PART II:
PERFORMANCE-RELATED SPECIFICATION

CONTENTS

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Hot-mix asphalt (HMA) is a material that has been used extensively throughout the United States and the world as a reliable and cost-effective pavement surfacing for highways, streets, parking lots, and airfields. Despite its popularity, it is generally acknowledged that HMA is a sensitive material in that its performance can be greatly affected by many materials and construction (M&C) factors, as well as traffic and environmental variables.

Historically, HMA mix design and construction procedures have tended to be more of an art than a science. They have relied heavily upon the expertise of the engineer and mix designer and the experience and commitment to quality of the contractor. The advent of polymer and crumb rubber additives, as well as recycling, has made these activities more complex. For example, in the case of mix design, the introduction of the additives has called into question the adequacy of currently used test procedures and equipment.

1.1.1 Technology Advancements

During the last two decades, advances in HMA technology have occurred in several important areas including materials (particularly binders), mix design, and construction specifications. With regard to materials, the practice of including various polymers, crumb rubber, and other additives has helped address the temperature sensitivity issues associated with asphalt binders. In the area of mix design, there have been developments in the use of aggregate gradations which rely more on the aggregate structure for load distribution, for example, stone matrix. In addition, considerable emphasis was placed on mix design in the Strategic Highway Research Program (SHRP). The Superpave (*1*) mix design method proposed the use of new laboratory equipment, test procedures, and prediction models to better account for the individual and interactive effects of the following:

- Mix characteristics (including asphalt type, asphalt content, aggregate type, aggregate gradation, and compactive effort).
- Environment (temperature and moisture).
- Loading.

Its development is still in progress.

The third area of recent advancement in HMA technology has been in performance-related specification (PRS) development. This research has taken place under the sponsorship of both the National Cooperative Highway Research Program (NCHRP) and the Federal Highway Administration (FHWA). A major research project dealing with HMA pavement construction was accomplished by Shook et al. (*2*) in 1993. This study focused on the development of improved relationships between M&C factors and other performance-related measures of HMA quality. Since 1990, however, the most significant enhancements to the fundamental PRS technology have taken place under FHWA sponsorship by Darter et al. for portland cement concrete (PCC) pavements (see references 3, 4, 5, 6, and 7). By focusing on issues related to pay adjustment and field application, this research has produced a PRS model, *PaveSpec*, for PCC pavement construction that is both defensible and implementable (*7*).

1.1.2 Performance-Related Specifications

In a synthesis for NCHRP, Chamberlain (*8*) offered a diagram (Figure 1) which helps distinguish between the different levels of construction specifications that take pavement performance into consideration. On one level are specifications where the M&C factors used to control quality (also known as acceptance quality characteristics [AQCs]) are primarily tied to performance through intuition, engineering judgment, or both. Minimum density, for example, is an AQC used in a specification where the connection to better performance is intuitive, that is, higher densities generally mean better performance. Like many other types of specifications, there is a consequence to the contractor for not satisfying the specification. Typically, the problem would have to be corrected (to the satisfaction of the engineer), the defective pavement would have to be removed and replaced or the contractor would have to accept a reduced payment.

On the second level are specifications where the AQC is a measure of performance. These are specifications that describe how the finished product should perform over time. Examples of performance specifications are "no cracking after 10 years," "permanent deformation less than 10 mm after 5 years," and "PSI greater than 3.0 after 15 years." Because of the time factor involved with performance specifications, they have not, so far, been widely accepted in the United States. The ones that have been used typically take the form of warranty or guarantee specifications, under which the contractor agrees to build and maintain the pavement for a specified period of time.

In between these two levels are performance-related and performance-based specifications. A performance-based specification (PBS) is one in which the AQCs are tied to performance through prediction models and through various tests intended to measure fundamental engineering characteristics, for example, layer thickness, tensile strength, and shear strength; initial pavement condition, that is, smoothness; or both. It should be emphasized, however, that the selection of a particular AQC, by itself, does not make the specification performance-based. There must be the connection to performance through some valid empirical or mechanistic prediction model that accounts for the effect of deviations of the as-constructed AQC level from the as-designed AQC level. The difference in predicted performance between the as-designed and as-constructed pavement is then used as a basis for a contractor pay adjustment (PA). Also, depending on the as-constructed AQC level, the potential for no payment or removal and replacement still exists.

A PRS is similar to a PBS in that a performance prediction model (or models) is required and the effect of deviations of the as-constructed AQC level from its target, as-designed level are considered. However, an AQC is less directly tied to pavement performance in a PRS than in a PBS. Consequently, the prediction models used tend to be more empirical and secondary prediction relationships (between an AQC and a fundamental engineering property or some other predictor of performance) may be required to establish the link to performance. Examples of AQCs in a PRS for HMA pavement construction are asphalt content, air void content, and various aggregate gradation parameters.

