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16. Abstract

The problem of providing pavements that perform as designed is a major concern among state transportation agencies. In the face of budget restrictions, it is imperative that the expected performance be achieved when a highway is put into service. Of importance to addressing this problem is the recognition that performance should drive not only the design process but also the construction process. This approach would necessitate the development of materials and construction specifications that are tied to pavement performance and the development of test equipment and procedures to evaluate the quality of the contractor's work based on predicted performance.

Project 1708, "Predicting Hot-Mix Performance from Measured Properties," aims to develop rational, reliable, and practical test procedures for evaluating the quality of the finished pavement based on predicted performance. To accomplish this objective, the Texas Department of Transportation established a three-phased work plan that calls for: 1) conducting a detailed review of recent and ongoing related studies at the state and federal level (Phase I); 2) identifying mixture-, construction-, and structural-related properties that are significant predictors of pavement performance and are under the contractor's control (Phase II); and 3) identifying/modifying existing procedures or developing new procedures that relate the properties identified in Phase II to the expected field performance (Phase III).

This report represents the culmination of Phase I research activities. This report presents: 1) detailed review of the state-of-knowledge with respect to test methods for measuring construction quality indicators of relevance to this study; 2) available models to establish the impact of the contractor's operations and decisions on expected performance; and 3) a proposed work plan for Phase II and Phase III to develop rational and practical test methods for evaluating the quality of hot-mix asphalt concrete pavements.

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas Department of Transportation (TxDOT), or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The engineers in charge of the project are Emmanuel G. Fernando, P.E. # 69614, and Tom Scullion, P.E. # 62683.

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The work reported herein was conducted as part of a research project sponsored by the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The objective of the study is to develop rational, reliable, and practical test procedures for evaluating the quality of hot-mix asphalt concrete pavements based on predicted performance. The researchers gratefully acknowledge the support and guidance of Mr. Miles Garrison, acting project director at the time of this report, and Mr. Maghsoud Tahmoressi, who initiated this project.

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CHAPTER I

INTRODUCTION

BACKGROUND

Performance-related specifications (PRS) have been the subject of a number of investigations at the federal level. Transportation Research Circular 457, Glossary of Highway Quality Assurance Terms (Committee on Management of Quality Assurance, 1996), defines these specifications as:

> *"Specifications that describe the desired levels of key materials and construction quality characteristics that have been found to correlate with fundamental engineering properties that predict performance."*

Implementation of performance-related specifications requires that the key quality characteristics used to establish conformance are measurable factors controlled by the contractor's operations or decisions in hot-mix asphalt (HMA) construction. Most of the studies conducted to date have focused on:

- identifying materials and construction variables that, on the basis of existing models and experience, are determined to be significant predictors of pavement performance and over which the contractor has control;
- evaluating relationships between materials and construction variables and predicted pavement performance; and
- establishing test procedures for verifying the quality of the contractor's work that are primarily based on laboratory testing of compacted mixtures and/or field cores.

While significant work has been accomplished in the above areas since development of the conceptual framework for performance-related specifications in NCHRP 10-26A, most of the studies conducted to date have focused on evaluating compositional, volumetric, and fundamental engineering properties of HMA molded specimens or field cores through laboratory testing. To move forward in the implementation of performance-related specifications, test procedures must be established so that the key quality characteristics affecting predicted pavement performance may be measured reliably, accurately, consistently, and expeditiously in the field. Significant advances made over the years in methods and equipment for pavements and materials testing make the development of these tests a much more achievable objective.

Automated and nondestructive test equipment already in place within the Texas Department of Transportation (TxDOT) offer the potential for improved quality control and quality assurance (QC/QA) tests. These automated devices include ground penetrating radar (GPR), the falling weight deflectometer (FWD), the portable seismic pavement analyzer (P-SPA), inertial profilers for measuring ride quality and laser-based systems for measuring surface texture and skid resistance. In addition, equipment for materials characterization has recently become available that is suitable for application in a mobile field laboratory. What is needed is to investigate QC/QA applications of these techniques and to develop acceptance criteria applicable for asphalt concrete mixtures used in Texas. This report proposes a work plan for this investigation.

RESEARCH OBJECTIVES

Project 0-1708, "Predicting Hot-Mix Performance from Measured Properties," was initiated by TxDOT to develop simple, practical, and reliable test procedures for evaluating the quality of finished asphalt concrete pavements on the basis of predicted performance. To accomplish this goal, a three-phased work plan has been established that calls for:

- conducting a detailed review of recent and ongoing related studies at the state and federal level (Phase I);
- identifying mixture-, construction-, and structural-related properties that are significant predictors of pavement performance and are under the contractor's control (Phase II); and
- identifying/modifying existing procedures or developing new procedures that relate the properties from Phase II to the expected field performance (Phase III).

TxDOT will use the results to develop performance-related specifications for asphalt concrete pavements and to support the implementation of such specifications in the state.

Development efforts will concentrate on QC/QA test methods for new flexible pavements and will target the following areas:

- identification of key quality characteristics consisting of mixture-, construction- and structural-related properties that are significant predictors of field performance;
- rational and practical test methods for measuring construction quality characteristics; and
- performance-related acceptance criteria.

SCOPE OF REPORT

This report documents the Phase I review of recent and on-going studies pertaining to performance-related specifications that are at the heart of Project 0-1708. The objectives of this review are to:

- establish the state-of-knowledge with respect to test methods for measuring construction quality indicators and the models available to establish the impact of the contractor's operations and decisions on expected performance; and
- propose a work plan, based on the findings from the literature review, to develop rational and practical test methods for evaluating the quality of hot-mix asphalt concrete (HMAC) pavements on the basis of predicted performance.

This report represents the culmination of the Phase I research activities. It is divided into three main parts:

- Chapter I provides the impetus for Project 0-1708 and defines the research objectives.
- Chapter II presents a work plan for developing performance-related tests in Phase II and Phase III.
- The appendix provides a detailed review of the current QC/QA practice with respect to evaluating segregation, longitudinal joint density, ride quality and construction uniformity; recent and on-going projects pertaining to performance-related specifications; and models for relating construction quality indicators to predicted pavement performance.

CHAPTER II

PROPOSED WORK PLAN FOR PHASES II AND III

INTRODUCTION

Before proposing a work plan to develop test methods for evaluating HMAC pavements based on predicted performance, it is necessary to initially establish the state-of-practice with respect to QC/QA testing of HMAC pavements. This testing will determine the baseline for charting the additional development efforts necessary to accomplish the research objectives. With this in mind, researchers prepared a summary of QC/QA test methods for measuring construction quality indicators of relevance to this study. Tables 1 and 2 identify the test methods from our literature review.

Each table presents information in the same format. For a given construction quality indicator, measurement techniques are classified into test methods that are:

1. currently used by TxDOT,
2. currently used by other agencies, and
3. under development.

Quality control procedures summarized in Table 1 refer to tests that are primarily intended to ensure that the contractor's process will meet the targets specified in the design plans. These tests are typically conducted as material is produced in the plant or placed at the construction site. In this way, any deviations from target values are captured in a timely fashion before a substantial quantity of material is produced or placed in the field that does not conform to specifications. Depending on the test results, the contractor may be required to review and modify his or her process to conform to specifications, or even to cease operations until he or she can show, to the satisfaction of the engineer, that subsequent production or placement of material will meet the specified values.

On the other hand, the quality assurance procedures in Table 2 are primarily conducted for acceptance testing of the finished HMAC pavement. These tests need not necessarily be made as material is produced or placed, but are typically conducted within a reasonable time after placement of the final surface. Test results are then used to determine

Table 1. Summary of Quality Control (QC) Procedures for New HMAC Pavements.

Quality Indicator	Currently Used by TxDOT	Currently Used by Other Agencies	Under Development
Segregation	Visual, nuclear density gauge (special provision to Item 340)	Infrared	Lasers, GPR, P-SPA
Longitudinal joint density	—	Density tests on cores	Infrared
Ride quality	Straightedge, profilograph, inertial profilers	Straightedge, profilograph, inertial profilers	—
Thickness	Visual, estimate from application rate	Cores	GPR, P-SPA
Density (other than at joints)	As per Item 3146, density is checked during production testing; also, in-place air voids content is checked during placement testing.	Nuclear density gauge, laboratory testing of cores	GPR
Damage to asphalt cement during production	Discharge mixture temperature shall be within range specified of Item 3146; some districts perform penetration testing.	Mixing temperature and/or binder storage temperature are required to be within specified ranges; penetration testing of binders	—
Bonding between asphalt layers	Visual (check for dirt on surface)	Visual	University of Nottingham is evaluating the use of P-SPA.
Structural properties, e.g., modulus	—	—	P-SPA, portable test equipment such as those available from IPC

Table 2. Summary of Quality Assurance (QA) Procedures for New HMAC Pavements.

Quality Indicator	Currently Used by TxDOT	Currently Used by Other Agencies	Under Development
Segregation	—	Nuclear density	Lasers, GPR, P-SPA
Longitudinal joint density	—	Density tests on cores	GPR, P-SPA
Ride quality	Straightedge, profilograph, inertial profilers	Straightedge, profilograph, inertial profilers	—
Thickness	—	Cores	GPR, P-SPA
Density (other than at joints)	As per Item 3146, density is checked during production testing; also, in-place air voids content is checked during placement testing.	Nuclear density gauge, laboratory testing of cores	GPR
Damage to asphalt cement during production	—	—	—
Bonding between asphalt layers	—	—	University of Nottingham is evaluating the use of P-SPA.
Structural properties, e.g., modulus	—	FWD	FWD, P-SPA, portable test equipment such as those available from IPC

payment to the contractor. Tables 1 and 2 show that the procedures currently used may be variously described as:

- subjective, relying on visual observations and engineering judgment;
- approximate, involving no direct measurements of the quality indicator, such as estimating thickness using the application rate, or the surface profile using the straightedge or the profilograph;
- destructive, requiring cores to be taken from the job site and tested in the laboratory (as opposed to tests that are nondestructive and conducted in-situ);
- indirect, measuring material properties that are not directly used in pavement design but are surrogates or predictors of properties that are input to the design program, e.g, density, air void content, and gradation; and
- tedious and/or time consuming, such as measuring density profiles using the nuclear density gauge to detect segregation.

These observations show where improvements in test methods may be made so that construction quality is evaluated:

- based on predicted performance,
- using parameters that are direct inputs to pavement design and are under the contractor's control, and
- based on measurements from nondestructive tests conducted in-situ.

To realize these improvements, researchers identified tools presently used by TxDOT which may, with further development, be adapted for QC/QA applications. These existing capabilities, identified in the last columns of Tables 1 and 2, include ground penetrating radar, the portable seismic pavement analyzer, and the use of lasers for surface texture measurement. At the present time, these devices are primarily used in Texas for pavement evaluation, forensic investigations, and pavement research. There is a need to investigate the application of these devices for QC/QA purposes through pilot field and laboratory tests on mixtures used in Texas. For this purpose, we propose to conduct demonstration projects in Phase II to identify which existing test methods may be used successfully on TxDOT mixtures that would merit further development in this study. We expect that this will require software and/or hardware modifications to adapt the tests for QC/QA applications and provide TxDOT with what it needs to implement performance-related specifications. These requirements do not only include performance-related tests that are practical to use but also analysis procedures for evaluating the effects of deviations from target values on predicted pavement performance. To relate construction quality indicators to predicted performance, it will be necessary to identify the performance-related parameters that are affected by these quality indicators and are under the contractor's control. This identification will help to establish the parameters that need to be measured during construction. These parameters are then used in existing models to predict performance.

Table 3 reflects our current thinking on how construction quality indicators may relate to pavement performance. For each quality indicator, the table identifies performance-related parameters affected by varying levels of the given indicator. Also identified are the distress types that may occur due to changes in these parameters and the methods for predicting performance based on measurements of the quality indicators during construction. Researchers used the information summarized in Tables 1 to 3 to develop the work plan presented in the following section.

WORK PLAN

Task A. Performance Model Selection

A necessary element of a performance-related specification is the set of models to evaluate the quality of the as-built pavement based on predicted service life. Since the benchmark for this evaluation is the service life associated with pavement design, it is logical that construction quality be evaluated consistent with the design procedure used. In Texas, flexible pavements are designed using the FPS-19 computer program. FPS-19 uses layered elastic theory along with a serviceability loss model to design HMAC pavements satisfying the desired service life. Since TxDOT is presently using FPS-19 for flexible pavement design, it is prudent that we include it in developing the performance-related QC/QA tests in this project. FPS-19 has already undergone extensive checks in Project 0-1869 and is compatible with existing TxDOT practices and specifications. However, other models to evaluate the effects of segregation, longitudinal joint density, ride quality, and construction uniformity should also be considered in developing the performance-related QC/QA tests which are called for in this study. In particular, the following applications need to be investigated:

Table 3. Relating Construction Quality Indicators to Predicted Pavement Performance.

Quality Indicator	Performance-Related Parameter	Anticipated Distress	Prediction Method
Coarse segregation	Modulus, permeability, loss of fines, permanent deformation parameters	Fatigue cracking, rutting, structural problems due to weakening of underlying layers, raveling,	Predict change in fatigue coefficients due to change in modulus; model softening of underlying layers due to moisture infiltration; use FPS, VESYS, or FLEXPASS to predict fatigue cracking and rutting.
Longitudinal joint density	Permeability, permanent deformation parameters	Structural problems due to weakening of underlying layers	Use FPS to predict effect of weaker base and subgrade; use VESYS or FLEXPASS to predict rutting in each layer.
Ride quality	Surface profile	Load-associated distress, i.e., fatigue cracking and rutting	Model effect of surface profile on predicted dynamic loading which affects the expected 18-kip ESALs.
Thickness	Thickness	Load- and non-load associated distress	Model effect on predicted pavement response, e.g,, induced stresses and strains under traffic and environmental loadings.

Table 3. Relating Construction Quality Indicators to Predicted Pavement Performance (continued).

Quality Indicator	Performance-Related Parameter	Anticipated Distress	Prediction Method
Density	Modulus, permanent deformation parameters	Load-associated distress, i.e., fatigue cracking and rutting	Predict change in fatigue coefficients due to change in modulus; use FPS, VESYS, or FLEXPASS to predict fatigue cracking; use VESYS or FLEXPASS to predict rutting in HMAC layer.
Damage to asphalt cement during production	Modulus, loss of cohesion	Fatigue cracking	Predict change in fatigue coefficients due to change in modulus; use FPS, VESYS, or FLEXPASS to predict fatigue cracking.
Bonding between asphalt layers	—	Load-associated fatigue cracking	Model effect on predicted pavement response, e.g,, induced stresses and strains under traffic.
Structural properties	Modulus, permanent deformation parameters	Load-associated distress, i.e., fatigue cracking and rutting	Predict change in fatigue coefficients due to change in modulus; use FPS, VESYS, or FLEXPASS to predict fatigue cracking; use VESYS or FLEXPASS to predict rutting in each layer.

- the use of finite elements to model the horizontal and vertical variation in performance-related material parameters and the boundary condition at joints,
- vehicle simulation of dynamic loading to model the effects of surface profile,
- characterization of permanent deformation properties and prediction of rutting in individual layers, and
- the use of fracture mechanics to model the effects of mixture properties on the development of fatigue and thermal cracking.

Based on the literature review, we propose to compare and evaluate FPS-19, VESYS, FLEXPASS, and the Level III Superpave finite element models in this task. These models range from mechanistic-empirical to mechanistic and will provide us with the utmost flexibility in modeling the performance of asphalt mixtures used in the state as well as new materials that may be introduced in the future. It is noted that the findings from WesTrack were considered in identifying the models to be evaluated in this task. However, only the Level 1 procedure from this project was available at the time of this report. Further, the Level 1 models are largely empirical and developed based on correlating the observed performance of the WesTrack pavements with the properties of the materials used. Thus, the applicability of the models to Texas conditions and materials is questionable.

Researchers therefore selected performance models that have a wide range of applicability and which permit us to predict performance based on fundamental material properties that can be measured in-situ or in the laboratory. The evaluation in this task will be made using a common database of pavement cross-sections, materials, and traffic loadings. Its objective is to establish the significance of differences between the models so that appropriate recommendations may be made with respect to evaluating the effects of construction quality indicators of interest to this project. For example, the effects of segregation may be predicted using a layered elastic analysis where a reduced modulus for the entire asphalt layer is used or by a finite element analysis where the horizontal and vertical variation of modulus is modeled. By comparing the predictions, we hope to identify where simplifications may be made or, alternatively, where sophisticated and elaborate methodologies will have to be used to model the effects of the construction quality indicators of interest to this study. From the results of this evaluation, models shall be selected for developing test methods to evaluate construction quality based on measured properties that are performance-related and under the contractor's control. Researchers will identify performance-related properties through the sensitivity analysis in Task B.

Task B. Sensitivity Analysis of Predicted Performance

Once the models are selected, a sensitivity analysis is proposed to identify the parameters that significantly influence the performance predictions. This analysis will cover parameters that are under the contractor's control. For each construction quality indicator given in Table 3, the analysis shall identify the parameters that significantly influence the anticipated modes of distress. Each model parameter will be varied one at a time over a realistic range. The resulting change in the predicted service life will then be used to establish the significance of each model parameter.

Researchers recognize that the sensitivity of predicted performance to a given parameter may depend on the levels of the other independent variables. Thus, the analysis shall be conducted at different levels. We propose to vary each variable one at a time holding the others at a given level. This procedure will then be repeated at one or two additional levels. The findings are therefore expected to identify not only the significant predictors of pavement performance, but also the conditions under which the effects are significant. In practical terms, the findings may prove useful in developing guidelines on the scope of tests necessary for a given project.

We will identify which of the significant parameters are presently tested, either directly or indirectly, in the current QC/QA specification and establish how these parameters are affected by construction quality indicators of interest in this study. Table 3 will then be updated accordingly. Subsequent development efforts will focus on how the performance-related parameters may be characterized, either by direct measurements, or by indirect methods through surrogate properties that are more easily determined in-situ or in the laboratory. Construction quality will then be evaluated using the measured properties with the selected performance models from Task A.

Task C. Test Methods to Characterize Performance-Related Parameters

This task will evaluate test methods for quality control and quality assurance during pavement construction. While in-situ nondestructive test methods are preferred, we expect that tests on cores or molded specimens will still be necessary, particularly for monitoring the production of the asphalt mix. Also, while a direct characterization of a performance-related parameter is desirable, it is likely that some parameters may have to be estimated indirectly using surrogate variables that are easier to test in-situ or in the laboratory. This will likely require relationships for predicting fundamental material properties using basic mixture variables that are measured under current practice. Thus, this task aims to:

- evaluate in-situ and laboratory test methods for characterizing performance-related parameters that are feasible for construction implementation, and
- investigate relationships between basic mixture variables and fundamental material properties that are input to the models selected in Task A.

There are two types of activities for which tests are needed during construction. As hot-mix asphalt is being produced or placed in the field, or shortly after it is laid, quality control tests are carried out so that any deviations from the target specifications may be captured in a timely fashion before a substantial quantity of non-conforming material is produced or placed at the site. This activity is then followed by a quality assurance phase where the impact of exceeding or not meeting the required specifications is translated to bonuses or penalties for the contractor. Each phase has its special requirements as illustrated below.

Referring to the quality control activities relevant to seismic methods in Table 1, the main purpose would be to determine the baseline modulus and to define acceptable limits for different types of quality indicators. Since seismic methods yield similar moduli in the laboratory and in the field, these limits can be developed in the laboratory by testing specimens that simulate the undesirable conditions for each quality indicator. The limits may then be used for quality control as illustrated in Figure 1. In this example, measurements of asphalt concrete modulus are made in the field using the P-SPA. The data are then compared against the allowable limits. Measurements that consistently fall outside these limits would indicate a need for the contractor to review his or her operations to pinpoint the reasons for the deviations and to make adjustments accordingly.

In the quality assurance stage, payment to the contractor is determined. For a performance-related specification, this payment is based on the predicted change in pavement life due to deviations from the target values established in the design. This determination is

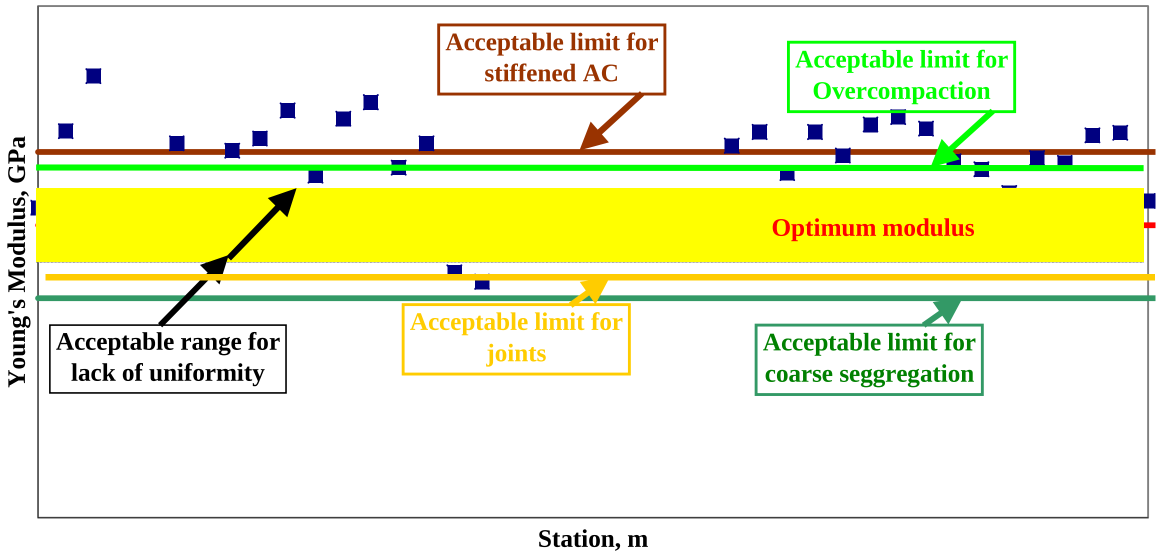

Figure 1. Example Application of P-SPA For Construction Quality Control.

made consistent with the design procedure used. In the example given, if seismic tests are used to determine the asphalt concrete modulus for acceptance testing, the measurements must be converted into the corresponding FWD modulus or laboratory dynamic modulus. The reason for this conversion is that seismic tests provide low-strain, high-strain rate moduli, whereas the FWD or the dynamic modulus test proposed by the American Association of State Highway and Transportation Officials (AASHTO) yield high-strain, low- strain rate values. The success of using seismic tests for QC/QA will therefore depend on how well we can establish the relationship between the seismic modulus and the FWD or dynamic modulus used in pavement design. To evaluate this relationship, researchers propose to use a number of methods to determine the modulus over a wide frequency range that include the uniaxial creep test, the frequency sweep test, and the resonant column test. These tests shall be conducted for various asphalt concrete mixtures used in the state and at different temperatures. The objective is to establish the factor by which to adjust the seismic modulus to the design modulus for a given mix and temperature.

In addition, researchers have accumulated a large database of measured seismic, FWD, and laboratory moduli from WesTrack and NCHRP Study 10-44A. These data will also be used in this task to evaluate the relationships between seismic and design moduli. The WesTrack data are from 26 different sites whereas the data from NCHRP 10-44A are from ten sites located in three states that were tested twice (once during winter and another during summer).

It is noted that the FWD may be used to estimate the asphalt concrete modulus assuming that a specification based on this material property is developed from this study. Using the FWD will yield the design modulus directly. Alternatively, transportable and relatively inexpensive equipment have recently become available that are suitable for use in a mobile field laboratory. Examples are the SERVOPAC gyratory compactor and the rapid triaxial tester shown in Figures 2 and 3, respectively. Both test units were developed by International Process Controls (IPC) Limited of Australia.

Researchers propose to investigate field applications of these devices in Task E of the work plan. The gyratory compactor in Figure 2 can be used to mold specimens in the field using material sampled from the paver. Compacted specimens are then tested using the apparatus shown in Figure 3. With this device, the specimen to be tested is simply placed inside the triaxial cell which is then coupled to the load frame for testing. Sensors for measuring horizontal and vertical deformations are integrated with the cell thereby eliminating the set up time associated with instrumenting a test specimen in conventional geotechnical triaxial tests. Innovative testing devices such as these will be necessary to implement performance-related specifications and the AASHTO 2002 Guide which is expected to use the dynamic modulus for design. While this material property may be estimated nondestructively, such as through the FWD or by seismic wave propagation techniques, laboratory characterizations will still be necessary for mix design, and perhaps for construction quality control and quality assurance, particularly for projects with thin surface layers.

In summary, the major products of this task are expected to be:

- a catalog of test methods for measuring performance-related parameters that include nondestructive tests conducted in-situ as well as tests on cores or molded specimens;
- relationships between moduli values determined from seismic, FWD, and laboratory tests; and

Figure 2. SERVOPAC Gyratory Compactor Developed by IPC Limited of Australia.

Figure 3. The Rapid Triaxial Tester Developed by IPC Limited of Australia.

- relationships for estimating fundamental material properties from basic mixture variables that are easier to measure in-situ or in the laboratory and are tested under current practice.

It is likely that several procedures may be available for measuring a particular performance-related parameter. Consideration will be given to integrating the test methods into a hierarchical framework consisting of different levels that are tied to the number of 18-kip ESALs a project is designed to serve over its lifetime.

Task D. Relationships between Construction Quality Indicators and Predicted Performance

This is a big task that incorporates model development and laboratory investigations. In particular, researchers anticipate that additional development work will be required in evaluating the effects of ride quality and construction uniformity on predicted pavement life. For modeling purposes, laboratory investigations will be needed not only to provide the required input data to the selected models but to establish the effects of construction quality indicators such as segregation and longitudinal joint density on fundamental material properties that control the development of the anticipated modes of distress for the given indicators (see Table 3). In terms of fatigue cracking, it will be important to evaluate the effects of these indicators on the coefficients, K_1 and K_2, of the equation relating strain level to number of allowable load repetitions:

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

where,

N_f = number of load applications to failure

ε_t = predicted tensile strain at the bottom of the asphalt concrete layer

Many existing design procedures use Equation (1) to predict fatigue life for a given pavement design. Traditionally, the fatigue coefficients, K_1 and K_2, are determined from beam fatigue tests. However, these tests are difficult to implement for QC/QA applications. Fortunately, relationships for predicting these coefficients have been developed that permit estimates to be made using results from simpler tests. These relationships are given in the following equations, which are based on fracture mechanics theory (Tseng and Lytton, 1990):

$$K_1 = \frac{\left(d^{1-\frac{n}{2}}\right)\left[1-\left(\frac{c_0}{d}\right)^{1-nq}\right]}{A\left(1-nq\right)\left(rE\right)^{n}} \tag{2}$$

$$K_2 = n \tag{3}$$

where,

d	=	thickness of the asphalt concrete layer
c_0	=	initial crack length
E	=	asphalt concrete modulus
r, q	=	constants that relate the stress-intensity factor at the crack tip to the geometry of the sample, loading, and crack length
A, n	=	fracture parameters of the Paris and Erdogan (1963) equation

Equation (2) shows that K_1 is a function of material parameters, A, n, and E, and the specimen geometry. On the other hand, the coefficient, K_2, is a function only of the fracture parameter, n. Thus, the effects of segregation and density on fatigue life are expected to depend on how they influence A, n, and E. From the preceding relations, it is observed that modulus and thickness are important to achieving the desired fatigue life because of:

- their influence on the fatigue coefficient, K_1, and
- their effects on the predicted horizontal strain at the bottom of the AC layer.

Based on theoretical work done by Schapery (1973) and experimental studies conducted by Germann and Lytton (1979), the fracture parameter, n, may be estimated from the slope of the creep curve, m, using the relation:

$$n = \frac{2}{m} \tag{4}$$

Witczak (1993) also proposed the following relationship for predicting n using the asphalt concrete modulus:

$$n = \left[\frac{1.972}{0.151041 + 0.002775\,(3301.4 - 501.76 \log E\,)^{0.6897}} \right] - 1.998 \tag{5}$$

Finally, the following functional form of a relationship for predicting *n* in terms of the slope of the creep curve *m* was used in the SHRP A-005 project by Lytton et al. (1993):

$$n = g_0 + \frac{g_1}{m} \tag{6}$$

The coefficients g_0 and g_1 in the above equation were determined for different environmental zones by calibration to field performance data collected on SHRP GPS sections. Equation (6) is used in the Level III Superpave performance model for fatigue cracking.

