \varepsilon_f - S_t)}{M_R} \tag{4}$$

$$EE = \tfrac{1}{2} S_t (\varepsilon_f - \varepsilon_0) \tag{5}$$

$$DCSE = FE - EE \tag{6}$$

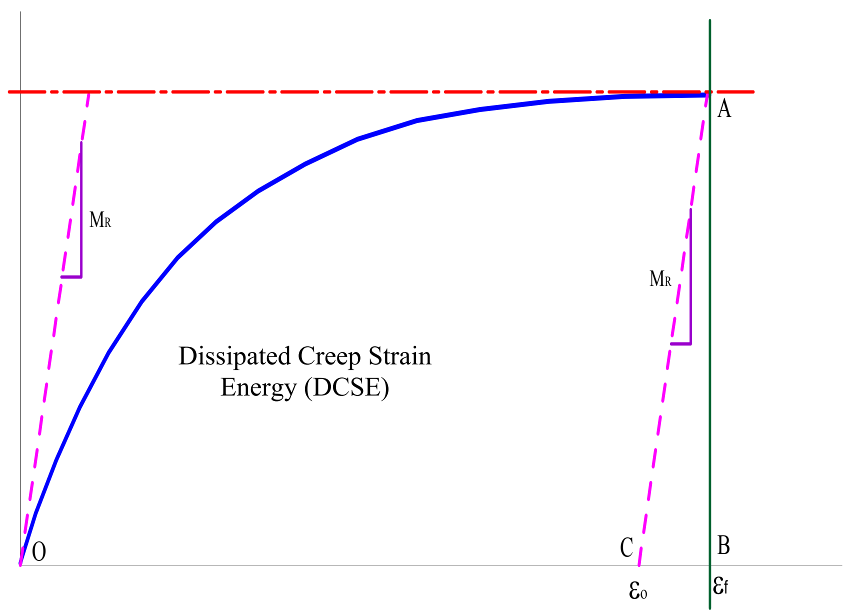

Figure 6
Typical stress-strain curve for DCSE determination

Flow Number (FN) Test

This test was conducted according to the Annex B of the NCHRP Report 513 *[11]*. Laboratory fabricated cylindrical samples 3.96 in. in diameter and 5.91 in. in height were tested without confinement. The flow number test was performed at a constant single temperature of 54.4°C and a stress level of 30 psi. A repeated dynamic load for 10,000 repetitions with a loading cycle of 1.0 second in duration and consisting of 0.1 second haversine load followed by 0.9 second rest period was applied to determine the permanent deformation characteristics of paving materials.

Dynamic Modulus |E*| Test. The dynamic modulus test was conducted on unconfined cylindrical samples in accordance with AASHTO Standard TP 62-03 *[10]*. The stress-to-strain relationship under a continuous sinusoidal loading for linear viscoelastic materials is defined by a complex number called the "complex modulus" (E*). The absolute value of the complex modulus, |E*|, is defined as the dynamic modulus. Mathematically, dynamic modulus is defined as the maximum (i.e., peak) dynamic stress (σ_o) divided by the peak recoverable strain (ε_0):

$$|E^*| = \frac{\sigma_o}{\varepsilon_o} \tag{7}$$

A sinusoidal compressive stress was applied to the samples at 4, 25, 37.8, and 54.4°C with loading frequencies of 0.1, 0.5, 1.0, 5, 10, 25 Hz at each temperature to achieve a targeted vertical strain level of 100 microns. An increasing order of temperature (starting with the lowest temperature and proceeding to the highest one) was maintained throughout the test. Testing at a particular temperature began with the highest frequency of loading and proceeded to the lowest one. Triplicate samples were tested for each mixture type considered in this study.

Laboratory Testing Program of Unbound Granular Base Materials

Materials. Three types of unbound granular materials were considered in this study. These materials included: limestone (LS), sandstone (SS), and a 75 percent limestone and 25 percent coarse sand blend (LS-CS). The tested limestone and sandstone materials were taken from selected samples used in the construction of base course layers in Louisiana. The gradation curves of the considered materials are shown in Figure 7. It is noted that there were differences between the gradation curves of the considered materials. The LS, SS, and LS-CS materials have maximum dry unit weights of 138.8, 136.5, and 143.9 lb/ft^3, respectively, and optimum moisture contents of 6.5, 7.1, and 5.9 percent, respectively, as measured by the standard Proctor test.

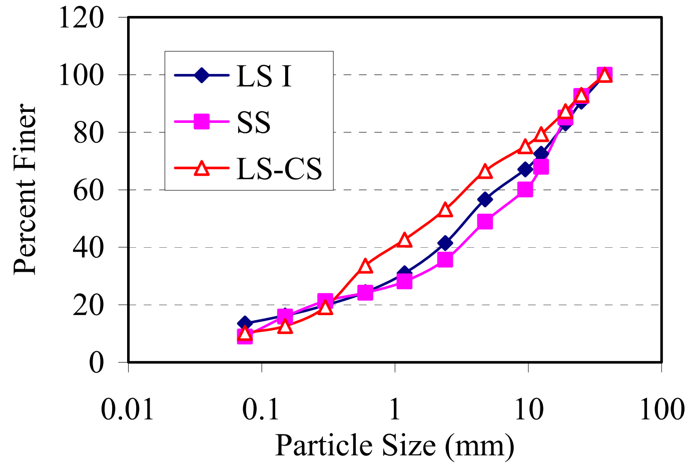

Figure 7
Gradation curves of unbound granular materials

Testing Program. Triaxial tests were also used in this study to characterize the mechanical properties of base course granular materials at their field construction conditions and examine their response under different loading conditions. The Static Compression (SCT) test and two types of Repeated Load (RLT) tests were used for this purpose. The two types of RLT tests included resilient modulus and single-stage and multi-stage RLT tests. The triaxial tests conducted in this study are briefly described below.

Testing Setup

All triaxial tests were performed using the Material Testing System (MTS) 810 machine (Figure 8) with a closed loop and a servo hydraulic loading system. The applied load was measured using a load cell installed inside the triaxial cell. This type of set up reduces the equipment compliance errors as well as the alignment errors. The capacity of the load cell used was ± 22.25 kN. The axial displacement measurements were made using two linearly variable differential transducers (LVDTs) placed between the top platen and base of the cell to reduce the amount of extraneous axial deformation measured compared to external LVDTs. Air was used as the confining fluid to the specimens.

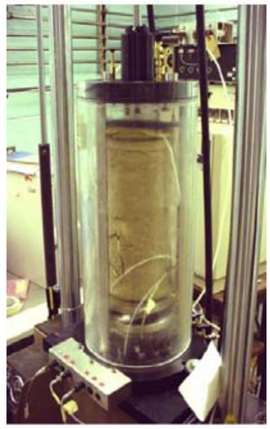

Figure 8
Triaxial test setup

Sample Preparation
All samples were fabricated with 5.91-in. diameters and 12-in. heights using a split mold. Each material was first mixed with the required amount of water needed to achieve its corresponding optimum moisture content as measured in the Standard Proctor test. The material was then placed within a split mold and compacted using a vibratory compaction device to achieve the maximum dry density measured in the Standard Proctor test. To achieve a uniform compaction throughout the thickness, samples were compacted in six 2-in. layers. Samples were compacted to the target moisture contents and dry densities within ±0.5% and ±1% of the target value, respectively. Samples were enclosed in two latex membranes with a thickness of 0.012 in. prior to testing.

Static Triaxial Compression (SLT) Tests. Drained triaxial compression tests were first performed to obtain the shear strength properties of the different materials considered. The triaxial compression tests were performed at three different confining pressures (2, 7, and 10 psi). The strain rate used in those tests was less than ten percent strain per hour to ensure that no excess pore water pressure developed during testing.

Repeated Load Triaxial (RLT) Tests. RLT tests were conducted to examine the stiffness and structural response of the considered granular materials under loading conditions similar to those encountered in the pavement structure. In these tests, a repeated axial cyclic stress with a haversine-shaped load-pulse and fixed magnitude was applied to the tested samples. The load pulse used in this study has a 0.1-sec load duration and 0.9-sec rest period. The resilient and permanent deformations of the samples were continuously measured during this test to compute resilient and permanent strains, respectively. During a RLT test, cyclic deviator and confining stresses along with vertical deformations were recorded. Two different types of RLT tests were used in this study to describe the behavior of the considered granular materials. The following sections describe the procedures followed in these tests.

Resilient Modulus Tests

Resilient modulus tests were performed in accordance with the AASHTO-T307 standard method for determining the resilient modulus of base course materials *[15]*. In this test method, the samples are first conditioned by applying 1,000 load cycles with a deviator stress of 13.5 psi and a confining stress of 15 psi. The conditioning step removes most irregularities on the top and bottom surfaces of the test sample and suppresses most of the initial stage of permanent deformation. This step is followed by a sequence of loading with varying confining and deviator stresses. The confining pressure is set constant, and the deviator stress is increased. Subsequently, the confining pressure is increased, and the deviator stress varied. Resilient modulus values are calculated at a specified deviator stress and confining pressure values.

In order to determine the resilient modulus parameters of tested samples, the average value of the resilient modulus for each stress sequence was first calculated. A regression analysis was then carried out to fit all test data to the generalized constitutive model given in equation (8), which was adopted by the new Mechanistic-Empirical Design Guide (MEPDG) *[16]*.

$$M_r = p_a k_1 \left(\frac{\theta}{p_a} \right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1 \right)^{k_3} \tag{8}$$

where, M_r is the resilient modulus; θ is the bulk stress, τ_{oct} is octahedral shear stress; P_a is a normalizing stress equal to atmospheric stress (P_a = 14.7 psi), and k_1, k_2, k_3 are coefficients of tested material.

Single-Stage RLT Tests

Single-stage RLT tests were performed to determine the permanent and resilient deformations of the considered materials at a different number of load cycles. The test consisted of applying 10,000 load cycles at a constant confining pressure of 3 psi and a peak cyclic stress of 30 psi. The value of the confinement pressure was chosen to match the field measurement of the lateral confining pressure within the base course layer that was reported in different studies *[17]*. The peak cyclic stress was selected based on a previous finite element study *[17]*. Tests were stopped after 10,000 load cycles or when the sample reached a permanent vertical strain of 7 percent. All samples were conditioned before the tests in a way similar to that used in the resilient modulus tests. It is noted that the single-stage RLT procedure is similar to those followed in previous studies *[17]*, *[18]*.

Performance Evaluation Using MEPDG

The new MEPDG software was used to compare the performance of three pavement sections that incorporated LS I asphalt treated mixture layer(s) to a typical section designed for intermediate traffic volume. The cross section of the evaluated pavement sections is presented in Figure 9. Section 1, the control section, consisted of four layers: 2 in. of ½ in. Superpave Level 2 wearing course, 2 in. of ¾ in. Superpave Level 2 binder course, 4 in. of crushed limestone base course, and 10 in. of soil cement stabilized base course. In section 2 and 3, the LS I asphalt treated mixture replaced the ¾ in. Superpave Level 2 binder course mixture and the crushed limestone base course material. Finally, in section 4, the binder course layer was eliminated and the asphalt treated mixture was used as a base course layer.

The analysis was conducted for a 20-year design period. The initial two-way average annual daily traffic (AADT) was assumed to be 2,000 vehicles/day with 20 to 25 percent trucks in the design direction and 95 percent trucks in the design lane. The default values in the MEPDG software for vehicle class distribution, number of axles per truck of each class, and axle configuration were used in the analysis. Monthly adjustment factors were set to 1.0. The traffic growth rate was 5 percent per year. Level I input was used for the HMA and asphalt treated mixtures layers, while, Level II inputs were used for the base, subbase, and subgrade layers. The input parameters for the HMA wearing and binder course mixture were obtained from values reported in a previous study. In addition, the asphalt treated mixture and the unbound granular base material input parameters were obtained from the results of a test conducted in this study. Finally, the input parameters of subbase and subgrade layers were based on experimental studies that were previously conducted by the research team. Table 3 presents a summary of the input parameters used in the MEPDG analysis.

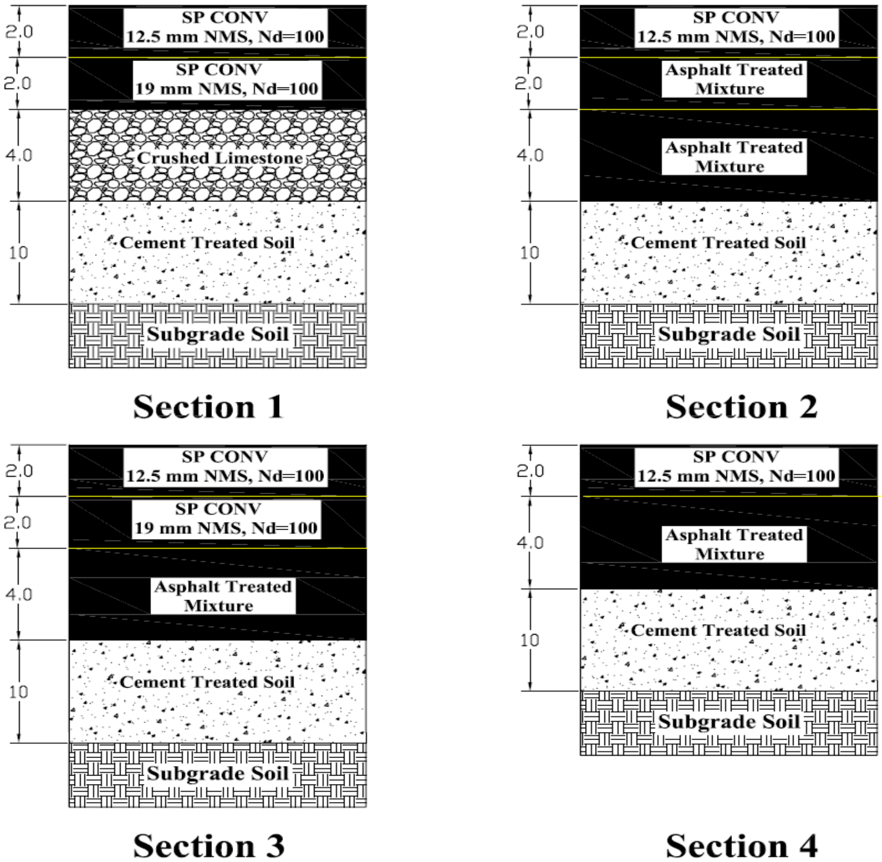

Figure 9
Pavement sections evaluated In MEPDG

Table 3
MEPDG input parameters of pavement layers

Material	Input level	Input Parameter
½ in. Superpave Level 2 Mixture	Level I	E* at 6 frequencies and 5 temperatures
½ in. Superpave Level 2 Mixture	Level I	E* at 6 frequencies and 5 temperatures
LS I Asphalt treated Mixture	Level I	E* at 6 frequencies and 5 temperatures
Crushed limestone Base	Level II	M_r = 40 ksi
Subbase	Level II	M_r = 25 ksi
Subgrade	Level II	M_r = 5 ksi

Part II: Field Study

Projects Overview

Four overlay rehabilitation projects in Louisiana, Figure 10, were selected in Part II of this study to evaluate the constructability of the asphalt treated mixtures designed in Part I. The selection of projects was coordinated with the LADOTD construction and research personnel. In each of the four projects, a one mile test section of the roadway shoulder was constructed using one of the asphalt treated mixtures designed in Part I. Figure 11 presents the cross section of four test sections evaluated in this study.

The first project was located on the LA 3127 Highway near St. James, LA. In this project 2 in. were milled of two existing 12-ft. roadway lanes with 10-ft. shoulders. Two inches of Level II binder was placed on the roadway. For this research project, 2-in. of the proposed asphalt treated mixture were placed on the 1-mile section of the shoulder of the north bound lane between stations 80+00 and 130+00. A 2-in. layer of Level A shoulder mixture was placed on top of the asphalt treated layer as shown in Figure 11.

The second project was located on US Highway 425 near Rayville, LA. Two inches of a 2-lane roadway with 10-ft. shoulders were milled in this project. Two inches of Level II binder course HMAC and a 1.5 inch Level II wearing coarse HMAC were placed on the roadway. In addition, 3.5 in. of the asphalt treated mixture was placed on both shoulders of the 1-mile test section. It is noted that the full depth of the ATM layer was placed in one lift, which was thought to be ideal by the contactor.