When a pavement construction specification is prepared in which multiple AQCs are included (as is usually the case), the individual specifications associated with each AQC may vary from intuitive to performance-based. Although there are no rules for establishing the level of an overall specification containing multiple AQCs, the specification is best characterized on the basis of the predominance of the type of AQCs included. As will be described in Chapter 2, the *HMA Spec* software developed in this study primarily generates PRSs because of its emphasis on conventional AQCs such as asphalt content, air void content, and gradation.

The primary benefit of a PRS (or PBS) is that it permits engineers to prepare practical specifications for pavement construction that focus heavily on the M&C factors that have the most effect on long-term performance. By considering a multitude of costs associated with design, construction, and future performance of the pavement, a PRS provides an equitable means of rewarding or penalizing the contractor for the "as-constructed" pavement delivered. In the case of the comprehensive PRS involving multiple AQCs, the contractor on a given project may be penalized for not meeting the specification on one AQC (say average asphalt content below target by 0.2 percent) and rewarded for exceeding the requirement for another (say average HMA thickness above target by 10 mm [0.4 in.]). Assuming that the predicted performance of the pavement is dependent more on HMA thickness than on asphalt content, it is likely (for this simple example) that the net effect would be a slight reward for the contractor and some assurance for the client (highway agency) that its funds were spent well.

1.2 PROJECT OBJECTIVES

The official title of the WesTrack project was "Accelerated Field Test of Performance-Related Specifications for Hot-Mix Asphalt Construction." The contract was awarded in 1994 by the FHWA, Contract No. DTFH61-94-C-00004. In 1998, the Transportation Efficiency Act for the 21st Century (TEA-21) transferred much of the Highway Trust Fund support for research from the federal government to the states, necessitating NCHRP take over sponsorship of the project in its last year. In this transition, tasks were redefined and funding reallocated, but the primary objectives of the project remained the same. These were (1) to continue development of PRSs for HMA pavement construction by evaluating the impact on performance of deviations in M&C properties, for example, asphalt content, voids, and aggregate gradation from design values in a large-scale, accelerated field test and (2) to provide an early field verification of the Superpave mix design method.

Accomplishment of these objectives was achieved primarily through the construction, trafficking, monitoring, and performance evaluation of 34 experimental HMA pavement sections located on a 2.8-km (1.75-mi) closed loop test track. The track, located approximately 100 km (60 mi) southeast of Reno in the Nevada desert, was loaded in an accelerated fashion over a 2½-year period to achieve the equivalent of five million 80-kN (18-kip) single-axle load applications.

Following is a brief discussion of how the two objectives were satisfied.

1.2.1 Improved System of Performance-Related Specifications

The previous step in the evolution of PRS for HMA pavements was completed in the early 1990s by Shook and others (*2*). The study focused on the development of so-called secondary prediction relationships that helped establish the effect of certain M&C factors on pavement performance. It also resulted in a prototype system (in the form of a Microsoft Excel spreadsheet) that could be used to determine a contractor PA based on the quality of the pavement delivered. From a user's standpoint, the system had some significant weaknesses because it did not actually produce a specification nor did it adequately consider the effect of certain key M&C factors that are essential to quality construction.

By design, the WesTrack study produced a vast amount of data that can be analyzed and used to develop a variety of useful products and other significant findings. In terms of achieving the primary project objective, the key product of the WesTrack study is a prototype PRS for HMA pavement construction with the following attributes:

- Developed as a Microsoft Windows-based software package called *HMA Spec* (ready for beta testing).
- Capable of producing a specialized specification in document form.
- Compatible with the *PaveSpec* PRS software developed for rigid pavements.
- Capable of considering multiple M&C factors (including HMA surface thickness, initial smoothness, asphalt content, air void content, and aggregate gradation) for the purpose of evaluating quality.
- Uses models developed from WesTrack data analyses as a basis for better treating the effects of air void content, asphalt content, and gradation on predicted pavement performance.
- Uses life-cycle cost as the basis for contractor PAs that are considered defensible.
- Incorporates a statistically valid methodology to account for the effects of stochastic variability in both the as-designed and as-constructed pavement.
- Offers the flexibility of incorporating alternate models for pavement performance prediction.
- Can be used for both the state-of-the-practice and state-of-the art application levels.
- Designed for use at either the simple or sophisticated operating level.

Although the *HMA Spec* software is a major step forward in the development of PRS for HMA pavement construction, it should be emphasized that the program described here is a prototype and is not yet suitable for adoption in routine practice. As in the series of FHWA studies associated with the development of the PRS for PCC pavements, additional work is required to beta test *HMA Spec*, validate it as part of projects involving "shadow" specifications, and verify it in rigorous field trials (see references 3, 4, 5, 6, and 7).