With respect to the fracture parameter *A*, a number of relationships have been proposed by various researchers. Among them are:

Molenaar (1983):

$$\log A = 4.389 - 2.52 \log(E\, \sigma_m\, n) \tag{7}$$

Lytton et al. (1993):

$$\log A = 4.389 - 2.52 \log(10{,}000\, \sigma_m\, n) \tag{8}$$

Uzan (1997):

$$\log A = -6.3245 - 2.0741\, n \tag{9}$$

Based on the preceding review of models for predicting fatigue life, it is evident that test methods are needed to characterize or estimate the following performance-related parameters:

- asphalt concrete modulus,
- slope of the creep compliance curve,
- tensile strength, and
- asphalt concrete thickness.

The first two parameters may be determined by conducting frequency sweep tests on cores or molded specimens at different temperatures or at the temperature assumed in the pavement design. Once these tests are done, a uniaxial tensile test under monotonic loading or an indirect tensile test may be conducted to determine the tensile strength of the test specimen. It is noted that the frequency sweep test and the indirect tensile test do not take much time to run and are relatively easy to learn and simple to perform. With new test equipment such as that shown in Figures 2 and 3, it is possible to conduct the tests in the field and to program the test procedures for automated data collection and data reduction.

Obviously, other methods for measuring or estimating the required material parameters will have to be investigated in order to develop a specification that is based on sound engineering principles and which can be implemented in practice. As part of the work plan to develop a methodology to relate construction quality indicators to predicted fatigue life, a test program will be conducted in Task D to:

- verify the equations for predicting fatigue parameters and identify the equations which provide the best agreement with laboratory test data,
- calibrate the equations for mixtures used in the state, and
- develop a database of fatigue parameters that may be used in the absence of actual test data.

With respect to predicting pavement rutting, a popular model that is used to relate plastic strain with number of load repetitions is the VESYS model (Kenis, 1977) given by the equation:

$$\varepsilon_a = IN^S \tag{10}$$

where,

ε_a	=	accumulated or permanent strain
N	=	cumulative load repetitions
I, s	=	model parameters determined from permanent deformation tests

Equation (10) is implemented in the FLEXPASS and Level III Superpave permanent deformation modules. Note that the model defines a linear relationship between the logarithm of the permanent strain and the logarithm of the number of load repetitions. The parameter, I, is the arithmetic value of the intercept, and s is the slope of the line. From these parameters, α and μ are determined from the following relations:

$$\alpha = 1 - s \tag{11}$$

$$\mu = \frac{I\ s}{\varepsilon_r} \tag{12}$$

where, ε_r is the resilient strain at the 200th load repetition of a permanent deformation test. A conceptual illustration of data from this test is given in Figure 4, where it is observed that the total strain at a given load repetition consists of permanent and recoverable components.

The parameters α and μ are used to determine the permanent strain per load repetition which are then accumulated to predict the increase in rutting with load cycles. These parameters are found to vary with temperature, moisture, and material type. On a practical basis, determining these parameters from permanent deformation tests will be difficult to implement in a performance-related specification. Even though equipment are available to do these tests in the field, one test takes a significant amount of time to run, about 2.8 hours for 10,000 cycles at 1 second per cycle. There are, however, equations for estimating these parameters that have been reported in the literature. In particular, Lytton (1990) has shown that the slope *s* of the log-log relationship between permanent strain and cumulative load cycles is equal to the slope *m* of the creep compliance curve, which is determined much more easily from frequency sweep tests.

The relationship for predicting μ is more complicated. However, note that μ is a function of the intercept *I* of the log-log relationship between permanent strain and cumulative load cycles. Physically, *I* corresponds to the permanent strain at the first load repetition. Thus, if *s* is determined from another relationship, i.e, using the slope *m* of the creep curve, it may not be necessary to run the test at a large number of load repetitions to estimate μ. Thus, researchers propose a laboratory program of permanent deformation and frequency sweep tests to:

- verify the relationship between *m* and *s* for AC mixtures used by TxDOT,
- establish a rational but practical methodology for estimating α and μ, and
- develop a database of permanent deformation parameters that may be used in the absence of actual test data.

With respect to evaluating the impact of ride quality on predicted performance, it will be necessary to investigate relationships between measured surface profile and vehicle dynamic loading. This investigation will involve modeling the response of a standard truck to the as-built surface profile which can be measured using lightweight inertial profiling equipment that have become available. In fact, TxDOT recently introduced a new ride specification based on measurements of surface profile with these lightweight devices. The

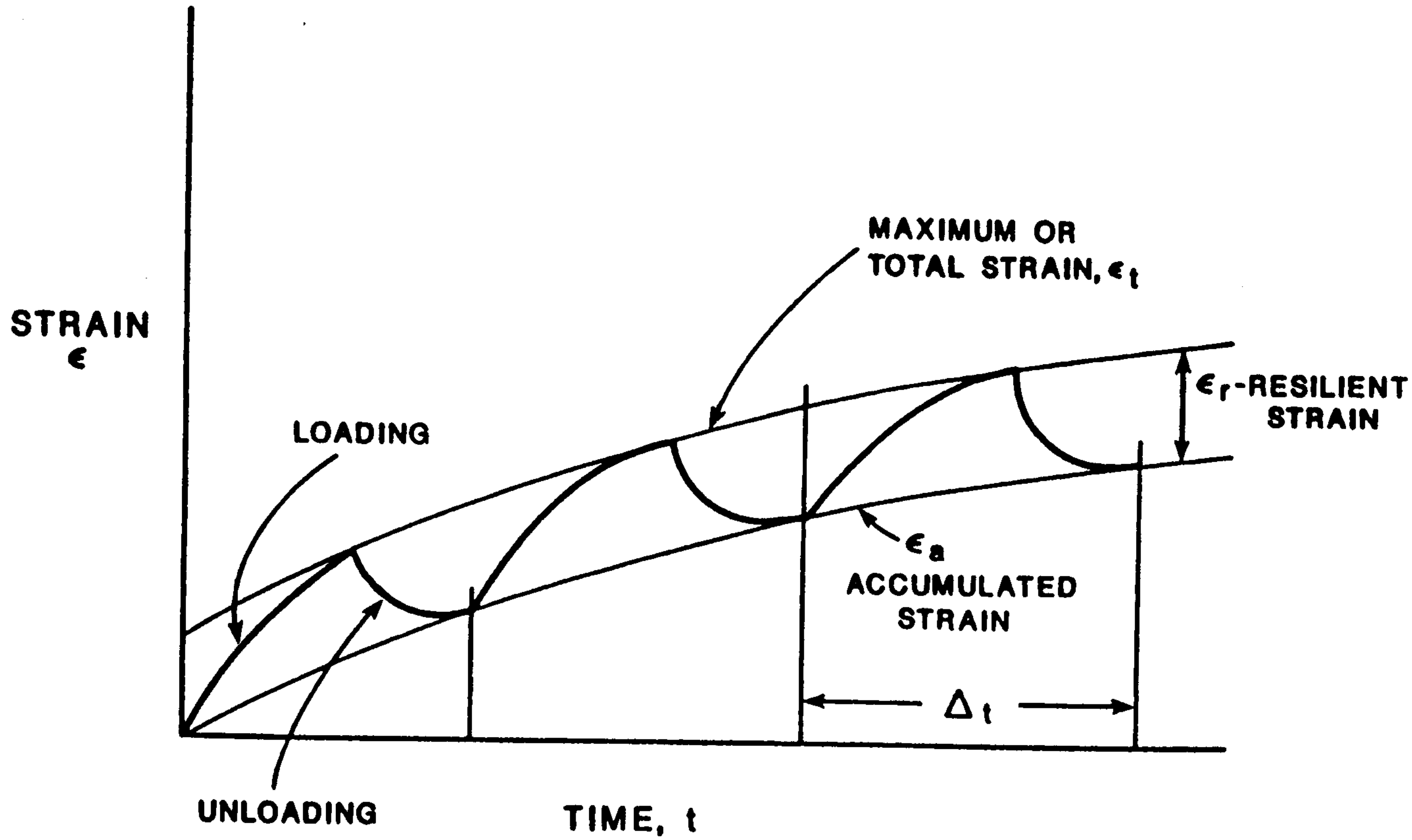

Figure 4. Conceptual Illustration of Data From Repeated-Load Permanent Deformation Test (Hoyt et al., 1987).

investigation of vehicle dynamic response will yield theoretical pavement damage factors due to dynamic loads induced by various profiles. If this investigation shows that the current profile specification effectively eliminates damaging dynamic loads, then no further development will be necessary to evaluate the effect of initial ride quality. In this case, our recommendation will be to continue implementing the profile specification for HMAC pavements that has recently been introduced. On the other hand, if the investigation shows that the existing specification allows pavements with profile wavelengths that are detrimental in terms of the predicted dynamic load response, then our recommendation will be to develop a truck index for new construction that may be used to evaluate the initial profile in terms of predicted performance. Similar work has been conducted by Fernando (1998), who proposed the following index for evaluating the initial surface profile of overlaid pavements:

$$\Delta = \left[\frac{1 + zCV_0}{1 + zCV_1} \right]^n - 1 \tag{13}$$

where,

Δ	=	predicted change in pavement life due to differences between the as-built and target profiles
z	=	the number of standard deviations corresponding to a given percentile of the predicted dynamic load distribution
CV_0	=	coefficient of variation of predicted dynamic loads for the target profile
CV_1	=	coefficient of variation of predicted dynamic loads for the as-built profile
n	=	fracture parameter of the Paris and Erdogan (1963) equation

Fernando (1998) derived the above equation based on reflection crack growth which is considered the dominant mode of distress for asphalt concrete overlays. Note that $\Delta = 0$ when $CV_0 = CV_1$ (i.e., the target and as-built profiles) are the same. However, if the as-built surface is rougher than the target, (i.e., $CV_1 > CV_0$), Δ is negative indicating a reduction in predicted pavement life because of the higher impact loading. Conversely, if the as-built surface is smoother than the target ($CV_1 < CV_0$), Δ is positive indicating an increase over the design life. Note that the predicted change in pavement life also varies with the fracture parameter *n* which also influences the predicted fatigue life. Thus, Eq.(13) shows that, aside from surface smoothness, the design, production and placement of the overlay mix are also important to building overlays that last their design lives.

Task E. Field Investigations

Researchers propose to select a number of HMAC paving projects on which measurements of performance-related parameters identified in Task B will be made using the test methods identified in Task C. In addition, material samples shall be taken for sample preparation and testing in the laboratory. There are a number of objectives in this task:

- support the development of test methods for quantifying segregation, longitudinal joint density, ride quality, and construction uniformity;
- establish sections for long-term performance monitoring and verify the effects of the above construction quality indicators on predicted pavement performance;
- collect field samples in potential defect areas so that the engineering properties can be measured in the laboratory. These properties can then be used to calculate their impact on pavement life, as the first step in assessing a penalty for each defect; and
- determine what problems exist with implementing each of these new procedures in the field.

Projects will be selected from among the QC/QA projects already programmed by TxDOT. Researchers propose to select monitoring areas within each job and then conduct an extensive series of testing before, during and after placement of the HMA surfacing layer. This testing will include but not be limited to the following activities.

1) Pre-Construction Testing

These tests will be performed if the project is an overlay of an existing facility rather than part of new construction. The goal is to characterize the variability in the existing structure so that its impact on long term pavement performance can be estimated. This test will include:

- a condition survey,
- a GPR survey to look for buried stripping in old HMA, and
- a profile survey to measure existing levels of roughness.

TxDOT's GPR van (Figure 5) will be used to collect radar data on the field projects established for this task. In addition, profile data will be collected using one of TxDOT's inertial profilers.

2) Measurements and Testing during Placement of HMA (QC Applications)

The primary tool here will be an Infra-Red (IR) camera to measure the variation in surface temperature of the mat prior to compaction. The low temperature areas will be noted for future coring.

TTI also proposes to establish a mobile laboratory on site with a gyratory compactor so that control samples can be made for future laboratory testing. Six-inch diameter by eight-inch high samples of the HMA will be molded at both the average mat temperature and at the low mat temperature as recorded by the IR camera. The difference in engineering properties of samples compacted at the range of field temperatures can then be established in the laboratory. The lab tests that will be conducted will include engineering properties such as strength, fatigue cracking potential, permanent deformation properties, and permeability.

The reason for establishing the field laboratory to make big samples is because of the difficulty in measuring material properties on cores taken from thin layers. By molding specimens and then taking cores it will be possible to compare results from both sets of samples.

While the HMA is being placed, researchers will also note the end of each load of HMA and denote locations where the paving operation stopped.

3) Measurements Taken Shortly after Compaction (QA Applications)

The tools to be used here will be the P-SPA for measuring the modulus of the HMA layer, the GPR for measuring both thickness and density, the falling head permeameter for measuring permeability on cores taken from the projects, and the lightweight inertial profiler for overlay smoothness. In addition, we plan to use the surface texture laser system currently under development by TxDOT to estimate texture depth which has been found to correlate

Figure 5. TxDOT's GPR Van.

with different levels of segregation in NCHRP Project 9-11. For this purpose, we have communicated with the Pavements Section about using its laser-based texture measuring system in this study. The objective is to determine if we can use TxDOT's system to measure segregation for quality assurance purposes. We were told that the system can be mounted on one of the department's multi-function vehicles for the tests planned in this task.

With respect to the permeability measurements, there is currently no TxDOT test method for measuring the permeability of compacted asphalt concrete mixtures. However, a test procedure was proposed by Izzo and Button (1997) in Project 0-1238 which we will consider using in this project. Alternatively, Florida has a standard test method designated as Florida Method of Test, FM 5-565, that uses the falling head permeameter illustrated in Figure 6 to determine the water conductivity of molded asphalt concrete specimens or cores. We will decide which test procedure to use after consulting with the project monitoring committee.

Areas to be tested will be the average and low mat temperature areas as identified by the IR camera, locations where the paving operation stopped, and the longitudinal construction joint. Longitudinal profile measurements will be made on both wheelpaths.

4) Field Coring and Laboratory Testing

For validation purposes it will be critical to coordinate the coring locations with the location of the QA tests described above. The intent will be to take these cores to the laboratory and measure the standard QA properties, e.g., density and air voids; other basic mixture properties such as asphalt content, penetration, and Hveem strength; and engineering properties (permeability, fatigue, and rutting parameters). Cores will be taken in areas judged to be representative of the entire mat, areas where possible segregation occurred, and areas close to the longitudinal construction joints.

The field core properties will be compared to those measured on the samples compacted in the field laboratory during HMA placement. The laboratory properties will be used to estimate changes in predicted pavement performance and to form a basis for developing a rational bonus and penalty system. This development will be accomplished using the models developed in Task D.

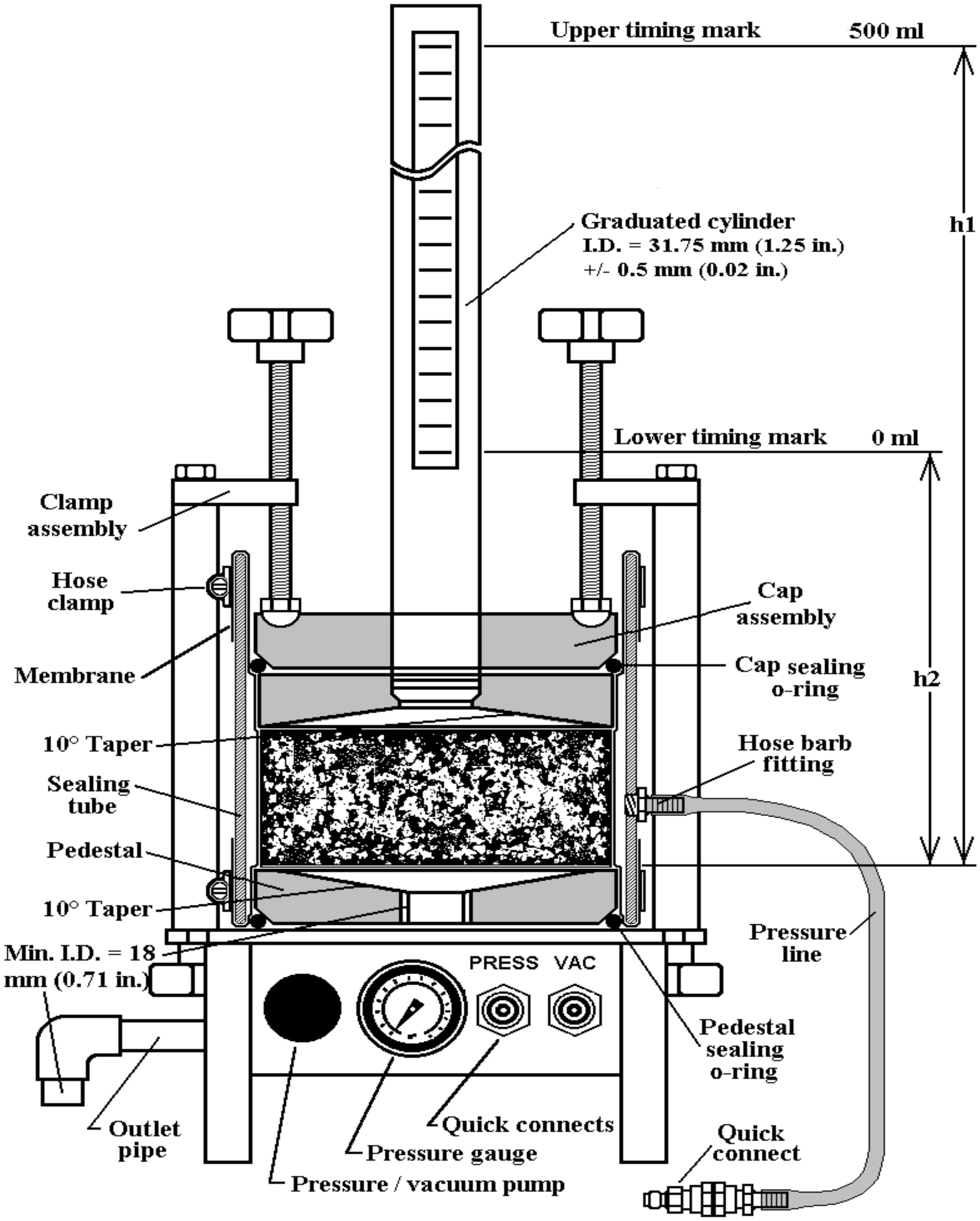

Figure 6. Florida Permeability Testing Apparatus (Florida Sampling and Testing Manual, 2000).

Task F. Model Verification

To verify the models that will form the basis for developing the acceptance schedule in a performance-related QC/QA specification, we propose to use the accelerated pavement analyzer (APA) to test the different specimens obtained from the demonstration projects in Task E. The intent is to rank the materials in terms of resistance to fatigue and permanent deformation as determined from the APA tests. The fatigue and permanent deformation properties determined for these materials will also be used with the models from Task D and the rankings obtained based on the performance predictions will be compared to the rankings from the APA tests. If the rankings are close, we would then have demonstrated that the models produce realistic results.

It is recognized that the proposed plan will not provide a true verification. However, we believe that this can only be accomplished with a long-term monitoring effort. For this purpose, we recommend that a monitoring program be established by TxDOT to periodically survey the demonstration projects established in Task E. We will establish the data requirements for this monitoring effort as part of this task. Long-term field performance data will then be available that will allow TxDOT to conduct a field verification of the models in later years using the baseline data collected in Task E.

Task G. Develop Computer Program for Evaluating As-Built HMAC Pavements

This task is expected to produce a Windows (TM)-based program to evaluate the quality of as-built HMAC pavements based on segregation, longitudinal joint density, ride quality, construction uniformity, and the other quality indicators of relevance to this study. The program shall be written to permit an integrated analysis of data from various sources, such as the FWD, P-SPA, GPR, inertial profilers, and other field and laboratory test methods established in this study. To consider the variability in measured properties, it will be necessary to incorporate Monte Carlo simulation techniques into the program so that the expected variability in predicted performance can be evaluated. This incorporation will require information on the distributions of the parameters that are input to the performance models. In this regard, characterization of these distributions will most likely be simplified if nondestructive methods such as GPR, P-SPA, and FWD can be used in practice to measure the required parameters. However, for those parameters that will require laboratory tests, a sampling procedure will be necessary to characterize the variability of the given parameter. This issue will have to be addressed in order to implement a computer program that models the stochastic nature of the parameters used to predict performance.

Task H. Draft New Test Specifications for QC/QA

As necessary, researchers will write new test specifications for implementing the products of this study. Draft documents will include:

- new test methods or procedures for measuring construction quality indicators;
- materials and pavement design specifications; and
- procedures that will have to be put in place to ensure that the proposed test methods will provide accurate, repeatable, uniform, and consistent measurements during construction.

Task I. Pilot Implementation

We propose to work with a number of districts to conduct a pilot implementation on selected HMAC paving projects. We plan to meet with district and contractor representatives to introduce the proposed test methods and specifications in Task H and the computer program developed in Task G. The purpose of this pilot implementation is to field test the procedures developed. This field test may result in further changes to the proposed test methods and specifications to streamline subsequent implementation efforts. It will be important to select QC/QA projects where tests will also be conducted under the existing specifications. This information will permit us to demonstrate to TxDOT the potential benefits of implementing the products from this research project.

Task J. Documentation of Research Work

The research team will document results from this project through the following:

- new test methods and specifications for evaluating the quality of as-built HMAC pavements;

- a comprehensive report that covers the development and verification of performance models to evaluate HMAC pavements based on predicted performance; evaluation of relationships between basic mixture variables and fundamental material properties; correlations between properties measured by different test methods; set of mixture-, structural-, and construction-related factors that significantly affect pavement performance and are under the contractor's control; and the pilot implementation in Task I;
- a user's guide to the computer program for evaluating as-built HMAC pavements; and
- a project summary report describing and referencing the test methods developed and providing recommendations for implementation and further development.

CHAPTER III

SUMMARY

Project 0-1708, "Predicting Hot-Mix Performance from Measured Properties," aims to develop rational, reliable and practical test procedures for evaluating the quality of HMAC pavements based on predicted pavement life. The first phase in this development was a review of the current state-of-knowledge with respect to test methods for measuring construction quality indicators and the models available to establish the impact of the contractor's operations and decisions on expected performance. This review showed that advances made over the years in methods and equipment for pavements and materials testing make the development of performance-related tests a much more achievable objective.

Automated and nondestructive test equipment already in place within TxDOT offer the potential for improved quality control and quality assurance procedures. These automated devices include ground penetrating radar, the falling weight deflectometer, the portable seismic pavement analyzer, inertial profilers for measuring ride quality, and laser-based systems for measuring surface texture and skid resistance. In addition, equipment for materials characterization has become available that is suitable for field applications.

What is needed is to investigate QC/QA applications of these techniques and to develop acceptance criteria applicable for the asphalt concrete mixtures used in Texas. Researchers propose to conduct this investigation in the next phase of the research project. Specifically, researchers propose to monitor construction projects in the state and to collect data with which to evaluate QC/QA applications of test equipment. The plan is to monitor three such projects during the second year of the study with additional projects selected in the third year based on the available funding.

The intent is to establish whether certain signatures are observed from the test data which may be used to measure the construction quality indicators of relevance to this study. Test data will be checked against corresponding measurements made on cores or molded specimens to establish the applicability of the selected nondestructive tests for QC/QA purposes. Results will establish a catalog of test methods for measuring performance-related parameters that include nondestructive tests in-situ as well as tests on cores or molded specimens. Since several procedures may be available for measuring a particular parameter, consideration will be given to integrating the test methods into a hierarchical framework consisting of different levels that are tied to the design number of 18-kip ESALs.

In addition to developing test methods for measuring performance-related parameters, it will be necessary to relate these measurements to predicted pavement life in order to establish a rational basis for pay adjustments during acceptance testing. The literature review has shown that existing models are available for establishing these relationships. However, it will be necessary to verify and calibrate these models for the asphalt concrete materials used by TxDOT. This work will require laboratory characterizations of these materials to establish rational and practical test methods for determining the required parameters and to develop a database of material properties that may be used in developing the acceptance criteria. To verify the models, we propose to use the accelerated pavement analyzer to test the different specimens obtained from the field projects. The intent is to rank the materials in terms of resistance to fatigue and permanent deformation as determined from the APA tests and to compare these rankings with the corresponding results from the performance models. While this will establish whether the models produce realistic results, we recognize that a more rigorous verification will require a long-term monitoring effort that is beyond the scope of the present study. Consequently, we recommend a long-term monitoring program to collect data on the field projects for use in later years to verify the models using the baseline data collected in this study.

Research products from this project are expected to include new test methods and specifications for evaluating the quality of HMAC pavements based on predicted performance. Implementation of these products will require training schools to inform and teach TxDOT engineers about the performance-related tests developed from Phase II and Phase III. Meetings will have to be held with industry representatives regarding changes in the QC/QA test protocols. In view of the potential far-reaching impact that the research results may have, implementation of the new test methods and specifications will most likely have to be done in stages over a period of time.

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APPENDIX - LITERATURE REVIEW

INTRODUCTION

Existing literature that provides a background on the main issues concerned with Project 0-1708 was reviewed and is presented in this appendix. To start with, available material that provides some basic information regarding construction-related problems and how they have been dealt with in the past has been summarized in the section entitled "Construction Problems in Hot-Mix Asphalt." The main topics covered in this section include:

- segregation,
- ride quality,
- longitudinal joint density, and
- non - uniformity in layer thickness.

Next, a detailed review of the recent and ongoing related studies was done and a summary of each of the studies reviewed is compiled in the section entitled "Review of Ongoing and Recently Completed Studies that Relate to Project 0-1708." This section is followed by a section on responses to agency surveys. This section has been compiled from various studies that are related to Project 0-1708. In conclusion to the literature review, a section on the specifications of the Federal Aviation Agency is presented as an example of the agency specifications.

CONSTRUCTION PROBLEMS IN HOT-MIX ASPHALT

An extensive review of available material was undertaken in order to identify the causes, the available identification and measurement techniques, and the effects of these problems on the performance of the pavement.

Segregation

Segregation refers to the non-uniform distribution of coarse and fine aggregate, which are major components of an asphalt concrete mixture (AASHTO, 1997). This term would also imply that there is a non-uniform distribution of the asphalt cement, since more asphalt cement is required to coat finer particles due to their high surface area (Brock and Jakob). Segregated mixtures generally do not conform to the original job mix formula and, as a result, such areas may exhibit poor structural and textural characteristics, which in turn result in poor performance (Cross and Brown, 1993). Although segregation can occur in any hot-mix asphalt concrete mixture, it is more prevalent in coarser mixtures and especially gap-graded mixtures. Segregation may occur anywhere in the manufacturing and construction phases, starting at aggregate stockpiling and ending with improper operation of the paver (Williams et al., 1996). Therefore it is important to make sure that the material is handled properly during the entire manufacture and placement of the HMAC. It is also equally important to check for segregation at early stages after placement, so that it can be rectified immediately and its effects on pavement performance reduced (Williams et al., 1996). AASHTO has identified five basic types of mix segregation (AASHTO, 1997). These include:

- Truck end segregation

  Occurrence: Spots occur longitudinally on either side of the lane being paved. These spots are composed of coarse aggregate separated from the mix.

  Cause: Improper truck loading and unloading, silo segregation, or running the hopper empty due to loads.

- Truck end segregation/one side

  Occurrence: Spots occur longitudinally on one side of the lane being paved.

  Cause: Improper loading of the batcher on the hot storage bin.

- Center line segregation

  Occurrence: Along the centerline of the pavement.

  Cause: Concentration of coarse aggregates in the center of the mat as it rolls underneath the auger chain drive.

- Joint edge segregation

  Occurrence: Outer edge of the pavement being placed.

  Cause: Insufficient speed of augers on the paver causing the coarse aggregates to roll to the outside of the mat.

- Random segregation

Occurrence: Random.

Cause: Unidentifiable, but usually due to improper mixing.

Figures A-1 through A-5 illustrate the five basic types of segregation.

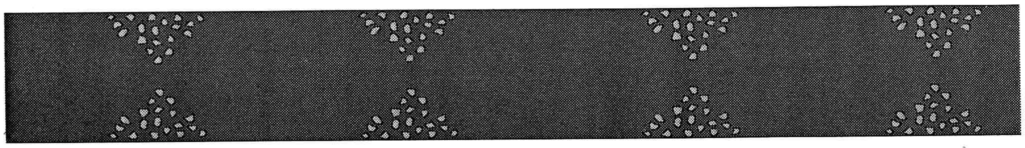


Figure A-1. Truck End Segregation (AASHTO, 1997).