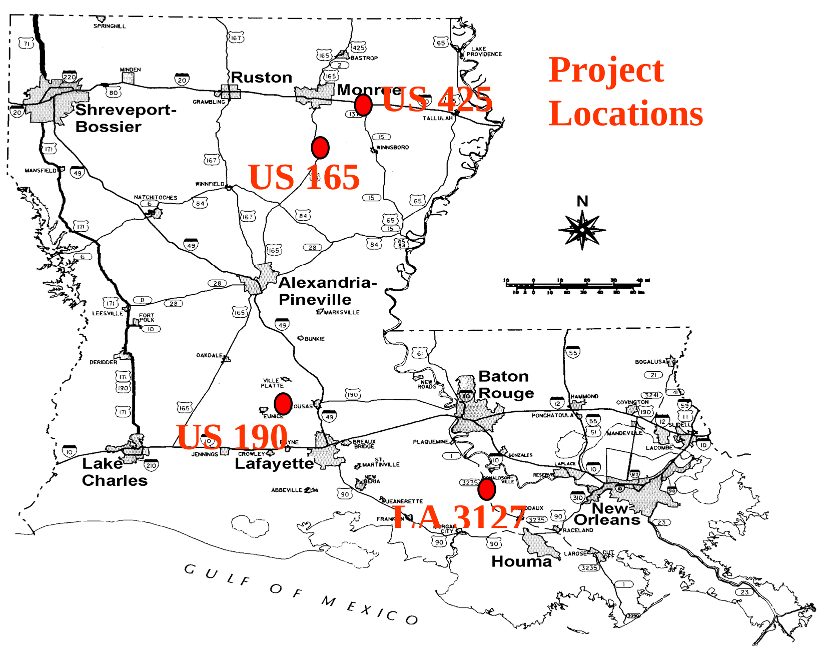

Figure 10
Highway projects investigated in this study

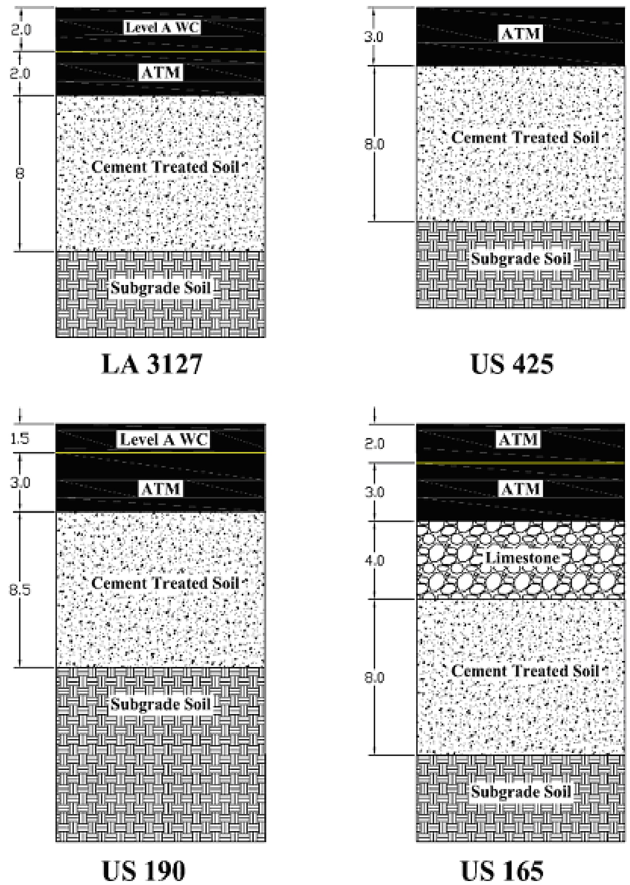

Figure 11
Cross-section of tested section

The third project was located on US Highway 190 near Lawtell, LA. It consisted of a four-lane divided roadway with 10-ft. shoulders. In this project, the asphalt treated mixture was used in the construction of a 1-mile test section of the shoulder. In this section, the shoulder structure consisted of an 8.5-in. cement treated base course (9 percent cement by volume), 4.5-in. layer of asphalt treated mixture placed in two lifts, and a 1.5-in. Level A shoulder mixture surface layer, Figure 11.

Finally, the fourth project was located on US Highway 165 near Rilla, LA. The US 165 project is a newly widened four lane divided roadway located near Rilla, La. It consists of two twelve foot roadway lanes with ten foot outside shoulders and 4 foot inside shoulders in each roadway. The planned construction for the shoulders consisted of 8 inches cement treated (6% cement by volume) base and 3 inch average Level A shoulder mixture with 2 inches Level A shoulder mixture for the surface. For our research study, the Asphalt Treated Aggregate Mixture (ATAM) replaced the planned Level A mixture for the length of the project in the south bound lane. The test section was placed between log mile 4.2 and 10.7, approximately 6.3 miles.

Materials. Two types of asphalt binders meeting LADOTD specifications, PG 70-22M and PG 64-22, were used in this study. It is noted that PG 70-22M was a SBS elastomeric polymer-modified binder. Limestone, Navoculite, and coarse sand were aggregates used in the mixtures evaluated in Part II. It is further noted that 20 percent of reclaimed asphalt pavement (RAP) was used in mixture used in US 165 project.

Mixtures Design. The aggregate structure of asphalt treated mixtures used in the construction of the test sections consisted of 75 percent of minus 1.5-in. sieve crusher run materials from each of the selected aggregate sources and 25 percent of coarse sand (CS). Figure 12 presents gradation curves of the aggregate of the evaluated mixtures. Three percent asphalt cement content was used for those mixtures. Thirty gyrations were used to determine the density to which mixtures should be compacted in the field. Figure 13 shows the rate of change in height with the number of gyration curves obtained for the different asphalt treated mixtures evaluated in this phase. It is noted that the curves for the different mixtures reached an asymptotic value at about 28 gyrations. This is consistent with the results obtained for the mixtures evaluated in Phase I. The job mix formula (JMF) of all field mixtures is summarized in Table 4.

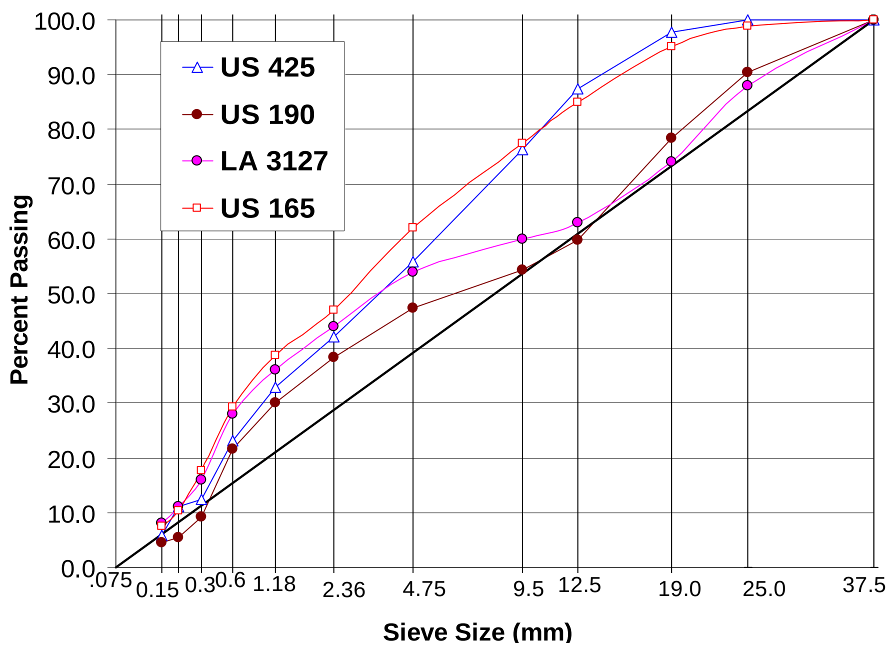

Figure 12
Aggregate gradation curves of mixtures used in the field

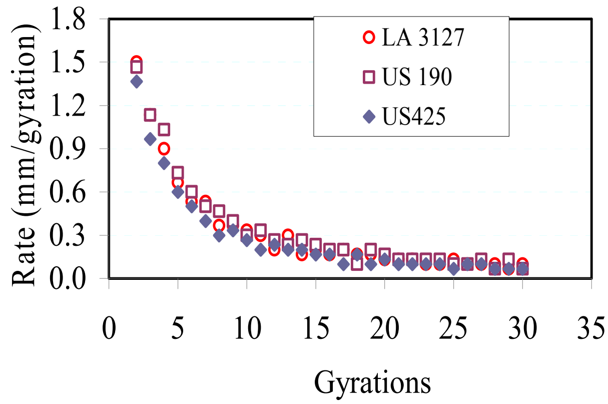

Figure 13
Rate of change in height with number of gyrations

Construction of Asphalt Treated Mixture Layer. Asphalt treated mixtures used in the construction of test sections evaluated in this study were produced, delivered, and placed using conventional equipment as used in typical HMA mixtures. A nuclear gage was used to determine the rolling pattern for each project. In general, the rolling pattern used in the compaction of asphalt treated mixtures included first performing compaction passes using vibratory steel roller; this is followed by finishing compaction passes using a static steel roller. Table 5 summarizes the rolling pattern used in the compaction of the asphalt treated mixture in each project. It is noted that the contractor and LADOTD engineers did not report any problem in the production, delivery, and placement of the asphalt treated mixtures. Figure 14 depicts pictures taken during the construction of the asphalt treated layer.

Laboratory Characterization of Mixtures in Part II

For each test section, sufficient loose mixtures were secured within paver extensions and plants and sent to the laboratory. Attempts were made to compact laboratory samples to an air void content similar to the roadway cores air void within ± 1 percent. Beam samples were compacted using a kneading compactor. In addition, cylindrical samples were compacted with the Superpave gyratory compactor (SGC).

Table 4
Job mix formula for asphalt treated mixtures in Phase II

Mixture Designation	LA 3127	US 425	US 190	US 165
Aggregate blend	75% LS II 25% CS	75% NV 25% CS	75% LS II 25% CS	60% LS II 20% RAP 20% CS
Binder type	PG70-22M	PG70-22M	PG 64-22	PG70-22M
Design binder content, %	3.0	3.0	3.0	3.0 (2.5+0.5 RAP)
Design air void, %	8.0	10.4	9.0	6.7
Metric	Gradation, (% passing)			
37.5 mm	100	100	100	100
25 mm	96	100	97	97
19 mm	87	100	84	84
12.5 mm	68	98	64	65
9.5 mm	59	89	49	52
4.75 mm	35	50	29	32
2.36 mm	23	29	22	24
1.18 mm	17	19	18	20
0.6 mm	13	13	14	15
0.3 mm	7	10	7	8
0.15 mm	4	–	4.3	4.9
0.075 mm	3.6	6.5	3.1	3.6

Table 5
Summary of rolling pattern used in each project

Project	Established Rolling Pattern
LA 3127	2 – Vibratory S.R., 2– Static S.R., 2– Pnuematic Tire
US 425	3 – Static Steel Roller
US 190	5 – Vibratory Steel Roller, 3 – Static Steel Roller
US 165	5 – Vibratory Steel Roller, 3 – Static Steel Roller

Figure 14
Construction of asphalt treated mixtures layers

Various laboratory tests were performed to examine the physical and mechanical properties of the asphalt treated mixtures that were used in the construction of field test sections. The laboratory tests performed in this phase included: permeability, ITS, LWT, Lottman, SCB, DCSE, dynamic modulus |E*|, and flow number (F_N) tests. Triplicate samples were used for each test except for LWT test, in which duplicate samples were tested. A description of those tests is provided in previous sections.

Field Non-Destructive Tests

Cores were obtained at 15 test points at each of the 1-mile test sections in the four selected projects. In addition, FWD, LFWD, and PSPA were performed at the test points to characterize in-situ properties of asphalt treated mixtures. Figure 15 presents the field tests layout used in this study. It should be noted that for LA 3127 only the FWD and PSPA were conducted. A description of the in-situ tests conducted in this study is provided next.

Falling Weight Deflectometer (FWD). The Dynatest Model 8000 was used in this study to conduct all FWD tests. This device applies a transient load with a frequency of 30 Hz to the pavement layer by dropping a weight from a specified height on an 11.81-in. circular loading plate with a thin rubber pad mounted underneath. Different load magnitudes can be generated by varying the mass of weight and drop height. A 9,000-lb. load level was used in this study. The pavement deformation induced by the applied load is obtained using sensors (geophones) located at a different distance from the center of the load plate. In this study, the deformation was obtained using nine sensors. Based on the measured load and deflections, the elastic moduli of the tested pavement layers were backcalculated using ELMOD 5.1.69 software *[19]*. A Linear backcalculation model with no seed values was used to backcalculate the FWD moduli.

Light Falling Weight Deflectometer (LFWD). The LFWD is a portable device, which is designed for estimating the elastic modulus of unbound materials in pavements. The Dynatest 3031 LFWD device was selected for testing in this study. The device consists of a 22 lb. drop weight that falls freely onto a loading plate that has a 5.9-in. diameter, producing a load pulse and one geophone sensor to measure the center surface deflection. The output from the test is called the dynamic deformation modulus E_{LFWD}, which is calculated from the center deflection measured based on the Boussineq elastic half space theory. The equation used to calculate the modulus E_{LFWD}, which was used in the subsequent section of analysis, is as follows:

$$E_{LFWD} = \frac{2(1-\nu^2)\sigma \times R}{\delta_c} \tag{9}$$

where, σ is the applied stress, MPa; R is the loading plate radius, mm; δ_c is the deflection measured under the plate, mm; and ν is the Poisson's ratio assumed (0.35).

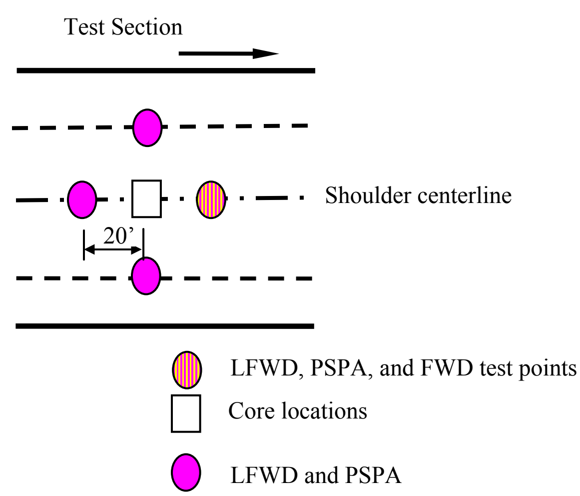

Figure 15
Field tests layout

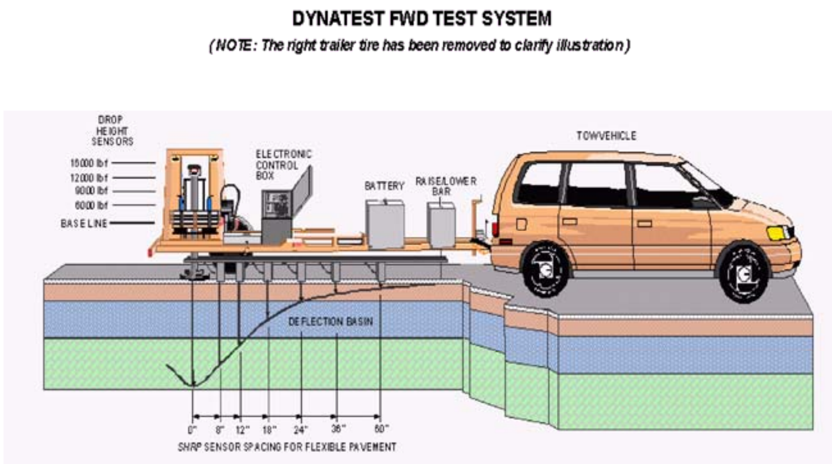

Figure 16
Dynatest Model 8000 (FWD) (LTRC, 2000)

Portable Seismic Pavement Analyzer (PSPA). The PSPA is a device designed to determine the average modulus of the top layer of pavements. It consists of two receivers (accelerometers) and a source packaged into a hand-portable system, which can perform high frequency seismic tests. The operating principle of the PSPA is based on generating and detecting stress waves in a medium. The Ultrasonic Surface Wave (USW) method, an offshoot of the Spectral Analysis of Surface Wave (SASW) method, is used to determine the modulus of the material *[20]*.

DISCUSSION OF RESULTS

The results of Part I and Part II of this study are presented in the following sections. It is noted that in the proceeding discussion the following designation will be used for the mixtures evaluated:

- **LS I**: Mixture with limestone I and PG 70-22M binder
- **LS II**: Mixture with limestone II and PG 70-22M binder
- **PLS I**: Mixture with porous limestone and PG 70-22M binder
- **SS**: Mixture with sandstone and PG 70-22M binder
- **GR**: Mixture with granite and PG 70-22M binder
- **NV**: Mixture with Navoculite and PG 70-22M binder
- **Ry**: Mixture with Rhyolite and PG 70-22M binder

Results of Part I: Laboratory Study

Results Laboratory Testing Program of Asphalt treated Mixtures-Phase I

Physical Properties Test Results. Table 6 presents a summary of the physical properties of asphalt treated mixtures aggregates. It is noted that all aggregates consisted of stones with at least one fractured surface. Furthermore, they exhibited similar fine angularity levels. The sandstone possessed a much lower sand equivalent value than other aggregates, which indicates that it has a higher clay-like content. As expected, the porous limestone aggregates had much higher absorption values compared to all other aggregates considered. In addition, this aggregate had the highest percentage loss as measured in the Micro-Deval test. This indicates that these aggregates have a lower ability to resist degradation during construction and under traffic loading. Figure 16 presents the relation between absorption and Micro-Deval loss for the considered aggregates. It is noted that the Micro-Deval loss increased linearly with the increase in absorption. Furthermore, excellent correlation with high coefficient of determination, R^2, of 0.9 exists between the two parameters.