1.2.2 Field Verification of *Superpave*

The intent of the original objective associated with field verification of the SHRP Superpave level III mix design procedure was for the WesTrack team to gather pavement performance data, extract and test suitable samples, and conduct an evaluation to help establish the procedure's validity. Unfortunately, in the 2 or 3 years following the commencement of the WesTrack project, researchers involved with the then FHWA-sponsored research project titled "Superpave Models—Development and Support" identified numerous flaws in the Superpave level III performance models and software which ultimately led to them being abandoned (*9*). With no level III method left to verify, work under this objective of the WesTrack project focused more on analyzing laboratory and field data to develop new performance models for Superpave or any other advanced HMA mix design procedure. However, the WesTrack project did investigate performance differences between coarse- and fine-graded mixes and reinforced the need for a simple performance test in the Superpave volumetric mix design method.

Although there are a number of ways to characterize pavement performance, for example, permanent deformation, fatigue cracking, low temperature cracking, roughness, and friction loss, the emphasis of WesTrack research was on permanent deformation and fatigue cracking. The reason for this was that the materials used for track construction, the asphalt cement and aggregate, were selected to minimize the adverse effects of low temperature cracking and moisture on pavement performance.

In addition to the distinction between models for the two key types of distress, it should also be pointed out that the models were developed at two basic levels: empirical (for use at the performance-related level) and mechanistic-empirical (M-E) (for use at the performance-based level). All models were developed for candidate use in the PRS software, *HMA Spec*, as well as for use in other advanced technology HMA mix design procedures.

1.3 DOCUMENTATION

Overall, this report for the WesTrack project is divided into four parts:

- Part I: Project Overview.
- Part II: Performance-Related Specification.
- Part III: WesTrack Database.
- Part IV: Observations and Lessons.

Part II describes the PRS for HMA pavement construction. It describes the development and use of the PRS and its accompanying *HMA Spec* software. Part I provides an overview of the accomplishment of the key WesTrack tasks and activities as well as the findings. Part III discusses the development and use of the WesTrack database, the primary record of WesTrack data for the future. Part IV provides a comprehensive summary of the observations and lessons in the $15-million, multifaceted study. These observations and lessons were not limited to the pavement performance aspects; they covered a wide array of topics including project management, trafficking, construction, pavement repair, laboratory testing, and public information.

Following is a description of the remaining chapters in Part II of this report. As indicated above, they describe in detail the development and application of the PRS for HMA pavement construction.

Chapter 2 provides an overview of the PRS. It discusses the general framework, explains the meaning of the system's two operating levels (basic versus advanced) and three application levels (state-of-the-practice, state-of-the-art, and future technology), describes the basic system output, and previews the component of the system that represents the advanced operating level, that is, the *HMA Spec* software.

Because one of the keys issues associated with PRS is relating a contractor's payment, that is, bonus or penalty, to the difference in predicted future performance between the as-designed and as-constructed pavement, Chapter 3 describes the concept and the details of the method of PA incorporated in the HMA PRS. This description is key to the understanding of how pay factors (PFs) are derived from measurements of the contractor's as-constructed quality levels. It is also essential to the understanding of the internal workings of the *HMA Spec* software.

Chapter 4 describes the pavement performance prediction models that were developed from WesTrack data for use in the PRS. These models are used for performance prediction in terms of two measures of pavement distress, fatigue cracking, and permanent deformation. They are primarily a function of HMA characteristics including asphalt content, air void content, aggregate gradation, and layer thickness.

Chapter 5 describes how life-cycle costs are calculated within the PRS to estimate the overall cost of both the as-designed and the as-constructed pavement. This description includes a discussion of how future maintenance and rehabilitation needs are determined and considered.

Chapters 6 through 9 provide information on the application and operation of the HMA PRS. Chapter 6 provides guidelines on determining the required PRS inputs. Chapter 7 provides guidelines for making decisions regarding PA based upon certain criteria. The PRS affects both the development of the construction specification and the determination of the final payment. Accordingly, Chapter 8 provides a step-by-step guide for generating the preconstruction output associated with the specification while Chapter 9 provides a step-by-step guide to determining the PAs for the as-constructed pavement lots.

Chapter 10 describes the development of a specification offered as a guide to state highway agencies (SHAs) which are considering a transition to PRS. Appendix C provides the actual Guide Specification. It should be noted that although the *HMA Spec* software permits the user to incorporate any specification, it currently uses the Guide Specification as a default.

The development of simplified PF relationships for use at the *basic* PRS operating level is describe in Chapter 11. These are the kind of relationships that are already being used by some SHAs in preparing PRSs for their pavement construction projects.

Finally, Chapter 12 provides a series of conclusions derived from the development of the HMA PRS as well as some recommendations for future research, field testing, and implementation.