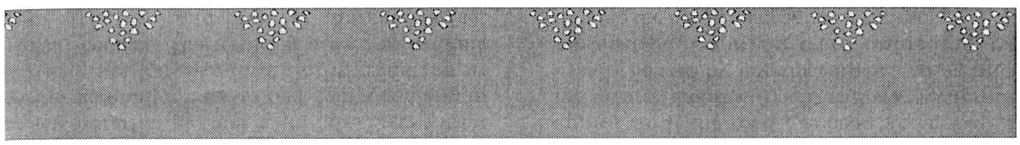


Figure A-2. Truck End Segregation/One side (AASHTO, 1997).

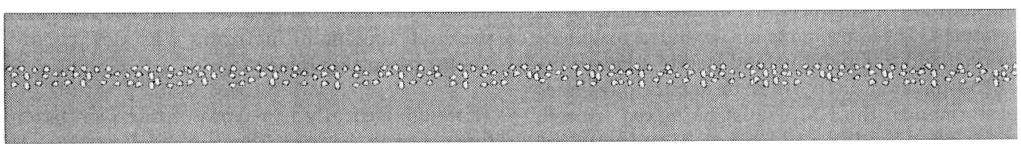


Figure A-3. Centerline Segregation (AASHTO, 1997).

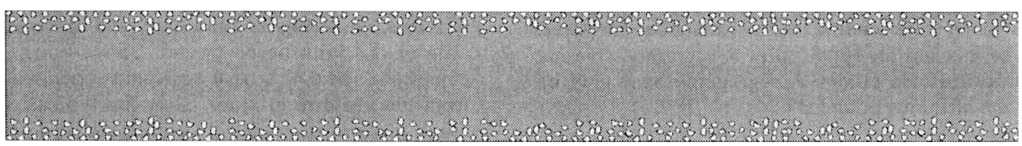


Figure A-4. Joint/Edge Segregation (AASHTO, 1997).

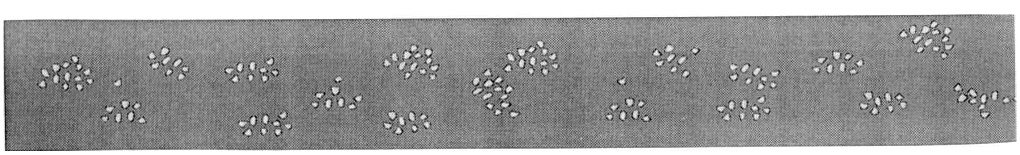


Figure A-5. Random Segregation (AASHTO, 1997).

Causes for Segregation

Mix Design

Improper mix design is one of the main factors that cause segregation (Kennedy et al., 1987). Mix design factors that influence segregation are:

- Asphalt content: In an effort to achieve higher stability, engineers have reduced the asphalt content of typical mixes. This reduction results in reduced cohesion, which in turn leads to segregation. Hensley (1992) has suggested that the design asphalt content should not vary by more that one-half percent (on the lower side), as the asphalt content determined for minimum voids in mineral aggregate (VMA) condition. This would ensure adequate cohesion in the mix. It has been indicated by Brock and May (1988) that by increasing the asphalt content by as little as 0.2 percent, the segregation potential of the mix may be reduced significantly.
- Gradation: It has been observed that coarse-graded and gap-graded mixes are more prone to segregation (Kennedy et al., 1987). Kennedy et al. suggested that mixes designed close to the maximum density gradation would have a reduced potential for segregation.
- Aggregate characteristics: The problem of segregation has been associated with blending of aggregates having widely different specific gravities (Cross and Brown, 1993). Segregation has been observed on some road sections in Alabama due to the blending of steel slag (bulk specific gravity of 3.138) with blended aggregates (bulk specific gravity of 2.588) (Cross and Brown, 1993).

Mix Production

- Improper stockpiling: Loading of aggregate on top of a conical stockpile causes the larger particles to roll down to the outside of the pile. It has been recommended that stockpiles should be constructed in layers and the slopes of the pile should be restricted to less than 30 percent to avoid segregation due to stockpiling (Hensley, 1992).
- Screening process in the batch plant: The screening action causes the larger particles to move toward the far ends of the bin (Hensley, 1992).

- Drum mixer: Larger particles may flow through the mixer at a slightly faster rate than the smaller particles. This problem is seen in gap-graded mixtures.
- Conveying the mix to the storage silo: The cool surface of the drag conveyor may cause the fine material to adhere to the conveyor, which may result in the hydroplaning of the drags. This would cause the coarser particles to ride over the drags, thereby increasing segregation (Brown et al., 1989).
- Storage in the silos.

Mix Transportation

Segregation can occur while the mix is transported to the job site. Improper truck loading (loading in one conical dump) can cause segregation. Segregation can also occur when the material is unloaded into the paver. Kennedy et al. (1987) suggested that the truck bed should be raised slightly before opening the tailgate to minimize the slow movement of the mix and aid in deluging the paver hopper.

Paving Operations

- Gradation segregation
  If the hopper is completely empty, coarse material will tend to accumulate in the wings and when the wings are dumped and a lateral band of segregated mix is laid down (Brown et al., 1989). Augers in bad condition will cause larger particles to congregate at the worn or broken locations.
- Segregation due to temperature differentials
  Segregation due to a thermal differential was first identified by Mr. Steve Read (University of Washington) in his study on the cause and potential solution for cyclic segregation. The mechanism of temperature segregation begins when a load of HMA is dumped into the paver. If a temperature differential exists in the mass, the cool material along the sides of the load is extended outward to the sides of the paver's hopper. As the pile in the hopper is discharged, the cool material falls inward to lie on top of the material

over the slat conveyors. The cool mix is subsequently conveyed back to the screed with the next load, but the screed is unable to consolidate this colder mix (Read, 1996).

Identification and Measurement of Segregation

Visual Identification: In the past, identification of segregation has been mainly through visual inspection, which may not be very accurate as some spots may not be visible untill months after the pavement has been opened to the traffic (Brock and Jakob). It would also be important to note that segregation occurring in the middle of a thick lift layer may not be identified by visual inspection (Killingsworth, 1999). The mixture may look normal behind the laydown machine. However, after several months of high-speed traffic, surface fines may be ripped away, revealing the bare rocks. In order to achieve consistency in visual identification, the Ontario Department of Transportation makes use of a distress identification manual to help identify different levels of segregation, and South Carolina has a team of experts who can identify segregated areas. Efforts have been made to develop more reliable methods to determine segregation.

Nuclear Density Gauge

The Kansas State Department of Transportation has included methods to detect segregation in their specification (Wilson, 1999). It has suggested the use of the nuclear gauge to measure density longitudinally along the pavement in order to identify variations in density. The assumption is that segregation will be seen as low density due to the presence of large air voids (Stroup-Gardiner and Brown, 2000). A maximum limit has been set for the variation in density. The state DOT has also specified that the difference in the mean density and the lowest density value should not exceed 2.5 pcf. This method has been found to be effective in detecting truck end segregation (Wilson, 1999).

A study conducted by Michigan State University to detect linear pattern segregation recommended that linear nuclear density profiles should be used for quality control procedures (Wolff et al., 1997). The study also noted that several sites that were visually identified as segregated areas did not indicate a significant change in density.

Thermal Imaging

'ASTEC Industries' has recently used an infrared camera in an attempt to detect segregation (Brock and Jakob). Infrared cameras can be used to help identify the thermal differential across a bed (Brock and Jakob). The main principle behind the use of an infrared camera is that all materials emit infrared radiation in the form of heat which can be detected by the infrared scanner (Stroup-Gardiner and Brown, 2000). These pulses are then converted into electrical pulses, which are then processed to form images of the thermal energy. This technology has been used to detect delaminations in bridge decks, defects in concrete, and asphalt overlay debonding (Pla-Rucki, 1985; Weil, 1989; Manning, 1986). It has also been shown that the areas with high air void content (segregated areas having coarse gradations) will cool faster and areas with a high fine aggregate content will retain heat longer (Pellinen, 1991; Lahtinen, 1991). It should be noted that this technique depends on the solar heat gain and hence its applicability is limited by environmental conditions. It should also be noted that this method is useful in detecting temperature differentials directly behind the paver and is not suitable for later detection (Stroup-Gardiner and Brown, 2000). It is not possible to distinguish between temperature segregation and gradation segregation using this technique (Stroup-Gardiner and Brown, 2000).

Sand Patch Test

The standard for this test is ASTM 965 (1997). This test has been used to quantify differences in the surface macrotexture (Brown, 1988). The results of this test have been reported to have good correlation with visual observations of non-uniform texture. Results show that if a maximum limit of 0.3 mm is placed on the macrotexture, 88 percent of the area with air voids greater than 10 percent due to either under-compaction or segregation is identified (Stroup-Gardiner and Brown, 2000). It is important to note that the limits on surface texture are mix-specific.

Ground Penetrating Radar (GPR)

GPR technology has been used in the past for computation of layer thickness and stripping in the lower layers of the pavement structure (Smith, 1993; Maser, 1992; Rmeili and Scullion, 1997). This technology uses the dielectric property of the material. A project in Finland noted a reduction in the dielectric value of the material at the ends of truckloads and also at places where the paver experienced problems (Saarenketo, 1997). A paper by Saarenketo (1997) discusses the use of GPR and dielectric probe measurements in pavement density control. Laboratory tests were carried out at Texas Transportation Institute and the field tests were performed in Finland. The method is based on the fact that the dielectric value of the asphalt concrete material would decrease as a result of high air voids (dielectric of air = 1, water = 81, aggregates = 4.5 to 6.5, asphalt = 2.6 to 2.8). The results of the lab tests showed a positive relationship between the dielectric value of the material and the dry density of the sample. The field tests showed that there was a parallel variation in the dielectric measurements of the inner and outer wheel paths. The dielectric values for the outer wheel path were found to be consistently higher than those for the inner wheel path. This difference can be explained by the fact that traffic load compaction on roads with a bilateral gradient is more pronounced on the outer part than the inner path. The study also shows that examination of a particular stretch that had a peculiarly low dielectric value indicated an extremely open surfaced pavement in that region. It must be noted that the study was based on the assumption that the asphalt content of the material would remain constant. If this were not the case, fluctuations in the dielectric value of the material would not necessarily indicate a change in the air voids. This technique was recently used by TxDOT on a project in El Paso. Figu re A-6 shows the use of radar measurements to detect segregation.

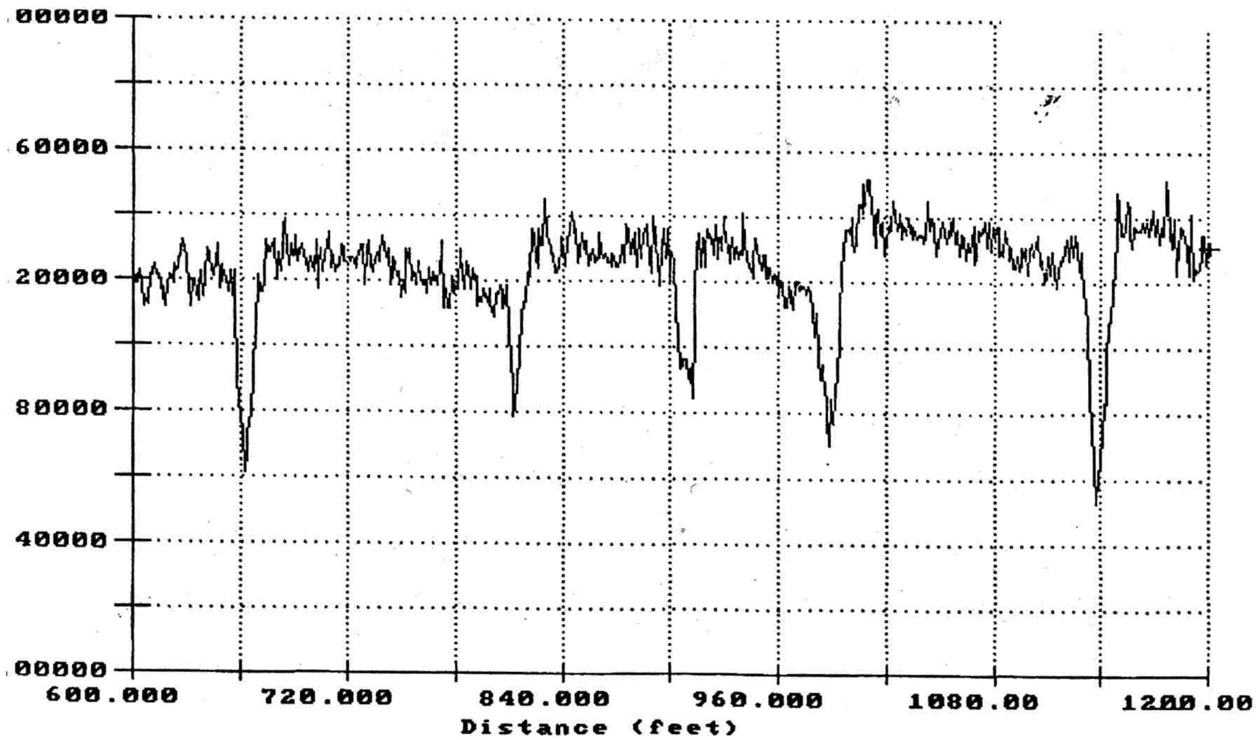


Figure A-6. Detection of Segregation Using Radar.

Laser Surface Texture Measurements

This method adapts a laser to produce infrared light, which is projected onto the pavement, and receiving lenses focus the scattered light onto photodiodes. "The diode receiving the most light corresponds to the distance to the surface" (Stroup-Gardiner and Brown, 2000). Determination of texture is done using a series of such measurements. It has been demonstrated that there is good correlation between the results of the laser tests and the sand patch tests (Cooper, 1974; Hallett and Wix, 1996). Examples of such profilometers include the ARAN profilometer, multi-laser profilometer system (MLP), and the ROSAN. A major advantage of this equipment is that it can be operated at highway speeds (50 mph). It is also very useful as it provides a quantifiable measurement of segregation. However, this technique can be used only on dry pavements. The technique is capable of measuring surface defects only.

Thin Lift Nuclear Density Gauge/Asphalt Content Gauge

Stroup-Gardiner and Brown (2000) report that a prototype of this instrument, which has been developed by Troxler, is an improvement over the traditional density gauge in that the depth of measurement is limited to the upper layers. As a result of this, variations due to the underlying layers are eliminated. Limited laboratory studies have indicated that there is a good relationship between the gauge readings and asphalt content. As the readings are dependent on the volume of voids in the HMAC, it should be effective in determining changes in density as well. The asphalt content can be determined in place, and hence, the percentage of non-uniform area resulting from aggregate-asphalt separation may be determined easily, using these data with density measurements. However, as concurrent density measurements are needed to completely use the data, two gauges per test may be required. The gauge is reported to be useful mostly during construction work, as it is sensitive to moisture content.

Seismic Devices (SPA and P-SPA)

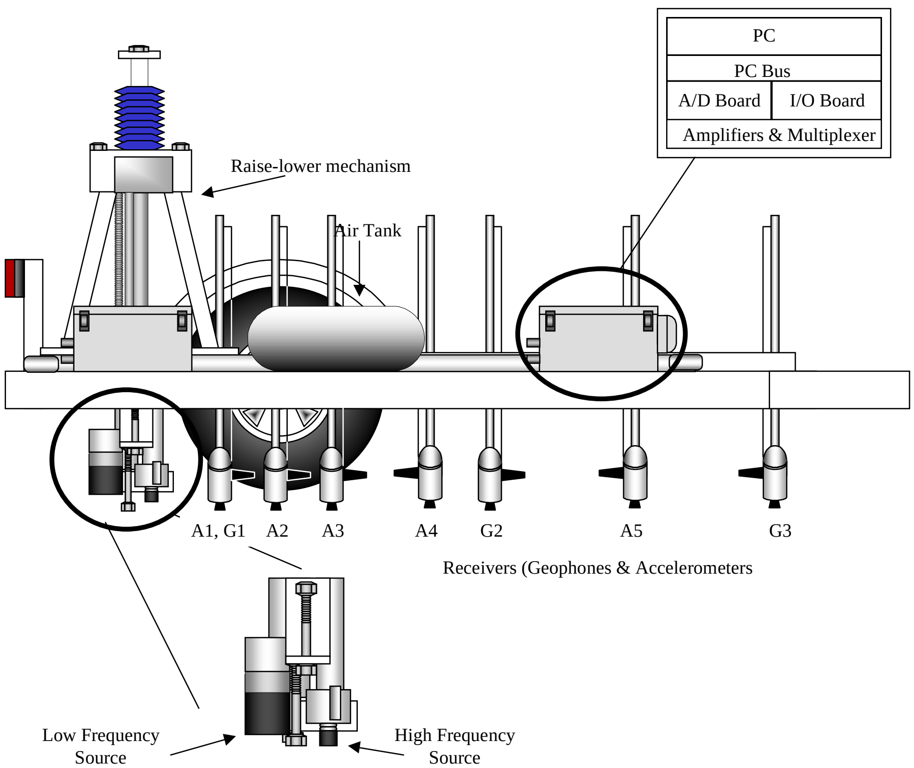
The seismic pavement analyzer, as shown in Figure A-7, is an instrument designed to determine the variation in modulus with depth of pavement sections. However, information obtained from the surface layers may be used to assess the impact of segregation on performance, as the stiffness values can be correlated to density (Stroup-Gardiner and Brown, 2000).

With the SPA, shear and/or Young's modulus of different layers can be estimated using one or all of the following methods:

- ultrasonic body waves (UBW),
- ultrasonic surface waves (USW),
- impulse response (IR),
- impact echo (IE) and
- spectral analysis of surface waves (SASW).

The SPA records the pavement response produced by high- and low-frequency pneumatic hammers on five accelerometers and three geophones. A computer controls data acquisition,

a) Device in Use



b) Schematic

Figure A-7. Seismic Pavement Analyzer.

instrument control, and interpretation. The quality of collected data is generally better than the quality of those collected manually because a computer controls the operation of the source and receivers.

The equipment has been used in several applications:

- Analyzing, in detail, pavement conditions in project-level surveys;
- Diagnosing specific distress precursors to aid in selecting a maintenance treatment; and
- Monitoring pavement conditions after construction as a quality control tool.

The operating principle of the SPA is based on generating and detecting stress waves in a layered medium. Each of the five tests and its areas of strength and weaknesses are summarized in Table A-1. The design and construction of the SPA are based on two general principles. First, the strength of each method should be fully utilized and, second, testing should provide enough redundancy to identify the properties of each layer within a pavement.

The ultrasonic-body-wave method can determine Young's modulus of the top pavement layer. Similarly, the ultrasonic-surface-wave method can be used to determine the shear modulus of the material. Measuring the stiffness of the slab at different locations using the impulse-response method can evaluate the condition of the support. The impact echo method can be used to determine the overlay delamination or to measure the thickness of the top layer. The SASW method can be utilized to determine the modulus and thickness of each layer in the pavement.

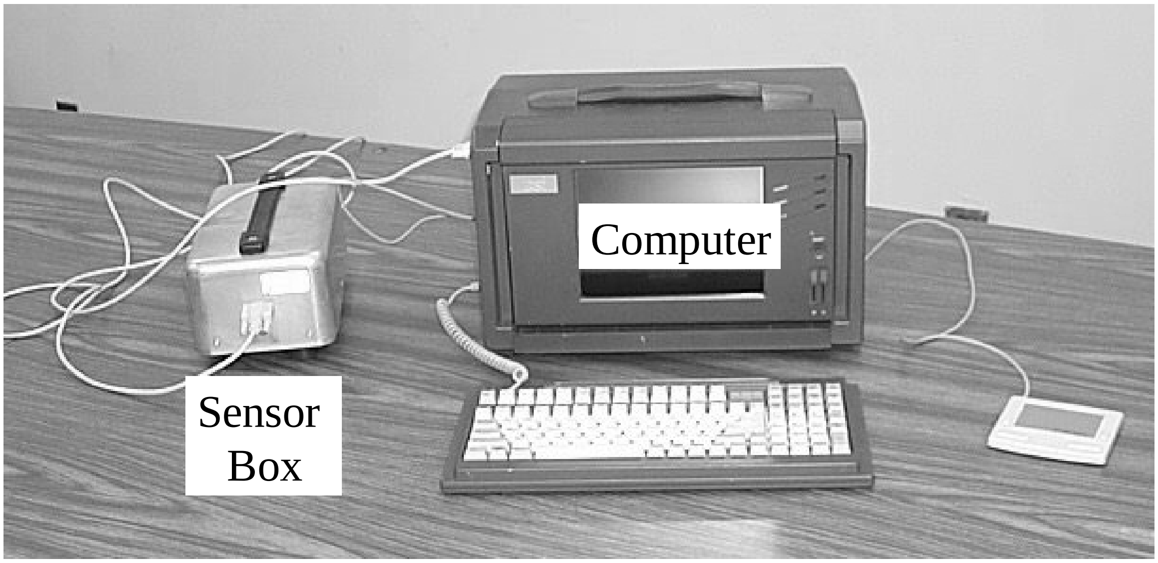
The P-SPA (see Figure A-8) consists of two transducers and a source packaged into a hand-portable system, which can perform the ultrasonic body wave, ultrasonic surface wave and impact echo tests. The device is operable from a computer. This computer is tethered to the hand-carried transducer unit through a cable that carries power to the accelerometers and hammer and returns the measured signal to the data acquisition board in the computer.

The major mechanical components of the P-SPA sensor box, as depicted in Figure A-8, are a near and far accelerometer and an electric source. The main structural member holding the transducers and source is a plate mounted to the base of the box. Rubber vibration isolators de-couple the accelerometers from the plate above 200 Hz. The source is directly mounted to the plate.

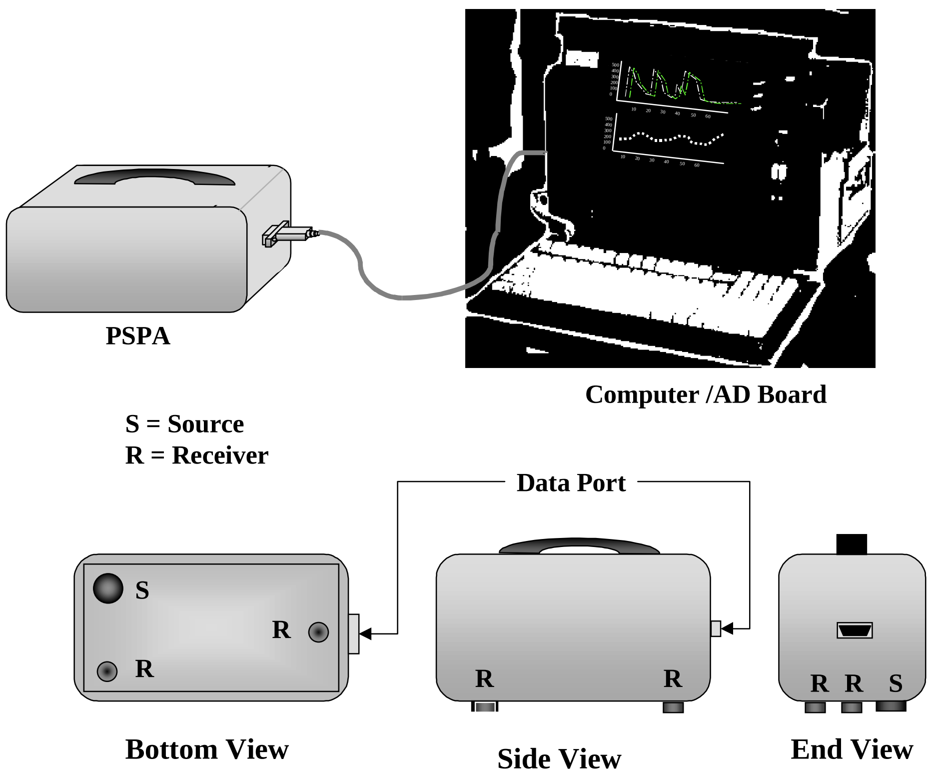
This instrument is very useful as it can determine, in place, variations in the performance-related material properties. It must be noted that the results are temperature dependent and hence normalization of the data is necessary. The P-SPA gives information necessary to determine the effects of various levels of segregation on performance.

Table A-1. Pavement Parameters Measured with Different Seismic Methods Used in SPA and P-SPA

Method	Primary Use/ Device	Strengths	Weaknesses
Ultrasonic Body Waves	Modulus of top layer (SPA and P-SPA)	• Rapid to perform • Simple data reduction	• Results may be affected by underlying layers • Sensitive to surface condition
Ultrasonic Surface Waves	Modulus of top layer (SPA and P-SPA)	• Sensitive to properties of top layer • Rapid to perform • Layer specific results	• For multi-course pavements, determination of layer-specific information is complex
Impact Echo	Thickness of top layer or depth to delaminated interface (SPA and P-SPA)	• Can determine thickness of the layer • Sensitive to delaminated interfaces	• Substantial contrast between the modulus of two adjacent layers is needed for sensitivity • For multi-course pavements, at least one core is needed for calibration • Applies only to pavements with thicker top layer
Impulse Response	Modulus of subgrade reaction of foundation layers or overall modulus of a pavement (SPA only)	• Powerful tool for rapidly locating weak spots in a pavement • May be used to estimate depth to stiff layer (in progress)	• For flexible pavements, the contribution of different layers are unknown • Results are affected by depth to rigid layer and water table
Spectral Analysis of Surface Waves	Modulus and thickness of each layer (SPA only)	• Provides the modulus profile in a comprehensive manner • More robust than deflection-based methods	• Data reduction are time consuming and complex • Automated analysis applicable only to simple structures



a) Device



b) Schematic

Figure A-8. Portable Seismic Pavement Analyzer.

Permeability

Stroup-Gardiner and Brown (2000) report that permeability tests may be useful only in defining various levels of coarse aggregate segregation, as the results of this tests are more dependent on the nature of the interconnected voids rather than the total void content. Research has indicated that the results of these tests have not been able to identify moderately segregated areas in fine and dense graded mixtures (Williams, 1996). The Florida Department of Transportation (FDOT) has implemented a falling head permeability test on cores which have the outer edges sealed in order to eliminate horizontal flow.

Aggregate Gradation and Asphalt Content

Studies on segregation have shown that there is a decrease in the asphalt content with a corresponding increase in coarseness (Bryant, 1967; Brock, 1986). Brown et al.(1989) have reported that segregated areas in 16 projects in Georgia have shown an asphalt content of 1-2% less than that in non-segregated areas (Brown and Brownfield, 1989).

Effects of Segregation on Performance

A study conducted by Williams et al. (1996) on HMA segregation included laboratory testing as well as field testing of four mixes with five levels of segregation. The laboratory segregation technique developed by Khedaywi and White (1995) was used to establish segregation levels for the mixes. All the mixes were first characterized by gradation asphalt content, density and air voids. Tests were carried out using air permeameters, nuclear density gauges, thermal imaging systems, and wheel tracking devices to evaluate performance of the segregated mixes.

Test slabs were prepared using a linear compactor specially designed for this study. Nuclear density tests and accelerated wheel track testing were carried out on this specimen. Effect of the base pavement types was considered, and both types of base layers were incorporated in the tests. Results from the wheel tracking tests indicate that the pavement performance is affected to a great extent in terms of rutting and stripping.

A study to evaluate the effect of segregation on fatigue performance was also undertaken by the above mentioned authors. They established a relationship between the number of cycles of failure of the segregated mixtures using laboratory beam fatigue tests. The results of this study indicate that the coarsely segregated asphalt mixture is associated with a low asphalt content and has a shorter fatigue life for the mix design tested. The finely segregated mixture exhibited a longer fatigue life; however, the lack of sufficient coarse aggregates in combination with the high asphalt content would make the mix more susceptible to rutting.

A study was carried out by Brown et al. (1993) to determine the effect of segregation on performance. The study selected five pavements from Alabama to evaluate how much segregation can be tolerated before premature raveling is likely. Severity of segregation and raveling was visually estimated and cores from the pavement were taken. Gradations of the cores were determined. Density measurements were made using a thin lift nuclear gauge. Results indicate that a variation in the percent passing No. 4 sieve of greater than 8-10 percent can lead to raveling. The study also developed a model to predict raveling from the macro texture and expected traffic.

Ride Quality

Ride quality of a pavement has been identified as a primary indicator of pavement performance (Asnani et al., 1993). Serviceability index, which is a measure of the functional performance of the pavement, is a function of roughness, cracking, rutting, and patching (Asnani et al., 1993). Since roughness is an indicator of all the other parameters, certain highway agencies compute the present serviceability index (PSI) based on roughness (Asnani et al., 1993). Controlling the initial roughness during construction can greatly improve the performance and the life of the pavement structure (Smith et al., 1997). Ksaibati et al. (1993) define roughness as “the vertical surface undulations that affect the vehicle operating costs and the riding quality of that pavement as perceived by the user.”