Aggregate Gradation Analysis. The design gradations of different mixtures considered in this study were evaluated using the power-law method suggested by Ruth et al. *[21]*. The power-law shown in equation (8) characterizes the slope and the intercept constants of the coarse aggregate (CA) and fine aggregate (FA) portions of the gradation. In this study, a sieve size of 2.36 mm was selected as a divider for the CA and the FA portions in the regression analysis. Table 6 presents the power law gradation parameters for the considered aggregate blends. It is noted that the higher n_{ca} value, the coarser the CA portion, while a higher n_{fa} value indicates that the FA portion of an aggregate gradation is finer. The

GR aggregate blend had the highest n_{ca} value of 0.55, while the NV aggregate blend had the lowest n_{ca} value of 0.16. In addition, the GR aggregate blend had lowest n_{fa} value of 0.38, while the LS I aggregate blend had the highest n_{fa} value of 0.38. This indicates that the GR aggregate blend had the coarsest CA and FA portion of the gradation. While NV, and LS I aggregate blends were the finest on the CA and FA portion of the gradation, respectively. It is worth noting that n_{fa} for the NV aggregate blend was relatively higher than that of other aggregates and was close to the n_{fa} value of the LS I aggregate blend.

$$P_{CA} = a_{CA}(d)^{n_{CA}} \text{ and } \quad P_{FA} = a_{FA}(d)^{n_{FA}} \tag{10}$$

where, P_{CA} and P_{FA} are the percent by weight passing a given sieve that has an opening of width; a_{CA} is the intercept constant for the coarse aggregate; n_{CA} is the slope (exponent) for the coarse aggregate; d is the sieve opening width, mm; a_{FA} is the intercept constant for the fine aggregate; and n_{FA} is the slope (exponent) for the fine aggregates.

Table 6
Physical and strength properties of aggregate sources and blends

Parameter	LS I	LS II	PLS	PLS II	SS	GR	NV	RY	Sand
G_{sb}	2.601	2.564	2.252	2.259	2.529	2.658	2.623	2.429	2.618
G_{sa}	2.707	2.712	2.537	2.599	2.642	2.719	2.665	2.591	2.643
Absorption (%)	1.5	2.1	5.0	5.8	1.7	0.9	1.6	2.6	0.4
Micro-Deval	12.6	13.0	28.5	33.9	11.6	5.9	6.1	10.2	NA
CAA(%)	100	100	100	100	100	100	100	100	NA
FAA(%)	42.9	43.0	37.1	37.09	46.2	44.7			43
Sand Eq. (%)	55	44	41	54.3	24	72			56
Power Law Analysis of Gradation of Aggregate Blends									
Parameters	LS I	LS II	PLS	PLS II	SS	GR	NV	RY	
a_{ca}	28.2	46.7	51.9	37.2	38.6	15.3	57.7	29.3	
n_{ca}	0.35	0.21	0.19	0.29	0.27	0.55	0.16	0.34	
a_{Fa}	33.0	40.0	36.8	31.763	41.7	14.8	27.7	27.4	
n_{Fa}	0.68	0.56	0.57	0.54	0.66	0.38	0.66	0.56	

CAA: Coarse aggregate angularity, FAA: Fine aggregate angularity, Sand Eq.: Sand Equivalent

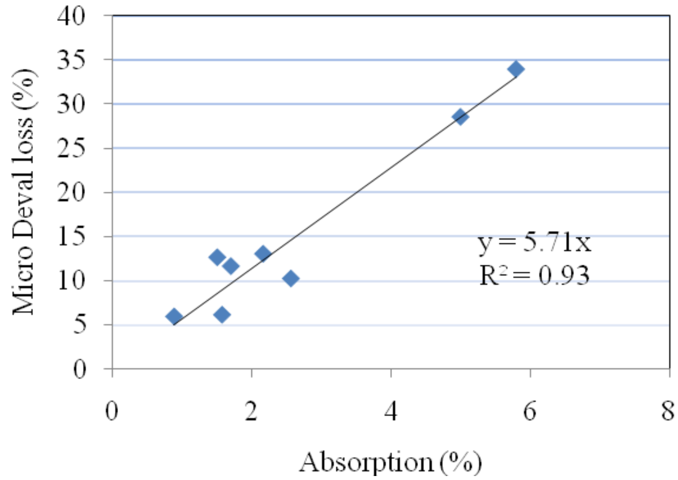

Figure 17
Absorption micro-deval loss relation

Indirect Tensile Strength (ITS) Test Results. Table 7 presents the mean and coefficient of variation of the indirect tensile strength values for mixtures at 25ºC. Higher ITS values are desirable as they correspond to a strong and durable mixture. In general, the coefficient of variation of the ITS values were less than 10 percent, which suggest that the test results were repeatable. It is noted that the porous limestone mixtures were the only ones that did not meet the screening criterion of having a minimum ITS value of 150 psi.

Table 7
Summary of ITS test results

Parameter	LS I	LS II	PLS I	PLS II	SS-70	GR	NV	RY
Mean (psi)	212	186	118	91	190	193	193	214
COV (%)	2	7	5	12	10	8	7.6	7.6

STD: Standard deviation; COV: Coefficient of variation

Loaded Wheel Tracking (LWT) Test Results. Figure 18a presents the rut depth of asphalt treated mixtures after 20,000 passes in the LWT test. In addition, Figure 18b compares the stripping inflection point observed in the LWT test for different mixtures considered in this study. Asphalt treated mixtures were considered to pass the LWT test if the rut depth of the specimen remained less than 12 mm after 20,000 passes. It is noted that the asphalt treated mixtures containing porous limestone aggregates were among the ones that exhibited the highest rut depth and lowest number of cycles to stripping. Thus, those mixtures were considered to be susceptible to moisture damage.

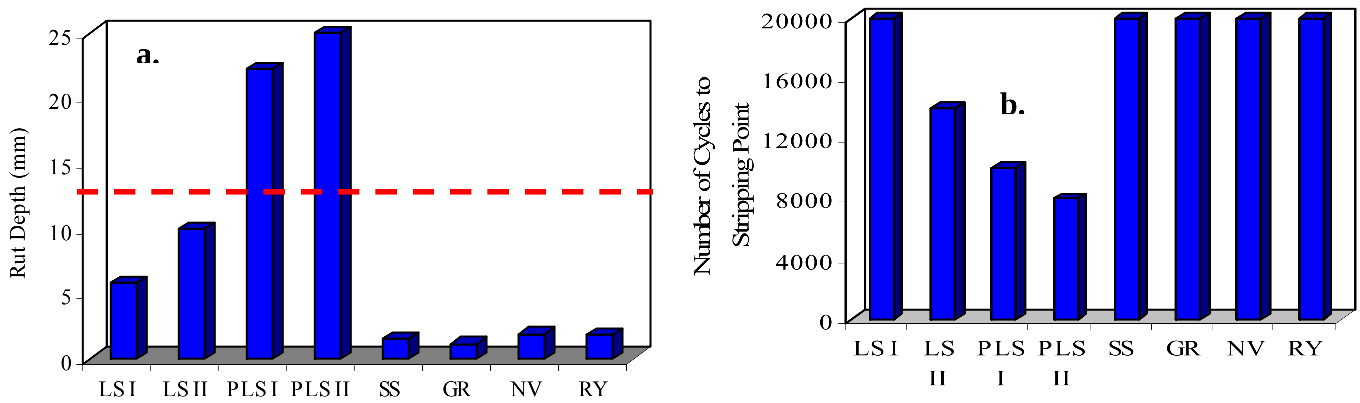

Figure 18
LWT test results: (a) rut depth (b) stripping inflection point

Summary of Laboratory Testing Program- Phase I

The test results of the screening procedure of Phase I of the laboratory testing program indicated that mixtures containing the porous limestone, PLS I and PLS II, were the only ones that did not meet ITS and LWT passing criteria. This suggests that the 3 percent asphalt content used in this study did not provide sufficient film thickness for binding the aggregate particles together in PLS I and PLS II mixtures. It is worth noting that those aggregate sources had the highest absorption value and percentage loss as measured in the Micro-Deval test among the aggregate source evaluated in this study. This may suggest that the absorption and results of the Micro-Deval test can be used as a screening criterion for selecting the aggregate types for asphalt treated mixtures.

Results of Laboratory Testing Program-Phase II

Permeability. Permeability of asphalt mixtures is an important factor that affects its durability. Mixtures with high permeability are believed to have a greater number of interconnected voids, allowing air and water to penetrate into the pavement structure. Air increases the rate of oxidation of the asphalt binder, which can lead to binder hardening and ultimately pavement cracking. The presence of water within asphalt mixtures leads to weakening the bond between the aggregate and binder, a phenomenon known as stripping. In this study, a falling head permeability apparatus was used to determine the rate of flow of water through asphalt treated mixtures. Figure 19 presents the average coefficient of permeability for all mixtures evaluated in this phase. It is noted that all mixtures except the NV mixture showed a good permeability level that is lower than $125*10^{-4}$ mm/sec. This indicates that the NV mixture may have durability and stripping problems in the field.

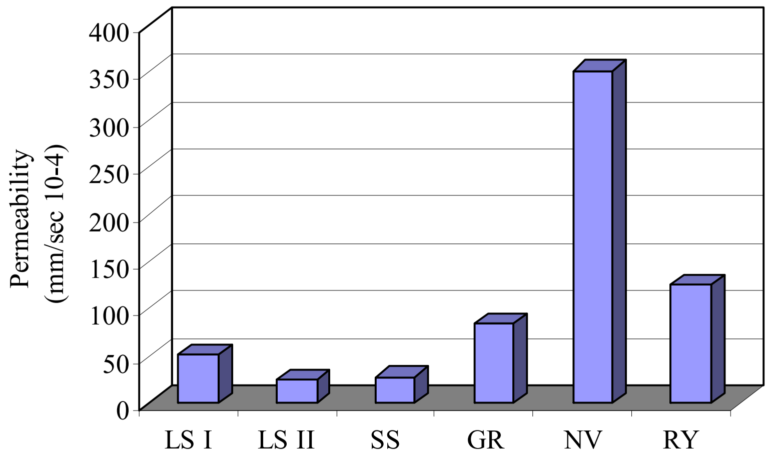

Figure 19
Results of permeability test

Modified Lottman Test Results. This test quantifies the asphalt treated mixtures sensitivity to moisture damage which is necessary to assure its durability. Moisture sensitivity is measured by the percentage of retained tensile strength ratio of the conditioned samples compared to the control samples. Louisiana requires a minimum retained tensile strength of 80 percent for HMA mixtures. Figure 20 presents the measured retained tensile strength values for asphalt treated mixtures evaluated. It is noted that all mixtures except the SS mixture had retained tensile strength values greater than 70 percent. Furthermore, LS II, GR, and RY had TSR values greater than 80 percent.

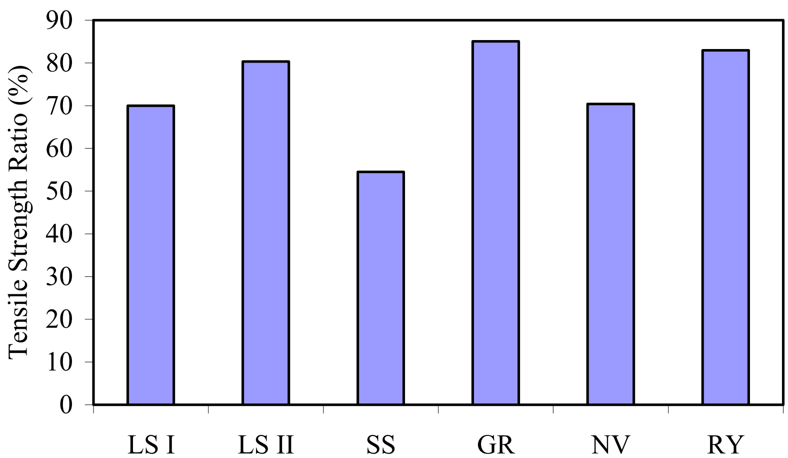

Figure 20
Modified Lottman retained tensile strength

Semi-Circular Bend (SCB) Test Results. Figure 21a shows the average computed critical fracture resistance (J_c) values for asphalt treated mixtures evaluated in this study. The J_c represents the fracture resistance of a material; therefore, the higher the value of J_c the better fracture resistance the material possesses. It is noted that significant differences in the J_c value existed among asphalt treated mixtures with different aggregate sources considered in this study. GR and NV mixtures had the highest and lowest J_c values of 1.35 KJ/m^2 and 0.24 KJ/m^2, respectively. Thus, GR and NV mixtures have the best and worst fracture resistance, respectively. Furthermore, mixtures containing limestone aggregates had higher J_c values than the SS mixture.

Considering critical fracture resistance data from other studies, a mixture achieving a J_c value greater than 0.5 KJ/m^2 is expected to exhibit good fracture resistance *[8], [14]*. This suggests that, based on the SCB test result, all considered asphalt treated mixtures except SS and NV mixtures would be expected to show satisfactory field performance against fracture resistance as all of them obtained J_c values higher than the minimum required value of 0.5 KJ/m^2.

One of the physical properties that have been linked to the asphalt mixtures cracking resistance is the film thickness. In general, studies showed that asphalt mixtures with low film thickness are generally considered to be more susceptible to oxidation, which causes the mix to become brittle, reducing cracking resistance *[22], [23]*. Figure 21b presents the J_c relationship with film thickness. It is noted that there is a trend in the two parameters, such that J_c increased with the increase in the film thickness.

The influence of aggregate gradation on the J_c value of the asphalt treated mixtures was also evaluated. Figure 21c and 21d present the relation between n_{ca} and n_{fa}, the power law parameters characterizing the coarse and fine portions of aggregate gradation, respectively, and the J_c value. It is noted that a strong relation exists between J_c and n_{ca} and n_{fa}, such that the coarser the CA and FA portions of the gradation the higher the J_c value.

Dissipated Creep Strain Energy (DCSE) Test Results. Dissipated creep strain energy is limiting energy that a mixture can stand before it fractures. Roque et al. reported that a DCSE value of 0.75 KJ/m^3 was the value to differentiate cracked and uncracked pavements *[9]*. Pavements having a DCSE value greater than 0.75 KJ/m^3 did not crack and vice versa. Therefore, mixtures having lower DCSE values are considered more vulnerable to cracking than mixtures having higher DCSE values when both mixtures are exposed to similar loading and environmental conditions.

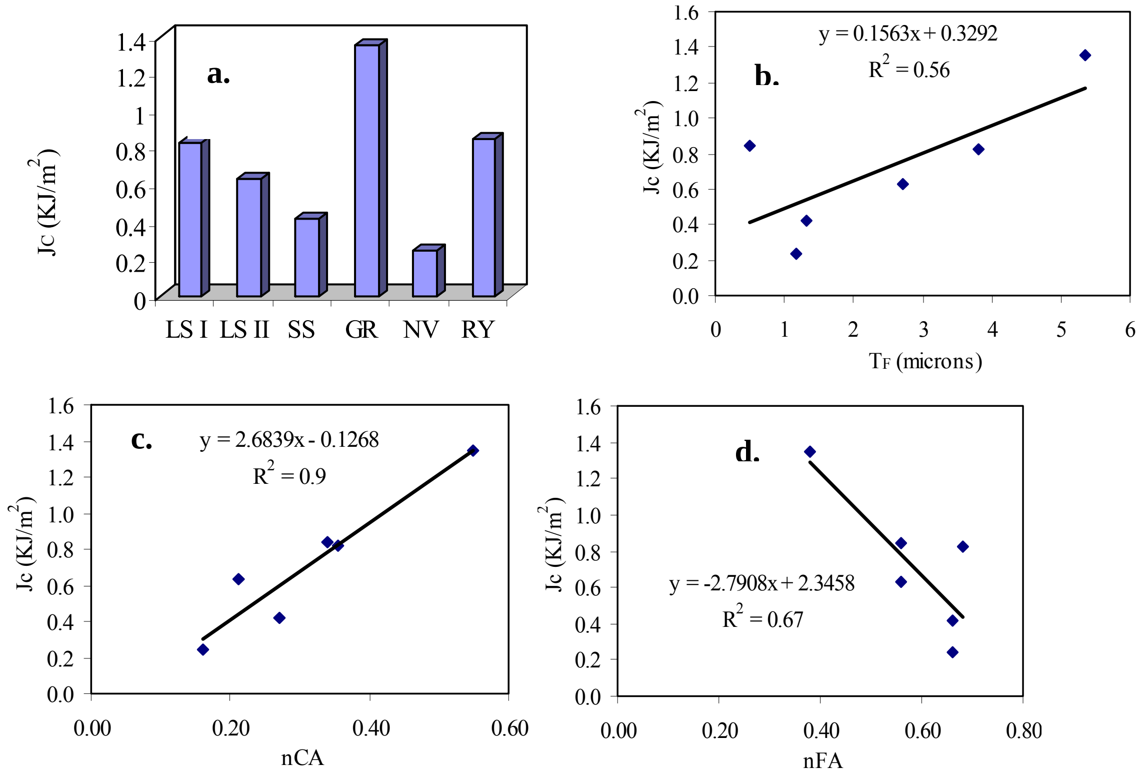

Figure 21
SCB results: (a) SCB values, (b) SCB relation T_F, (c) SCB relation with n_{ca}, and (d) SCB relation with n_{fa}

The DCSE test and calculation procedure followed in this study was introduced by Roque et al. *[9]*. Three samples were tested for each asphalt treated mixture. Figure 22a presents the mean DCSE values for mixtures evaluated in Phase II. In general, DCSE test results were consistent with those obtained from the SCB test. The GR mixture had the highest DSCE value of 1.4 KJ/m^3. Furthermore, NV and RY mixtures were the only ones that had DCSE values less than 0.75 KJ/m^3, indicating that all other mixtures would have satisfactory fatigue cracking resistance in the field.