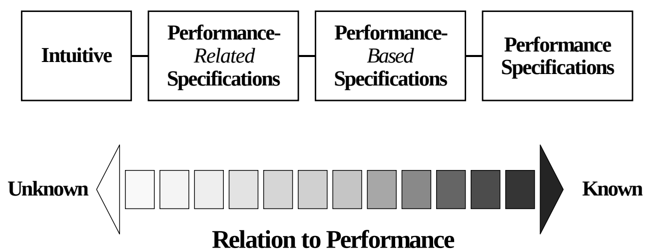


Figure 1. *Illustration of various levels of performance specifications (9).*

CHAPTER 2

PERFORMANCE-RELATED SPECIFICATION SYSTEM OVERVIEW

2.1 GENERAL FRAMEWORK

The PRS for HMA pavement construction developed in this project is considered a *system* because it incorporates a number of different models or components (working within a systems framework) to generate a specification for any given HMA pavement construction project. Because of its rigorous analytical nature and need for user interaction, the system is incorporated within a Microsoft Windows-based software package named *HMA Spec*. The computer code for the *HMA Spec* software was written in a modular fashion so that various key components can be interchanged or replaced in the future. For the easier changes (e.g., alternate performance prediction models or M&R decision criteria), features are available within the user interface that permit the user to specify a new or different model. For more sophisticated changes (e.g., revised overlay design methodology or cost model), individual modules (or subroutines) within the program may be rewritten and replaced. Considering the varying needs of the potential users of *HMA Spec*, a systems approach to PRS development was considered essential.

The major modules in the *HMA Spec* software are the following:

- Graphical User Interface (GUI)—This is a Microsoft Windows-based user interface that permits input of new project data and access to standard or default data for PRS operation.
- Control Module—This controls the step-by-step sequences for the two key paths within the program. It also performs the required Monte Carlo simulations and calculates the PA.
- Performance Prediction Module(s)—These are the various pavement performance prediction relationships used to forecast the future pavement distress state (in terms of permanent deformation, fatigue cracking, roughness, etc.) and trigger M&R activities for both the as-designed and as-constructed pavement.
- Life-Cycle Cost (LCC) Module—This model estimates future M&R costs for both as-designed and as-constructed pavements. The current LCC submodels include maintenance cost and rehabilitation cost. The program is amenable to the incorporation of future submodels to consider user costs and salvage value.
- M&R Module—This module characterizes the distress decision criteria (trigger values) as well as their related M&R treatments.
- Rehabilitation Design Module—This component is used to estimate the future rehabilitation needs (i.e., AASHTO structural number required for HMA overlay) of any given project. The current PRS methodology permits only one rehabilitation action during the analysis period. That rehabilitation is designed and evaluated as if it would last the remainder of the analysis period.
- PA Module—This module estimates the PA to the contractor's bid price to account for any strengths or deficiencies in the simulated as-constructed pavement section. The method is applicable to the as-designed and as-constructed pavement and requires a Monte Carlo simulation process. The details of this methodology are presented in Chapter 3.
- Guide Specification Module—This represents the default information needed to generate a comprehensive HMA pavement construction specification. The module does provide for direct interaction with the user.
- Stochastic Variability Module—This feature provides some statistical leeway to both the contractor and the owner agency so that neither a penalty or nor a bonus would be assessed if the predicted performance of the as-constructed pavement is within an acceptable range of the as-designed pavement.

The *HMA Spec* software performs two important functions associated with PRS. The first is the development of an actual construction specification that can be used by a contractor to prepare a bid and, if successful, carry out the work required in a manner that will be satisfactory to the client. The specification document produced by the program may be very similar to more traditional specifications, especially for M&C factors that are considered less performance-related and typically monitored through conventional quality control (QC) means. The specification will differ, however, for those key M&C factors that are considered performance-related. For these factors (also referred to as AQCs), the specification will provide the contractor with equations that define how payment will be adjusted when target values are not met (or exceeded). Analysis of these PA equations should provide the contractor with a sense of how much effort and attention should be devoted to achieving the target values.

The second important function of the *HMA Spec* software is the calculation of the final PA for any given pavement lot constructed along the project. Obviously, this function is exercised during or after pavement construction. Unlike the PA equations provided in the specification, this branch of the system provides the added feature of considering the inherent stochastic variability and uncertainty of the PA. This feature allows the agency and contractor to agree upon a range in predicted performance under which neither a bonus nor a penalty would be assessed.

2.2 OPERATING LEVELS

There are two primary operating levels within the PRS. The *basic* operating level is one that permits agencies to adopt a key output of the PRS, that is, a specific PF relationship, and incorporate it into its current method of specifications development. By comparison, the *advanced* operating level is more rigorous and is encompassed in large part by the *HMA Spec* software. It incorporates user-friendliness and other help features typically associated with Microsoft Windows software; however, its sophistication does require an advanced level of understanding by the user for proper operation.