Causes

The ride quality or smoothness of any pavement structure depends on various factors, which include the existing condition of the underlying material, construction process, non-uniform compaction of the asphalt concrete material, segregation, and non-uniformity in layer thickness. It is important to note that we are only considering the initial ride quality of the pavement. It can be said that the HMA mixture properties as well as the material components will also influence the smoothness, as these factors relate to various distresses responsible for deterioration in the ride quality of the pavement.

Measurement of Ride Quality

Determination of the ride quality of any pavement involves the measurement of the pavement roughness. In order to quantify pavement roughness, accurate measurement techniques are required. A variety of devices are available today to measure the road profile. These devices range from a hand-held dipstick to high-speed, vehicle-based profilers. The dipstick is generally considered to be a very accurate measure of the profile, which can be converted to roughness, but the method is very time consuming. Various forms of profilers are available and they differ in the sensor types. Ultrasonic, optic, and laser sensors have been used (Perera et al., 1996).

Available Devices to Measure the Longitudinal Profile

A profile equipment evaluation study was conducted by Fernando et al. (1997) for TxDOT. The profilers that were evaluated include: – Digital Profilite Model 300 (CSC), Walking Profiler (WPR), Lightweight Inertial Surface Analyzer (LISA), Lightweight Profilometer T6400, Construction Profiler (CPR), Laser Rut/Profiler (LRP), and the Surface Profiler (SP). The team compared the profiles collected by the various profilers. The results of this study are summarized below.

Repeatability of Profiles

- A comparison of the two devices that measure and integrate differential elevations (WPR and CSC) indicated that WPR showed better repeatability.

- A comparison of the lightweight profilers (T6400, LISA, CPR) indicated that the data from the T6400 were not as repeatable as those from the other two.
- A comparison of the inertial profilers indicated that the van-mounted and, in particular, the SP showed best repeatability.

Comparison of Computed International Roughness Indices (IRI)

- The computed IRIs from CSC and WPR tend to underestimate the corresponding statistic from the rod and level data.
- A comparison of the IRIs from the lightweight profilers indicated that LISA had the smallest average discrepancy, which indicates greater accuracy as compared to the other profilers.
- A comparison of the van-mounted inertial profilers indicated that LRP tends to underestimate the rod and level IRIs. However, both profilers show similar levels of accuracy.

Accuracy of Profile Measurements

- A comparison of CSC and WPR profiles indicated that the WPR profile is more accurate relative to the rod and level data.
- A comparison of the profiles from the inertial profilers indicated that the van-mounted profilers correlated best with the reference profiles.
- A comparison of the lightweight inertial profilers indicated that the CPR measurements correlated most favorably with the rod and level data.

A paper by Collins et al. (1996), indicates that Georgia Department of Transportation (GDOT) had been using a rolling straight edge for construction specifications to measure and control pavement smoothness until 1972. The CHOLE profilometer was used on an experimental basis but was dropped as it was too slow to be used as a specified construction control tool; however, it was found to be more accurate than the straight edge. Later, the PCA roadmeter was adopted to specify the smoothness requirements. The PCA roadmeter did not have the capability of producing a physical trace and, as a result, the May's meter, which determines the response of the trailer to the pavement profile by measuring the vertical movement between the axle and the chassis of the trailer, replaced it. In 1990, after the evaluation of many profilers, GDOT chose the South Dakota-type profiler for use in Georgia. The advantages of this profiler have been listed as:

- The measurement sensors are noncontact.
- There are no moving parts in the system to wear.
- The results are much less speed sensitive when compared to the results from other profilers.
- The results are not temperature sensitive.
- The actual profile is measured, not the vehicle's response.
- The ride quality resulting from the measured profile is calculated using standardized methods.
- The results are repeatable and comparable among units.
- The system is computer controlled, and data are stored on a disk.

A study was conducted in North Carolina (Hearne et al., 1996) using the California profilograph and the straight edge. Disturbances closely resembling sine waves, with wave-lengths near 4.6 m and amplitudes of about 5 mm were determined to be primary contributors to poor rideability on two major resurfacing projects in North Carolina. The most commonly used California profilograph has a very poor response to the disturbances with wavelengths in this vicinity. The straight edge detected roughness better during construction when compared to the California profilograph.

Profile Based Smoothness Specification

The Texas Department of Transportation has implemented a smoothness specification based on profilograph testing as a part of its quality control/quality assurance program. This specification for asphalt concrete overlays was developed by TTI (Fernando, 1998) and is based on the current generation of profiling equipment. Researchers evaluated the relationship between the pavement profile and predicted overlay life, assuming reflection cracking as the primary failure mechanism, and developed a relationship between the predicted change in pavement life associated with the placement of the overlay, dynamic load variability, and the fracture parameter, n, of the asphalt overlay mixture. Using this relationship, they developed two categories (A and B) of pavement evaluation. For thin overlays (<63 mm) which have no surface preparations, or where only spot level-ups are used, Group A test methods are alternatives to dropping the smoothness specification or using the straight edge as a check for the surface smoothness. Group B test methods evaluate the contractor's work based on the final surface profile and are applicable on projects where the overlay thickness is greater than 64 mm or in situations where surface preparation is adopted to correct existing surface distresses. The test methods proposed were evaluated using profile data collected from the district overlay projects. The results were found to be generally consistent between the different methods developed in the study.

Effects of Pavement Roughness on Performance

The effect of initial ride quality on the pavement performance was evaluated in the NCHRP 1-31 report. The results of this study indicate that the initial pavement smoothness has a significant effect on the future smoothness of the pavement, in 80 percent of the new constructions and in 70 percent of the overlay constructions. It was also shown that added pavement life could be achieved by improving the initial smoothness of the pavement. Results showed that at least a 9 percent increase in life corresponds to a 25 percent increase in smoothness from target profile index value of 5 in/mi.

Longitudinal Joint Density

Cause

As a result of lack of support in the lateral direction when the first lane is placed, the unconfined edge of the pavement is not compacted well enough and has lower densities when compared to the rest of the mat. For the second lane, the first lane that has already been placed and compacted serves as a confining edge. As a result of this, higher densities may be achieved at the confined edge of the newly paved lane. This higher density leads to a density gradient across the joint. It is found that the density at longitudinal joints is 1-2 percent less than the density measured at places away from the joint (Khandal and Mallick, 1996).

Measurement

Destructive Methods: Cores can be taken from the test sections and the air voids may be computed through maximum specific gravity determinations. The bulk specific gravity and thickness determinations can also be made on the cores. Results may be reported in terms of percent compaction, in-place air voids or density (Killingsworth, 1999).

Non-destructive measurements: Joint density may be determined using the nuclear density gauge (Killingsworth, 1999) and the pavement quality indicator (PQI). However, it must be noted that the nuclear instruments pose a challenge in measuring values on uneven surfaces (Sawchuk, 1997). Killingsworth (1999) points out that there is a need for specifying and determining the in-place air voids at the joint to ensure that proper construction practices were used to achieve the specified density.

Effects of Variations in Longitudinal Density on Performance

As joints are the weakest parts of the pavement structure, longitudinal cracks may form due to the stresses induced by traffic and environmental conditions (Khandhal and Rao, 1994). Improper compaction at the joints will reduce the durability of the pavement structure due to a high air void content, increased potential to raveling, and decreased mixture stability (Killingsworth, 1999). Moisture damage may also take place as a result of accumulated water in the voids and depressions at the joint.

Joint Construction Techniques

Suitable longitudinal joint construction techniques for multilane asphalt pavements can minimize the problems associated with low densities and surface irregularity (Khandal and Mallick, 1996). Longitudinal cracks form as a result of the density gradient encountered across the joint. Such density gradients arise as a result of lack of support in the lateral direction when the first lane is placed. It is found that the density at longitudinal joints may be 1-2 percent less than the density measured at places away from the joint.

Seven different joint construction techniques were adopted on I-25 in Colorado in 1994 (Khandal and Mallick, 1996). These construction techniques included various rolling procedures to compact the joint, provision of a vertical face with a cutting wheel, and the use of rubberized asphalt tack coat on the face of the unconfined edge. Two longitudinal joint construction techniques were used on I-79 in Pennsylvania the same year (Khandal and Mallick, 1996). These techniques included the conventional technique and the New Jersey type wedge joint, which uses a 3:1 taper at the unconfined edge of the first lane, the face of which is heated with an infrared heater before the adjacent lane is placed. A study of these pavements was conducted by Khandal et al. (1996) to evaluate the techniques and rank them. Cores were taken on the joint and at 305 mm away from the joint for density measurements. Teams of at least four engineers also visually inspected these joints in June 1995. The different construction techniques adopted on I-25 and I-79 are shown in Figures 1 and 2 respectively. From this study researchers concluded that of the seven types of LJCT evaluated in the Colorado project, LJCT 6 was the best. This technique had a 3:1 taper with a 25 mm offset. The cold side unconfined edge was constructed with a 25.4 mm vertical step at the top of the joint. The vertical face was not tacked. They also concluded that of the three rolling procedures adopted in the Colorado project, LJCT 3 appeared to be the best. In this rolling technique, compaction was started with the edge of the roller about 152 mm from the joint on the hot side. The lateral pushing of the material toward the joint during the first pass of the roller is believed to produce high density at the joint. The different joint construction techniques are illustrated in Figure A-9.

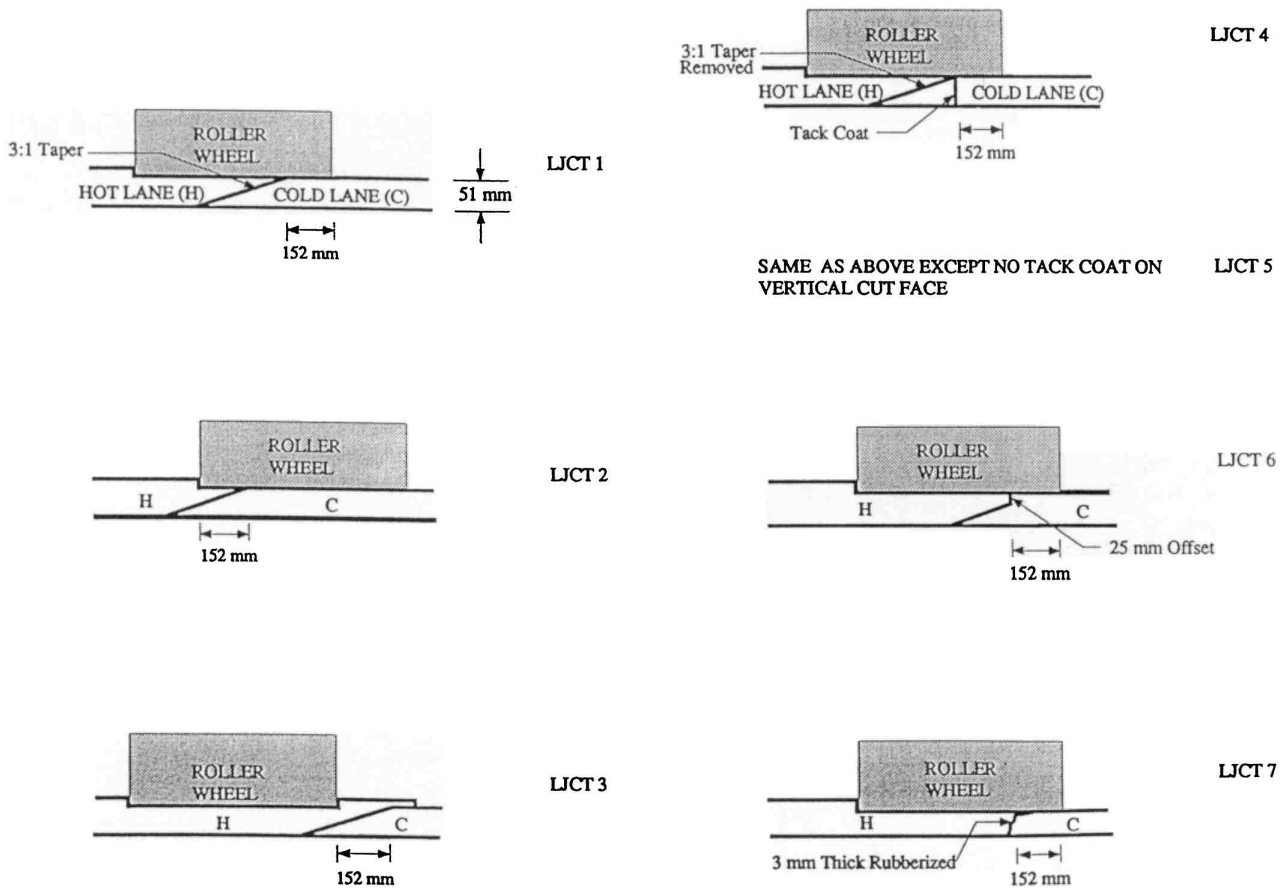


Figure A-9. Joint Construction and Rolling Techniques (Kandhal and Mallick, 1996).

Another study was reported by Kandhal et al. in 1994. This study was sponsored by the National Center for Asphalt Technology. The team evaluated seven joint construction techniques used on a project in Michigan and eight techniques used on a project in Wisconsin. Both the projects adopted a dense graded HMAC wearing course of 38 mm layer thickness. Table A-2 shows the gradation details.

Table A-2. Gradation Details (Kandhal et al., 1994).

Sieve Size in mm.	Percent Passing	
	Michigan	Wisconsin
19	100	100
12.5	100	97
9.5	88	-

The results of the Michigan study indicate that the wedge joint and the cutting wheel techniques give the highest joint densities. The results of the Wisconsin project show that the edge restraining device and the cutting wheel technique give the best results. The wedge joint is constructed by tapering the edge (at a slope of 1:12) of the lane paved first. Attaching a steel plate to the screed of the paver can form the taper. Compaction was done by limiting the roller to a maximum of 2 in. beyond the top of the unconfined edge. In the cutting wheel technique adopted, about 38-51 mm of the low-density edge at the joint of the unconfined lane is removed by cutting it off while the mix is still plastic. The edge-restraining device adopted in the Wisconsin project consists of a hydraulically powered wheel that rolls along the compactor's drum. It pushes the material at the unconfined edge towards the drum as it rolls along.

A paper by Buchanan (2000) presents a study conducted to evaluate the notched wedge joint (NWJ) construction technique. A comparison of the NWJ to conventional joint construction technique was based on results from projects in five states– Colorado, Indiana, Alabama, Wisconsin, and Maryland. Researchers examined the density of cores taken at the centerline and at 150 and 450 mm on either side of the centerline. Results from this study indicate that NWJ can significantly improve the density at the longitudinal joint.

Non-Uniformity in Layer Thickness

Causes

Non-uniformity in layer thickness is another problem that occurs in construction. The profile of the underlying layer has a significant effect on the thickness of the overlying layers. Non-uniform compaction of the HMAC material will result in varied layer thickness.

Measurement of Layer Thickness

Core samples from the pavement structure may be used to determine the thickness of the layer. However, this procedure proves to very expensive and time consuming and non-destructive testing procedures are desirable (McLellan and Hooper, 1978).

NCHRP Project 10-6 investigates the feasibility of non-destructive testing methods to determine pavement thickness (Howkins, 1968). The study considered three main evaluation techniques: acoustic, nuclear, and electrical methods. Based on the findings of the project, three different techniques were recommended. These include:

- Use of large mosaic ultrasonic transducers that can be used on any pavement type with thickness up to 10 in. The estimated accuracy of these transducers is $\pm$ 2 percent.
- Use of a short mechanical impulse source with ultra micrometer detectors. This technique can be applied to any pavement type in the hardened state. The estimated accuracy of this system is $\pm$2 percent.
- Placement of radioactive pellets before paving the road. This technique is applicable to both pavement types and the estimated accuracy level is $\pm$1 percent.

Spectral analysis of surface waves (SASW) may be used in order to determine layer thickness (Roesset et al., 1990). This test can also be used to find the stiffness of the layers. This is a modification of the "steady state rayleigh wave technique," which was introduced for measurement of elastic properties of pavement. SASW may be "used for near-surface profiling of pavement sites." This test involves the use of surface waves in order to evaluate the modulus profile of the entire pavement system. The equipment consists of source (which generates surface waves in the order of 1 to 50 kHz) and recording equipment (includes a waveform analyzer with a microcomputer). The test measures the dispersion of the surface waves in order to determine the surface wave velocity at various wavelengths. From a plot of the surface wave velocity vs. wavelength, the critical wavelength can be determined, which is a measure of the surface layer thickness.

A paper by McLellan et al. (1978) presents the use of a probe, called the permascope, to determine pavement layer thickness. The probe is placed on the surface layer directly above an aluminum foil under the layer. "The reduction in the self-inductance of the measuring coil in the probe is related to the distance between the probe and the coil." The results show that the accuracy of the method is $\pm$2 percent. However, this method yields accurate measurements only if the temperature of the material is less than 60 $^{\circ}$C.

Another study conducted by the above mentioned author examines the feasibility of using a rotating laser source with a level staff fitted with a moving optical receiver that is sensitive to the laser light (McLellan, 1982). The receiver is capable of sliding on the staff until it detects and locks on to the laser plane. Readings can be accurately obtained from the staff. The datum of the laser can be maintained constant for a wide area, and readings can be quickly obtained.

Ground penetrating radar technology has been used as a non-destructive pavement testing procedure to determine pavement thickness. As described below, very accurate predictions of layer thicknesses have been reported for new pavements. Reasonable estimates have been found for existing pavements.

A study was undertaken by Chung et al. (1991) to evaluate the feasibility of using impulse radar to determine the thickness. They used monostatic radar which had an antenna design based on the constant flare angle, variable width open horn. The system generated a monocycle pulse which had a pulse width of approximately 1 ns and a repetition rate of 5 MHz. Data were collected using the Ministry of Transportation-Ontario (MTO) radar-based DART for two pavement sections. The results of this study indicate that the computed thickness based on the radar evaluation correlates well with the actual thickness measurements obtained from the pavement structure.

A study was conducted by Maser and Scullion (1992) to evaluate the influence of asphalt layering and surface treatments on asphalt and base layer thickness computations (of in-service pavements) using radar. They used the Penetradar PS-24 radar system for data acquisition and the PAVLAYER software to analyze the data. They also studied the differences in results obtained by using Penetradar and Pulse Radar R-II systems. Core measurements were taken to determine the accuracy of the thickness values obtained using radar measurements. The results of this study indicate that it is possible to accurately determine overlay thicknesses as low as 1 inch. This ability to determine overlay thickness can be attributed to the differences in the dielectrics of the two layers. The overall accuracy level is ± 5 percent to ± 7.5 percent. A comparison of the two radar systems shows that the reflections from the base-subgrade interface are weaker for the PS-24 system, but high accuracy may be obtained while determining the overlay thickness.

A study was sponsored by Missouri Department of Transportation (MoDOT) to determine the accuracy of thickness measurements of new pavements, obtained using GPR (Wenzlick et al., 1999). This study includes measurements on both asphalt concrete (AC) and portland cement concrete pavements (PCC). Two AC pavements were evaluated, the thicknesses of which were 12 inches and 17 inches. The accuracy of GPR measurements on the 12-inch and 17-inch AC pavements was found to be 0.17 inches or 1.4 percent (30 cores) and 0.2 inches or 1.1 percent (49 cores), respectively. However, the GPR measurements on the 14-inch PCC pavement studied indicates that the accuracy of GPR is 0.39 inch or 2.8 percent (70 cores).

Mesher et al. (1995) evaluated a new GPR technology (Road Radar) to quantify pavements. The Road Radar uses "multiple antennas that provide accurate non-intrusive thickness measurements of multiple layers from 50 mm to greater than 2000 mm without the benefit of any destructive calibration procedures." The Road Radar unit is a self-calibrating unit with a rapid processing computer software. Several pavement sections were evaluated using the Road Radar, and the results indicate that there is excellent correlation between the core data and the radar data collected.

Table A-3 is taken from the NCHRP 9-15 (Killingsworth, 1999) report. It shows the accuracy levels of various studies conducted by different agencies.

Effects of Non-Uniform Layer Thickness on Pavement Performance

Non-uniform layer thickness may affect the surface profile and may result in insufficient layer thickness, as a result of which the pavement structure may not be structurally capable of catering to the traffic loads (Killingsworth, 1999). The ride quality is affected by any non-uniformity in the layer thickness (McLellan, 1982).

Table A-3. Accuracy Evaluation Studies Using GPR (Killingsworth, 1999).

Agency	Number of Sections			Number of Cores or Test Pits	Average Deviation (%)
	AC[a]	PCC[a]	AC/PCC[a]		
TxDOT[c]	12	1	--	90	5
Kansas DOT	11	--	3	73	7
Florida DOT	20	1	5	150	10
Washington DOT	1	1	1	5	8
Wyoming DOT	9	--	--	36	10
Mn/ROAD	15	10	--	74	5
USA-SHRP	10	--	--	68	7
US Air Force	6	6	1	13	6
US FHWA	--	2	2	10	5
Pforzheim (Germany)	26	--	--	35	8
Kent (UK)	5	--	--	76	5
TRL (UK)	3	1	--	115	6
Thüringen (Germany)	9	--	--	28	10
TOTALS	**127**	**22**	**12**	**773**	**7.07 (Mean)**

[a] AC = asphalt concrete; PCC = portland cement concrete; AC/PCC = AC over PCC

[c] DOT = Department of Transportation

Note: The term “Average Deviation (%)” indicates the percent deviation of the thickness computed by radar from the thickness obtained using cores.

REVIEW OF ON –GOING AND RECENTLY COMPLETED STUDIES THAT RELATE TO PROJECT 0-1708

The main task of the first phase of TxDOT Project 0-1708 was to conduct a detailed review of the on-going related studies at the state and federal level. The research team has identified six major projects, the findings of which will prove to be a guiding tool for future phases of Project 0-1708. The six major projects identified include:

- NCHRP Project 9-15 - Quality Characteristics and Test Methods for Use in Performance-Related Specifications of Hot-Mix Asphalt Pavements;
- FHWA DTFH61-94-C-00004 - Performance Related Specification;
- NCHRP Project 1-37 - AASHTO 2002 Design;
- NCHRP Project 9-11 - Segregation in Hot-Mix Asphalt Pavements;
- demonstration project conducted by TxDOT – Material Transfer Device Showcase in El Paso, Texas;
- NCHRP-96-ID032 – Testing and Trial Development of a Cost Effective and Real-Time Asphalt Pavement Quality Indicator System;
- FHWA-RD-91-070 - Performance-Related Specifications for Asphalt Concrete-Phase II; and
- NCHRP Project 332 - Framework for Development of Performance-Related Specifications for Hot-Mix Asphaltic Concrete.

A summary of each of the above-mentioned projects is presented below.

NCHRP 9-15

This project, which is entitled "Quality Characteristics and Test Methods for Use in Performance-Related Specifications of Hot-Mix Asphalt Pavements," was carried out with the following objectives:

- identify construction related quality characteristics of HMA pavements which affect the long term pavement performance;
- identify quality characteristics of as-mixed HMA that reflect compositional, volumetric, and fundamental engineering properties in terms of long term performance; and

- identify tests to measure the quality characteristics so that they may be used in performance related specifications.

The team began by reviewing available literature to identify properties that were currently considered to have potential influence on performance. The performance indicators identified are classified into three categories and they include:

- distress related indicators, which include rutting and cracking;
- durability related indicators such as moisture-induced damage, flushing/bleeding, and raveling; and
- functional indicators such as smoothness and skid resistance.

Some of the test devices identified from the literature search include:

- density gauges - nuclear, electric, continuous;
- smoothness equipment - profilograph, profilometers, dipstick;
- GPR;
- geosonar;
- stiffness gauge;
- thermal imaging;
- permeameter;
- laser surface texture;
- skid tester; and
- NDT - deflections(FWD, Dynaflect, Road Rater), other (SASW, impact echo, impulse response, wave propagation, acoustic, ultrasonic).

The team then reviewed several performance models for the performance indicators identified above. It found that most of the models required fundamental properties as input parameters. Models that relate material and construction variables to the fundamental variables may be obtained from the FHWA contract entitled "Performance Related Specifications for Asphalt Concrete - Phase II." The models identified by the team are shown in Table A-4 taken from pages 32, 33, and 34 of the report.

Table A-4. Summary of Performance Relationships with Independent Variables from Literature Review (Killingsworth, 1999).

Distress	Performance Model[1]	Classification	Independent Variables Related to HMA	Variation Considered	Comments
Permanent Deformation (Rutting)	VESYS Rut Depth (Kenis, 1983)	M-E	AC Mix Modulus AC Mix Poisson's Ratio $\propto$, μ of AC Mix	Yes, probability theory for input parameters	
	Superpave (Lytton, et al., 1993)	M	AC Mix Modulus AC Mix Poisson's Ratio Vermeer Plasticity Parameters Slope of Permanent Strain vs. Cycles Curve	Partially Considered	Inputs are determined @ various test temperatures. Slope determined from compliance curve.
	Shell (Van der Loo, 1978)	M-E	AC Mix Modulus AC Mix Poisson's Ratio Bitumen Viscosity Penetration and Penetration Index Stiffness Modulus of Mix	No	
	COSTOP1 (Von Quintus et al., 1984)	M-E	Elastic Modulus of AC Mix	Yes, Taylor series expansion of deterministic model.	Other inputs include equivalent pavement thickness and E subgrade.
	AAMAS NCHRP 338 (Von Quintus et al., 1991)	M-E	Unconfined Compressive Strength Resilient Modulus Static Creep Modulus GTM Shear Stress and Strains	No	Test specimens are conditioned.
	FAA Design Procedure (Baker et al., 1975)	M-E	Modulus of Elasticity or Dynamic Modulus Poisson's Ratio	No	Inputs are determined at various test temperatures.
	DAMA (Asphalt Institute)	M-E	Modulus of Elasticity Poisson's Ratio	No	Considers temperature dependency of AC Mix.
Fatigue Cracking	VESYS Cracking Model (Kenis, 1983)	M-E	AC Mix Fatigue Properties (K_1, K_2) AC Mix Modulus AC Poisson's Ratio	Yes, probability theory for input parameters	
	Superpave (Lytton, et al., 1993)	M	Fracture Properties A and η AC Mix Modulus AC Mix Poisson's Ratio Tensile Strength Cycles to Crack Initiation Slope of Permanent Strain vs. Cycles	Partially considered	Inputs are determined at various test temperatures. Slope determined from compliance curves Other empirical relationships included to estimate primary relationships.

Table A-4. Summary of Performance Relationships with Independent Variables from Literature Review (Killingsworth, 1999), continued.

Distress	Performance Model[1]	Classification	Independent Variables Related to HMA	Variation Considered	Comments
Fatigue Cracking (Continued)	ARE	M-E	AC Mix Modulus AC Mix Poisson's Ratio	?	
	Asphalt Institute	M-E	AC Mix Modulus Binder Content Air Voids AC Mix Poisson's Ratio	?	
	COSTOP1 (Von Quintus et al., 1984) Utilizing WATMODE Model (Meyer et al., 1977)	E	Elastic Modulus of AC Mix	Yes, Taylor series expansion of deterministic model	Other inputs required: E of base and subgrade; output is radial tensile strain at bottom of AC.
	AAMAS-NCHRP 338 (Von Quintus et al., 1991)	M-E	Resilient Modulus Indirect Tensile Strength Static Creep Modulus	No	Tests completed at various temperatures and on both conditioned and unconditioned specimens.
	KENLAYER (Huang, 1993)	M-E	Elastic Modulus of AC Mix Creep Compliance Temperature-Shift Factor Poisson's Ratio	No	Nonlinear elastic and linear viscoelastic cases may be considered.
Thermal Cracking (Low Temp.)	VESYS (Kenis, 1983)	M-E	AC Mix Modulus AC Mix Poisson's Ratio Others?	Yes	
	Superpave (Lytton, et al., 1993)	M	Creep Compliance (Indirect Tension) Tensile Strength at Low Temperatures Relaxation Modulus Parameters for Pavement Temperature calculation	Partially Considered	Indirect tensile tests at low temperatures utilized to obtain input parameters.
	COLD (Christison, 1972)	M-E	Modulus-Temperature Relationship Tensile Strength-Temperature Relationship Thermal Conductivity Heat Capacity Absorptivity/Emissivity Convection Coefficient	?	

Table A-4. Summary of Performance Relationships with Independent Variables from Literature Review (Killingsworth, 1999), continued.