The effect of film thickness on the DCSE value of asphalt treated mixtures was evaluated as shown in Figure 22b. It is noted that a good correlation (R^2 = 0.74) exists between the film thickness and the DCSE value. This suggests that the film thickness has a more pronounced influence on the fatigue cracking properties of asphalt treated mixtures at lower temperature. Significant differences in DCSE values between the evaluated asphalt treated mixtures suggest that the aggregate source also had significant effect on their fatigue cracking resistance. When examining the aggregate properties that affect the DSCE value, it was found that gradation parameters did not exhibit good relation with the DCSE value. However, a good correlation was found between the aggregate absorption and the DSCE, which is presented in Figure 22c. It is clear that higher DCSE values were obtained for aggregates that posses lower absorption values.

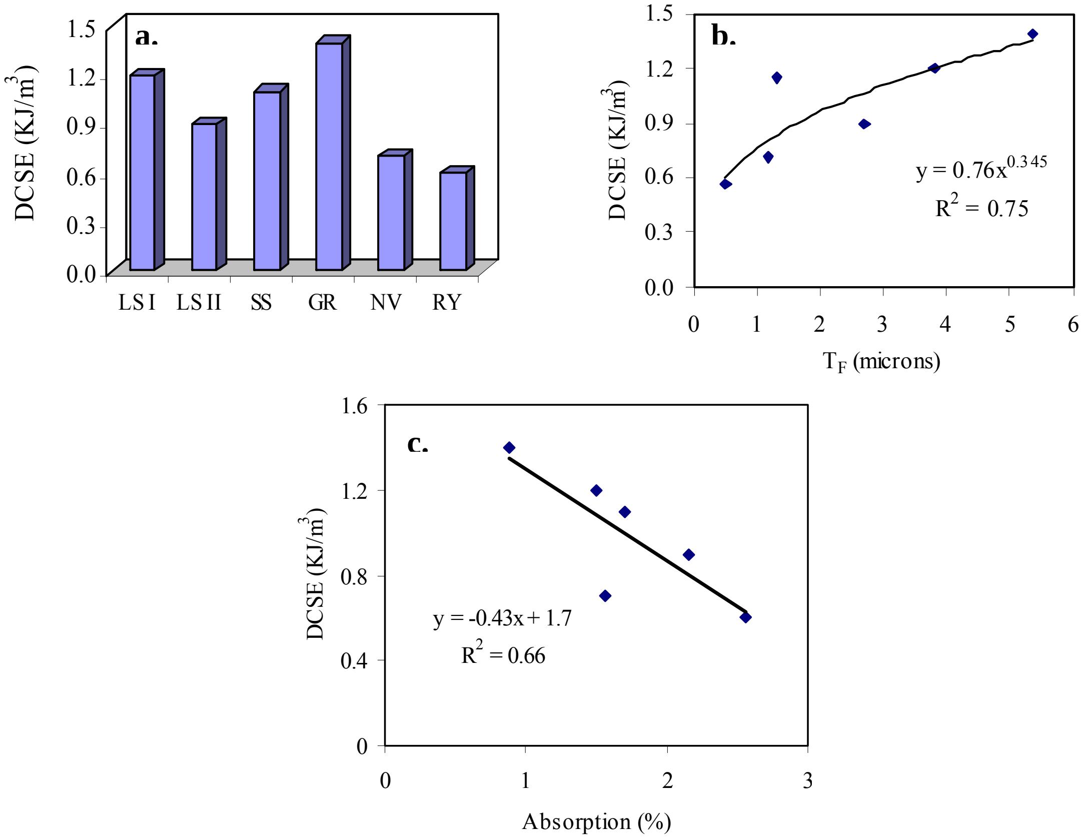

Figure 22
DCSE test results: (a) DCSE values, (b) DCSE relation T_F, and (c) DCSE relation with absorption

Flow Number (F_N) Test Results. Figure 23 presents the flow number results for the asphalt treated mixtures evaluated in Phase II of the laboratory test program. Each vertical bar represents the average flow number value of three samples. It is worth mentioning that, in general, the coefficient of variation of F_N test results were about 18 percent, which was higher than those obtained for other tests conducted in this study. Figure 23 shows that GR and NV mixtures had low flow number values. The low flow number value of the NV mixture may be attributed to its finer aggregate gradation (low n_{ca} and high n_{fa}), especially on the coarse portion of the gradation.

Although the GR mixture possessed a lower flow number value than other mixtures, it exhibited good rut resistance in the LWT test. This indicates that the response of this mixture at high temperatures was significantly affected by the confinement stress, especially since the F_N test was conducted without confinement. The coarse gradation of GR mixture (as indicated by the n_{ca} and n_{fa} parameters) may explain the significant effect that confinement had on its response at high temperature. This shows that for asphalt treated mixtures, the aggregate skeleton plays a major role in its performance at high temperatures.

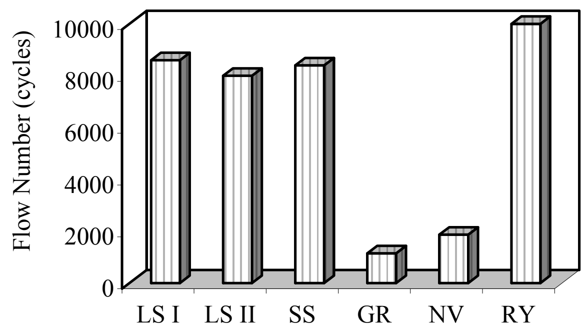

Figure 23
Flow number test results

Dynamic Modulus (E*) Test Results. Figures 24a-c show the dynamic modulus isotherms for all tested mixtures at different temperatures and frequencies, where each isotherm represents the average E* value of three samples. In general, the coefficient of variation of E* test results were about 15 percent. It is noted that E* values for all mixtures increased with an increase in frequency and a decrease in temperature. At low temperatures (4.4°C), E* isotherms maintained the pattern of inclined straight-line, which indicated that the mixture behavior was in the linear viscoelastic region at those temperatures. However, at intermediate and high temperatures (25°C and 54.4°C), E* isotherms gained a concave shape (Figures 24a and 24b) which represents the non-linear behavior in asphalt treated mixtures under compression. It is noted that significant differences in E* values were detected at 4°C; however, E* values of different mixtures were much closer at 54.4°C. It is noted that the LS II and SS mixtures consistently exhibited the highest and lowest E* values, respectively.

The variation of phase angles with the dynamic modulus is shown in Figure 24d for the six frequencies and five temperatures for each mixture tested in this study. The phase angle increased with increasing frequency, reached a peak, and then decreased. This response is different from that of the typical asphalt binder response in which the phase angle generally decreases with an increasing frequency. The reason for this is that at a high frequency (low temperature) the asphalt binder primarily affects the phase angle of asphalt mixtures, i.e., binder viscoelastic follows similar trend. However, at a low frequency (high temperature), it is predominantly affected by the aggregate, and therefore, the phase angle for asphalt treated mixtures decreases with a decreasing frequency or increasing temperature because of the aggregate influence.

A rut factor defined as $|E^*|/\sin\delta|_{5hz\ \&\ 54.4C}$ was computed from the dynamic modulus test to examine the permanent deformation (i.e., rutting) characteristics of the asphalt treated mixtures.

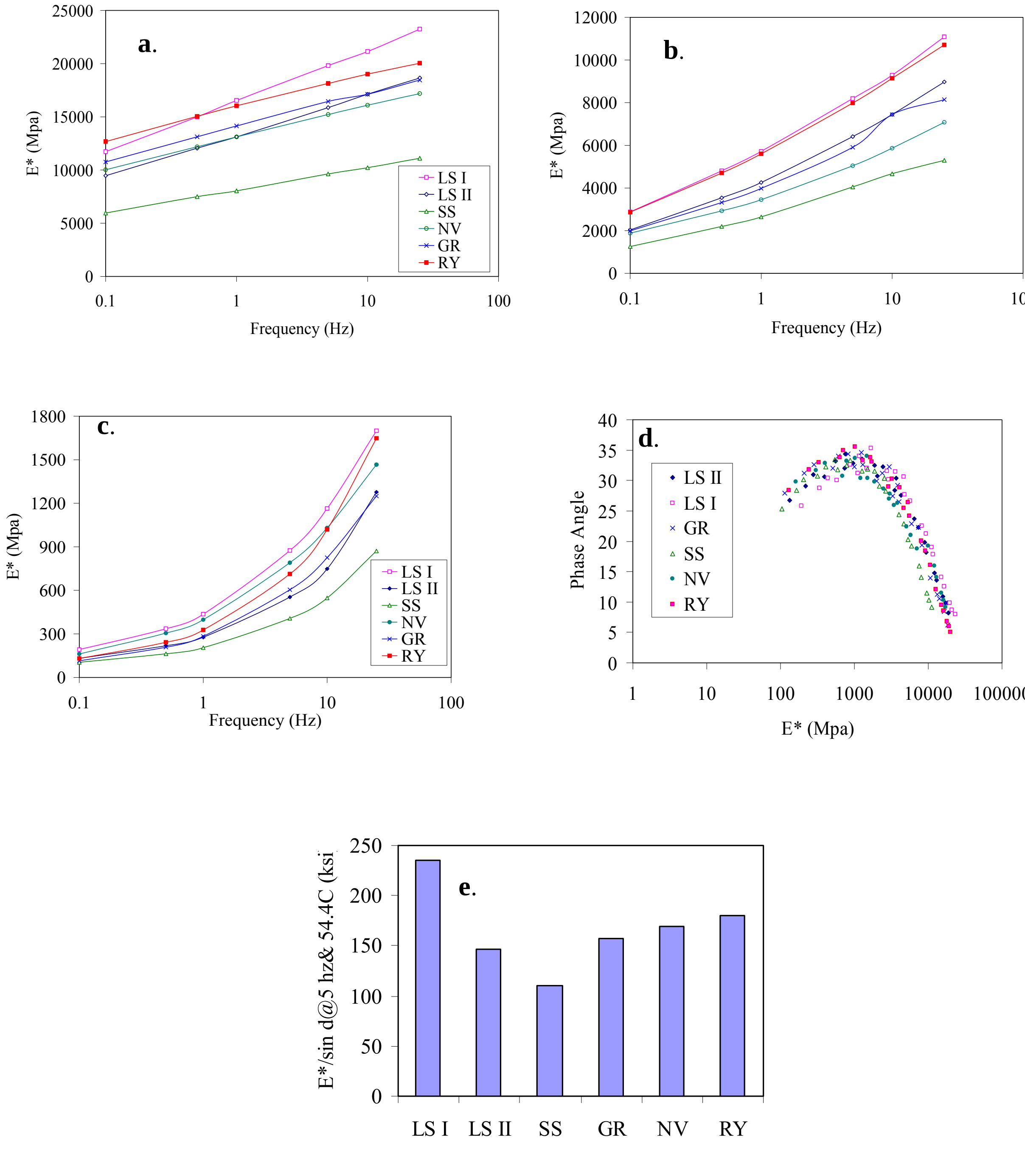

Figure 24
E* test results: (a) E* isotherms at 4.4ºC, (b) E* isotherms at 25ºC, (c) E* isotherms at 54.4ºC, (d) variation of phase angle with E*, and (e) rutting factor at 5Hz and 54.4ºC

Figure 24e presents the average rutting factor values at 5Hz and 54.4ºC for all mixtures included in this study. It is noted that all mixtures had rutting factor values comparable to those of good performing conventional base course HMA mixtures *[24]*. This indicates that those mixtures have rut resistance comparable to conventional base course HMA mixtures, which is consistent with results of other tests conducted in this study at high temperatures. Figure 24e shows that some difference was observed for mixtures with different aggregate sources. The highest and lowest rutting factor values were obtained for LS I-70 and SS-70 mixtures, respectively.

Effect of Binder Type on Asphalt Treated Mixtures Performance

The influence of the asphalt binder type on the high temperature rutting resistance of asphalt treated mixtures was evaluated using LWT and FN tests. Figures 25a and 25b present the results of the LWT and FN tests, respectively. For the same aggregate source, it is noted that mixtures containing PG 64-22 asphalt binder exhibited a higher rut depth than those with PG 70-22 asphalt binder. Furthermore, mixtures with PG 64-22 asphalt binder showed rut depths greater than 0.48-in. so did not meet the LWT criterion adopted in this study. Figure 25b shows that mixtures containing PG 70-22 asphalt binder exhibited much higher flow number values than those with PG 64-22 asphalt binder, so they had better rutting resistance. This is consistent with results of the LWT test, which suggests that the asphalt binder type has a considerable effect on high temperature properties of asphalt treated mixtures.

The effect of the binder type on fatigue cracking resistance of asphalt treated mixtures was also evaluated in this study using SCB and DCSE tests. Figures 25c and 25d compare the critical fracture resistance (J_c) and DCSE values for asphalt treated mixtures with the PG70-22M to those containing PG64-22, respectively. For the same aggregate source, it is noted that mixtures containing PG 64-22 binder exhibited lower J_c and DCSE values compared to those with PG 70-22M binder. This suggests mixtures with PG 70-22M possess better fatigue cracking resistance than that those with PG 64-22 asphalt. Thus, the asphalt binder also had a significant effect on fatigue cracking properties of asphalt treated mixtures.

Summary of Laboratory Testing Program-Phase II

Table 8 presents a summary of results of tests conducted in Phase II. It is noted that among all asphalt treated mixtures the NV mixture exhibited the worst rutting and fracture

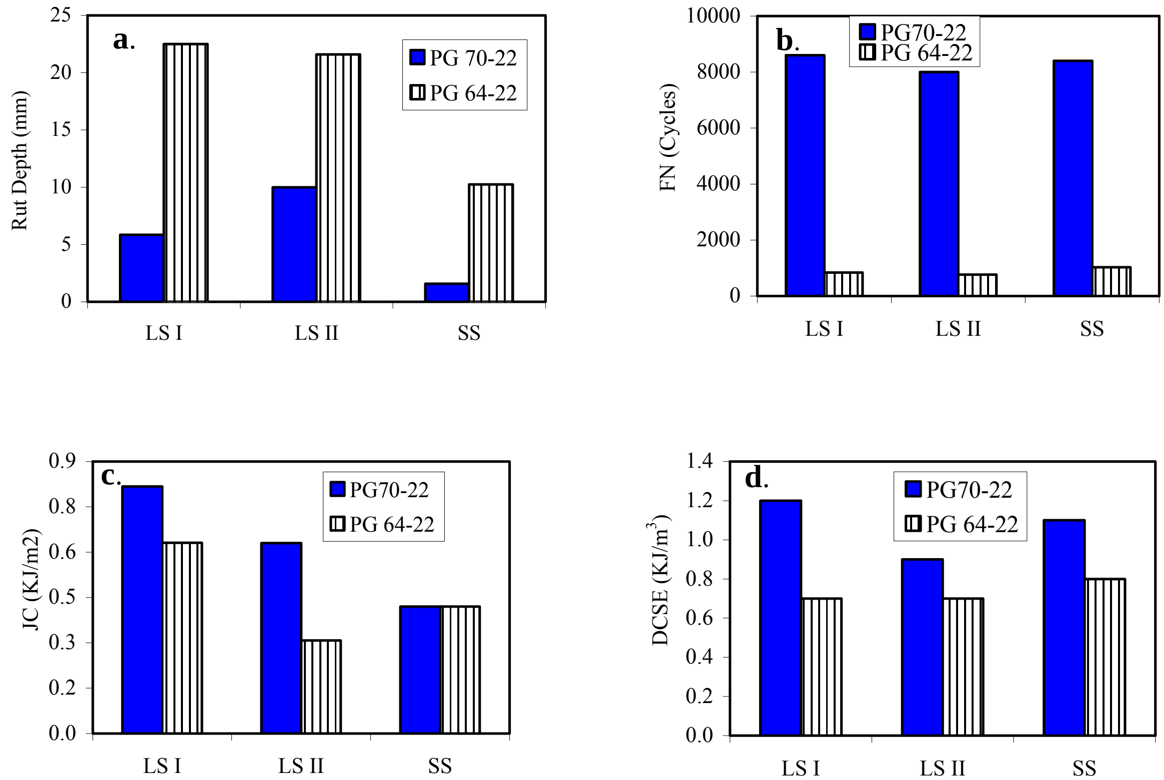

Figure 25

Results of partial test factorial: (a) LWT test; (b)FN test; (c) SCB test; (d) DCSE test

resistance. In addition, mixtures containing limestone aggregates, LS I and LS II, showed good laboratory performance similar to that of conventional base course HMA ones at high and intermediate temperatures. While the GR mixture showed the best fracture resistance properties at intermediate temperatures, it did not exhibit good permanent deformation resistance in the flow number test. This may be attributed to the coarse gradation of its aggregate, resulting in a more significant effect of the confinement stress on its properties at high temperatures. Based on the results of this phase, a guideline for the design of asphalt treated mixtures was developed, Figure 26a. As shown in Figure 26, for an aggregate source to be used in asphalt treated mixtures, it should have a maximum absorption and Micro-Deval loss values of 2 percent and 18 percent, respectively. Once the aggregate source met the absorption and Micro-Deval screening criteria, the gradation curve of the aggregate blend (75 percent of -1.5 crushed run aggregate material and 25 percent coarse sand) should be within the gradation limits shown in Figure 26b. It is also recommended that the n_{ca} parameter of the aggregate blend should be a least 0.2. For an asphalt treated mixture to be considered for use in the field, it should have a maximum rutting value of 0.48 in the LWT test and minimum ITS and TI values of 150 psi and 0.65, respectively.