New Jersey Department of Transportation (NJDOT) is an example of an SHA that uses a version of the basic approach in developing its specifications. Weed and others (*10*) applied a combination of statistical analysis and engineering judgment to develop a relationship which is intended to adjust the contractor's payment based upon how well the as-constructed levels of HMA surface thickness, asphalt content, and initial smoothness met their target values. This PF relationship is incorporated into the preconstruction specification for the contractor to consider in preparing its bid. It is also used to calculate the PA for each pavement lot after construction.

The PF relationship associated with the basic operating level in the PRS developed in this project is very similar to the NJDOT model, especially in how the relationship is used. The difference is in how the relationship is derived. Rather than relying solely on statistical analysis and engineering judgment, the *HMA Spec* software is used to derive one (or more) customized relationship(s) for a given environment, traffic level, and pavement structure. Also, by analyzing the output of the program, the PF relationship(s) is derived considering both the main effects and the interactions of the AQCs which have the greatest effect on pavement performance.

Chapter 11 of Part II provides an example of the development of a customized PF relationship for use at the basic operating level.

2.3 APPLICATION LEVELS

Application level is the term used to describe the level of "performance-relatedness" of the prediction models used in the PRS system. Recalling the definition offered for performance-type specifications in Chapter 1 of Part II, level 1 refers to the type of performance prediction models that are used in a performance-*related* specification. In addition, level 1 encompasses the types of sampling and testing procedures that are considered state-of-the-practice (e.g., for example, asphalt content, air void content, and gradation).

Application level 2 refers to the type of prediction models that would be used in a performance-*based* specification. These models are more mechanistic and dependent on fundamental engineering properties. Thus, level 2 encompasses the kinds of sampling and testing procedures that would be considered state-of-the-art (e.g., tensile strength, shear strength, and elastic modulus).

Table 1 provides a matrix illustrating the different application levels and their associated characteristics. This table also makes room for a third application level (level 3) which is intended for future development.

The emphasis of the research and development on this project has been on level 1 prediction models and the associated PRS. Consequently, level 1 prediction models are included for fatigue cracking and permanent deformation. However, some meaningful work was directed at the development of level 2 prediction models for use in a more performance-based specification. Chapter 4 of Part II describes the development of the level 1 and level 2 prediction models.

2.4 SYSTEM OUTPUT

The outputs of the PRS system can be divided into three categories. The first two are used in the preconstruction process, while the third is used after construction.

- Specification—For the advanced operating level, the *HMA Spec* software produces a complete construction specification for an HMA pavement in the form of an actual document. A Guide Specification is incorporated into the program for use as a default (see Chapter 10 of Part II and Appendix C). However, the user has the flexibility to copy and modify this specification or include a totally new one as an alternative (or as the default). The choice of the most appropriate specification must be made by the agency on the basis of engineering judgment.

  For the basic operating level, the user has the option of extracting and adapting the Guide Specification. However, in this scenario, the agency is more likely to retain the use of its current specification.
- Preconstruction PF Relationship—This relationship is derived through a rigorous Monte Carlo simulation process involving the predicted performance of the

as-designed and as-constructed pavements. By evaluating the sensitivity of this relationship, both the contractor and the highway agency can focus attention on the M&C factors that have the greatest effect on pavement performance.

At the advanced PRS operating level, the *HMA Spec* software has the capability of producing an individual PF relationship for any given project. For the basic level, a standard relationship associated with a given environment, traffic level, or pavement structure is used.

- Contractor Bonus/Penalty—The third and final key output of the PRS system is the final PA for any given pavement construction project. For the *HMA Spec* software, this is calculated on a lot-by-lot basis based on the measured quality levels of the as-constructed pavement. One of the features of the technology in the *HMA Spec* software is its ability to address variability in measurement and predicted performance. Overall, this feature provides an equitable basis for the agency and contractor to agree on a range of predicted performance (between the as-designed and as-constructed pavement) in which no PA would be made.

  For the case of the basic approach, the PF relationship used in the preconstruction specification would also be used to define the PA for the as-constructed lots.

Another desirable feature of a PRS system is the ability to produce operating characteristic (OC) curves. OC curves are plots of the probability of acceptance (at a given pay level) versus the mean of a given AQC. They are sensitive to the sampling and testing plan, particularly in terms of the number of sublots and the number of samples per sublot. Thus, they can provide the highway agency and the contractor with information regarding their respective risks (*7*). Unfortunately, the available time and resources did not permit the incorporation of this feature into the *HMA Spec* software.

2.5 *HMA SPEC* SOFTWARE

The *HMA Spec* software is a Microsoft Windows-based program designed to generate PRSs for HMA pavements. More specifically, *HMA Spec* was developed as a tool to assist SHAs in determining appropriate PAs to a contractor's bid price based on the difference in predicted LCCs of the as-designed and as-constructed pavement lots. Figure 2 shows the two general operating modes of the *HMA Spec* software used to accomplish this.