Distress	Performance Model[1]	Classification	Independent Variables Related to HMA	Variation Considered	Comments
Thermal Cracking (Low Temp.) (continued)	TC-1 (Shahin, 1972)	M-E	Air Voids Binder Content Volume Concentration of Aggregates Specific Gravity of Asphalt Specific Gravity of Aggregate Coefficient of Thermal Expansion Penetration Index Softening Temperature Absorptivity/Conductivity of AC Mix	?	
	AAMAS-NCHRP 338 (Von Quintus et al., 1991)	M-E	Resilient Modulus Indirect Tensile Strength Static Creep Modulus	No	
	OPAC (MTO, 1990)	M-E	Elastic Modulus Poisson's Ratio		Odemark subgrade surface used from elastic layer formulation of pavement structure.
Smoothness (Roughness or Serviceability)	VESYS Roughness (Kenis, 1983)	M-E	AC Mix Modulus Fatigue Properties (K_1, K_2) Permanent Deformation Properties ($\propto$, μ) Poisson's Ratio	Yes	Output is loss in PSI.
	COSTOP1 (Von Quintus et al., 1984) (Utilizing VESYS IV-B and WATMODE)	M-E	Elastic Modulus	Yes	Conglomeration of various models utilized to determine loss of serviceability.
	AASHTO (1986, 1993)	M-E	AC Mix Modulus	Yes, reliability	Model predicts loss in serviceability.
	LTPP P-020 (Simpson et al., 1994)	E	Asphalt Viscosity Air Voids	No	Many other variables that do not include AC mixture properties. Output is) IRI.
Skid Resistance	COSTOP1 (Von Quintus, et al., 1984)	E	HMA Aggregate LA Abrasion HMA Aggregate Moh's Hardness	Yes	Unvalidated empirical relationships predicting skid number.
Raveling (Disintegration)	AAMAS-NCHRP 338 (Von Quintus et al., 1991)	M-E	Indirect Tensile Strains	No	Bonding loss is determined from conditioned and unconditioned specimens.

The team also identified the regression models that were developed in the FHWA-RD-91-070 report. These models are shown in a later part (Performance-Related Studies (PRS)) of this appendix.

As quality control tests are meant to provide timely results to the contractor, such tests should be non-destructive in nature. However acceptance testing may be destructive in nature so that accurate results may be obtained. The project does not give any priority to the measurement of in-situ properties over the as-produced properties, even though the team members recognize the fact that in-situ properties are more beneficial to a PRS. They also pointed out that the design and construction specifications have so far been treated independently with little or no interaction between the processes. As a result of this, the criteria for acceptance for the design and construction are not the same. In order to integrate the design and construction processes, the team decided to evaluate those quality characteristics that would support the models developed for the AASHTO 2002 Pavement Design Guide. A list of the quality characteristics chosen for further evaluation is shown in Table A-5 taken from page 61 of the report. The following factors will be considered while evaluating the quality characteristics.

- Initial Ride Quality - underlying layer conditions, distance of measurement, and the device type.
- Segregation - mixture type, plant type, and construction operations.
- In-Place Air Voids - type of finishing roller, lift thickness, and device type.
- Longitudinal Joint Air Voids - joint construction method, type of finishing roller, and device type.
- Permeability - mixture type, maximum specified in-place density, and underlying layer condition.
- In-Place Stiffness with Non-Destructive Testing – mixture type, lift thickness, underlying layer condition, and surface temperature.
- Thickness – underlying layer condition and age, lift thickness, and device type.
- Dynamic Modulus of HMA (lab testing and use of Witczak’s predictive equation) - aggregate type, asphalt content, and nominal maximum aggregate size.
- Low Temperature Tensile Strength – asphalt grade, aggregate type, and air void content.

Table A-5. Quality Characteristics Selected for Further Evaluation in Phase II (Killingsworth, 1999).

Performance Category	Performance Indicator	Quality Characteristic	Test Device/Method
Functional	Smoothness	Initial Ride Quality	Full-Size Profilometer (IRI) Lightweight Profilometer (IRI) Walking Dipstick (IRI)
Durability	Raveling	Segregation: Temperature Diff. Surface Texture	Thermal Imaging Laser-Based Surface Texture Measurement
		In-Place Air Voids	Density Gauges (Nuclear, PQI, Rolling) Maximum Theoretical Specific Gravity
		Asphalt Content	Ignition Oven
	Moisture Induced Damage	Permeability k-Value	Field Permeameter
	Longitudinal Joint Deterioration	Longitudinal Joint Air Voids	Density Gauges (Nuclear, PQI, Rolling) Maximum Theoretical Specific Gravity
Structural	Fatigue Cracking and Permanent Deformation	In-Place Stiffness	Nondestructive Tests (FWD, SPA)
		Dynamic Modulus	Field Dynamic Modulus Test System Predicted Dynamic Modulus (using mixture properties)
		Lift Thickness	Ground Penetrating Radar
		Effective Asphalt Content	Ignition Oven Absorbed Asphalt
		Aggregate Gradation	Video Grading (including post-ignition gradation) Mechanical Sieving
		Asphalt Binder Viscosity	Dynamic Shear Rheometer
		In-Place Air Voids	See Above
	Thermal Cracking	Low Temp. Tensile Strength	Indirect Tensile Strength
		Fracture Temperature	Field Adapted Thermal Restrained Test

- Fracture Temperature from Restrained Tensile Tests – asphalt grade, aggregate type, and air void content.

FHWA DTFH61-94-C-00004 (WES TRACK PROJECT)

The main objectives of this project entitled “Accelerated Field Test of Performance-Related Specifications for Hot-Mix Asphalt Construction" are:

- to continue the development of performance-related specifications (PRS) for hot-mix asphalt construction by evaluating the impact on performance of deviations in materials and construction properties (e.g., asphalt content, air void content, and aggregate gradation) from design values in a large scale, accelerated field test; and
- to provide early field verification of the SHRP SUPERPAVE Level III mix design procedures.

In order to accomplish the above-mentioned objectives, 26 experimental sections were built on a stretch of 2.9 km (1.8 mi) oval track. Four triple-trailer combination units were operated on these tracks for a period of two years, and the performance of the track was monitored over this period.

The experimental design of the project includes asphalt content (three levels), aggregate gradation (fine, fine +, and coarse), and air voids (three levels). There was only one binder used in the experiment. The layer thicknesses of the sections are:

- 150 mm AC,
- 150 mm base, and
- 300 mm engineering fill subbase.

Table A-6 shows the combinations considered for the design.

Table A-6. Factorial Experiment Design *

Original 1995 Construction										1997 Rehabilitation		
Design	Aggregate Gradation Design											
Air Void	Fine			Fine Plus			Coarse			Coarse		
Content	Design Asphalt Contents (%)											
%	4.7	5.4	6.1	4.7	5.4	6.1	5.0	5.7	6.4	5.1	5.8	6.5
4		4	18		12	21/9		23	25		39	55
8	2	1/15	14	22	19/11	13	8	5/24	7	38	35/54	37
12	3/16	17		10	20		26	6		56	36	

*Numbers shown in each cell represent actual test section numbers

The research team has developed a performance-related specification for pavement construction using HMA. This system is designed as a Windows-based software package called “HMA Spec.” The program has two main components: pre-construction output and post-construction assessment. The pre-construction output generates a performance-related specification, and the post-construction assessment determines an appropriate pay factor. The PRS system components are designed in a modular fashion and are listed below:

- performance prediction,
- life-cycle cost (LCC),
- maintenance and rehabilitation (M&R),
- sampling and testing,
- overlay design,
- pay adjustment, and
- variability.

Factors that are considered in the PRS program from WesTrack include:

- thickness,
- initial smoothness,
- air voids,
- asphalt content, and
- gradation.

There are two levels of performance models developed for this project – level 1 and level 2. The level 1 models are regression models and the level 2 models are mechanistic-empirical (ME) models. The classification of the models developed and their applicability is illustrated in Figure A-10 taken from the draft report.

Laboratory tests and field tests were carried out to determine the elastic moduli of the pavement layers. Field tests were carried out using the FWD. Lab tests include:

- flexural fatigue tests, repeated load simple shear test at constant height (RSST-CH), and indirect tension tests for asphalt concrete; and
- triaxial compression tests and resilient modulus tests for the untreated base and foundation soil.

Figures A-11, A-12 and Table A-7 indicate how the field-tested and lab determined moduli for the HMA and underlying layers compare. The figures are based on data presented in the draft report from WesTrack.

The tests carried out to develop models for rutting include:

- RSST-CH on field-mixed, field-compacted (FMFC) specimens that were tested prior to the application of loads and also at the conclusion of application of loads; and
- RSST-CH on lab-mixed, lab-compacted (LMLC) specimens.

The models developed for rutting in the different mixes are of the general form:

Level 1

$\ln(rd)=a_0+a_1.P_{asp}+a_2V_{air}+a_3.P_{asp}^2+a_4.V_{air}^2+a_5.P_{200}+a_6.fa+a_7.\ln ESALs+a_8.T\ldots..+$(interaction terms amongst the variables)…+(indicator variables representing the three aggregate gradings used at WesTrack)…+(indicator variable for aggregate type, replacement sections)

where,

rd = rut depth in in. or mm.

P_{asp} = asphalt content, %

V_{air} = air void content, %

ESALs = number of 18,000 equivalent single axle loads

P_{200} = percent aggregate finer than 0.074 mm sieve

fa = percent aggregate passing the 2.36 mm sieve and retained on the .074 mm sieve

T = a measure of temperature

$a_0\ldots..a_n$ = regression constants

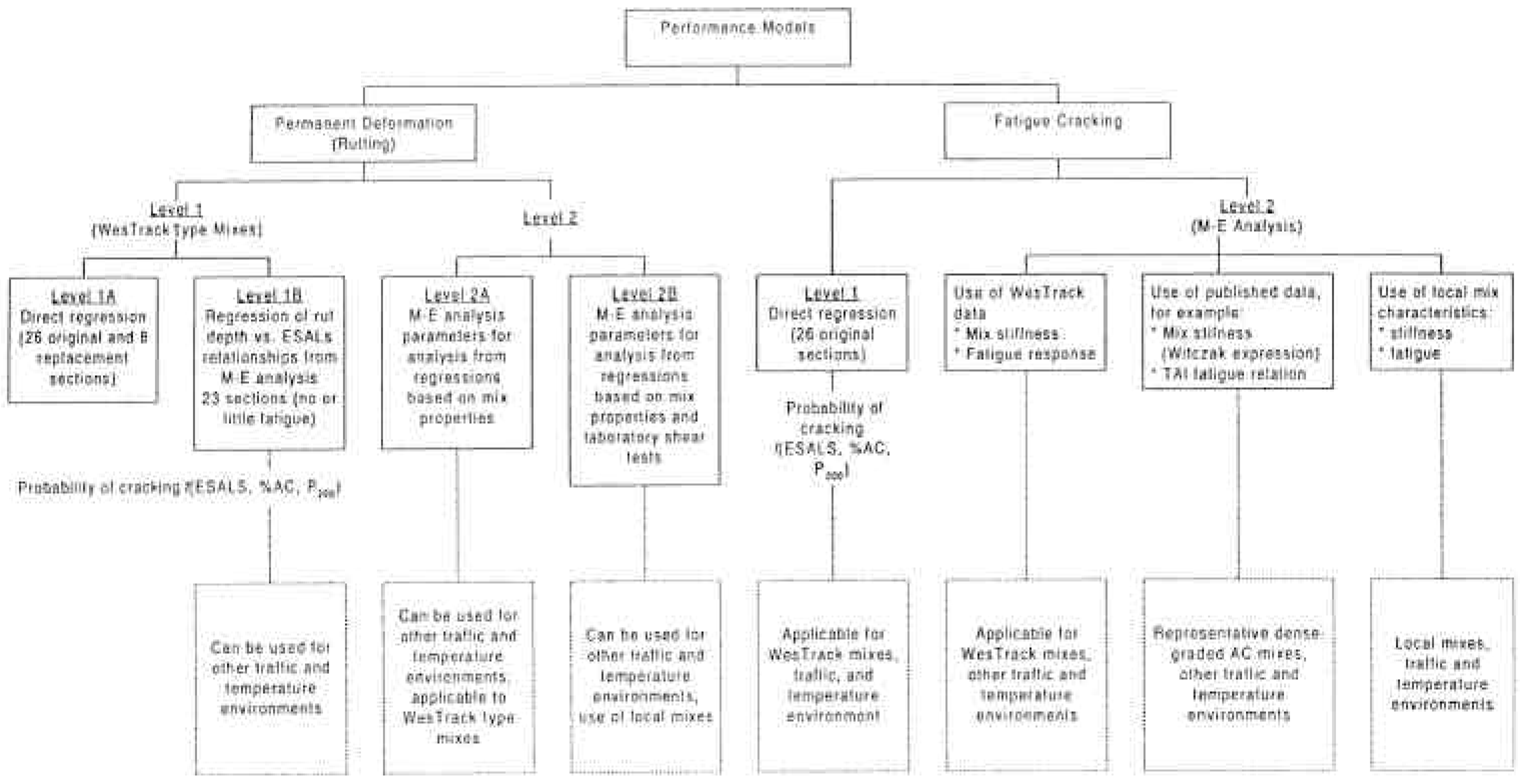


Figure A-10. Performance Model Framework (FHWA DTFH61-94-C-00004, 2000).

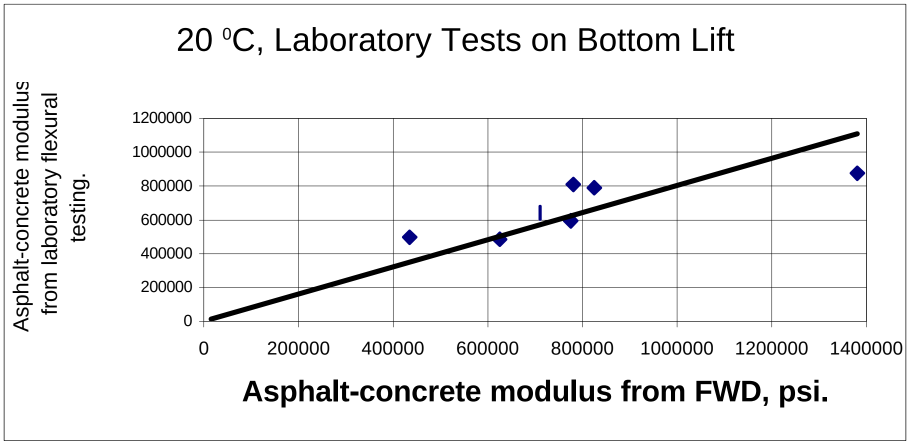


Figure A-11. Comparison of Laboratory Flexural Stiffness Values at 20 °C vs. Moduli Determined From FWD Measurements (FHWA DTFH61-94-C-00004).

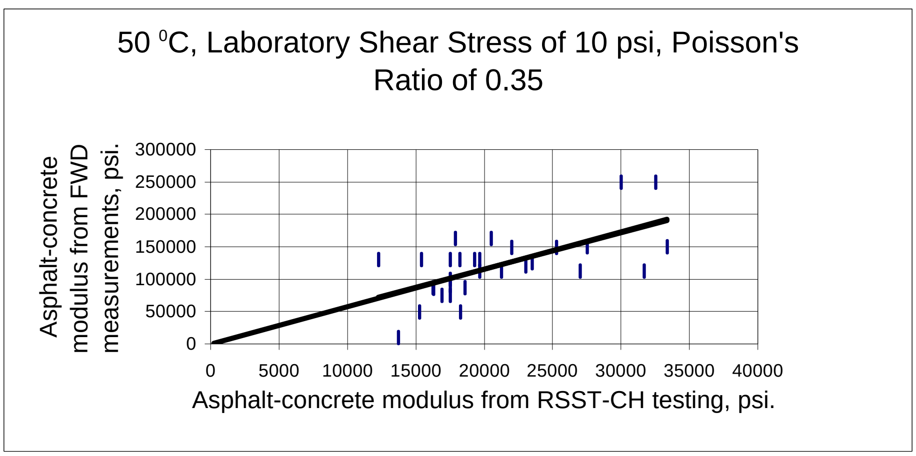


Figure A-12. Comparison of Stiffness Values from RSST-CH Tests at 50 °C and FWD Measurement (FHWA DTFH61-94-C-00004).

Table A-7. Comparison of Laboratory and FWD Moduli (psi) for Base and Foundation Soils (FHWA DTFH61-94-C-00004).

Layer	South Tangent		North Tangent	
	Laboratory	FWD	Laboratory	FWD
Base	13,000	15,100	12,100	13,500
Engineered fill, top	14,100		6,800	
Engineered fill, bottom	20,000	16,700	20,400	21,000
Foundation soil	11,000		16,800	

Level 2A

$$rd_{ac} = K(\gamma_j^i)$$

where,

rd_{ac} = rutting due to shear deformation

$(^i = a \exp(bJ)\ (^e n^c$

K = 5.5 for a 150 mm. (6in.) layer

a, b, and c are constants

n = number of load repetitions;

J = repeated shear stress;

$(_e$ = is the elastic shear strain

Models developed for fatigue cracking, include:

- Level 1

Probit Model-Developed for crack initiation. This model was selected as it permits the use of performance data collected from all the 26 sections. It should be noted that the probit model was not adopted in the HMA Spec software as it did not provide a direct measure of the cracking.

Fine and fine plus mixes

$$Prob(INCR=1)=\Phi(-49.502+4.788.\ln(ESALs)-5.245.P_{asp}+1.148.V_{air}-2.301P_{200})$$

Coarse mixes

$$Prob(INCR=1)=\Phi(-47.151+5.293.\ln(ESALs)-5.996.\ P_{asp}+0.45.\ V_{ai})$$

where,

INCR=indication of cracking, equal to 1 when cracking is observed and zero otherwise.

Φ is the cumulative density function of the normal distribution.

Composite Model-Developed for crack propagation. This model was adopted in the HMA Spec software.

Fine-graded mixes

$$FC(\%)=[1.2313+0.071655*\log(W18)+0.2358*\log(\varepsilon)+0.061193*\log(E^*)-0.034086*AC+0.0074593*AV-0.014954*P_{200}]^{154.04}$$

Coarse-graded mixes

$$FC(\%)=[1.2850+0.07478*\log(W18)+0.2461*\log(\varepsilon)+0.06386*\log(E^*)-0.036791*AC+0.002761*AV]^{147.73}$$

- Level 2

The following models have been developed based on laboratory fatigue tests on FMFC mixes.

Fine mixes

$$\ln N_f=-27.0265-0.1439V_{air}+0.4148P_{asp}-4.6894\ln(\varepsilon_t)$$

Fine-plus mixes

$$\ln N_f=-27.3409-0.1431V_{air}+0.4219P_{asp}+0.0128\ln T-4.6918\ln(\varepsilon_t)$$

Coarse mixes

$$\ln N_f = -27.6723 - 0.0941V_{air} + 0.6540P_{asp} + 0.0331\ln T - 4.5402\ln(\varepsilon_t)$$

where,

N_f = fatigue life

V_{air} = air voids, %

P_{asp} = asphalt content, %

T = temperature at 150 mm, 0 °C

ε_t = maximum tensile strain

Low temperature cracking was observed at WesTrack even though the lowest temperature recorded is –10 ^{0}C. However, the fracture temperatures from thermal stress restrained specimen test (TSRST) of the specimens tested were all less than –10 ^{0}C. They found better correlation of observed low temperature cracking with the predictions from the COLD program which stands for Computation of Low Temperature Damage. Evaluation of low temperature cracking was made using data from Mn/Road, WesTrack, and Hybrid pavements, i.e., WesTrack binder in Mn/Road environment.

The inputs to the Life Cycle Costs (LCC) model include:

- materials and construction factors, which include- %air voids, %asphalt, thickness, initial smoothness, and P_{200};
- environmental factors, which include only pavement temperature at present;
- traffic;
- base course and road bed characteristics;
- actual cost for the overlay or mill and fill treatments; and
- factors that account for the time value of money.

PRS software uses Monte Carlo simulation to generate LCC and consider variability. It must be noted that only factors that can be controlled by the contractor can be varied during the Monte Carlo simulation. It uses Witczak's equation to get the modulus of the asphalt concrete mix.

Table A-8 shows the guide specification for HMA tests.

Table A-8. Hot-Mix Asphalt Tests (FHWA DTFH61-94-C-00004).

Test Designation	Test Method Number	
	AASHTO	ASTM
Bulk specific gravity of compacted HMAC-SSD method	T166	D2726
Bulk specific gravity of compacted HMA-parafilm	T275	
Bulk specific gravity of compacted HMA-parafilm		D1188
Percent air voids of compacted HMA	T269	D3203
Theoretic max specific gravity of HMA	T209	D2041
Superpave volumetric mix design (Spec)	MP2	
Superpave volumetric mix design of HMA	PP28	
Mixture conditioning of HMA	PP2	
SHRP gyratory compactor	TP4	
Sampling HMA	T168	D979
Sampling compacted HMA Resistance of HMA to moisture damage	T283	D5361 D4867
Thickness of compacted HMA		D3549
Nuclear density		D2950
Asphalt content by nuclear method	T287	D4125
Asphalt content by solvent extractor	T164	D2172
Asphalt content by ignition method		D6307
Marshall and Hveem mixture design	R12	
Marshall stability	T245	D1559
Hveem stability	T246	D1560
California kneading compactor	T247	D1561

AASHTO 2002 DESIGN GUIDE

The proposed design procedure for AASHTO 2002 guide is based on the hierarchical approach concept. The design procedure involves the development of the master curve using time-temperature superposition principle. The dynamic moduli values may either be obtained by actual testing or from Witczak's predictive equation shown below.

$$\log E = -1.249937 + 0.29232\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\rho_4 - 0.058097V_a$$
$$-0.802208\left(\frac{V_{beff}}{V_{beff} + V_a}\right) + \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.005470\rho_{34}}{1 + e^{(-0.6033\grave{}3 - 0.313351\log(f) - 0.393532\log(\eta))}}$$

where,

E = dynamic modulus, 10^5 psi

η = bitumen viscosity, 10^6 Poise

f = loading frequency, Hz

V_a = air void content, percent

V_{beff} = effective bitumen content, percent by volume

ρ_{34} = cumulative percent retained on the 19 mm sieve

ρ_{38} = cumulative percent retained on the 9.5 mm sieve

ρ_4 = cumulative percent retained on the 4.76 mm sieve

ρ_{200} = percent passing the 0.075 mm sieve

The statistics for this equation are shown in Table A-9, which is taken from page 2-45 of the report.

Table A-9. Summary Statistics for the Witczak Dynamic Modulus Prediction Equation (AASHTO, 2000).

Statistic	Value
Goodness of fit	$R^2 = 0.96$ Se/Sy = 0.24
Data points	2750
Temperature range	0 to 130 ^{o}F
Loading rates	0.1 to 25 Hz
Mixtures	205 total 171 with unmodified asphalt binders 34 with modified binders
Binders	23 total 9 unmodified 14 modified
Aggregates	39
Compaction methods	Kneading and gyratory
Specimen sizes	Cylindrical 4 in by 8 in or 2.75 in by 5.5 in

The design process may be classified into three different levels of reliability which are listed below.

- Level 1–This approach has high reliability and is appropriate for analysis of special problems. The design involves extensive testing, which includes the binder tests for G* and δ and mixture tests for dynamic modulus. The rolling thin film oven test (RTFOT) is used for short term aging of the binder. Binder tests include:
  1. penetration tests at 15 °C and 25 °C;
  2. brookfield viscosity test at 60, 80, 100, 121.1 ,135, 176 ^{0}C;
  3. softening point;
  4. absolute viscosity at 60 °C;
  5. kinematic viscosity at 135 °C; and
  6. dynamic shear rheometer test to obtain G* and the phase angle δ, at temperatures of 40,55, 70, 85, 100, 115, and 130 °F and loading rate of 10 rad/sec.

The mixture testing includes dynamic modulus frequency sweep tests for five temperatures and four rates of loading. Test temperatures are 10, 40, 70, 100, and 130 °F. Test frequency includes 0.1, 1, 10, 25 Hz.

Results from the penetration test and the dynamic shear rheometer test are converted to viscosity values using the relationships shown below.

$$\log \eta = 10.5012 - 2.2601\log(Pen) + 0.0389\log(Pen)^2$$

$$\eta = G^*/10\ (1/\sin \delta)^{4.8628}$$

where,

η = viscosity, cP

G^* = binder complex shear modulus

δ = binder phase angle

The relationship between binder viscosity and temperature is established using the following relationship.

$$\log\log\eta = A + VTS\log T_R$$

where,

η = viscosity, cP

A = regression parameters

T_R = temperature in Rankine

The parameters A and VTS are found by linear regression. The lab test data is then shifted to form a smooth master curve using the following relation.

$$\log E^* = \delta + \left(\frac{\alpha}{\left(1 + e^{\beta + \gamma\log(t) - c(\log(\eta) - \log(\eta Tr))}\right)}\right)$$

where,

E^* = dynamic modulus

t = time of loading

η = viscosity at temperature of interest

η_{Tr} = viscosity at reference temperature

$\alpha,\beta,\delta,\gamma,c$ = fitting parameters

- Level 2–This approach is appropriate for most of the cases and has medium reliability. Tests are conducted only for the binder and the mixture modulus is determined using Witczak's predictive equation. The binder tests carried out are the same as those for Level 1.
- Level 3–This process has a low reliability level and is applicable to lower risk projects where testing is not involved. Witczak's dynamic modulus equation is used with typical temperature-viscosity relationships established for all binder grades specified in AASHTO MP1. The actual mixture data that are required as input to the dynamic modulus equation include binder viscosity and loading rate. The other inputs can be obtained from representative data for similar mixtures. Typical temperature-viscosity relationships have been established for 38 binder grades that are included in the AASHTO MP1. The viscosity at any temperature can be calculated using the equation shown below.

$$\log \log \eta = A + VTS \log T_R$$

where,

η = viscosity, cP

A = regression parameters

T_R = temperature in Rankine

The master curve can be developed using Witczak's dynamic modulus equation, and an appropriate temperature-viscosity relationship developed.

The analysis is based on linear elastic theory and YULEA, which is a multi layer elastic theory (MLET) software, and will be used for all general analyses. However, the finite element approach may be used in cases of non-linear materials and special gear configurations. The use of DSC2D, which is a two-dimensional finite element program, is recommended for the adoption in the guide.

One of the major issues that concerned the team was the adoption of dynamic modulus as a primary test protocol to characterize the modulus response of the mixtures. As a comparison of dynamic modulus (E*) to resilient modulus (Mr), Dr. Witczak has pointed out the following :

- The complex modulus consists of two parts: the real part, which represents the dynamic modulus and the imaginary part that represents the phase angle, which is an indicator of the elastic-viscous properties of the mix.
- The phase angle may be an important input to two potential candidate saphalt concrete (AC) fatigue relationships that are being considered for use in the design guide.
- The existence of a reliable predictive equation for the dynamic modulus has made the hierarchical approach for the design guide feasible.
- A working predictive system for the aged in-place E* value already exists for direct utilization in the 2002 guide.
- The AASHTO 2002 guide will be 100 percent compatible with the performance grading (PG) based binder specifications and test parameters being used in the future Superpave implementation system.
- Since both the time of load and the temperature in the E* test are treated in a full factorial mode, the results can be analyzed through a master curve. This treatment would not be possible with Mr testing.
- The Mr vs. the AASHTO layer coefficient (a_i) relationship in the AASHTO 2002 guide is incompatible with theory, and the relation given below agrees much better with the present day stress distribution theories.

$$a_i = a_s * \left(\frac{\left(E_i * \left(1 - u_s^{\,2}\right)\right)}{E_s * \left(1 - u_i^{\,2}\right)} \right)^{\frac{1}{3}}$$

  The subscript "i," denotes the material in question, and "s" denotes an arbitrary standard material having known modulus and layer coefficient.
- The SHRP experts who analyzed the diametral technology agreed that, due to localized failures at loading plates at high temperatures, Mr computations may be highly inaccurate.
- Theoretically the complex modulus in compression and in tension should be the same. However, this is not true in practice, and the current state of art, which uses elastic analysis, is better served by the utilization of compression modulus rather than the tensile parameter. Hence, the use of compressive E* value in the mechanistic models is a better choice when compared to the use of a tensile Mr value.

The team was also concerned whether the modulus values obtained by the E* test and those obtained by the back-calculation of FWD data were comparable. It was pointed out by one of the research team members that "the E* test is fundamentally and theoretically more comparable and compatible to the FWD back-calculated moduli of AC mixtures than the Mr test." Table A-10 shows how the Mr values correlate with the FWD data. The information shown is from the TRB Workshop on the 2002 Guide for Mechanistic Pavement Design held during the 79th Annual TRB Conference in January 2000.