Table 8
Summary of Phase II test results

Test	LS I	LS II	SS	GR	NV	RY
Permeability					X	
TSR			X			
TI					X	X
E*			X			
E*/sin δ			X			
FN				X	X	
SCB			X		X	
DCSE					X	X

X Unsatisfactory performance

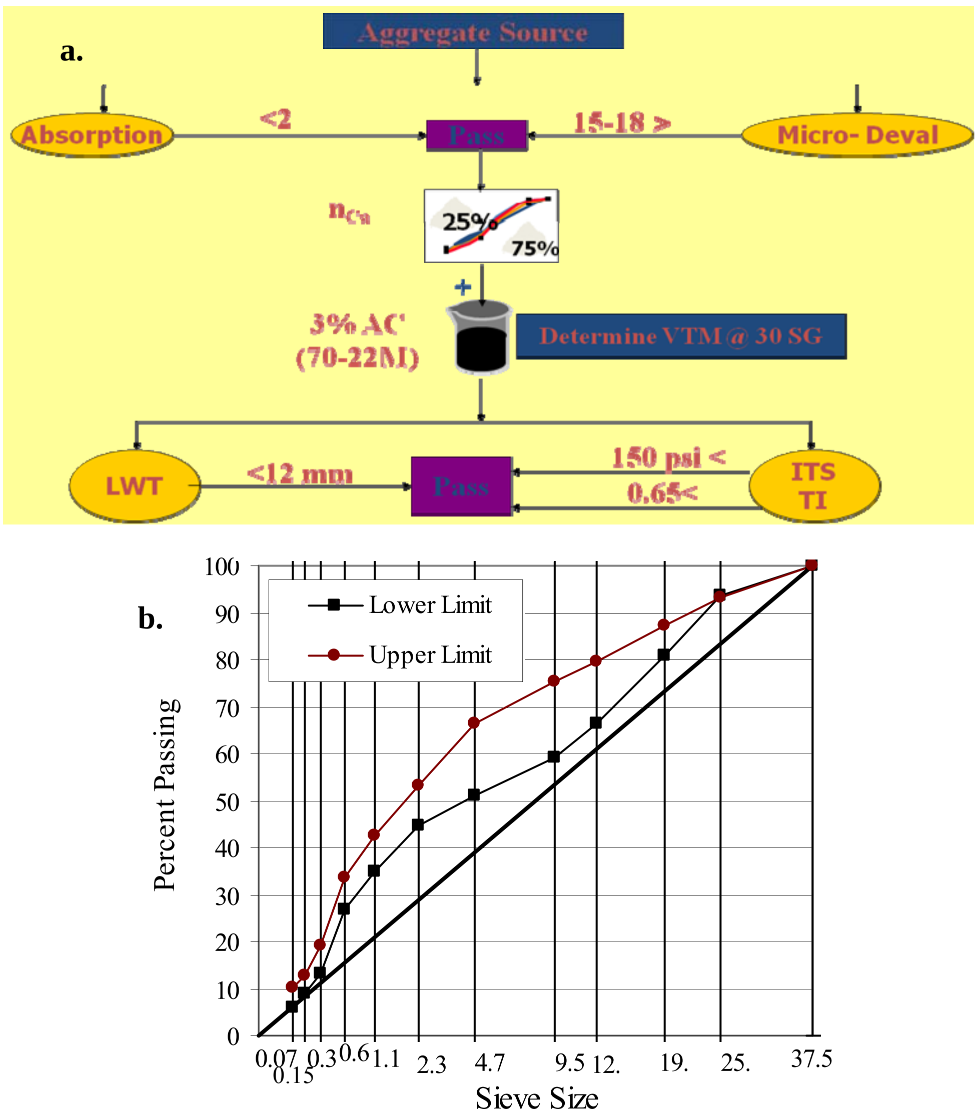

Figure 26
Guideline for design of asphalt treated mixtures

Results of Laboratory Characterization of Unbound Granular Materials

The following sections present the results of the static and repeated load triaxial tests that were conducted on the three unbound granular materials considered in this study at their optimum field compaction conditions.

Static Triaxial Tests Results. The results of triaxial compression tests were used to obtain the ultimate and residual (critical state) shear strength at each of the three confining stresses used in this study. Figures 27a and 27b present the ultimate and residual shear strength lines in the p-q space, respectively. In addition, the figures show the peak and residual strength friction angle that was determined from the results using equations. In general, it is noted that the considered unbound granular materials have similar shear strength properties at the field optimum compaction conditions. However, the granite had a slightly higher peak and residual shear strength friction angle than the sandstone and limestone. In addition, it had no cohesion, which may be attributed to its low fine content.

$$M_p = \frac{6\sin(\phi_p`)}{3-\sin(\phi_p`)} \tag{11}$$

$$M_{cs} = \frac{6\sin(\phi_{cs}`)}{3-\sin(\phi_{cs}`)} \tag{12}$$

where, M_p is slope of line connecting peak shear strength in p-q space; M_{cs} is slope of line connecting residual or critical state shear strength in p-q space; ϕ_p is peak friction angle; and ϕ_{cs} is residual strength friction angle.

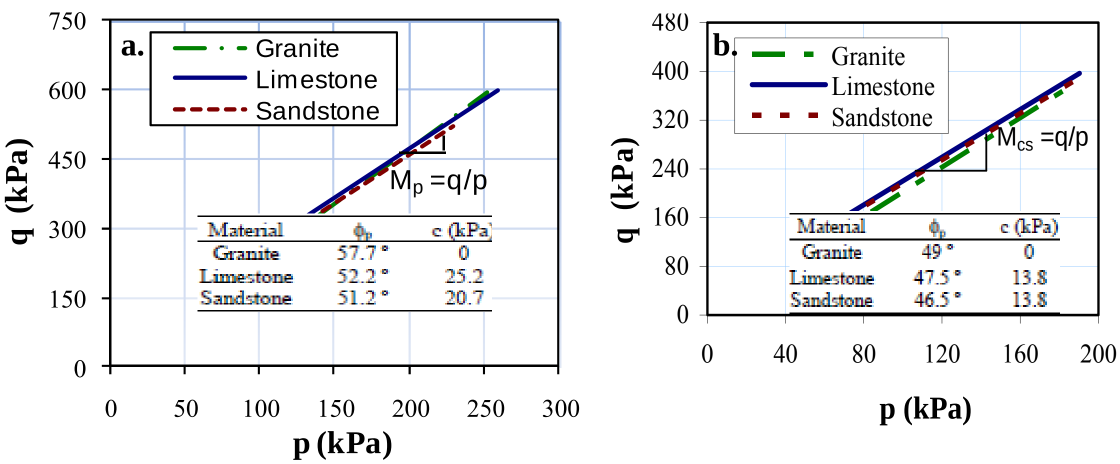

Figure 27
Results of SCT test: (a) peak shear strength (b) residual shear strength

RLT Test Results.

Resilient Modulus Test Results

Triplicate samples were tested at the optimum field condition for each material considered. The mean value of the resilient modulus for the last 10 cycles of each stress sequence was first computed from each of the resilient modulus test results; a regression analysis was then carried out to fit the data of each test to the generalized constitutive model given in equation (12) and determine the k_1-k_3 coefficients for the different tested samples. Figures 28a-c present the k_1-k_3 coefficients for the different granular materials considered, respectively. It is noted that for all tested samples, the k_1 coefficient had positive values, which is expected since the k_1 coefficient is proportional to the stiffness of a material. Furthermore, the limestone had the highest k_1 coefficient values followed by the granite and sandstone materials.

Figure 28b shows that granite had a higher k_2 coefficient compared to the limestone and sandstone materials, which had similar values. The k_2 coefficient describes the stiffening or hardening (higher modulus) of the material with increase in the bulk stress. Thus, the effect of the bulk stress is more pronounced for granite compared to other materials evaluated in Figure 28b. This can be attributed to its coarser gradation and lower fine content. Figure 28c shows that the average k_3 coefficients had a negative value for all tested materials. This observation was expected since k_3 coefficients describes the softening of the material (lower modulus) with the increase in the shear stress. It is noted that the sandstone material had very low k_3 values compared to other materials tested in this study. This suggests that the sandstone material exhibited less softening with the increase in the applied shear stresses.

Single-Stage RLT Test Results-Resilient Strain

Single-stage RLT tests were conducted on triplicate samples. The mean vertical resilient strain curves obtained from those tests for the three materials evaluated are presented in Figure 29a. The resilient strain had a similar trend in all materials, such that it initially increased then decreased as the number of load cycles increased until reaching an asymptote at about 6,000 load cycles, reaching a steady resilient response. The reason for this behavior is that during the primary post-compaction stage, the sample accumulated more deviatoric strain in the horizontal direction (perpendicular to the direction in which the cyclic load is applied), causing the Poisson's ratio to decrease slightly; this resulted in an increase in the sample stiffness and, hence, a decrease in the resilient strain. It should be noted that the number of cycles needed for the sample to reach a steady resilient response increases as the imposed deviatoric stress is increased.

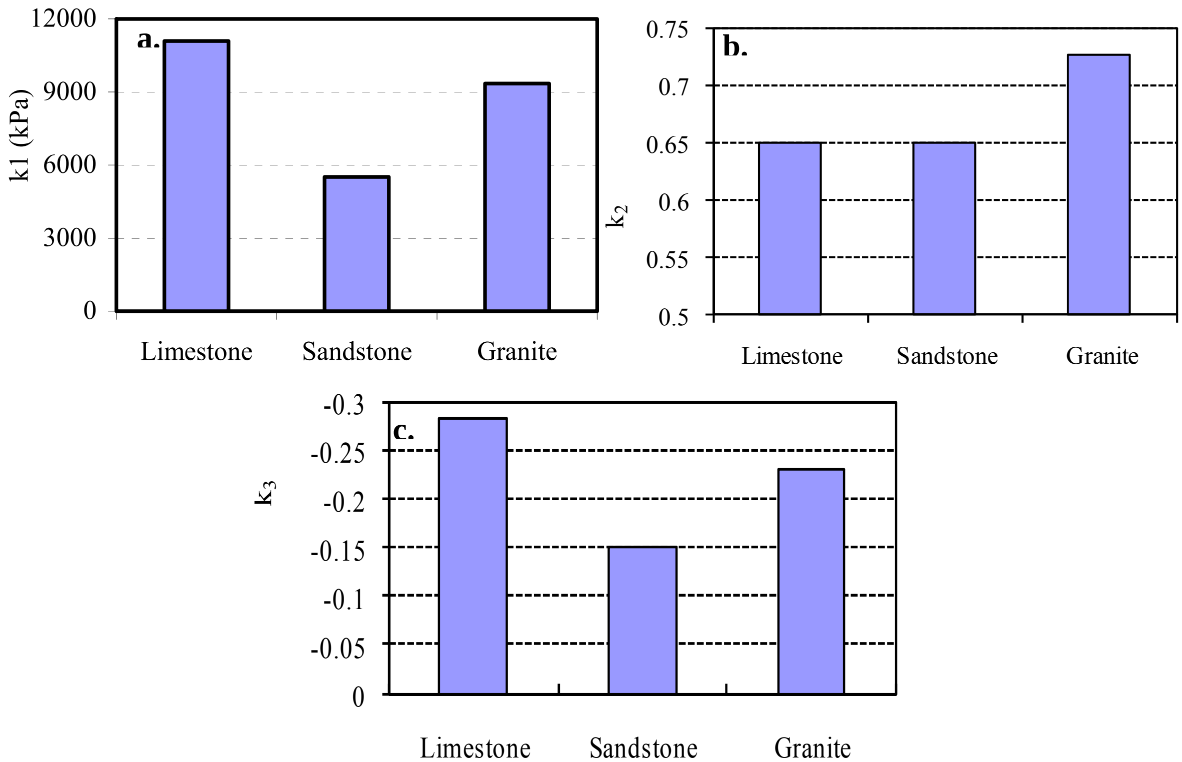

Figure 28
Resilient modulus coefficients of tested materials: (a) k_1, (b) k_2, and (c) k_3

It is noted that the sandstone material had a much higher resilient strain than the other two materials, creating a much smaller resilient modulus. Furthermore, the limestone had a lower resilient strain than the granite. These results are also illustrated in Figure 29b, which presents the mean resilient modulus value measured after 10,000 cycles in the single-stage RLT tests and those predicted using the universal resilient modulus model (Equation 8) based on k_1-k_3 coefficient obtained from the M_r RLT test. This figure shows that the predicted values were very similar to those measured, indicating the reliability of this model prediction.

Single-Stage RLT Test Results-Permanent Strain

Figures 30a presents the mean vertical permanent strain curves for the three materials considered in this study. It is noted that the primary and secondary stages were only reached during the single-stage RLT test. The sandstone experienced the largest permanent strain. Furthermore, the limestone had accumulated a greater permanent strain than the granite. The three materials had similar behavior during the initial load cycles, hence, during the primary post-compaction stage; however, the differences among the materials in the permanent strain behavior were detected during the secondary stage. This suggests that differences in permanent strain did not mainly result from discrepancies in the materials' initial voids and

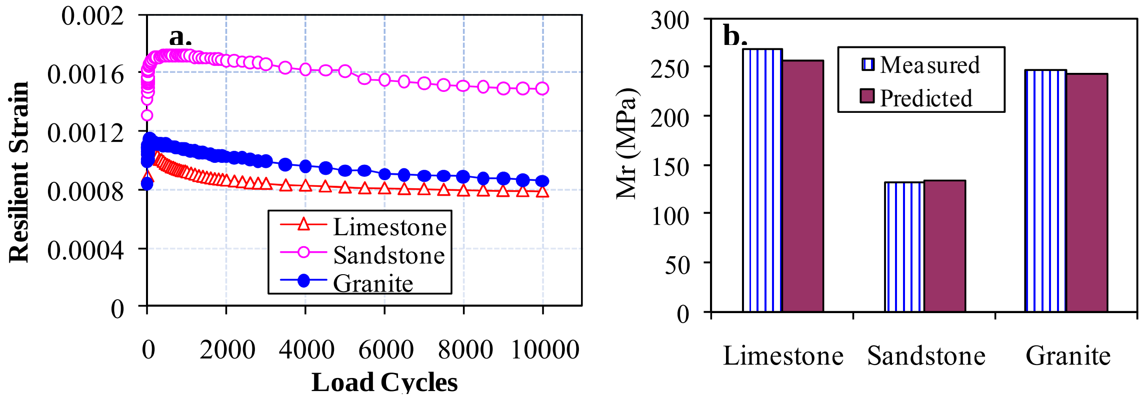

Figure 29
Results of single-stage RLT test (a) resilient strain variation of load cycles and (b) measured and predicted resilient modulus values