HMA Spec was developed using standard Windows features such as menus, buttons, list boxes, and drop-down lists, referred to as controls. Figure 3 is a screen capture from the *HMA Spec* software showing various standard Windows controls. As indicated in this screen capture, the forms contain various controls including a menu, various buttons, a tab control, and a tree view control.

Figure 3 also indicates that the *HMA Spec* software was designed and developed to accommodate multiple projects and multiple specifications for each project. *HMA Spec* creates a Microsoft Access database for each project and the specifications for a given project are records within the project database. The only limiting factor to the number of projects and specifications that can be accommodated is disk space on the computer on which *HMA Spec* is installed.

Once a project database has been created, developing a specification within a project is accomplished through a *specification wizard*. This consists of a series of forms that guide the user of the *HMA Spec* software through the specification generation process, that is, the preconstruction branch of the flow chart shown in Figure 2. The output from this mode of operation is a specification document and the preconstruction PF relationship. Generation of this latter output is accomplished through a Monte Carlo simulation process.

Following generation of a specification, postconstruction assessment of the as-constructed pavement lot is accomplished by entering the means and standard deviations for each included AQC. The *HMA Spec* software allows entry of these values directly or values for the individual test results can be entered and the software will calculate the means and standard deviations. Once entered, the Monte Carlo simulation process is executed in the same manner as in the preconstruction mode of operations except that the values entered for the as-constructed pavement AQCs are used. The PF for the pavement lot is then determined based on the difference in predicted mean LCCs of the as-designed and as-constructed pavement lots.

WesTrack Technical Report NCE-9 (*11*) is a user's guide for the *HMA Spec* software. However, it does not provide details regarding the inner workings of the software, it merely describes how to use the software. Chapters 3 through 9 of Part II provide the details regarding the methodology of the PRS embodied in the *HMA Spec* software.

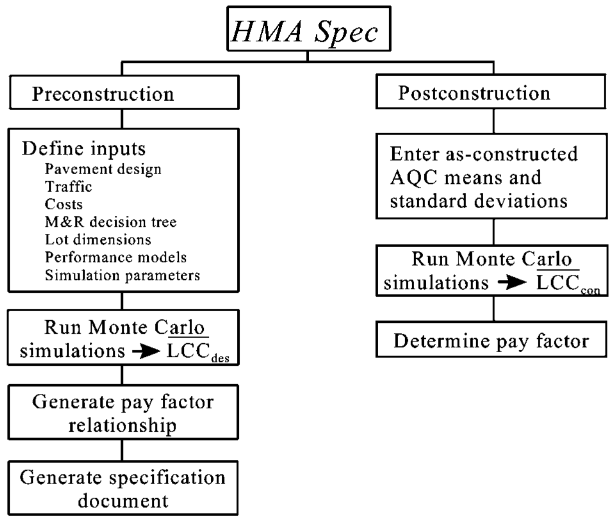


Figure 2. Flow chart showing general operating modes of the HMA Spec software.