Table A-10. Correlation between Mr and FWD Data.

Temperature(^{o}F)	Mr/E(FWD)
41	1.00
77	0.36
104	0.25

A histogram showing the correlation between dynamic modulus and FWD data is shown in Figure A-13. Figure A-14 shows how the ratio between FWD modulus and Witczak's predicted modulus varies with temperature while Figure A-15 shows the variation of the moduli values with temperature. These figures were presented at the AASHTO 2002 Workshop noted previously.

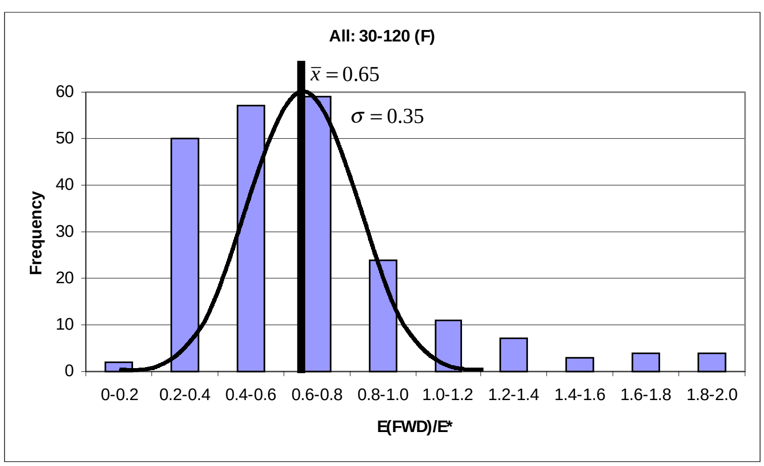


Figure A-13. Frequency Distribution of E(FWD)/E* Ratios.

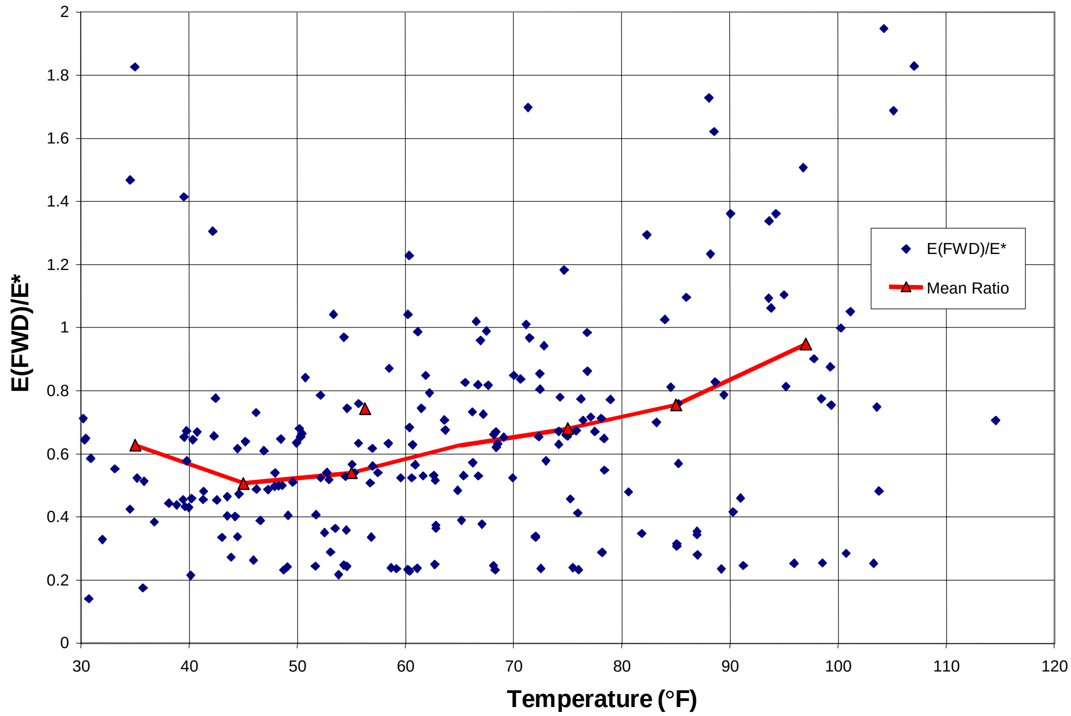


Figure A-14. Ratio of FWD Modulus to Witczak et al. Predicted E* Modulus Versus AC Pavement Temperature.

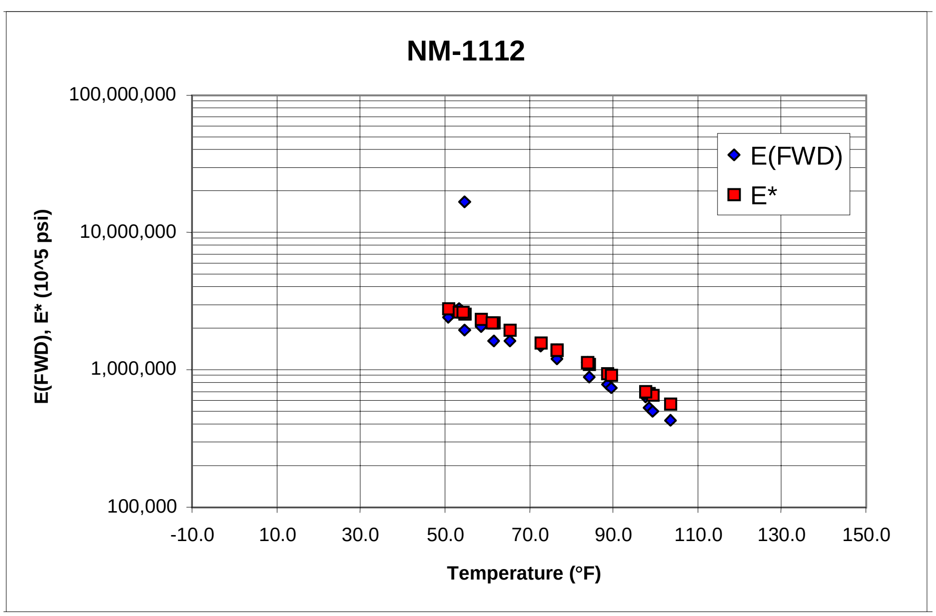


Figure A-15. FWD Modulus and Witczak et al. Predicted E* Modulus Versus AC Pavement Temperature.

It has also been pointed out that E* may provide a fundamental and accurate link to AC moduli measured from seismic testing, which adopt wave forms that provide stress pulses similar to the dynamic response of the E* test. Addressing the equipment requirement issue, one of the team members pointed out that recent improvements in pneumatic systems have made the task of accurately controlling the shape of the sinusoidal load at higher frequencies feasible. The cost of purchasing these testing units will vary between $40-$70 thousand. It was also pointed out that there is a definite problem with conducting complex modulus compressive tests on field samples. However, a SHRP team is investigating the possibility of using multiple stacked cores for compression modulus testing.

NCHRP PROJECT 9-11 FINAL REPORT

The main objective of the project titled "Segregation in Hot-Mix Asphalt Pavements" was to develop procedures to define, detect, and measure segregation so that its influence on performance could be evaluated. The literature review carried out for this project identifies current methodology used to detect and measure segregation. The detection methods identified include:

- visual identification,
- sand patch test, and
- nuclear density gauges.

The methods identified for measuring segregation include both non-destructive and destructive. The non-destructive methods are:

- permeability (for coarse gradations),
- nuclear density gauges (dependent on the changes in moisture content of the pavement and also on the gradation), and
- combinations of asphalt content measurements and density measurements.

The destructive method is:

- testing of cores for changes in asphalt content, gradation, density, and air voids.

The team has also identified some innovative technology for identifying and quantifying segregation. These include:

- thermal imaging,

- ground penetrating radar,
- thin lift nuclear asphalt content/density gauge,
- laser surface texture measurements , and
- seismic pavement analyzer.

The research team studied a total of 14 field projects, of which seven were recently constructed pavements, and seven were evaluated during construction. First, a visual survey was undertaken to identify and classify areas for no segregation, low level segregation, medium segregation, and high segregation. Locations were then marked for non-destructive testing. Field tests included laser texture measurements, portable seismic pavement analyzer measurements, nuclear densities, and infrared thermography. Cores from the segregated areas were taken to the lab to determine the bulk specific gravity, resilient modulus, tensile strength, theoretical maximum specific gravity, asphalt content and gradation.

Results From Lab Testing

Cores were classified as having no, low, medium, and high levels of segregation based on the correlation between asphalt content and gradation. The change in gradation associated with each level of segregation was determined. The results indicate that the difference in percent passing any sieve size was less than 5 percent for non-segregated cores; low level segregated cores had at least one sieve with a change of more than 5 percent; medium level segregated cores had at least two sieves with a change of more than 10 percent; and highly segregated cores had at least three sieve sizes with a change of more than 15 percent. Researchers also found that this method of classifying the cores matched with the visual observations about 60 percent of the times.

Results From Field Tests

Nuclear Density Gauge

The team used two different gauges-Troxler and Seamen gauges-and found that the Seamen gauge provided best results. The Seaman density gauge is equipped with a moisture gauge, which was used to measure the asphalt content based on changes in the hydrogen count. The pavement was evaluated along three longitudinal paths-shoulder, middle, and centerline. The team found a general trend of decreasing densities with increasing levels of segregation. However, there were a few readings that did not follow the general trend. Therefore the team members comment that the nuclear density gauge has had variable success due to the fact that a change in density may not be one of the best parameters to detect segregation. This finding is due to the fact that areas with fine segregation may produce an increase in density while segregation with coarser gradations may result in reduced densities. They also found that the air voids increased with an increase in the level of segregation. Asphalt content profiles from the moisture content readings indicate that areas of low asphalt content are not well correlated with areas having medium and high levels of segregation (testing of cores).

Infrared Thermography

The use of infrared thermography to detect segregation in recently constructed pavements did not appear to be very effective. This failure is due to the fact that the technology depends on the solar gain to highlight defective areas in the mat. The air voids act as insulators and trap the warm air while the dense areas act as good conductors of heat, thereby conducting the temperature away to the base and thus maintaining a lower temperature. The method is highly dependent on the environmental conditions and also the surroundings (e.g., blocks like shade from tree, etc.). Therefore the team decided to use this technology for detecting segregation only during construction, when the temperature differentials would be highly dependent on mix properties which govern the rate of cooling. A temperature histogram plot for the photographs shows three distinct patterns:

- narrowly distributed single mode that indicates a uniform mat temperature,
- widely distributed single mode that highlights localized cooler areas due to flipping of the paver wings, and
- bi-modal that highlights areas where the paver has been stopped for a significant length of time.

The team has also suggested that temperature segregation can be classified into two types. The first would be due to locally cooled or gradation segregated material in the truck and the second would be due to lengthy paver stoppage, which would result in a temperature differential of more than 20 °C.

Rosan$_v$ Laser Surface Texture Measurements

The Rosan$_v$ software has options to select the units for data collection. As the laser measurements were well correlated with the sand patch test, the team decided to use mean profile depth (MPD), a two-dimensional measurement, as the unit of measurement (ASTM E 1845). The MPD correlates to the estimated texture depth (ETD) by the following relationship (ASTM E 1845),

$$ETD = 0.2+0.8MPD$$

where ETD and MPD are in mm.

The average texture depth for every 500 mm of pavement length was plotted and compared to a plot of visually identified segregated areas. The comparison showed a good correlation between the measured and observed pavement texture. A histogram of the estimated texture depth (Figure A-16) shows that there is some overlap between none and the low levels of segregation and also between low and medium levels.

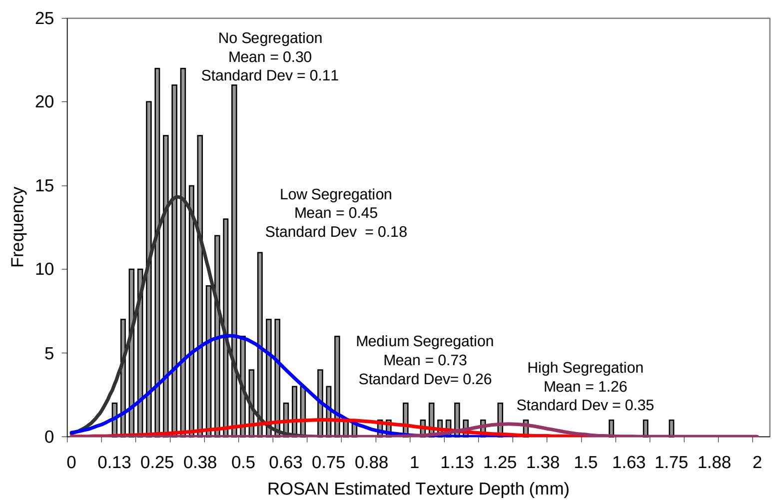


Figure A-16. Typical Histogram of Laser Texture Depth Measurements (Stroup-Gardiner and Brown, 2000).

Laboratory Study

The team conducted a laboratory study to estimate the influence of various levels of segregation on temperature susceptibility, moisture sensitivity, rutting potential, thermal cracking, and fatigue cracking. The testing was carried out on mixes that were segregated in the lab. These mixes were representative of two of the projects studied earlier. The team adopted the Superpave mix design procedure and the gyratory compaction technique to prepare the samples. Tests could not be conducted on the highly segregated samples because they fell apart.

Testing Carried Out By NCAT

They found that the permeability increased with increasing levels of segregation. Resilient modulus and the dynamic modulus of the mixtures were determined to estimate the mixture stiffness at various temperatures. Results indicate that there was little change in modulus at low levels of segregation, and the test temperatures did not influence the results. For medium level of segregation and test temperature of 4 °C, they found little change in the stiffness. However, they found that at higher temperatures the influence of segregation on stiffness seemed to be more pronounced. The dry tensile strength showed trends similar to the stiffness, but the wet tensile strength showed a continually decreasing strength pattern with increasing levels of segregation. The team members also noted that the influence of segregation on the modulus and dry tensile strength is not as pronounced as that seen when testing the cores. They concluded that this was due to the fact that uniformity could be achieved to a greater extent in the lab than in the field. It was only after moisture conditioning that the effect of segregation became more pronounced in the field.

The cohesion C was found to decrease with increasing levels of segregation. The angle of internal friction ϕ is relatively constant except for the highly segregated sample from one of the mixtures, where the internal friction value decreases significantly with a slight increase in C value. This relationship could be attributed to the loss in the interlock between aggregate particles, as there are few fines to fill the large voids between the aggregates. The octahedral shear stress did not seem to be influenced by the level of segregation for one of the mixtures. But the second mixture seemed to show about 40 to 60 percent lower shear stress tolerance with an assumed confining pressure of 300 kPa for medium and high levels of gradation segregation, respectively. They concluded that the effect of segregation on rutting potential is mix dependent and, in some cases, severe levels of segregation may cause rutting. Low temperature indirect tensile creep testing was also carried out, but the data obtained for these large aggregate mixtures were erratic. The DAMA program was used to determine the effect of segregation on the fatigue life. The assumption that a given level of segregation would occur in only one lift at a time was made. The results indicate that low level of segregation will reduce the fatigue life of the lift in which it occurs with very little effect on the layers above the affected one. Medium and high levels of segregation will have a more pronounced effect on the fatigue life and will affect all layers of the pavement. The results are summarized in Table A-11.

Table A-11. Influence of Segregation on Fatigue Life Using Output from DAMA Software (Stroup-Gardiner and Brown, 2000).

Lift	Percent Loss of Life Due to a Given Level of Segregation in a Given Lift, %		
	Low	Medium	High
Project 1-1			
Segregation in Wearing Course			
Wearing	38	81	99
Binder 1	3.2	0	(compression)
Binder 2	0	0	0
Segregation in Binder 1 Course			
Wearing	8	0	29
Binder 1	57	79	95
Binder 2	0	0	0
Segregation in Binder 2 Course			
Wearing	2	(compression)	11
Binder 1	(compression)	15	74
Binder 2	50	50	50
Project 6-1			
Segregation in Binder 1 Course	Not Evaluated		
Wearing		19	25
Binder 1		84	98
Binder 2		0	0
Segregation in Binder 2 Course			
Wearing		(compression)	(compression)
Binder 1		(compression	(compression)
Binder 2		50	50

Tests Carried Out at Purdue University

PURWheel

The research team also carried out tests using the PURWheel under conditions that relate to rutting and stripping. Test specimens were either compacted in the laboratory using a linear compactor or were obtained from in-service pavements. The test environment was either hot/wet or hot/dry. The test criterion was set at 20,000 wheel passes or 20 mm of deformation, whichever occurred earlier. The results indicate a minimal effect of segregation levels on rutting potential. The moisture condition did not seem to influence the effect on rutting. Testing under hot and wet conditions resulted in three times the amount of rut depth as that observed for hot and dry conditions for the no, low, and medium levels of segregation. The highly segregated samples failed even before 10,000 cycles were completed.

Resilient Modulus

The tests were carried out at a temperature of 60 °C with a confining pressure of 138 kPa. The modulus was determined after 200 applications of a 275 kPa-deviator stress that was applied at a rate of 1 Hz. The dry resilient modulus showed a general trend of decreasing modulus with increasing levels of segregation. Tests carried out on wet drained and un-drained specimens indicate no influence of drainage condition on the results.

Triaxial Test

Triaxial tests were carried out immediately after the resilient modulus tests. The results indicate a higher strength for the wet condition. This may be attributed to the reaction of pore water pressure. The results also indicate that the change in mix strength is not significant until a high level of segregation is reached. Team members point out that “a high level of segregation is needed before the permanent deformation of the coarse aggregate gradations will experience noticeable changes in rutting potential.”

The results of the testing carried out for the NCHRP 9-11Project are summarized below (Table A-12). This table is taken from page 108 of the report.

Table A-12. Summary of the Influence of Segregation on Mixture Properties (Stroup-Gardiner and Brown, 2000).

Mixture Property	Percent of Non-Segregated Mix Property by Level of Segregation			
	None	**Low**	**Medium**	**High**
Range of Temperature Differences, °C	<10	10 to 16	17 to 21	> 21
Surface Texture Ratios	< 1.16	1.16 to 1.56	1.57 to 2.09	> 2.09
Change in Mix Properties Expressed as a % of the Properties in the Non-Segregated Areas				
Permeability	Increased slightly	Increasing with level of coarse segregation		
Resilient Modulus[1]	Little or slightly increasing stiffness	70 to 90	30 to 70	< 30
Dynamic Modulus	Little or slightly increasing stiffness	80 to 90	70 to 80	50 to 70
Dry Tensile Strength	110	90 to 100	50 to 80	30 to 50
Wet Tensile Strength	80 to 90	75	50	30
Low Temperature Tensile Stress	No conclusions due to test method difficulties			
Loss of Fatigue Life when Segregation in Upper Lifts, %	Not Estimated	38	80	99
Rutting Potential	Not strongly influenced by gradation segregation Until a high level of segregation is seen			
Difference in Values Between Segregated and Non-Segregated Areas				
Gradations Minimum number of sieve sizes which are given % coarser	NA	1 sieve > 5	2 sieves > 10	4 sieves > 15
Change in Air Voids, %	NA	0 to 2.5	2.5% to 5.5	> 5.5
Change in Asphalt Content, %	NA	-0.3 to -0.75%	-0.75 to -1.3	> 1.3

[1]: Reflects results from testing both cores and laboratory-prepared samples.

Note: The surface texture ratio or the texture ratio refers to the ratio of the texture in the segregated area to that in the non-segregated area.

The range of temperature difference shown for different levels of segregation in the above table refers to the temperature differentials from the infrared camera.

Pavement Condition Surveys

As a part of their study, researchers surveyed existing pavements exhibiting signs of segregation-related distress. This survey covered six states–Alabama, Washington, Minnesota, Georgia, Texas, and Connecticut. Based on what they observed, researchers concluded the following:

- Initial low density due to temperature segregation results in periodic rutting, increased longitudinal, and fatigue cracking.
- Raveling occurs as a result of gradation segregation, and the rutting observed in temperature segregation due to densification from traffic is not seen in gradation segregation.
- According to the DOT staff, the loss in pavement life for a segregated pavement having an anticipated (non-segregated) life of 12-15 years was estimated to be 3-7 years.

In conclusion, the team reported the following:

- The nuclear density and the asphalt content measurements are not sensitive enough for the identification and measurement of segregation.
- Relating infrared measurements to mixture properties produced the results which are tabulated in Table A-13.

Table A-13. Relationship Between Measurements from Infrared Camera and Mix Properties (Stroup-Gardiner and Brown, 2000).

Mixture Property	Level of Segregation							
	No		Low		Medium		High	
	Temperature Change (°C)	Property	Temperature Change (°C)	Property	Temperature Change (°C)	Property	Temperature Change (°C)	Property
Air Voids	<10	Within 2% of average	10-16	Within 2-4.5% of average	16-21	Within 4.5-6.5% of average	>21	>6.5% of average
Asphalt Content	<10	Within 0.3% of average	10-16	Within 0.3-0.8% of average	16-21	Within 0.8-1.3% of average	>21	>1.3% of average
Resilient Modulus Ratios	<10	90%	10-16	70-90%	17-21	50-70%	>21	<50%

Conclusions from the laser texture measurements are shown below in Table A-14.

Table A-14. Results from Laser Texture Measurements.

Property	Low		Medium		High	
Texture Ratio (TR)	1.36		1.76		2.59	
Standard Deviations for TR	.15		.22		.42	
	ETD Ratio	Change	ETD Ratio	Change	ETD Ratio	Change
% Change in Air Voids	<1.6	2.5	1.6-2.2	2.5-5.3	>2.2	>5.3
% Change in Asphalt Content	<1.6	0.75	1.6-2.2	0.75-1.4	>2.2	>1.4
Resilient Modulus Ratio	<1.6	100-65%	1.6-2.2	65-25%	>2.2	<25%

Based on forward step-wise linear regression, the team proposed an equation for estimating the texture. The estimated texture depth (ETD), which is a measure of the mean texture, is given by the following equation.

$$ETD = 0.01980\ (\text{Max. agg. Size}) - 0.004984\ (\%\ \text{passing}\ 4.75\ \text{mm}) + 0.1038\ Cc - 0.004861Cu$$

where:

Cc = coefficient of curvature

Cu = coefficient of uniformity

MTD SHOWCASE IN EL PASO

TX DOT recently carried out demonstration project in El Paso. The main objectives of the project were to evaluate the effectiveness of different material transfer devices (MTD) in eliminating segregation and to compare different techniques to measure and quantify segregation. The MTDs evaluated include:

- Barber-Greene, Model BG-650
- Blaw-Knox, Model MC-330

- Cedarapids, Model CR 461
- Lincoln, Model 880-HP
- Roadtec, Model SB-2500B

The techniques evaluated to measure and quantify segregation include:

- In-Place Density

  -Nuclear Density Gauge

  -Road Cores

  -Ground Penetrating Radar (GPR)

- Infrared Thermal Imaging
- Visual Rating
- Smoothness or Ride Data

  - Profilograph
  - Profiler

Results

Results of the Nuclear Density Tests:

The mix used on this project had a maximum allowable density range of 8.0 lb/c.f and a maximum allowable decrease in density of 5 lb/c.f., as per the special provision proposed by TxDOT. Table A-15 indicates the capability of density gauges in determining segregation.

Table A-15. Summary of Findings from Nuclear Density Profiles (Tahmoressi et al., 1999).

MTD	Number of segregated locations not detected by density gauge	Number of segregated locations detected by density gauge
Barber-Greene	1	5
Roadtec	5	3
Lincoln	1	5
Cedarapids	2	4
Blaw-Knox	3	3

The density of the core did not correlate well with the nuclear gauge densities. The R^2 value for the plot was 0.2984. The bad correlation of the data may be due to the fact that the density gauge was not calibrated for the particular mix used on the project.

Thermal Imaging

The ability of various MTDs to produce a uniform mix temperature behind the paver was evaluated using the infrared camera to measure mat temperature. The results are shown in Table A-16.

Table A-16. Comparison of Various MTDs' Ability to Produce a Uniform Mix Temperature Behind the Paver.

MTD	Uniform temperature distribution in longitudinal direction	Uniform temperature distribution in transverse direction	Occurrence of low temperature spots
Barber-Greene	√	×	Individual
Roadtec	√	√	Rare
Lincoln	√	×	Occasional
Cedarapids	√	×	Several
Blaw-Knox	√	×	Individual

Results from GPR Study

The GPR evaluations and the resulting dielectric plots indicate that the Roadtec MTD proved to be the best choice among the MTDs evaluated in this study. Next in rank was the Barber-Greene. The surface dielectrics correlate well with the core densities.

Results from Visual Rating

Five raters rated the different MTDs. There was a significant amount of variation in the five different ratings. The results were normalized and evaluated. The visual rating indicates that the performance of the Roadtec is better as compared to the other MTDs at two locations.

Ride Quality

Results of the profilograph tests indicate that the use of Barber-Greene MTD resulted in the lowest profile index (PI) value of 2.8 in/mi. Roadtec, with a PI of 7.2 in/mi, followed it. Results from the profiler study indicate that Cedarapids provided the highest present serviceability index (PSI) and also the lowest international roughness index (IRI).

Conclusions from the Study

- None of the MTDs were completely successful in eliminating problems related to segregation.
- Screed extensions induced segregation in this project.
- Using MTDs having larger on-board mix storage capacity can reduce truck end segregation.
- Identification of segregation by using density profiles does not seem to be a very effective method.
- GPR has the potential to identify and quantify segregation.

NCHRP PROJECT 96-ID032

Testing and Trial Deployment of a Cost-Effective and Real-Time Asphalt Pavement Quality Indicator System

Phases I and II of this project have been completed. The pavement quality indicator (PQI) developed has been reported to be an effective electronic sensing instrument developed for the purpose of determining the density of the asphalt mat. Measurements of density can be obtained instantaneously, and hence it is possible to obtain a large number of readings on any site and provide "real time" feedback to the paving crew for timely corrective action. The prototype design adopts an electronic capacitance-based sensor system. The prototype developed was used to test lab samples and also for field-testing. Results show that with an accuracy level of $\pm$2 lbs/cu.ft., 58 percent of the PQI readings fall within the acceptable range and only 3 percent of the nuclear gauge readings fall within the acceptable range. However, it was found that measurements at the joints were a problem. If the data taken at joints were removed from the analysis, 84 percent of the PQI values and 6 percent of the nuclear gauge values fell within the ±2 percent acceptable range. Design improvement to eliminate this problem was to be carried out in the phase III of the project. Hence the usefulness of PQI in Project 0-1708 will have to be determined based on the results of Phase III of this study.

FHWA-RD-91-070

The main objective of the project entitled "Performance-Related Specifications for Asphalt Concrete–Phase II" was to continue the development of PRS by:

- conducting laboratory tests to determine the relationships between materials and construction (MC) variables and fundamental response variables (FMRV), and the relationship between FMRV and pavement performance indicators; and
- developing a detailed plan (experimental design, construction details, and data collection and analysis) for an accelerated field test at a test track facility.

The experimental variables for the laboratory study include:

- asphalt cement–two types were included, one with high temperature susceptibility and the other with low temperature susceptibility;
- aggregates–two types were included, a non-stripping crushed granite and a stripping granite.
- Asphalt content–three levels, one at optimum and the other two at ± 0.75 percent deviating from the optimum;
- levels of compaction–high, medium, and low. These levels were chosen to produce a range in air void content and were maintained constant for all samples compacted at each level. Therefore, the air voids and voids in mineral aggregate (VMA) were uncontrolled variables. Target air voids for different levels of compaction are shown in Table A-17.

Table A-17. Compaction Levels and Corresponding Air Voids Ranges (Shook et al., 1993).

Level of Compaction	Air Void Content(%)
High	1-5
Medium	5-8
Low	8-12

- Aggregate gradation–three basic gradations were used, and the percent passing the No. 30 (600 µm) and No. 200 (75 µm) sieves were varied at the three levels, producing nine different combinations.
- Additive (lime)–two levels

Primarily, kneading compaction procedure was adopted and the gyratory compaction was used only for the mix-design check test. Some specimens were aged and moisture conditioned. The specimens were conditioned using the Lottman accelerated conditioning procedure. Tests conducted include:

- resilient modulus at 77 °F;
- indirect tensile strength at 0 °F and 77 °F;
- diametral fatigue at 77 °F; and
- diametral creep at 104 °F.