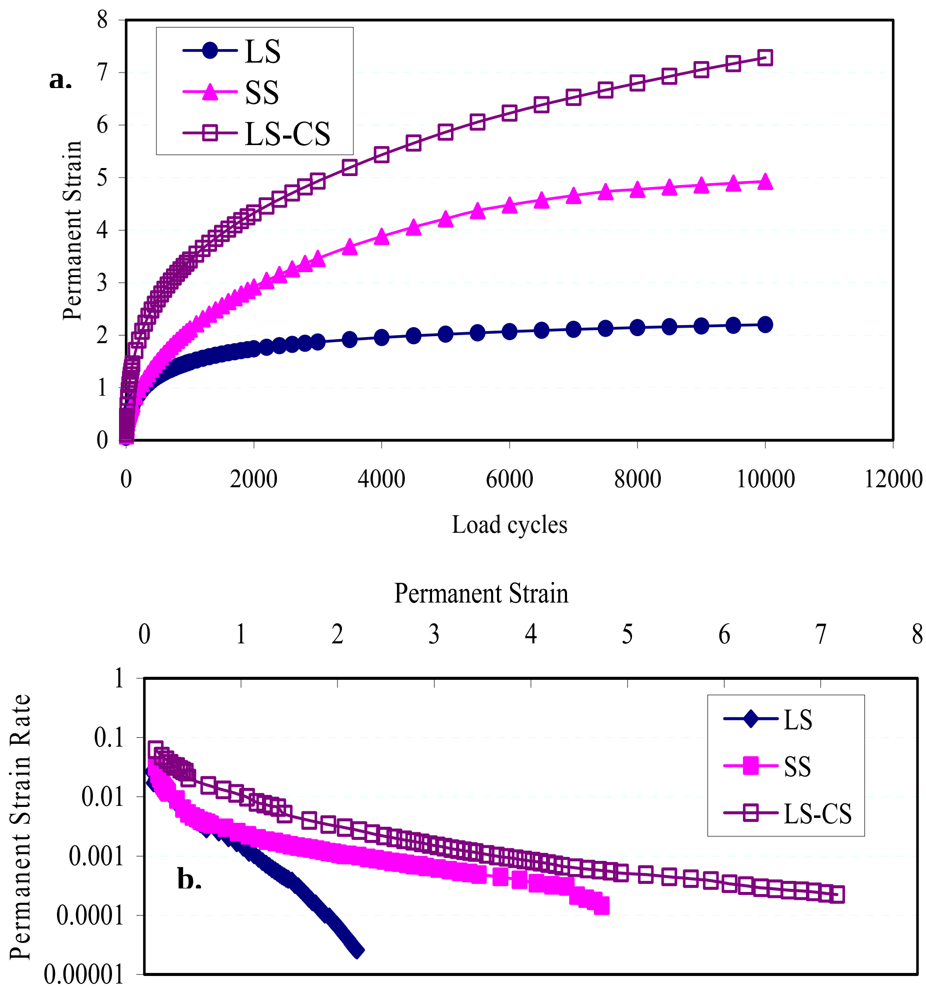

Figure 30
Permanent strain in single-stage RLT test: (a) vertical permanent strain variation with number of cycles and (b) vertical permanent strain rate versus vertical permanent strain

density conditions, but rather from the properties that affect the rotation and sliding mechanisms of the aggregate particles which result in the permanent deformation in the secondary stage. Those properties include particle surface friction and shape. Since the single-stage RLT test included applying 10,000 cycles, it is of interest to examine the long term structural stability of tested samples. For this purpose the relation of the accumulative vertical permanent strain with the vertical permanent strain rate was examined in Figure 30b. In general, all materials had a high permanent strain rate during the first load cycles, yet the permanent strain rate decreased with each load cycle. However, the permanent strain rate of limestone decreased more rapidly than other materials and reached lower values at the end of the RLT test. The permanent strain rate of the sandstone and limestone-coarse sand blend also decreased but at a much slower rate. It is worth noting that the vertical permanent strain rate curve of the limestone-coarse sand blend reached to an asymptote suggests the permanent strain rate will continue at constant rate leading to failure.

Comparison between Asphalt Treated Mixtures and Unbound Granular Materials

The results of tests conducted on asphalt treated mixtures and unbound granular base materials were used to investigate the effect of asphalt treatment on the behavior of unbound granular base materials. Stiffness and permanent deformation resistance were used to conduct this investigation. Figure 31 compares the resilient modulus of the considered unbound materials to the dynamic modulus of the LSI and SS asphalt treated mixtures measured at temperature of 25ºC and frequency of 1Hz (the frequency used in the resilient modulus test). It noted that the asphalt treated binder increased the moduli of unbound aggregate significantly. This suggests that the asphalt treated base layer will be much stiffer than that of unbound granular base material, thus will be able to better distribute the loads to the underlying layers.

Figure 32 shows permanent deformation curves obtained from the single stage RLT and flow number tests conducted on the unbound materials and the asphalt treated mixtures evaluated in study. It is noted that although the same stress level was used in the single stage RLT and flow number tests, the asphalt treated mixtures exhibited a much lower permanent strain than the unbound materials. Furthermore, the LS-CS blend, the same aggregate blend used the LS I asphalt treated mixture, did not only show much higher permanent strain that the LS I asphalt treated mixture but also exhibited an unstable behavior as indicated by the permanent strain curve. This indicates that the asphalt binder significantly improved the permanent deformation resistance of unbound granular materials, thereby their performance.

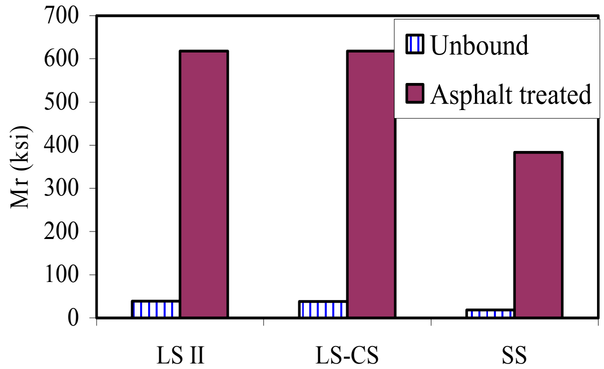

Figure 31
Modulus of the unbound base materials and asphalt treated mixtures

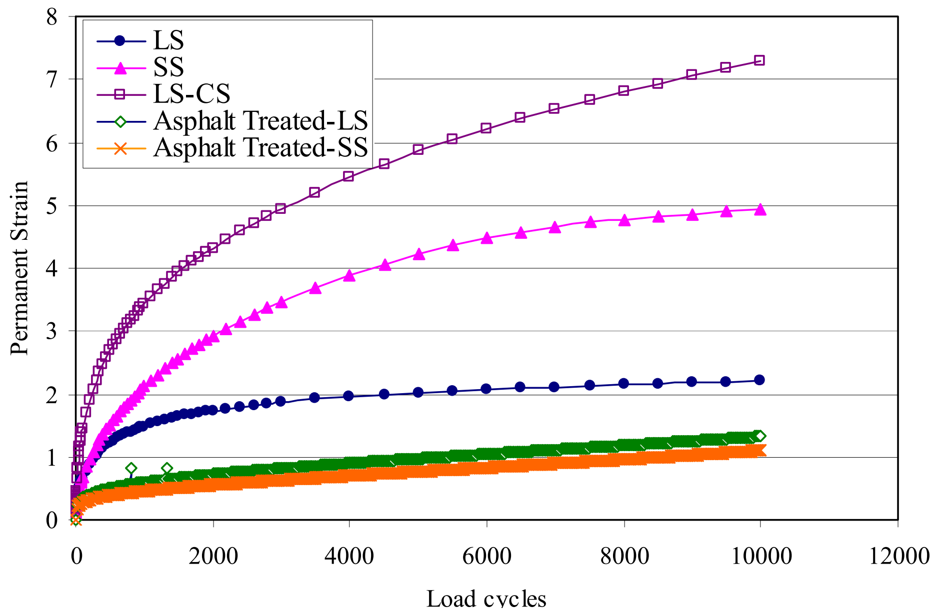

Figure 32
Permanent strain curves for the unbound base materials and asphalt treated mixtures

Economical Analysis

A comparison between the costs of asphalt treated mixtures and conventional HMA base course mixtures were conducted. The result of this comparison is shown in Table 9. It is noted that the asphalt cement binder, aggregate, and sand unit prices were estimated based on their current market value provided by their suppliers in Louisiana. The weighted averages based on current LADOTD construction projects are also included. The cost of materials used in the asphalt treated mixture is $7.2 per ton cheaper than conventional HMA mixtures. This corresponds to about a 16 percent reduction in price, or a cost of $71.40 per ton based on the weighted average of conventional HMA mixtures. It is estimated that plan thicknesses would need to be reduced by approximately ½ to be comparable with unbound base layers.

The current weighted averages, reported by LADOTD, for 8½” thick class II base courses were found to be between $14 and $26 ($Ave. 20) per square yard. Based on the criteria above, the estimated cost of for a 4.5-in. thick ATM base course is $17.70 per square yard. Therefore, in this analysis, the ATM base course mixture could be considerably less in cost to unbound base courses.

Table 9
Results of cost comparison analysis

	Conventional HMA Mixture			Asphalt treated mixture		
Material	Unit Price $/ton	Percent in Mixture	Price $/ton	Unit Price $/ton	Percent in Mixture	Price $/ton
Binder	600.0	4.0	24.00	600.0	3.0	18.00
Aggregate	25.0	81.6	20.40	25.00	72.8	18.20
Sand	10.0	14.4	1.40	10.00	24.2	2.40
Total		100.0	45.80		100.0	38.60
Weighted Ave.			85.00			*71.40

***Calculated based on percent difference of materials costs**

Results of MEPDG Performance Evaluation Analysis

The MEPDG software was used to predict the distress parameters for the four selected projects evaluated in this study. The distress parameters included rutting and fatigue cracking. For analytical purposes, comparisons were made between ATM’s, conventional HMA mixtures and unbound base mixtures. Figure 33 presents the total rutting curves predicted from the MEPDG analysis. It is noted that the use of the asphalt treated mixture in the binder and base course layers resulted in a significant reduction in the total rutting through the pavement service life. In addition, replacing the unbound granular base layer with an asphalt treated mixture layer resulted in reducing the total rutting more than 33 percent. Figure 33 shows that section 4 when the asphalt treated layer replaced the binder course and crushed limestone layers, exhibited lower rutting than the control section, section 1.

Figure 34 shows the fatigue cracking at the end of the design period for the four sections evaluated. It is clear that the asphalt treated mixture improved the pavement performance by reducing the fatigue cracking developed in the pavement section. Thus the results of the MEPDG suggest that the asphalt treated mixture can be used to extend the service life of a pavement structure and reduce its design thickness.

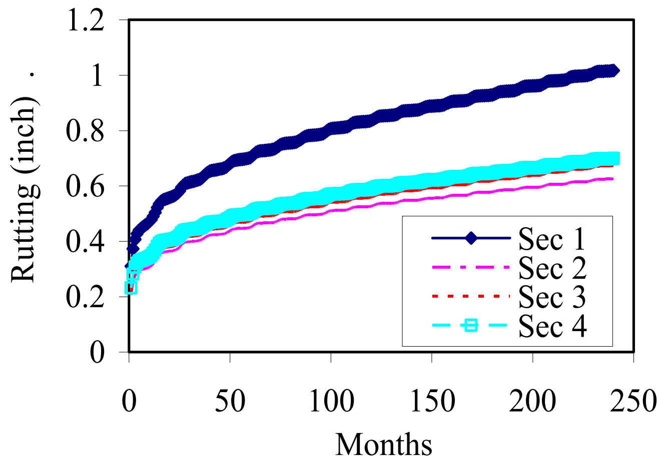

Figure 33
Rutting curves predicted using MEPDG

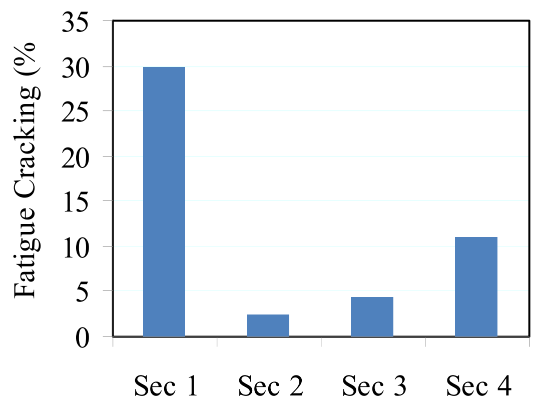

Figure 34
Fatigure cracking predicted using MEPDG

Results of Part II: Field Study

Evaluation of the Constructability of Asphalt Treated Mixtures

One of the objectives of Part II of this study was to evaluate the ability to produce the proposed asphalt treated mixture in the conventional HMA plant and compact it using available HMA compaction equipment. Field test sections were built using asphalt treated mixtures that were produced in various HMA plants. No problems were reported in the production of the asphalt treated mixture. The constructability of asphalt treated mixtures was also examined by comparing the design air voids with those achieved in the field. Table 10 shows the air void measurements of roadway cores for all mixtures. The average, standard deviation, and coefficient of variation were calculated for each mixture within each test section. It is noted that the field measured air void was comparable to that in the design. This suggests that the design air void can be achieved in the field, thus the proposed asphalt treated mixture can be constructed using typical HMA compaction equipment and method. Table 10 shows that coefficient of variation of the air void measurements ranged between 14.7 and 4.5. Furthermore, the LA 3127 test section had relatively higher variation as compared to the other sections. One reason that can explain this result was the weaker subgrade on which this section was built, which is indicated by the higher deflection of FWD seventh sensor, Figure 35.

Results of Laboratory Evaluation of Asphalt Treated Mixtures in Part II

Analysis of Aggregate Gradation. The design gradations of the mixtures evaluated in Part II were examined using the power-law method suggested by Ruth et al. described previously *[21]*. Table 11 presents the power law gradation parameters for the aggregate blends of the evaluated mixtures. The aggregate blend of the US 190 mixture had the highest n_{ca} value of 0.55, while the US 165 aggregate blend had the lowest n_{ca} value of 0.24. In addition, the LA 3127 mixture aggregate blend had the lowest n_{fa} value of 0.52, while the US 190 mixture aggregate blend had the highest n_{fa} value of 0.69. By comparing the power law gradation parameters of the aggregate blends of the mixtures evaluated in Part II, with those obtained for mixtures in Part I, it is noted that for the same aggregate source, significant differences in n_{ca} exists between the aggregate blends of mixtures evaluated in the two phases. In general, aggregate blends of mixtures examined in Part II had higher n_{ca} values than those considered in Part I. This indicates mixtures examined in Phase II had a coarser gradation.

Table 10
Summary of air void measurement results

Project	Air Void	STD	%CV	Design Air Void
LA 3127	7.6	1.1	14.7	8.0
US 425	10.7	0.5	4.5	11.0
US190	9.6	0.5	5.0	8.0
US165	8.9	1.0	11.6	6.7

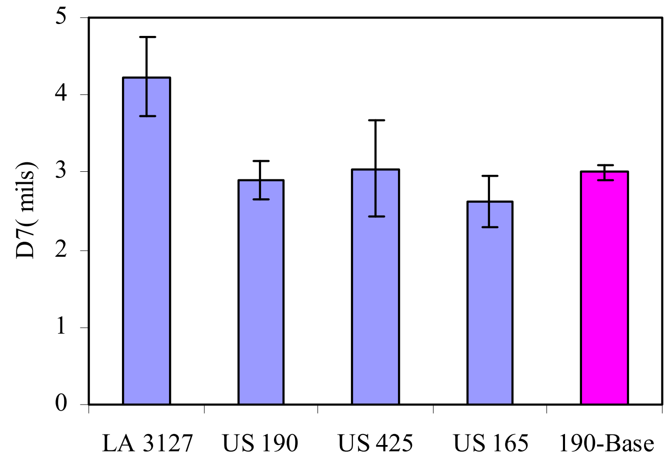

Figure 35
Variation of the d7 deflections FWD

Permeability. Figure 36 presents the average coefficient of permeability for all the mixtures evaluated in this phase. It is noted that all mixtures, except the US 425 mixture containing the Navoculite aggregate, showed a good permeability level that is lower than $125*10^{-4}$ mm/sec. This is consistent with results of Part I, which showed that the NV mixture exhibited high permeability.

Table 11
Results of aggregate gradation analysis

Project	a_{ca}	n_{ca}	a_{fa}	n_{fa}
LA 3127	30.91	0.31	31.26	0.52
US 425	38.60	0.29	28.08	0.57
US 190	23.89	0.40	24.20	0.69
US 165	44.44	0.24	33.29	0.56

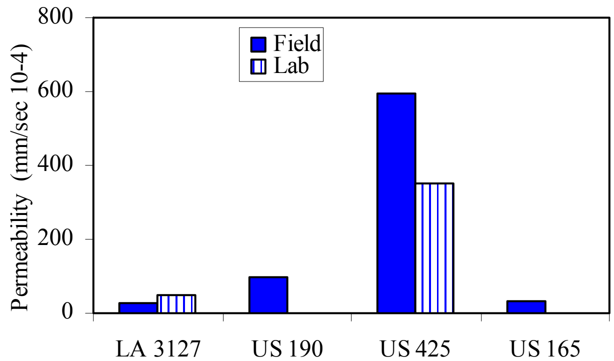

Figure 36
Results of permeability test

Indirect Tensile Strength (ITS) Test Results. Table 12 presents the mean and coefficient of the variation of ITS, IT strain, and toughness index (TI) measurement for the considered mixtures at 25ºC. Higher ITS values are desirable as they correspond to a strong and durable mixture. It is worth noting that the lower the TI value, the amount of energy is absorbed by the mixture under tensile strain, which eventually increases the chances of developing fatigue cracks. In general, the coefficients of variation of the ITS and IT strain and TI values were less than 10 percent, which suggest the test results were repeatable. It is noted that the US 425 mixture had a much lower ITS value than the other mixtures tested. However, all mixtures exhibited similar TI values, which were greater than 0.60, a minimum value observed for fatigue resistant mixtures.