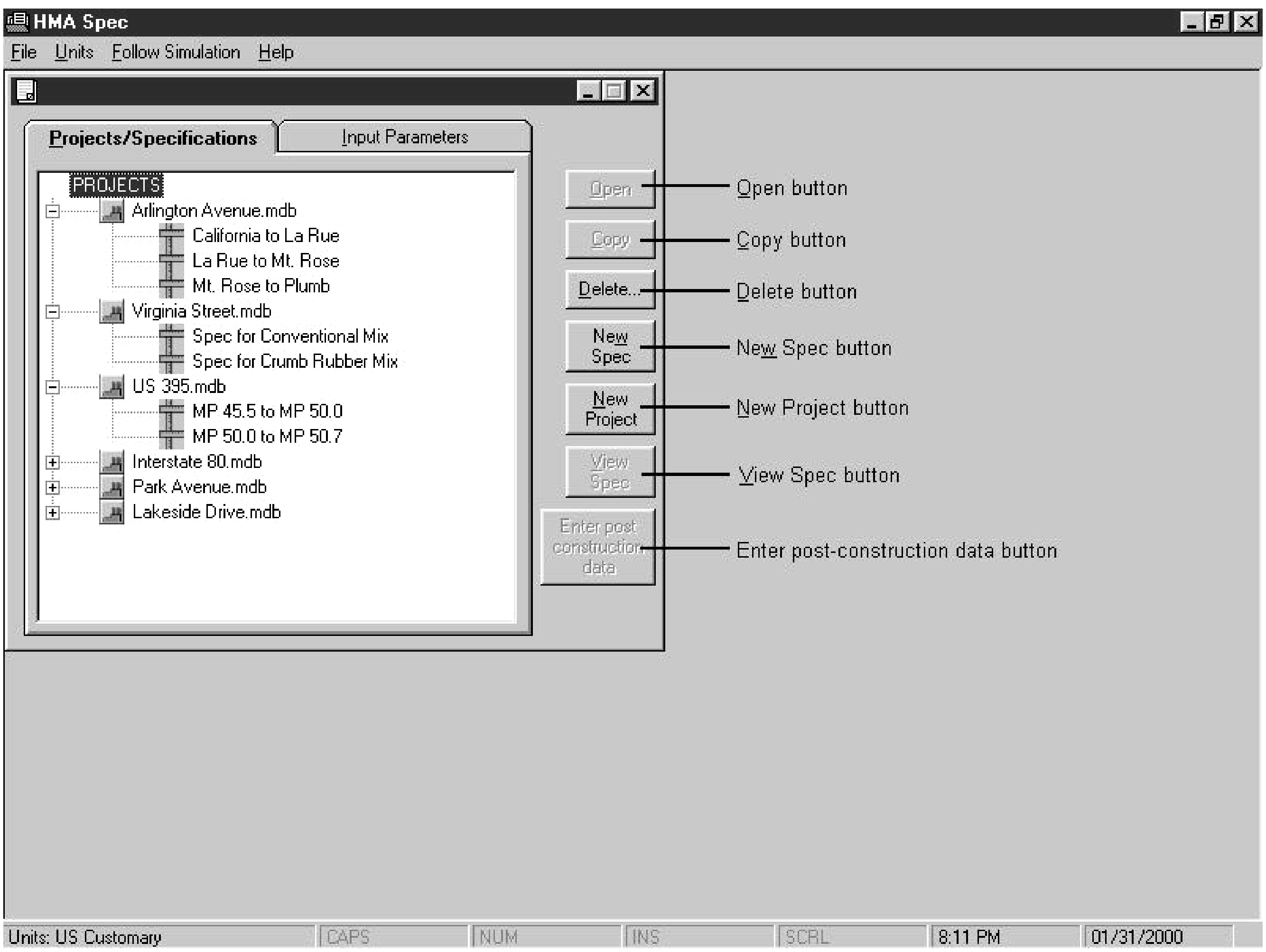


Figure 3. HMA Spec screen capture showing various standard Windows features.

TABLE 1   Application level matrix

Level	Model Basis	AQC Test Measure: HMA Surface Thickness	Initial Smoothness	Air Void Content	Asphalt Content	Gradation (parameters)	Simple Shear Test (parameters)	Beam Fatigue (parameters)	HMA Stiffness/Modulus	Thermal Stress Restrained Specimen Test	Other ?
1. State-of-the-Practice	Empirical	✓	✓	✓	✓	✓					
2. State-of-the-Art	Empirical Mechanistic	✓	✓	?			✓	✓	✓	✓	?
3. Future Technology	Mechanistic Empirical	✓	✓				✓	✓	✓	✓	

CHAPTER 3

METHOD OF PAY ADJUSTMENT

3.1 LIFE-CYCLE COST AS THE BASIS FOR PAY ADJUSTMENT

The fundamental basis for PA in a comprehensive PRS should be the difference between the predicted LCC of the as-designed (target) pavement and the predicted LCC of the as-constructed pavement. The major benefit of this approach is that it provides a rational, defensible process through which an adjustment can be made to a contractor's payment based on the effect that M&C quality have on the estimated cost of maintaining (and using) the pavement in the future. The concept of using LCC as a basis for PA in a PRS was first demonstrated (for asphalt concrete [AC] pavements) by researchers at Pennsylvania State University under NCHRP Project 10-26 in the late 1980s (*12*). The concept continued to evolve through research by ARE Inc. in furthering the development of PRS for asphalt pavements for the FHWA (*2*) and by ERES Consultants for the FHWA in the development of PRS for rigid pavements (*3*, *7*). By relating key M&C factors (e.g., thickness, smoothness, asphalt content, and air void content) to performance, LCC, and ultimately payment, an incentive-disincentive process is established whereby material suppliers and contractors are induced to achieve better quality M&C. The only real disadvantage of the LCC-based approach to PA is that it is unfamiliar and complex, particularly when construction variability and uncertainty are also considered.

This chapter describes, in detail, the method by which adjustments to contractor payment are made within the PRS for hot-mix asphalt (HMA) pavement construction developed under this project. It should be noted that this method is very similar to the approach used in the latest rigid pavement PRS (*7*); however, there are some meaningful differences in how the various M&C factors are quantified and in how the risk and uncertainty are evaluated.