The results from the laboratory tests are shown in Table A-18.

Table A-18. Significant M&C Variables (Shook et al., 1993).

Dependent Variable	Independent Variables
Resilient Modulus (MR)	Compaction, percent passing sieve #30, asphalt content, asphalt type, %-passing sieve #200
Tensile Strength (TS)	Compaction, percent passing sieve #30, asphalt content, asphalt type, %-passing sieve #200
MR (32 days)/ MR (1day)	VMA, compaction
TS (32 days)/ TS (1 day)	Compaction, percent asphalt content
Index of Retained Modulus (IRM)	Additive, percent asphalt content, percent passing sieve #30, and percent passing sieve #200
Index of Retained Strength (IRS)	Additive, percent passing sieve #30

The final regression equations developed from the lab testing are shown in Table A-19. The compaction index (CI) represents the three different levels of compaction adopted for the experiment. The different levels of compaction have been assigned numerical values as shown below:

Low = -1, Medium = 0, High = 1.

The equation for predicting CI was developed by the team using SSPS statistical analysis program. The ratio MR (32 days)/ MR (1 day), represents the ratio between the aged conditioned and the unconditioned resilient modulus. A similar ratio is presented for the tensile strength. Equations for the index of retained modulus and index of retained tensile strength are also shown in the table. Researchers found that moisture conditioning was not significant in the equations. The effects of the MC variables on the FMRVs is shown in Figures A-17 to A-27 that have been taken from the report.

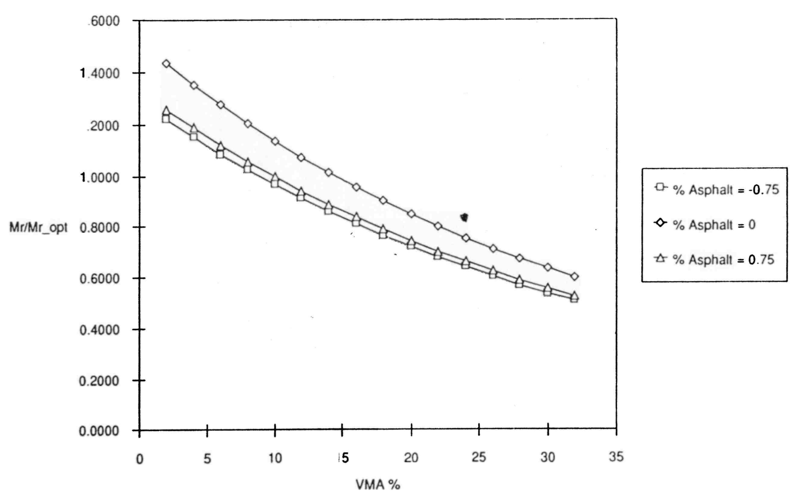


Figure A-17. Effect of VMA and Percent Asphalt Deviation on the Ratio of Predicted to Optimum Resilient Modulus (FHWA-RD-91-070, 1993).

Table A-19. Final Regression Models Relating HMA Quality Characteristics to Fundamental Response Variables (FHWA- RD-91-070).

DEPENDENT VARIABLE	EQUATION	N	R’	SE
Compaction Index (CI)	2.19087-0.05206(VMA) – 0.23405(%VOIDS) + 0.00340623(%#30)(%VOIDS)-0.02298(%#200)(%ASPHDEV) – 0.00882088(%#30)(%ASPHDEV)	105	0.85	0.34898
AC Type In (MR)	5.32928+0.64468(CI) + 0.94522(ASPHTYP) – 0.03965(VMA) + 0.02207(%ASPHDEV) – 0.26202(%ASPHDEV)^2 – 0.0012691(%#200) + 0.001484(%#200)(VMA)	108	0.84	0.38278
AC Type In (TS)	3.47901 + 0.74038(CI) + 0.51266(ASPHTYP) + 0.02932(VMA) + 0.12752(%ASPHDEV) – 0.15695(%ASPHDEV)^2 + 0.04984(%#200) – 0.001939(%#200)(VMA)	107	0.87	0.27457
AC Penetration In (MR)	7.60425 + 0.02189(%ASPHDEV) – 0.26264(%ASPHDEV)^2 – 0.02624(ASPHPEN) – 0.03926(VMA) + 0.64515(CI) – 0.000543256(%#200) – 0.001453686(%#200)(VMA)	108	0,84	0.38258
AC Penetration In (TS)	4.71325 + 0.12722(%ASPHDEV) – 0.15764(%ASPHDEV)^2 – 0.01423(ASPHPEN) + 0.02949(VMA) + 0.74065(CI) + 0.05005(%#200) – 0.00194589)(%#200)(VMA)	107	0.87	0.27440
In MR (32 days) MR (1 day)	0.18944 + 0.0020579(%#200)(VMA) – 0.01049(%ASPHDEV)(VMA) + 0.00046623(%#30)(VMA)	95	0.42	0.2307
In TS (32 days) TS (1 day)	0.50560 – 0.0091774(CI)(%#30) – 0.0052624(VMA)	93	0.29	0.278
IRM	41.42601 – 69.58340(ADITV) + 34.55498(ASPHTYP)(ADITV) + 3.69456(VMA) + 28.91298(CI)(ADITV)	97	0.44	29.615
IRS	85.78256 – 1.52260(%#30)(ADITV) + 3.86562(ASPHTYP)(VMA) – 1.89002(ASPHTYP)(%#30)	96	0.37	35.608
log (N)	2.92100 – 2.6401 log (S) + 2.22575 log (TS)	96	0.69	0.48751

NOTE:

CI	-	Compaction index
MR	-	Resilient modulus at 77 °F (25 °C)
TS	-	Tensile strength at 77 °F (25 °C)
R^2	-	Coefficient of determination
% VOIDS	-	Percent air voids (percent)
% ASPHDEV	-	Percent deviation from optimum asphalt content (percent)
SE	-	Standard error
% # 30	-	Percent passing No. 30 (600 µm) sieve (percent)
% # 200	-	Percent passing No. 200 (75 µm) sieve (percent)
ASPHPEN	-	Penetration value at 77 °F (25 °C)
ASPHTYP	-	Asphalt type (temperature susceptibility), 0 = low and 1 = high
ASPHPEN	-	Penetration value at 77 °F (25 °C)
IRM	-	Index of retained modulus
NCYC	-	Number of repetitions to failure
IRS	-	Index of retained strength
S	-	Applied stress level for fatigue analysis
VMA	-	Voids of mineral aggregate (percent)
N	-	Number of samples

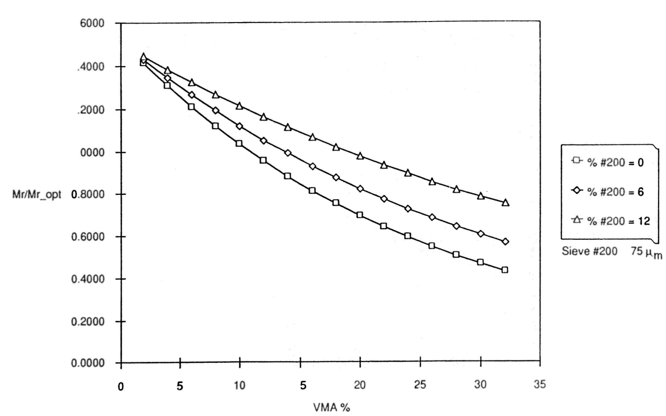


Figure A-18. Effect of VMA and Percent Passing Sieve #200 on the Ratio of Predicted to Optimum Resilient Modulus (FHWA-RD-91-070, 1993).

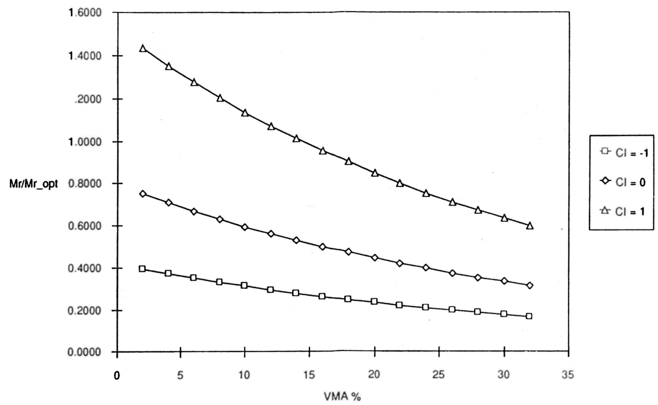


Figure A-19. Effect of VMA and CI on the Ratio of Predicted to Optimum Resilient Modulus (FHWA-RD-91-070, 1993).

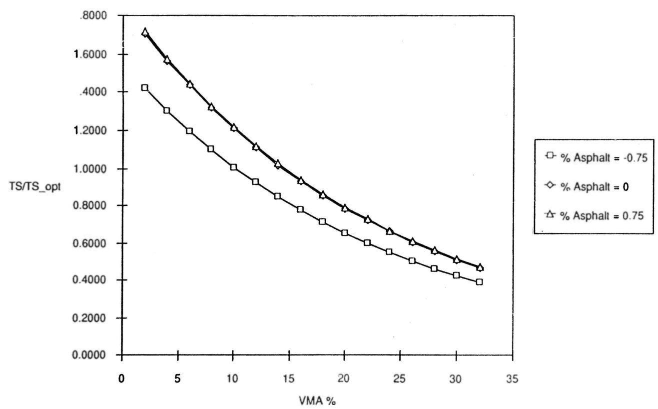


Figure A-20. Effect of VMA and Percent Asphalt Deviation on the Ratio of Predicted to Optimum Indirect Tensile Strength (FHWA-RD-91-070, 1993).

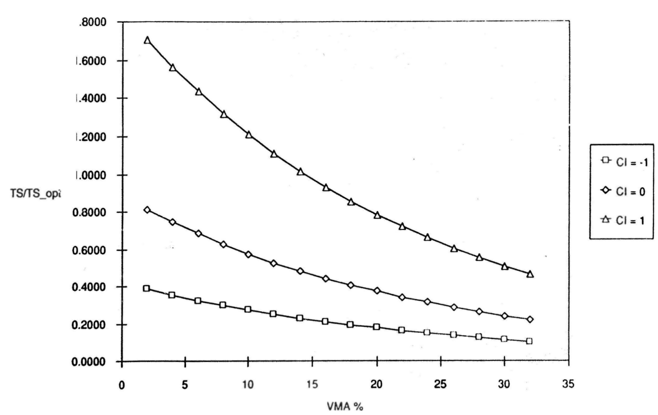


Figure A-21. Effect of VMA and CI on the Ratio of Predicted to Optimum Indirect Tensile Strength (FHWA-RD-91-070, 1993).

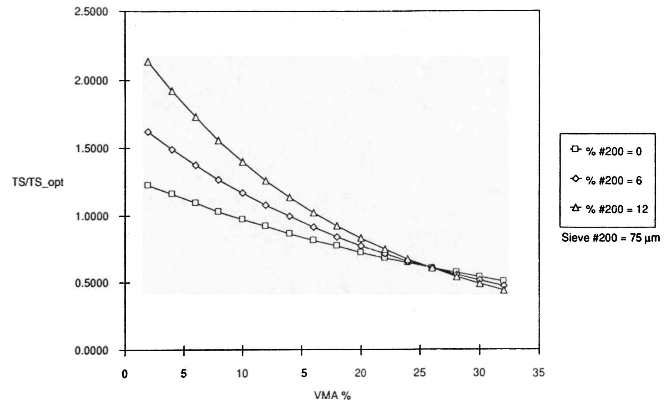


Figure A-22. Effect of VMA and Percent Passing Sieve #200 on the Ratio of Predicted to Optimum Indirect Tensile Strength (FHWA-RD-91-070, 1993).

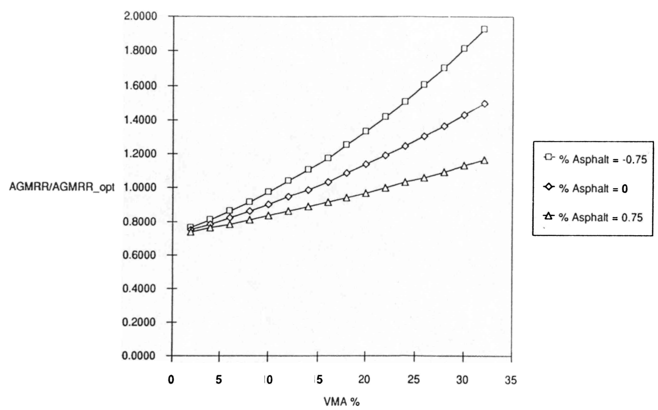


Figure A-23. Effect of VMA and Percent Asphalt Deviation on the Ratio of Predicted to Optimum Aged Resilient Modulus (FHWA-RD-91-070, 1993).

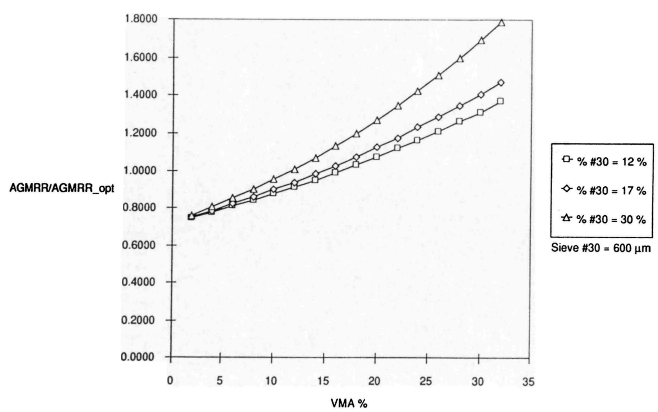


Figure A-24. Effect of VMA and Percent Passing Sieve #30 on the Ratio of Predicted to Optimum Aged Resilient Modulus (FHWA-RD-91-070, 1993).

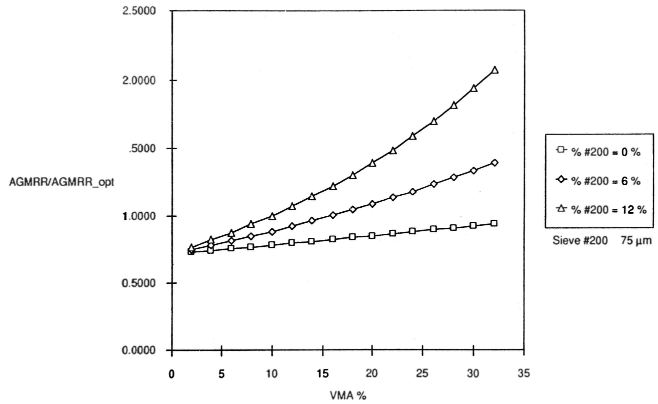


Figure A-25. Effect of VMA and Percent Passing Sieve #200 on the Ratio of Predicted to Optimum Aged Resilient Modulus (FHWA-RD-91-070, 1993).

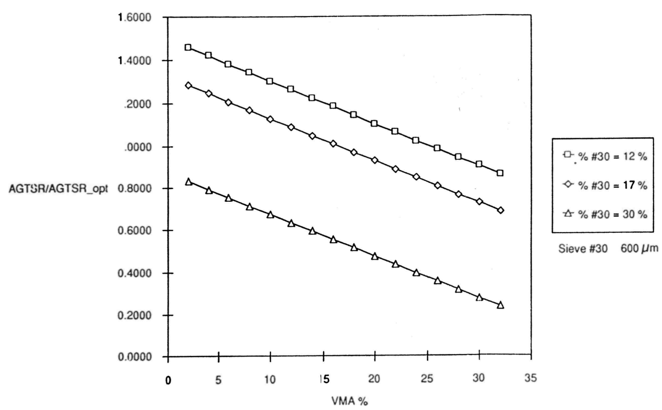


Figure A-26. Effect of VMA and Percent Passing Sieve #30 on the Ratio of Predicted to Optimum Aged Indirect Tensile Strength (FHWA-RD-91-070, 1993).

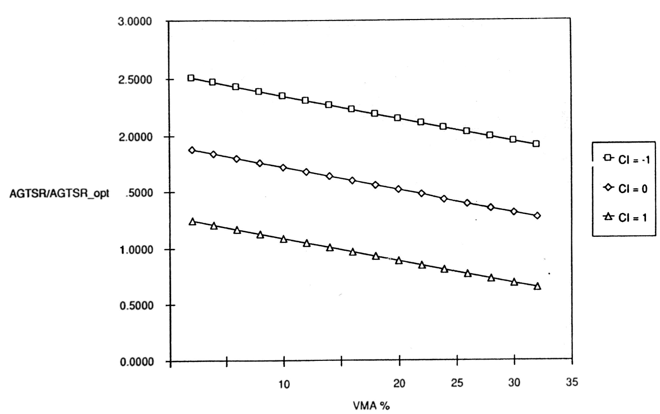


Figure A-27. Effect of VMA and CI on the Ratio of Predicted to Optimum Aged Indirect Tensile Strength (FHWA-RD-91-070, 1993).

NCHRP REPORT 332

The main objectives of this report entitled “Framework for Development of Performance-Related Specifications for Hot-Mix Asphaltic Concrete” are:

- to develop a conceptual framework for statistically based PRSs that can be applied in general to highway materials and their associated construction processes; and
- to demonstrate the validity of the conceptual framework.

In order to accomplish the above objectives, the team identified MC variables that relate to construction, evaluated currently available performance models, and identified the MC variables the affect performance and can be controlled by the contractor.

The generalized conceptual framework developed for this study is shown in Figure A-28. The flow diagram indicates that two different paths can be followed to develop a PRS. One algorithm would be to use the B-C-D-F-G-H path, which would consider only the “as-constructed structural responses” of the pavement. The team did not adopt this method, as it does not consider the effect of M&C variables. The second algorithm, which was adopted by the team, follows the path B-A-E-F-G-H.

The generalized framework as applicable to hot-mix asphalt concrete is shown in Figure A-29, which is taken from the NCHRP 332 report. The design algorithm includes:

- pavement design factors such as thickness, percent compaction and allowable roughness; and
- target values for mixture properties like percent asphalt, gradation, and Marshall stability.

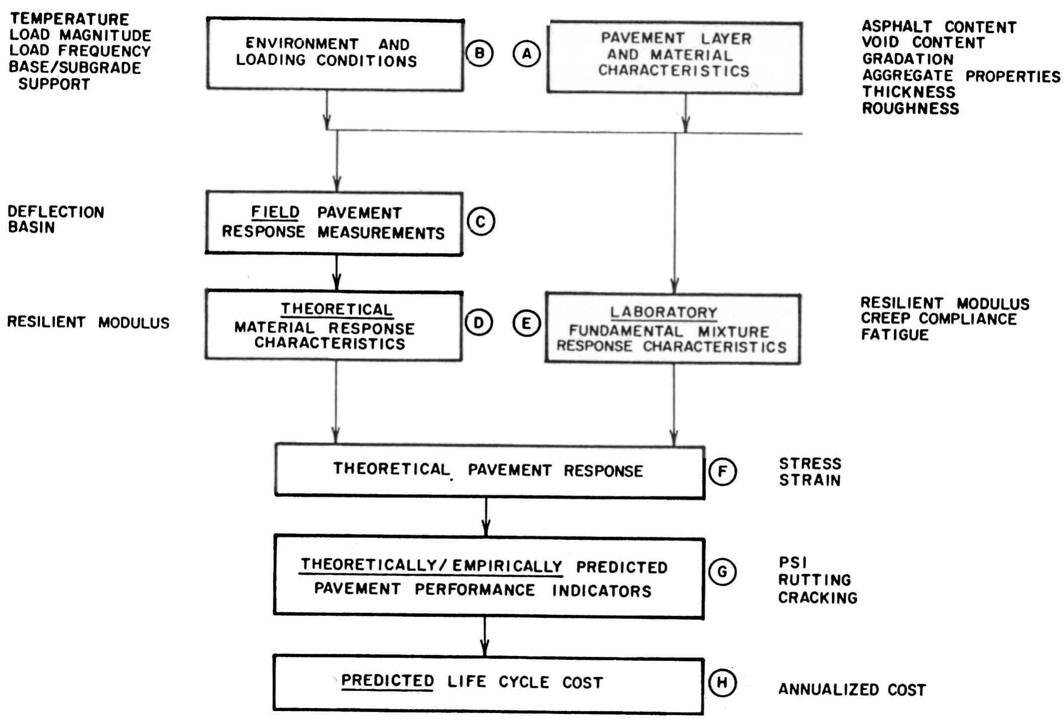


Figure A-28. Generalized Conceptual Framework (Anderson et al., 1990).

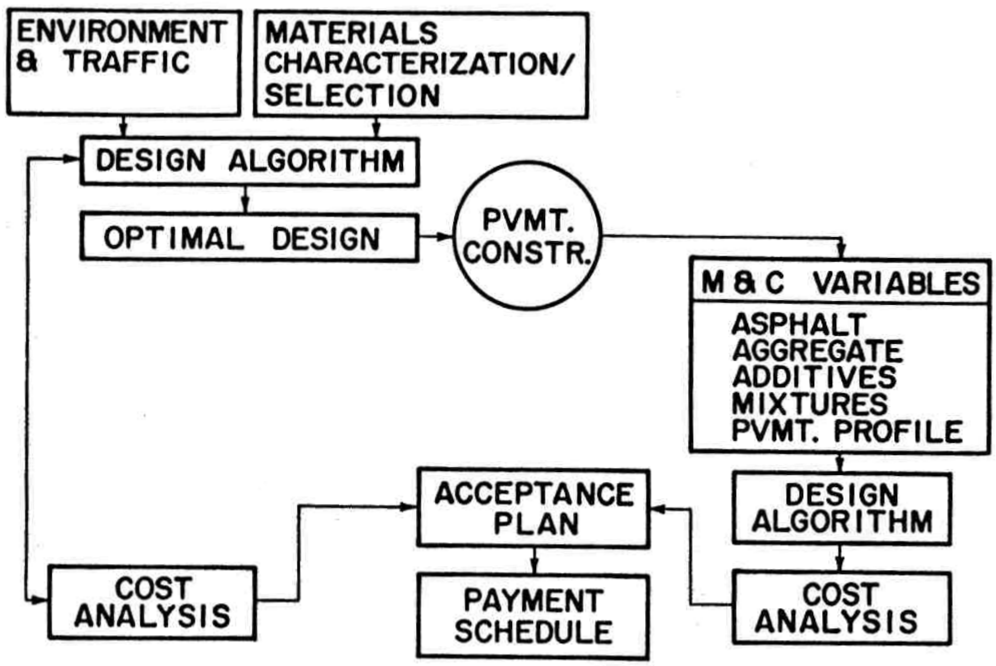


Figure A-29. Generalized Framework for a Performance-Based Specification for Hot-Mix Asphaltic Concrete (Anderson et al., 1990).

A computer program called "PERSPEC," which relates MC variables to annual costs, was developed by the team. It is a demonstration of the conceptual framework and the development of a payment schedule based on performance criteria. A demonstration of this program is shown in the report with three sets of hypothetical data for MC variables and a bid price of $10/yd^2. The flow diagram for the program and the three sample runs as reported in the report are shown below (Figure A-30 and Tables A-20, A-21).

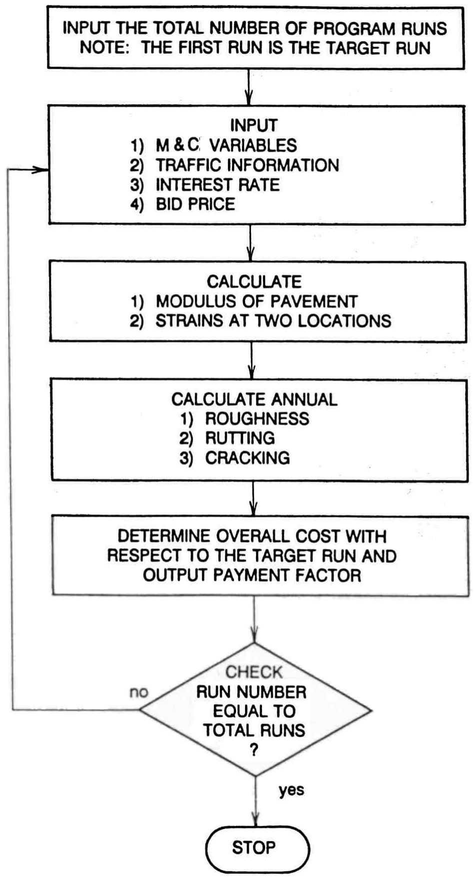


Figure A-30. Flow Diagram for PERSPEC Computer Program (Anderson et al., 1990).

Table A-20. Three Sample Runs of the PERSPEC Program (Anderson et al., 1990).

Run No.	Condition	Percent Passing No. 200 Sieve	Percent Air Voids	Percent AC
1	Target	7	5.5	6.5
2	Below target	4	3	5
3	Above target	10	8	8

Table A-21. Payment Values as Determined by PERSPEC (Anderson et al., 1990).

Run	Condition	Annual Cost ($1/yd^2)	Pavement Life	Payment ($1/yd^2)
1	Target	3.21	4	10.0*
2	Below target	3.96	4	7.36
3	Above target	2.33	6	11.0**

*Bid price

** Payment calculated by PERSPEC algorithm was $14.51/yd^2.

An upper limit of the bid price plus 10 percent was used to limit the payment to $11.0/yd^2.

The laboratory studies included tests for diametral complex modulus, tensile strength, and creep and fatigue properties. All testing was carried out on Marshall specimens. For determining complex modulus, creep, and fatigue parameters, the repeated diametral testing was conducted at 77 °F with a 10 Hz haversine load. The team encountered problems when it tried to determine the phase shift, and hence researchers were able to obtain only |E*|. The main objectives of the experiments were:

- to determine the usefulness of the dynamic modulus predictive equation (Witczak et al.) when the target values are not achieved; and
- to develop and evaluate regression models based on MC variables for predicting the tensile strength of the mixture.

The experimental study included the following MC variables in their sensitivity analysis to the effect of the variables on complex modulus:

- ercent passing the #200 sieve–target and above target;
- percent air voids–target and above target; and
- percent asphalt cement-target and below target.

The results indicate that, regardless of the levels of the other independent variables, and for the range of values used in the analysis, the percent passing the #200 sieve had the least effect on complex modulus, the percent air voids had a larger effect than the percent passing #200 sieve, and the percent asphalt cement had the greatest effect.

The data for the unaged mixtures had an R^2 value of 0.31 for the correlation between the measured and the predicted complex modulus. Hence, team members comment that Witczak's method may not be generally acceptable for mixtures that have non-conforming MC variables. However, when they considered individual asphalt-aggregate combinations, they found that there was a good correlation (R^2=0.87) between the measured and the predicted modulus for one particular combination. This correlation would suggest that Witczak's method might work on some particular asphalt-aggregate combinations. The results of the study have been summarized in the plots shown below (Figures A-31, A-32, A-33) which have been taken from NCHRP Report 332. Figure A-31 represents the relationship between the measured modulus and the modulus predicted using Witczak's equation.

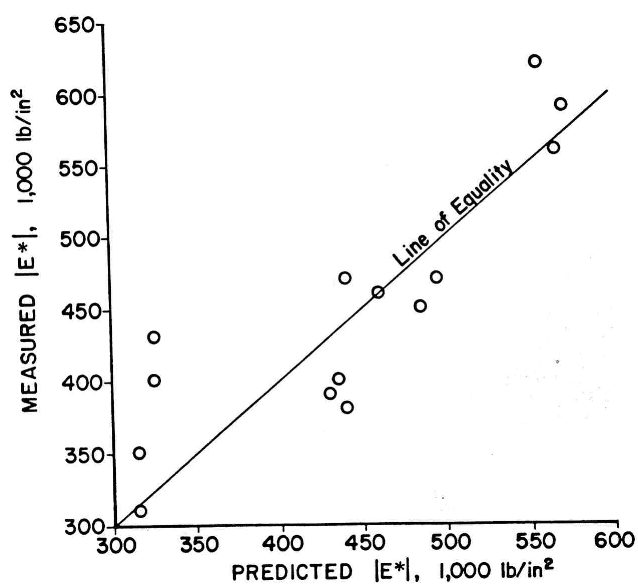


Figure A-31. Measured Modulus vs. Predicted by Witczak (Anderson et al., 1990).