Figures 37 compares the ITS, IT strain, and TI for mixtures evaluated in Phase II to those with same aggregate source and binder type that were examined in Phase I, respectively. It is noted that the same mixture type differences exist between those evaluated in Phase I and II. In general, for field mixed laboratory compacted mixtures (i.e., mixtures evaluated in Phase II), except US 425, had higher ITS, IT strain, and TI values compared to those mixed and compacted in the laboratory. This may be attributed to the differences in asphalt content as well as gradation that existed between those mixtures.

Table 12
Summary of ITS test results

Mixture	ITS (psi)		IT Strain		TI	
	AVG	COV	AVG	COV	AVG	COV
LA 3127	238	3	0.29	11.6	0.63	2.3
US 425	108	19	0.22	9.2	0.68	6.3
US 190	266	6	0.44	13.0	0.66	7.2

COV: Coefficient of variation

Loaded Wheel Tracking (LWT) Test Results. Two samples were tested for each mixture, and the mean rut depth was reported as the rut depth of that mixture. Figure 38a presents the rut depth of asphalt treated mixtures after 20,000 passes in the LWT test. In addition, Figure 38b compares the stripping inflection point observed in LWT test for the different mixtures considered in this study. It is noted that only the US 190 mixture had a rut depth greater than 0.47 inch, so it did not meet the LWT criterion adopted in this study. Furthermore, this mixture showed striping potential within the first 10,000 cycles, while all other mixtures did not exhibit susceptibility to moisture damage. It is worth noting that US 190 was the only mixture among those evaluated in Part II that had PG 64-22 binder. This is consistent with results of Part II, which indicated that mixtures with a higher binder grade that contain elastomeric polymer modification performed better when compared to mixtures with PG 64-22 binder.

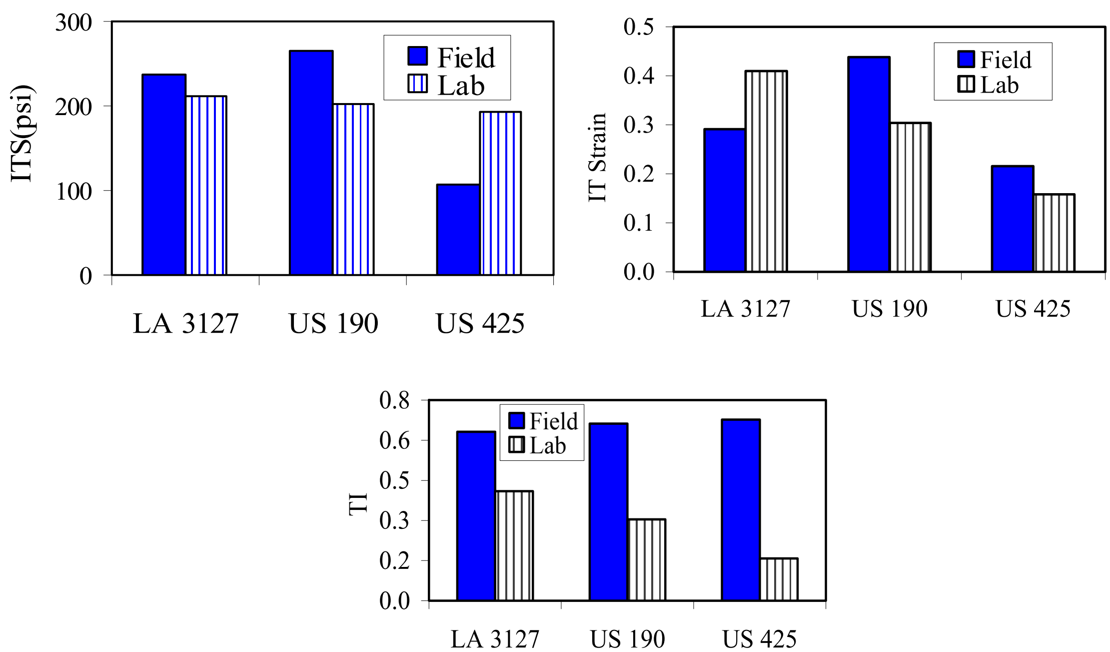

Figure 37
ITS test results

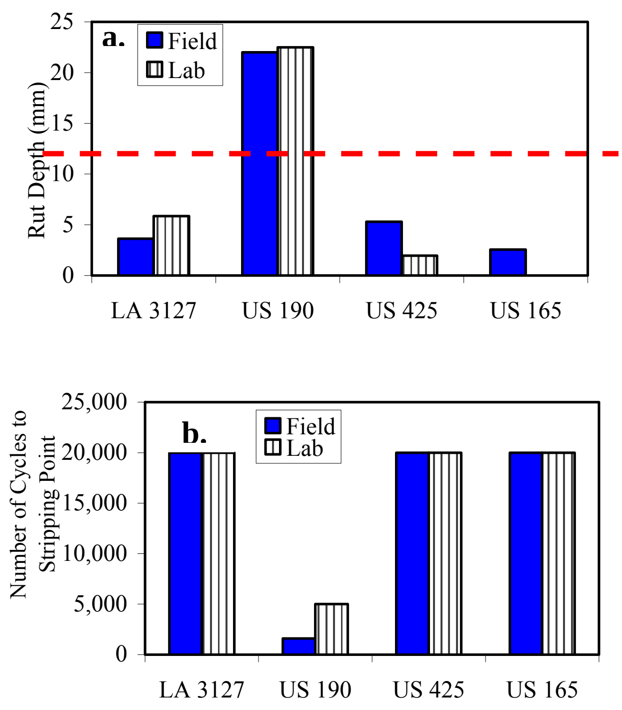

Figure 38
LWT test results: (a) rut depth and (b) stripping inflection point

Figures 37a and 37b also compare the LWT test results for mixtures evaluated in Part II to those with the same aggregate and binder types that were examined in Part I. It is noted that, in general, the mixtures containing the same aggregate and binder types in Part I and II had similar LWT test results.

Modified Lottman Test Results. Figure 39 presents the measured retained tensile strength values for the asphalt treated mixtures evaluated in Part II of this study. It is noted that LA 3127 and US 165 mixtures had retained tensile strength values greater than 80 percent. Furthermore, the US190 mixture had the lowest TSR value of 54 percent. Figure 39 compares the results of Part II with those obtained in Part I. It is noted that the mixtures containing the same asphalt and aggregate types had similar TSR values.

Semi-Circular Bend (SCB) Test Results. Figure 40 shows the average computed critical fracture resistance (J_c) values for asphalt treated mixtures evaluated in Part II of this study. It is noted that significant differences in J_c values exist between the asphalt treated mixtures used in construction of the field test sections.

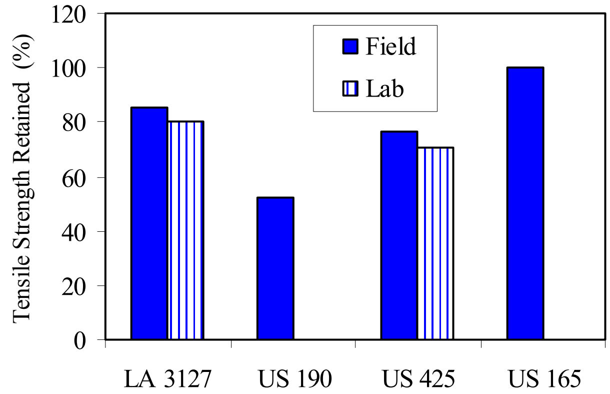

Figure 39
Modified Lottman retained tensile strength

LA 3127 and US 425 mixtures exhibited the highest and lowest J_c values, respectively. Although LA 3127 and US 190 mixtures contained similar aggregate type and gradation, the LA 3127 mixture had a much higher J_c value than that of the US 190. This may be attributed to the use of higher grade binder in the LA 3127 mixture, PG70-22. This again demonstrates the effect of binder type on the fatigue resistance of asphalt treated mixtures. It is worth noting that the US 165 mixture that contains 20 percent RAP exhibited good a J_c value greater than 0.6 KJ/m^2, which is the minimum value observed for fatigue resistant mixtures. Figure 40 also presents a comparison between similar mixtures in evaluated in Part I and Part II. It is noted that the mixtures evaluated in Part II had a higher J_c value compared those examined in Part I. This may be attributed to the coarser gradation Part II mixtures possessed as indicated by the higher n_{CA} values they had compared to their corresponding mixtures in Part I.

Dissipated Creep Strain Energy (DCSE) Test Results. Figure 41 presents the mean DCSE values for mixtures evaluated in Part II. In general, the DCSE test results were consistent with those obtained from the SCB test. The LA3127 mixture had the highest DSCE value of 1.4 KJ/m^3. Furthermore, the US 425 mixture was the only one that had DCSE values less than 0.75 KJ/m^3, the minimum value for fatigue resistant HMA mixtures. This suggests that all other mixtures would have satisfactory fatigue cracking resistance in the field. It is worth noting that the US 190 mixture exhibited a high Jc, which may be attributed to the high film thickness it possessed. This is consistent with Part I test results, which indicated that a good correlation exists between the film thickness and the DCSE value.

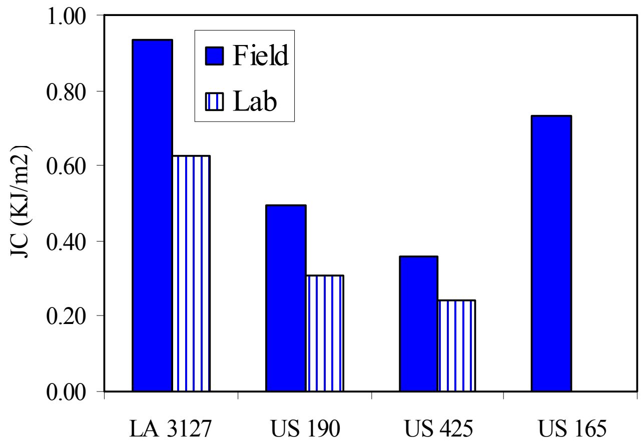

Figure 40
SCB test results

Flow Number (F_N) Test Results. Figure 42 presents the flow number results for the asphalt treated mixtures evaluated in Part II of the laboratory test program. Figure 42 shows that all mixtures except for US 165 mixture, exhibited low flow number values. The low flow number value of the US 190 mixture may be attributed to the binder type it contained, PG 64-22. This is consistent with LWT test results and the results of the Phase I testing program.

A comparison of the flow number test results for mixtures evaluated in Part II to those in Part I is shown in Figure 42. It is noted that only the LA 3127 mixture exhibited a different flow number value than its corresponding mixture evaluated in Part I. The coarser gradation of LA 3127 mixture (as indicated by the n_{ca} parameter) may explain the lower flow number it possessed. This suggests that that coarse graded mixture is more sensitive to confinement at high temperatures. It is recommended that the flow number test be conducted at different confinement stresses to evaluate the response of such mixtures.

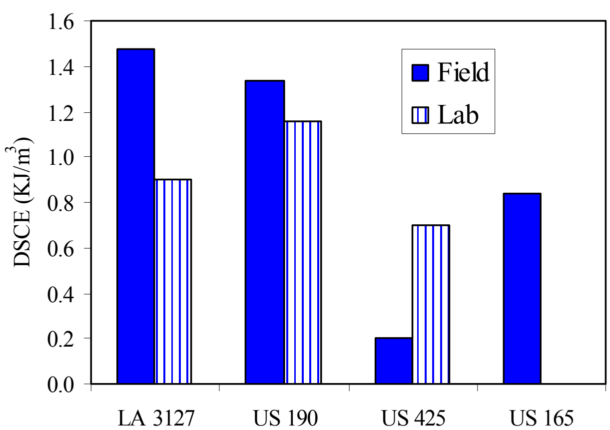

Figure 41
DCSE test results

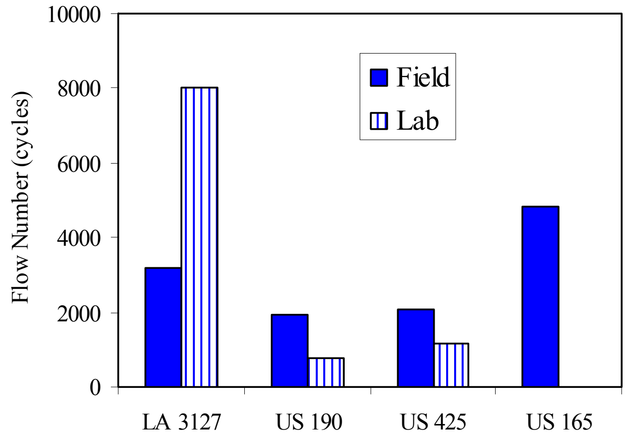

Figure 42
Flow number test results for all mixtures

Dynamic Modulus (E*) Test Results. Figures 43a-c show the dynamic modulus isotherms for Part II mixtures at different temperatures and frequencies, where each isotherm represents the average E* value of three samples. In general, the coefficient of variation of E* test results were about 20 percent. It is noted that E* values for all mixtures increased with an increase in frequency and a decrease in temperature. It is noted that the US 425 mixture consistently had the lowest E* values at different frequencies and temperatures. Furthermore, the US 190 mixture, which contains PG 64-22 binder, had relatively good E* values at low temperatures (4.4°C); however, E* values for this mixture decreased more rapidly with the increase in temperature as compared with other mixtures, such that the US 190 mixture E* isotherms were similar to that of US 425. This demonstrates the effect of the binder type on the response of asphalt treated mixtures at high temperatures.

Figure 43d presents the average rutting factor values at 5Hz and 54.4ºC for all mixtures included in this study. It is noted that, in general, mixtures evaluated in Part II had low rutting factor values comparable to those of good performing conventional base course HMA mixtures *[24]*. This indicates that those mixtures have a rut resistance comparable to conventional base course HMA mixtures, which is consistent with the results of other tests conducted in this study at high temperatures. Figure 41d shows that some difference was observed for the mixtures with different aggregate sources. The highest and lowest rutting factor values were obtained for LS I-70 and SS -70 mixtures, respectively.

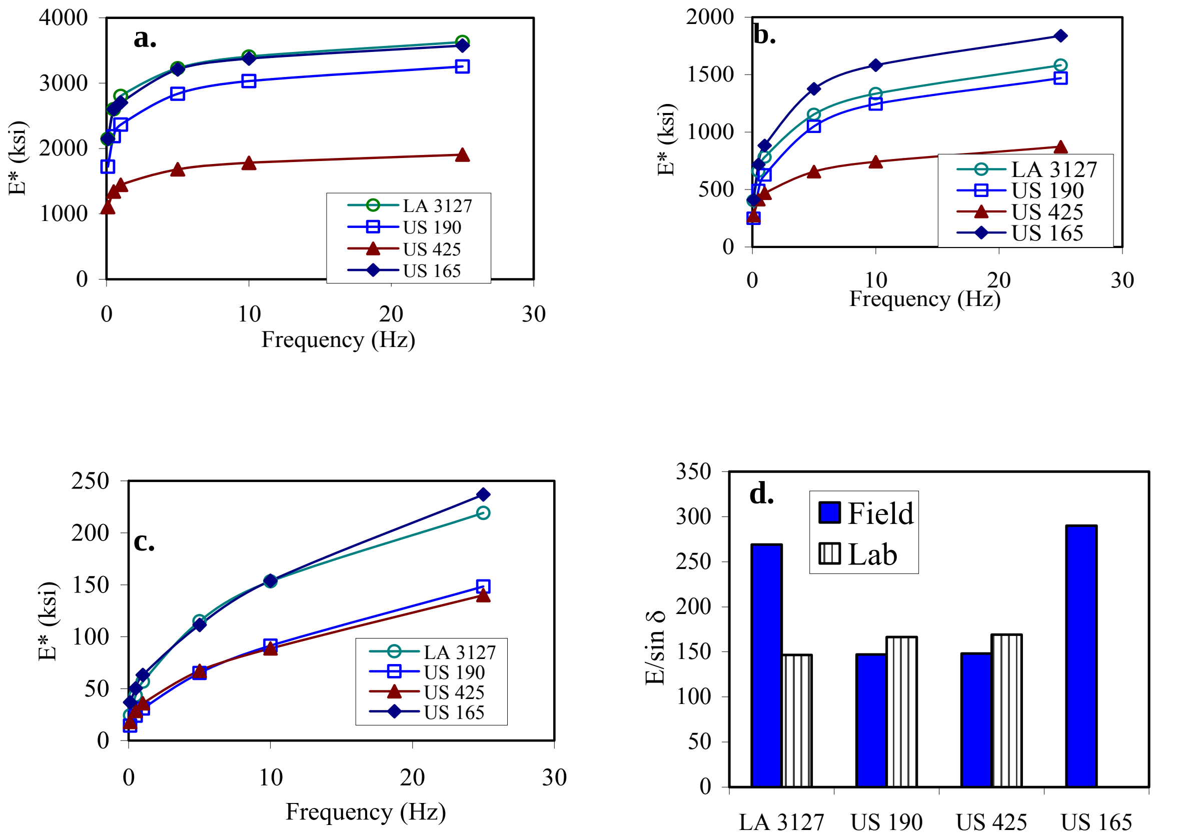

Figure 43
E* test results: (a) E* isotherms at 4.4°C, (b) E* isotherms at 25°C, (c) E* isotherms at 54.4°C, and (d) rut factors@5Hz and 54.4ºC

Results of Field Non-Destructive Tests

The mechanistic properties from field tests include FWD backcalculation modulus, LFWD deformation modulus, and PSPA modulus. It is noted that the field test results were corrected to 25°C using the following equation *[25]*:

$$E_{25} = \frac{E_T}{1.35 - 0.014 \times T} \tag{13}$$

where, E_{25} is the modulus at 25°C, MPa; E_T is the modulus at test temperature T, MPa; and T is the pavement mid depth temperature, °C.

The pavement mid depth temperature was obtained using the BELLS3 model [25] as shown in the following equation:

$$T = 0.95 + 0.892 \times IR + \{\log(d) - 1.25\}\{-0.448 \times IR + 0.621 \times (1\text{-day}) + 1.83 \times \sin(hr18 - 15.5)\} + 0.042 \times IR \times \sin(hr18 - 13.5) \quad (14)$$

where, T is the pavement temperature at depth d, °C; IR is the infrared surface temperature, °C; Log is the base 10 logarithm; d is the depth at which mat temperature is to be predicted, mm; 1-day is the average air temperature the day before testing, °C; sin is the sine function on an 18-hr clock system, with 2π radians equal to one 18-hr cycle; and hr18 is the time of day, in 24-hr clock system, but calculated using an 18-hr asphalt concrete.

FWD Test Result. Table 13 presents the mean of HMA layer moduli from FWD backcalculation and its coefficient of variation percentage (%CV) of the mixtures considered. To minimize variations in the applied load, the deflection measurements were normalized to a standard contact pressure of 550 kPa. Deflections d1 (center load plate), d7 (1,500 mm from load), and deflection difference d1-d6 (d6 at 900 mm from load) can be interpreted as indicators of overall pavement condition, capping and subgrade condition, and asphalt layer condition, respectively *[26], [27]*. The loading frequency of this device is approximately 30 Hz. The US 425 and US 190 mixtures had the highest and lowest FWD backcalculated moduli value of 199.7 and 1546.2 ksi, respectively. In general, FWD moduli of the ATM mixture test section were similar to those obtained to conventional HMA base course mixtures that were reported in previous studies. In general, a high variation in FWD backcalculated moduli was observed. The US 190 mixture had the highest coefficient of variation value of 48.7 percent and the US 165 mixture showed the lowest value of 23.6 percent. It is noted that a similar variation was reported when the testing section was constructed with the conventional HMA mixture. One factor that may contribute to the high variation is the FWD backcalculated layer modulus sensitivity to thickness, stiffness, and environmental conditions of the various layers.

Table 14 presents the mean of FWD deflections and its coefficient of variation percentage (%CV) for each tested mixture. US 190 and US 425 mixtures had the lowest and highest D1 and D1-6 deflections, respectively. In addition, the LA 3127 mixture exhibited the highest %CV among the mixture evaluate for d1 (31.2%) and d1-d6 (38.5%). It is worth noting that the LA 3127 test section the highest d7 values, which may explain the high variation observed. It is noted that the variation in the measured deflections are lower than the variations in the backcalculated ATM layer modulus.

Table 13
FWD test results—backcalculated HMA modulus

Test Section	FWD (ksi)	CV%
LA 3127	248.0	29.3
US 425	199.7	39.5
US190	1546.2	48.7
US165	410.6	23.6

Table 14
FWD test results—sensors deflections

Test Section	FWD deflections (mil)					
	d1		d1-d6		d7	
	Avg.	%CV	Avg.	%CV	Avg.	%CV
LA 3127	16.3	22.7	11.5	32.2	3.7	16.2
US 425	18.1	8.3	14.9	11.4	2.2	18.2
US190	9.2	21.7	5.6	30.4	2.6	7.7
US165	12.4	20.2	9.3	22.6	2.2	13.6
Min		8.3		11.4		7.7
Max		31.2		38.5		18.2

LFWD Test Result. Table 15 presents the mean of LFWD deformation modulus and its %CV of the mixtures considered. The LFWD was not available during the evaluation of the LA 3127 project. It is noted that all moduli were corrected to 25°C using equations (2) and (3). The highest %CV was for US 190 (30.8%), whereas the lowest %CV was 8.2% for the US 425 mixture. In general, the results are consistent with FWD test results, such that the US 425 and US 190 mixtures exhibited the lowest and highest LFWD modulus values, respectively. It is worth mentioning that the LFWD modulus values of the ATM mixtures used in US 190 and US 165 test sections were similar to those of good performing HMA base course mixtures that were reported in previous studies *[26, 27]*.

PSPA Test Result. Table 16 presents the mean of PSPA modulus and its %CV of mixtures evaluated in Phase II of this study. The moduli were corrected to 25°C using equations (2) and (3). The loading frequency of this device is approximately 49,500 Hz. In general, it is noted all ATM mixtures considered had similar PSPA modulus values. However, the LA 3127 mixture had the highest PSPA modulus. The range of %CV of the PSPA modulus was between 14.4% and 27.5%.

Table 15
LFWD test results—deformation modulus

Project	LFWD (ksi)	
	Avg.	%CV
US 425	86	8.2
US190	114	30.8
US165	107	15.9
Min		8.2
Max		30.8

Note: Device was not available for LA 3127

Table 16
PSPA test results—modulus

Mix Designation	PSPA Modulus (ksi)	
	Avg.	%CV
LA 3127	1935.5	14.4
US 425	1688.4	22.6
US190	1716.5	27.5
US165	1877.4	22.7
Min		14.4
Max		27.5

Comparison between FWD and LFWD Test Results. Figure 44 presents the relationship between the LFWD modulus and FWD deflection measurements (d1 and d1-d6). It is noted that a general trend exists, such that the LFWD modulus increased with a decrease in the FWD deflections of d1 and d1-d6 for the tested sections. However, a strong correlation does not exist between LFWD modulus and FWD deflection. This may be attributed to differences in the layer underlying the asphalt treated mixture.

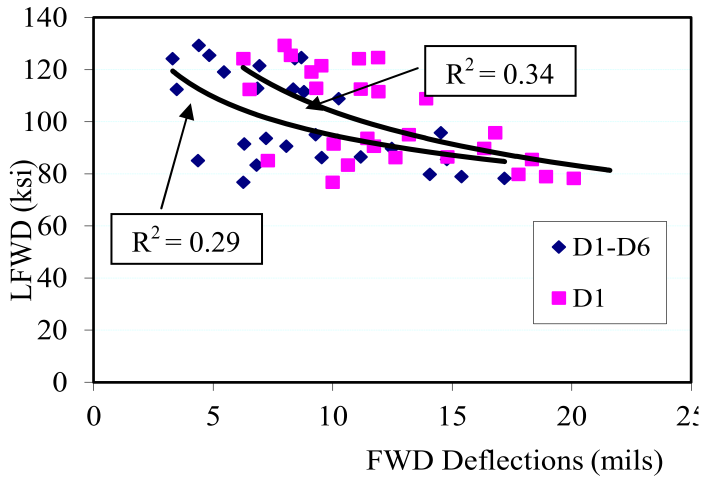

Figure 44
Relationship between FWD deflections and LFWD deformation modulus

Comparison between In-situ Modulus and Laboratory Dynamic Modulus. The behavior of asphalt mixture is dependent on factors such as temperature, mode of loading, and frequency of loading. The time dependency of asphalt mixtures can be described through a master curve. The dynamic modulus values of an asphalt mixture measured over a range of temperatures and frequencies of loading can be shifted into a master curve. The dynamic modulus values can then be determined using the master curve at any temperature and frequency. For this study, a master curve was developed for each mixture evaluated in the field. The master curve was then used to compute the E* at a temperature of 25°C and the operating frequency of each of the in-situ test devices used (i.e., 49,500 Hz for PSPA and 30Hz for FWD and LWD). Figures 45a-c present the relationship between the computed dynamic modulus values and results of the PSPA, FWD, and LFWD tests, respectively. It is noted that, in general, a good agreement exists between the computed E* values and in-situ test measurements, where the E* increased with an increase in PSPA and LFWD and a decrease in the FWD (d1-d6) measurements. Furthermore, good correlation exists between E* and each of the PSPA and LFWD moduli. This suggests that PSPA and LFWD can be used as a quality control tool for evaluating the in-situ modulus of asphalt treated layers.

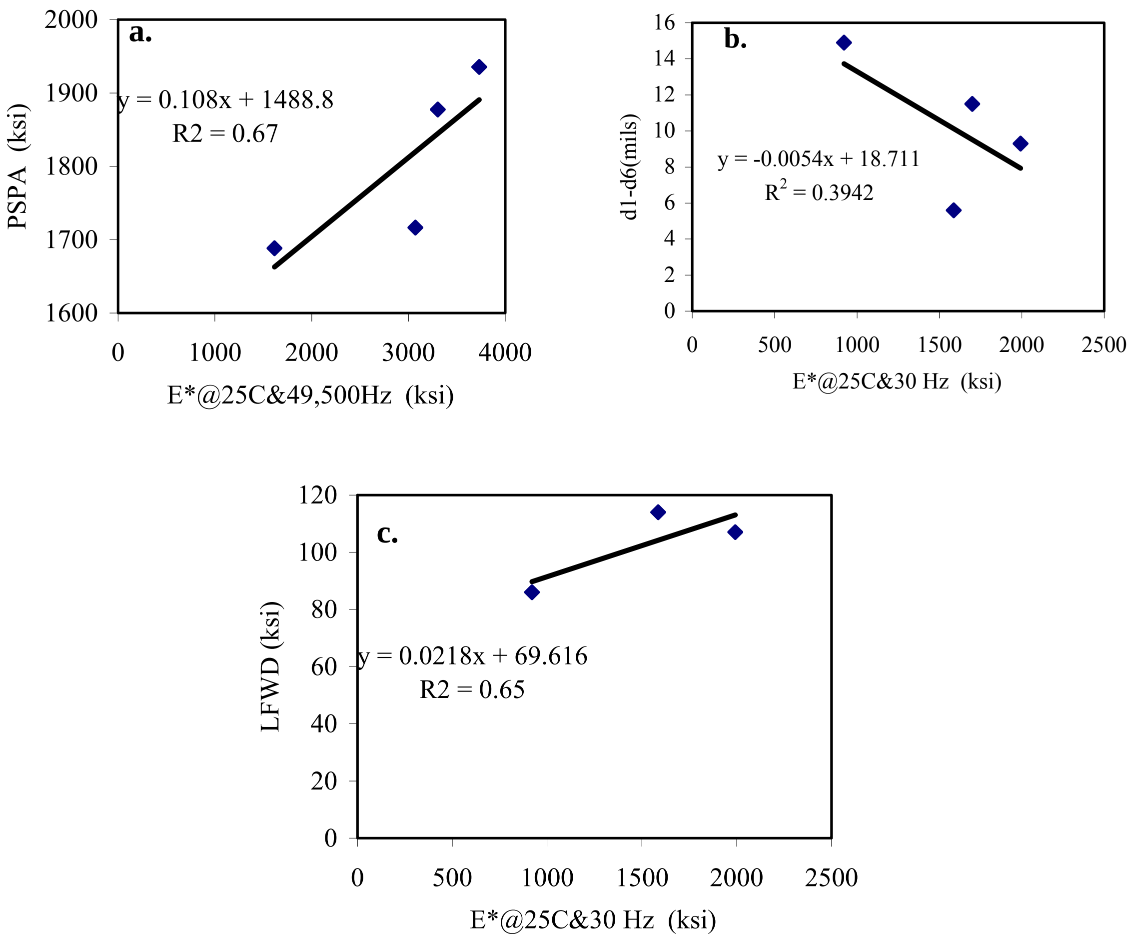

Figure 45
Comparison between lab and in-situ moduli: (a) PSPA modulus and E*, (b) FWD d1-d6 deflection and E*, and (c) LFWD modulus and E*

CONCLUSIONS

In this study, a simplified design methodology for asphalt treated mixtures that are durable, stable, constructible, and cost effective was developed through the examination of the performance of mixtures that have different aggregate gradations than typically available sources. The study was conducted in two parallel parts, Part I and Part II. Part I examined the behavior and performance of asphalt treated mixtures as well as unbound granular base materials. While Part II evaluated asphalt treated mixtures from ongoing field projects. Based on the results of this study, the following conclusions can be drawn:

- The asphalt treated mixtures can be successfully produced in conventional HMA plants and constructed in the field.

- The asphalt treated mixtures exhibited similar in-situ moduli to those of conventional HMA base course mixtures.

- Asphalt treated mixtures containing limestone aggregates, LS I and LS II, showed the best laboratory performance among all other mixtures evaluated in this study. Furthermore, their performance was similar to conventional base course HMA at high and intermediate temperatures.

- The asphalt binder type has a significant effect on fatigue cracking and rutting resistance of asphalt treated mixtures, such that asphalt treated mixtures with higher binder grade that contain elastomeric polymer modification perform better when compared to those with PG 64-22 asphalt binder.

- Based the MEPDG analysis, asphalt treated mixtures can be used to extend the service life and/or reduce the design thickness of pavement structures.

- Asphalt treated mixtures with porous limestone aggregates did not show good performance at intermediate and high temperatures.

- Among all asphalt treated mixtures, mixtures containing Novaculite aggregates exhibited the worst rutting and fracture resistance.

- The power law parameters characterizing the coarse and fine portions of aggregate gradation, n_{ca} and n_{fa}, respectively, were found to significantly affect the critical strain energy release rate, J_c, of asphalt treated mixtures.

- The DCSE value of the asphalt treated mixtures was found to have good correlations with film thickness and aggregate absorption.

- The results of the E* and FN tests indicated that the aggregate skeleton plays a major role in the response of asphalt treated mixtures at high temperatures.

- The cost of asphalt treated mixtures evaluated in this study was $7.2 per ton cheaper than conventional HMA base course mixtures. This corresponded to about 16 percent price reduction.

RECOMMENDATIONS

A simplified design methodology for durable, stable, constructible, and cost effective asphalt treated mixtures was developed based on the results of this study. The following initiatives are recommended in order to facilitate the implementation of this study:

- Allow the use of the proposed asphalt treated mixtures in construction of the wearing course layer of the roadway shoulder. The minimum thickness of the asphalt treated layer should be 3-in.
- Implement the proposed asphalt treated mixtures in the construction of base course layers in flexible and rigid pavements.
- Apply a minimum structural layer coefficient (.30) equal to the current Asphalt Base Course mixtures (Unmodified Binders) when using this mixture.
- Allow the use of the proposed asphalt treated mixtures in pavement widening and patching.

Future Work

- Evaluate the performance of asphalt treated mixtures with smaller nominal aggregate size.
- Evaluate the performance of asphalt treated mixtures with a high content of recycled materials.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AADTT	Average annual daily truck traffic
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
ATM	Asphalt treated mixture
CA	Coarse aggregate
CS	Coarse sand
DSCE	Dissipated Creep Strain Energy
E*	Dynamic Modulus
EE	Elastic energy
FA	Fine aggregate
FE	Fracture energy
FHWA	Federal Highway Administration
FN	Flow number
FWD	Falling weight deflectometer
GR	Granite
HMA	Hot mix asphalt
HMAC	Hot mix asphalt cement
ITS	Indirect tensile strength
JMF	Job mix formula
LADOTD	Louisiana Department of Transportation and Development
LAPA	Louisiana Asphalt Pavement Association
LFWD	Light falling weight deflectometer
LS	Limestone
LTRC	Louisiana Transportation Research Center
LVDT	Linearly variable differential transducer
LWT	Loaded wheel tracking
MEPDG	Mechanistic Empirical Pavement Design Guide
MTS	Material Testing System
NCHRP	National Cooperative Highway Research Program
NV	Novaculite
PG	Performance graded
PLS	Porous limestone
PSPA	Portable Seismic Pavement Analyzer
RAP	Reclaimed asphalt pavement

RLT	Repeated load triaxial
RY	Rhyolite
SBS	Styrene Butadiene Styrene
SCB	Semi-circular bend
SCT	Static compression test
SGC	Superpave gyratory compactor
SS	Sandstone
T_F	Film thinkness
TI	Toughness index
TSR	Tensile strength ratio
USW	Ultrasonic Surface Wave

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