The modulus equation obtained as a result of the tests the team carried out is shown in the equation below and has R^2 and coefficient of variation values (CV) of 0.8 and 12 percent respectively. Correlation between the measured modulus and the modulus predicted by the regression equation developed in this study is shown in Figure A-32.

$$\text{Modulus} = 1{,}653{,}142 - 12{,}689*\text{Aggregate} - 95{,}491*\text{Asphalt} - 6{,}838*\text{Dust} - 182{,}668*\text{AC} - 77{,}058*\text{AV} + 11{,}049(\text{AC}*\text{AV})$$

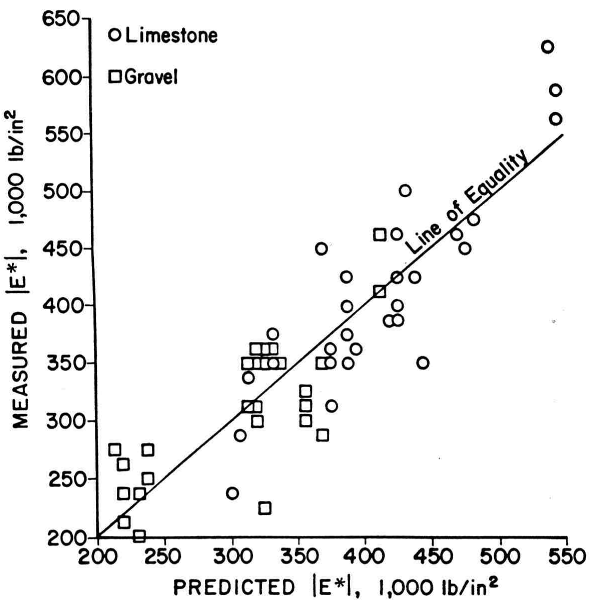


Figure A-32. Measured Modulus vs. Predicted by Regression (Anderson et al., 1990).

The tensile strength equation obtained from the experimental study is shown below and has R^2 and CV values of 0.82 and 11 percent respectively. Figure A-33 indicates the relationship between measured tensile strength and the predicted tensile strength.

Tensile Strength = 613+32(Aging)-9(Dust)-57(AC)-13(AV).

As a part of the study, the team has identified condition indicators and corresponding variables that affect pavement deterioration (Table A-22). It also identified performance models available for predicting pavement performance based on condition indicators that have been identified (Table A-23). A summary of findings can be seen in the tables shown below which are abstracted from the report.

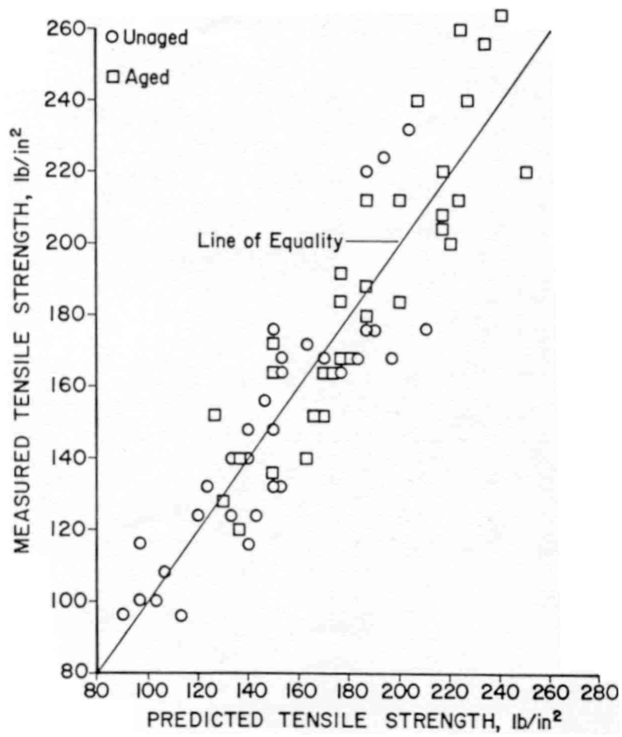


Figure A-33. Measured Tensile Strength vs. Predicted by Regression (Anderson et al., 1990).

Table A-22. Condition Indicators Identified as Being Significant for Development of Performance-Based Specification (Anderson et al., 1990).

Condition Indicator	Method of Measurement	Variables Affecting Deterioration in Pavement Condition				
		Materials	Plant	Construction	Environmental	Traffic
Cracking 1. Fatigue	Visual/video surveys	• AG • AS • AT • AD	• AC • Agr • AV	Density -(%max). Thickness	Precipitation Freeze-thaw Temperature	X
2. Thermal	Visual/video surveys	• AG • AS • AT • AD	• AC • Agr • AV	Density -(%max). Thickness	Temperature	-
3. Shrinkage	Visual/video surveys	• AG • AS • AD	• AC • Agr • AV	Density -(%max).	Temperature	-
Rutting	Measurement of transverse profile	• AG • AS • AT • AD	• AC • Agr • AV	Density -(%max). Thickness	Precipitation Freeze-thaw Temperature	X
Roughness	Measurement of longitudinal profile	May be influenced by materials and plant variables that affect cracking and rutting		Initial roughness after construction	Precipitation Freeze-thaw Temperature	X
Skid Resistance	Locked wheel skid number	• AG • AS • AT	• AC • AV	Initial skid resistance after construction	Precipitation Temperature	X
Raveling	Visual/video surveys	• AG • AS • AT • AD	• AC • AV • ATr	Segregation	Precipitation Temperature	X

Table A-22. Condition Indicators Identified as Being Significant for Development of Performance-Based Specification (Anderson et al., 1990) continued.

Condition Indicator	Method of Measurement	Variables Affecting Deterioration in Pavement Condition				
		Materials	Plant	Construction	Environmental	Traffic
Moisture Damage	• Loss of modulus • Coring/ destructive testing	• AG • AS • AT • AD	• AC • AV • ATr	Density -(%max).	Precipitation Temperature	X
Wear resistance	Currently not considered in pavement condition surveys	• AT	• AC • AV • Amount of coarse and fine aggregate	Density -(%max).	Temperature Freeze-thaw	X

AG – Asphalt grade

AV – Air voids

AD – Additives

AC – Asphalt content

AT – Aggregate type

ATr – Aggregate treatment

AS – Asphalt source

AGr – Aggregate gradation

Table A-23. Summary of Performance Models for Flexible Pavements.

Distress Mode	Example Models	Input Parameters Related to Asphalt Mix
Fatigue Cracking	ARE	AC mix modulus AC mix Poisson's ratio
	Asphalt Institute	AC mix modulus AC mix Poisson's ratio Binder content Air voids content
	VESYS Cracking Model	AC mix fatigue properties (k1, k2) AC mix modulus AC mix Poisson's ratio
Low-Temperature Cracking	Cold	AC mix modulus-temperature relationship AC mix tensile strength-temperature relationship Thermal conductivity of AC mix Heat capacity of AC mix Absorptivity of AC mix Emmissivity of AC mix Convection coefficient of AC mix
	Shahin-McCullough Model for Low-Temperature Cracking	Air voids content Binder content Volume concentration of aggregates Specific gravity of asphalt Specific gravity of aggregate AC mix coefficient of thermal expansion Asphalt penetration index Asphalt softening temperature Absorptivity of AC mix Conductivity of AC mix
Thermal Fatigue Cracking	Lytton-Shanmugham Model	Ring and ball softening point Air voids content Binder content Volume concentration of aggregates
	Shahin-McCullough Model for Fatigue Cracking	Air voids content Binder content Volume concentration of aggregates Specific gravity of asphalt Specific gravity of aggregate AC mix fatigue properties (k1, k2) Asphalt penetration index Asphalt softening temperature AC mix Poisson's ratio

Table A-23. Summary of Performance Models for Flexible Pavements (continued).

Distress Mode	Example Models	Input Parameters Related to Asphalt Mix
Rutting	VESYS Rut Depth Model	AC mix modulus AC mix Poisson's ratio Permanent Deformation properties of AC mix
	Shell	AC mix modulus AC mix Poisson's ratio Bitumen viscosity Bitumen penetration Penetration index
	AGIP (Italian Asphalt Pavement Design Procedure)	Creep compliance function for AC mix
PSI/Roughness	PDMS	AC mix modulus AC mix Poisson's ratio
	AASHTO	AC mix modulus
	VESYS Roughness Model	AC mix modulus AC mix fatigue properties (k1, k2)
	Fernando Model	AC mix modulus AC mix Poisson's ratio

Comments

Fatigue cracking

- The ARE model was developed by correlating observed performance with theoretically determined pavement response parameters (tensile strain, tensile stress).
- The AI model was developed based on laboratory fatigue testing of asphalt concrete samples. Therefore these models require a shift factor to account for the differences between the lab and field conditions. Such shift factors are condition specific.

Thermal Cracking

- The Shahin-McCullough model relates the number of thermal cycles to failure to the thermal tensile strain.

- The model developed by Lytton and Shanmugham is based on fracture mechanics and computes the change in stress intensity factor due to daily changes in temperature cycles. The fracture parameters are determined empirically from the asphalt consistency. As detailed temperature data and large amounts of computer time are required for the operation of the model, it is not suitable for design of individual roadway segments, but it can be used to develop empirical design equations that are applicable to a set of local conditions.

Rutting

- The Shell model predicts the number of load applications required to reach a pre-defined, unacceptable level of rutting. Hence, this model is not useful where quantification of actual rutting is required.
- The VESYS model is a mechanistic model that predicts the total pavement deformation at any specified number of load applications. Its use has been limited to research applications as extensive lab testing and arbitrary correction factors are required to obtain estimates of rut depth.
- The AGIP model is based on the linear viscoelastic theory. Creep compliance of each material in the pavement structure has to be determined through lab tests. Material properties are used as inputs to a computer program which computes the permanent deformation in each layer.

PSI/Roughness

- The AASHTO model is an empirical model that predicts the number of 18 kip equivalent single axle loads (ESALs) before the PSI drops to a pre-defined serviceability level.
- The Pavement Design Management System (PDMS) model was developed based on correlation between observed AASHO performance data theoretically determined pavement response parameters.
- The VESYS model uses a mathematical relationship between the slope and rut depth variances to predict the progression of pavement roughness. The slope variance of the

roughness model is combined with individual predictions from the rut depth and cracking models to obtain PSI values. They mention that the comparison of VESYS results with observed performance in test sections has been erratic.

Most of the models reviewed required the establishment of relationships between the "pavement layer and material characteristics" and FMRVs to predict pavement performance. Since lab testing to establish such relationships would be expensive and time consuming, available predictive equations were reviewed by the team. These include:

- Van der Poel's nomograph, which is used to determine the stiffness modulus of bitumen;
- relationship developed by Heukelom and Klomp, which relates bitumen stiffness to the mix stiffness;
- nomograph developed by Bonnaure et al., which is used to predict the mix stiffness; and
- the AI equation that predicts the absolute value of the complex modulus of the mix.

A sensitivity analysis was carried out for the roughness model developed by Fernando et al. The model predicts the pavement roughness as a function of cumulative number of load applications. The model is shown below.

$$\log_{10}(1+SV) – (\beta_0+\beta_1\log_{10}N)/(1+\beta_2\log_{10}N)$$

where:

SV = slope variance

β_0 = initial pavement surface roughness

β_1 = $-0.035-0.220\beta_0-0.035\log_{10}V_3-0.050\log_{10}(1+H_1)$

β_2 = $-0.354+1.232\beta_1+0.269\sqrt{\beta_0}-31.958V_5-0.026\log_{10}T_2+0.007\log_{10}(1+H_2)$

H1 = thickness of AC layer, inches

H2 = thickness of base layer, inches

V3 = $\varepsilon_{sg3}-\varepsilon_{sgmax}$

V5 = $\varepsilon_{sg2}-\varepsilon_{sg1}$

T2 = $\varepsilon_{acmax}- \varepsilon_{ac2}$

ε_{sgmax} = maximum vertical compressive strain at the top of the subgrade directly underneath the wheel load

ε_{sgi} = vertical compressive strain at the top of the subgrade located along the longitudinal direction at a distance ‘i’ feet from the maximum

ε_{acmax} = maximum tensile strain at the bottom of the asphalt concrete layer and directly underneath the tire

ε_{ac2} = tensile stain at the bottom of the asphalt concrete layer located along the longitudinal direction at a distance of 2 feet from the maximum

For the sensitivity analysis, the pavement was considered as a three-layered structure with a surface course of AC, a granular base layer, and the subgrade. Factors that were considered in the factorial experiment include:

- initial present serviceability index (PSIi), with levels of 3.6, 3.9, 4.2;
- AC modulus, with levels of 300,000, 450,000, 600,000 psi.;
- asphalt concrete thickness, with levels of 3, 5, 7 in;
- granular base thickness, with levels of 4, 7, 10 in;
- coefficient (k1) of the base resilient modulus-bulk stress relationship, with levels of 3000, 6000, and 9000;
- exponent (k2) of the base resilient modulus-bulk stress relationship, with levels of 0.2, 0.5, and 0.8;
- coefficient (m1) of the subgrade resilient modulus-deviatoric stress relationship10,000, 20,000, 30,000; and
- exponent (m2) of the subgrade resilient modulus-deviatoric stress relationship -1.0, -0.6, and -0.2.

The initial surface roughness values were assumed to be 0.38, 0.53, and 0.68. The findings of the analysis indicate:

- The service life predicted by the model was found to be sensitive to the initial PSI, AC modulus, and the coefficients m1 and m2.
- In general, the researchers found predicted service life improved with increases in levels of the following factors, if other factors were held constant:

  -AC thickness

  -Initial PSI

  -AC modulus

  -Subgrade m1 and m2

The degree of improvement depends on the particular levels at which the other factors are held constant.

- The effects of base related variables depend on levels of other design factors and are relatively small, when compared to those of the other design factors.
- As a result of the influence of stress dependency of unbound materials, there seemed to be an indication of the existence of optimum values for base related variables.

AGENCY SURVEY RESPONSES COMPILED FROM OTHER RESEARCH PROJECTS

A survey of the highway agencies would provide useful information regarding the exact state of quality control and quality assurance measures that are currently adopted by the industry. It would be important to obtain information regarding the following:

- quality characteristics that are measured for acceptance;
- equipment currently used for the measurement of the quality characteristics;
- drawbacks of the currently used test methods;
- awareness of new technology that can be used to measure the quality characteristics; and
- the willingness to adopt new technology as a part of the agency specifications.

While reviewing the material of recent and on-going studies related to Project 0-1708, it became evident that most of these issues have been addressed in the surveys carried out by these studies. Hence it was decided that a compilation of the survey responses would be useful to the project. This compilation is presented below.

NCHRP 9-15 Interim Report

The questionnaire that was sent out to various state DOTs and other highway agencies consisted of the following five questions.

- Please identify five important hot-mixed asphalt quality characteristics (consider both as-produced and in-place) that affect long-term pavement performance and can be utilized in a PRS (see examples on pages 3 and 4). Also indicate how significant that characteristic is to the development of pavement distress (5 = very significant and 1 = slightly significant).

- List all pertinent test methods available to measure the quality characteristics discussed in Part 1.
- Which parameters and tests currently being used for quality assurance (in your situation) can be used for a PRS? Also indicate if the tests are practical, timely, and affordable.
- What steps need to be taken to improve on current test methods that are not practical, timely, and affordable?
- Are you aware of any new test devices that could be adopted for use in a PRS? Please specify the device, the property (or properties) that it measures, known manufacturers, and any other useful information regarding the applicability of the device to a PRS.

The responses to this survey as summarized by the interim report are shown below (Figure A-34, Tables A-24 and A-25).

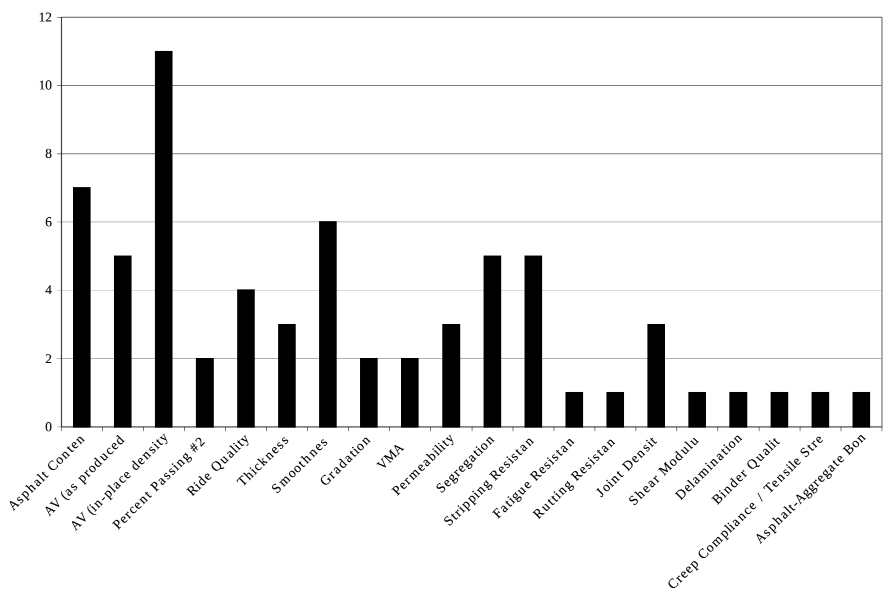


Figure A-34. Quality Characteristics Noted by Survey Respondents as Important to PRS (Killingsworth, 1999).

Table A-24. Test Devices/Methods for Measuring Quality Characteristics from Survey Respondents (Killingsworth, 1999).

Quality Characteristic	Methods of Measuring Quality Characteristic
Asphalt Content	Ignition Oven, Nuclear, Solvent Extraction, Batch Printout Ticket, Spot Check Method
As-Produced AV	Rice Gravity, Bulk Specific Gravity, Gyratory Compactor
In-Place AV (Density)	Cores, Nuclear, PQI,
Percent Passing #200	Extracted Gradation, Sieve Analysis
Ride Quality	Straight Edge, Profilometer, Profilograph
Thickness	Cores, Radar
Smoothness	Rolling Straightedge, California Profilograph, Lightweight Profilometer, Full-Size Profilometer, Walking Dipstick
Gradation	Sieve Analysis (Ignition), Mechanical Sieves
VMA	Marshall or Superpave "Volumetrics Criteria"
Permeability	FLDOT Permeameter, Test Device Manufactured by Soil Test in 1950's
Segregation	Thermal Imaging, Gradation, AC, In-Place Density, Sand Patch, In-Place Gradation Differences, ROSAN
Resistance To Stripping	In-Place Density, AASHTO T-283, Modified Lottman
Fatigue Resistance	Asphalt Pavement Analyzer
Rutting Resistance	Asphalt Pavement Analyzer
Joint Density	Specific Gravities, Nuclear Gage, Cores, PQI
Shear Modulus	SST, Field Adapted Shear Device
Delamination	Tack Coat Bond–Cores
Binder Quality	After Production and Storage-"Recovered Properties"
Creep Compliance/Tensile Strength	Universal Test Machine (Field Adapted)
Asphalt–Aggregate Bond	No Test Available

Table A-25. Needs Identified by Survey Respondents for Tests Associated with PRS (Killingsworth, 1999).

Needs	Respondents
Improvements to Gyratory Compactor (e.g., Calibration Method Needed)	1
Test Device that Provides Quick Strength Parameter	5
Improvements to In-Place Density Determinations	3
NDT Thickness Measurements	2
Improved Evaluation of Segregation	4
Improved Test Method for Air and/or Water Permeability	2
Improvements to Maximum Theoretical Specific Gravity Test Protocol	1
Quick Method for Fatigue Life	1
Accurate Method for Finding BSG of Fine Aggregate for VMA	3
Accurate Precision Statement for Ignition Ovens and Types of Aggregate	1
Test Method for Determining the Quality of Bond Between Layers	1
Improved Evaluation of Longitudinal Joints	1

Responses to the question regarding the practicality and timeliness of the tests identified for the parameters indicate that most agencies find that the currently used tests are satisfactory for their present needs. However, some agencies commented that the sieve analysis and the determination of maximum theoretical specific gravity are time consuming. In addition, the later mentioned test (maximum theoretical specific gravity), was also very variable.

The main comments on the response forms are tabulated in the NCHRP 9-15 interim report as shown below.

1. There are problems associated with current methods used for test device validation and comparison to the established criteria (further explanation not provided by respondent).
2. There may not be enough evidence to define the difference between smooth and super smooth pavements.
3. The industry could utilize existing reference labs to identify candidates for improving tests.
4. The research community needs to work with the AASHTO subcommittee on materials and ASTM to identify test alternatives and areas that require improvement.
5. A national competition for improved test methods may spurn creative ideas.

6. Most tests allow too much dispersion on the operator's part for sample preparation which in turn causes variability.
7. All acceptance tests in PRS should be in-situ.
8. Research and development should be conducted to improve a test until it becomes practical, timely, and affordable.

NCHRP 9-11 FINAL REPORT

The report contains responses to a survey carried out by Williams et al. (PP30). The main items in the survey are related to current segregation specifications and guidelines, training to detect and minimize segregation, methods to quantify segregation, moisture sensitivity testing, and future interest in training to minimize segregation. The questions on the survey form and a summary of the responses are shown below.

Questions:

1. Does your agency have any specifications or guidelines for the prevention of segregation in hot-mix asphalt (HMA) during the phases of production and placement?
2. Does your agency train technicians in any trouble-shooting procedures to minimize segregation in the production and placement of quality HMA?
3. Does your agency make any attempts to quantify the degree of segregation (i.e., testing, visual evaluation) when it is known to exist?
4. Does your agency have a reduction in pay factor for stripping? If so, what is the basis for deciding the reduction?
5. Would your agency be interested in training material or presentations concerning procedures to minimize segregation in HMA production and placement?

Responses to the questionnaire are shown in Table A-26.

Table A-26. Responses to Questionnaire.

Question Number	Response Yes	Response No	Comments
1	30	13	♣ Extracted asphalt content and gradation (random-not specifically for visually segregated areas) ♣ Contractor requirement to prevent and correct segregation ♣ Inspectors located at HMA plant and paving sites and inspector training ♣ Specifications (Standard operating procedures, guidelines, and checklists) ♣ Require or eliminate specific equipment and construction practices ♣ Pay factor for density (in development) ♣ Change to smaller top size aggregate gradations ♣ Stockpiling requirements ♣ General statements that "segregation of the mixture will not be acceptable" or "roadway must be uniform and smooth"
2	37	6	♣ Both state and contractor technicians trained to minimize segregation during production, hauling, and placement ♣ Intermittent workshops conducted by consultants ♣ Various asphalt plant and paving technician certification courses ♣ On-the-job training ♣ District level training sessions
3	26	17	♣ Visual evaluation only (most frequent response) ♣ Selective sampling and testing for density, asphalt content and/or gradation ♣ Visual plus nuclear gauge readings
4	3	39	♣ Lottman-type testing during mix design ♣ Raveled sections after construction removed ♣ and replaced at contractor's expense
5	33	6	♦ Segregation has not been a problem ♦ Already offer various courses

♣ Comments included with "yes" answers.

♦ Comments included with "no" answers.

A summary based on supplemental information provided by the respondents is documented below.

1. Universal recognition of the importance of controlling segregation.
2. Visual identification is the most common method of identification of segregation, but this method is highly subjective.

3. In order to eliminate the subjective evaluation of the finished product, many state agencies specify “good construction practices.”
4. Many state agencies are exploring a number of tests to objectively identify segregation.
5. As none of the methods are adopted, it reflects that there is a lack of correlation between the results of various tests and segregation.

An international survey was also conducted. The responses are tabulated in Table A-27.

Table A-27. Responses to International Survey.

Country	Test Methods for Identification and Measurement	Methods to Minimize Segregation
Australia	1. Visual identification 2. Tests for surface smoothness as it reflects segregation 3. Density tests 4. Tensile strength tests	1. Use of MTD 2. Methods outlined in NCHRP Report 386 and AASHTO document on segregation when large stone mixtures are used 3. Limiting the use of mixes prone to segregation (maximum aggregate size < 20 mm).
Scandinavian countries, Switzerland, and Denmark	1. Infrared thermography 2. GPR	-
England, Finland, Belgium, Netherlands, and France	1. Surface friction measurements 2. Lab tests during mix design to determine the mix segregation potential is being evaluated	1. Limiting the use of mixes prone to segregation (maximum aggregate size < 20 mm)
New Zealand	1. Nuclear Density Gauges 2. Subjective testing by wetting the pavement surface	1. “Best practices” construction techniques are specified (ISO9002)
South Africa		1. “Best practices” construction techniques are specified

NCHRP 1-31

This study conducted a survey of the highway agencies (SHA) and contractors in order to document the current state specifications and procedures for measuring initial smoothness and to record the viewpoints of the contractor and the state highway agencies concerning initial smoothness specifications and smoothness measuring equipment.

The responses of this survey indicate that almost every SHA uses some form of initial smoothness specification. Twenty-eight SHAs reported that they use a ride specification and 19 SHAs use a bump specification for new AC pavements. Twenty-six SHAs use a ride specification and 20 SHAs use a bump specification for an AC overlay on an existing AC pavement. Nineteen SHAs reported that they use a ride specification for an AC overlay over an existing PCC pavement.

Twenty-one agencies reported that they had some form of an incentive/disincentive payment schedule for new AC pavements. Critical incentive limits for new AC pavement range from 3 to 7 in/mi. For disincentives, the range is 7-10 in/mi. Table A-28 indicates the breakdown of roughness measuring equipment used on new AC pavements and AC overlays.

Table A-28. Use of Roughness Measuring Devices.

Equipment	Percentage Using the Equipment
Non-contact inertial profilometers	3
Light weight non-contact inertial profilometers	1
California profilograph	28
Ames profilograph	12
Rainhart profilograph	1
Mays meter installed in vehicle	3
Mays meter mounted on trailer	7
Straight edge	41
Rolling straight edge	1
Straight line	3

AGENCY SPECIFICATION

Finally, as a conclusion to the literature review, it would be interesting to note the measures adopted by the user agencies to specify their acceptance criteria. This material would provide useful information regarding the various measures that are currently adopted in the agency specifications to ensure the quality of the final product is up to the mark. As an example of such specifications, Table A-29 presents a summary of the acceptance criteria specified by the Federal Aviation Administration (FAA) for construction-related properties such as air voids, mat density, joint density, thickness, and smoothness.

Table A-29. Acceptance Criteria.

Property	**Associated Standards**	**Percent Within Limits (PWL) for Complete Payment**	**Specification Tolerance**			
			Gross Wt. of Aircraft >60,000 Lbs. or Tire Pressure >100 psi.		**Gross Wt. of Aircraft <60,000 Lbs. or Tire Pressure <100 psi.**	
			Upper Limit	**Lower Limit**	**Upper Limit**	**Lower Limit**
Air Voids	ASTM D 3203 ASTM D 2726 ASTM D 1188 ASTM D 2041	90 or more	2.0	5.0	2.0	5.0
Mat Density	ASTM D 2726 ASTM D 1188	90 or more	96.3	-	96.3	-
Joint Density	ASTM D 2726 ASTM D 1188	90 or more	93.3	-	93.3	-
Thickness	Measured on core samples	To be determined by the engineer	NA	NA	NA	NA
Smoothness	Straight Edge	85*	NA	NA	NA	NA

*When more then 15 percent of all measurements within a lot exceed the specified tolerance, the contractor shall remove the deficient area and replace it with new material.

Note 1: The finished surfaces of the pavement shall not vary more than 1/4 inch for surface and 3/8 inch for base course.

Note 2: Quality control for in-place density–nuclear density gauge may be used in accordance with ASTM D 2950.

In addition to the FAA specifications, the Kansas specification for segregation was also reviewed, and a brief summary is presented in this section. The Kansas Department of Transportation (KDOT) adopts nuclear density profiles to detect segregation. If the laydown machine continues without any stops, the engineer determines the starting point for the profile. If the laydown machine stops, the zero point is located at the point where it stops. Profiling begins approximately 10 feet behind the screed (zero point). Density readings are taken every 5 feet along the longitudinal direction. The following stipulations are included in the specification:

- "When checking for truck load segregation, the longitudinal distance from centerline may vary, but not the transverse distance.
- When checking for longitudinal streaking, the longitudinal distance from centerline will vary. This is done so the profile will cross over the longitudinal streaks. Determine the transverse distance from centerline to the longitudinal segregation. Start the profile approximately 2 ft farther transversely than the center of the longitudinal streak. End the profile approximately 2 ft less transversely than the center of the longitudinal streak. The approximate distance (2 ft) from the center of the streak to start and end the profile will be determined by the engineer."

The minus #30 aggregate from the mix should be used to fill in any surface voids and three, one-minute readings are taken with the density gauge and averaged (none of the readings should vary by more than 1 lb/ft^3). For surface and base courses, the drop in density (average to lowest) must be less than 2.5 lbs/cu ft. The maximum density range (highest to lowest) must be less than 5.0 lbs/cu ft.