There are three other points worth noting before providing the detailed description of the method of PA:

- PF is defined as the proportion of the contractor's original bid price that will be paid depending on the quality of the in-place pavement. A PF greater than 1.0 means that the contractor delivered a better quality product than that specified and may be entitled to a bonus. A PF less than 1.0 means that the contractor delivered a poorer quality product than that specified and is, therefore, likely to be assessed a penalty. PF is determined directly from the contractor's bid price and the calculated difference in LCC between the as-designed and as-constructed pavement.
- AQCs refer to those key M&C factors that have the most effect on pavement performance and are, therefore, used to evaluate quality and determine PF. Example AQCs that can be used in the PRS for HMA pavement construction include HMA thickness, initial smoothness, asphalt content, air void content, and aggregate gradation.
- There are two facets of PF assessment that should be understood: The first (discussed in Section 3.3) pertains to information that must be supplied to the contractor as part of the construction specification and that apprises him or her of the sensitivity of PF to the different AQCs. This helps focus the contractor's attention on those factors that can maximize performance. The second (discussed in Section 3.4) provides the details of the final PA for the as-constructed pavement lot. This takes into consideration the deviation of the means (averages) of the as-constructed M&C factors from the as-designed (target) M&C factors specified. In addition, it considers the effect of greater variability in these factors than expected.

3.2 COMPONENT MODELS

The analytical engine within the PRS contains a number of models to determine the LCC associated with both the as-designed (target) and the as-constructed pavement lots. This section provides a brief description of these models; detailed discussions are provided in Chapters 4 and 5.

3.2.1 Life-Cycle Cost Model

The conceptual framework for the LCC model is shown graphically in Figure 4. The figure indicates that the model comprises five main components as follows:

- Inputs.
- Pavement performance prediction relationships.

- M&R decision trees.
- Estimation of future costs.
- Computation of the LCC.

Inputs into the model include those factors that influence pavement performance and the computation of the net present value of future costs associated with M&R activities performed during the duration of the LCC analysis period. Inputs that influence pavement performance include M&C factors, environmental factors, traffic, and base course and roadbed soil characteristics. Inputs that influence the net present value of future costs associated with M&R treatments include the actual costs for the treatments, as well as factors to account for the time value of money.

It must be emphasized that many inputs into the LCC model are used to characterize the prevailing conditions and, therefore, are not candidates for control during construction. Weather (environmental conditions), roadbed soil characteristics (which are influenced by environmental conditions), and traffic are examples of these uncontrollable factors. Despite the fact that they can have a significant influence on performance, they are treated as constants for PRS application because they are beyond the control of the contractor. As constants, they have the same effect on the predicted performance of the as-designed and as-constructed pavements.

There are certain inputs, however, that can be directly controlled by the contractor. In particular, these include M&C factors. For example, the base course material within a pavement structure can be supplied to meet a specified gradation and compacted to meet a specified relative density given reasonable tolerances about the specified target values. Similarly, the HMA mat can be constructed to a specified thickness, aggregate gradation, binder content, and air void content given reasonable tolerances about the specified target values for each of these material characteristics. Given that only certain inputs into the LCC model can be controlled by the contractor, it is logical that PAs be based on only the factors under direct control of the contractor.

The LCC model uses the inputs to predict pavement performance in terms of one or more pavement distresses. The distress levels are predicted annually and used as input to an M&R decision tree to trigger an appropriate treatment (either a rehabilitation or "do nothing"). The future cost of the treatments and their timing are used to calculate the net present value of all future costs for a given set of economic parameters. These values are then summed to calculate the LCC for the as-designed and as-constructed pavement.

The current PRS model applies one major rehabilitation during the analysis period; thus, the effect of the contractor's efforts on construction QC on his or her payment is determined primarily by the difference in rehabilitation need between the as-designed and the as-constructed pavement. Further discussion regarding pavement performance prediction models and M&R decision trees is provided in Sections 3.2.2 and 3.2.3 of Part II, respectively. Chapter 5 of Part II provides a more detailed description of the LCC model.

3.2.2 Pavement Performance Prediction Models

The prediction of pavement performance as determined by the occurrence of distresses such as fatigue cracking, rutting, and serviceability loss (see block labeled Pavement Performance Prediction Models in Figure 4) is a key element in the LCC model and is, therefore, key to development of PFs. A number of models to predict the occurrence of distress have been developed in the past. However, none are considered adequate in accounting for the effects of asphalt content, air void content, or mixture gradation in the PRS for HMA pavement construction (*14*). Consequently, substantial effort was expended to develop improved models based on the information derived from the WesTrack project. Efforts have concentrated on models to predict fatigue cracking, rutting, and roughness increase. In addition, for the fatigue cracking and rutting models, two levels of sophistication were employed in their development and are referred to as level 1 and level 2 models. Level 1 models are derived from regression analyses of materials, traffic, and performance data obtained from WesTrack and, hence, are empirical in nature. Level 2 models involve use of laboratory tests and layered elastic analyses calibrated to the WesTrack performance data and, thus, are M-E in nature.

Chapter 4 of Part II provides detailed descriptions of the models developed for use in the PRS for HMA pavements.

3.2.3 Maintenance and Rehabilitation Decision Trees

Another key component of the LCC model is the M&R decision tree. This component of the model takes the output from the pavement performance prediction models and determines the type of treatment required (if any) to repair the predicted distress or distresses. An example of a simple M&R decision tree is shown in Figure 5. In this tree, decisions are made based on the amount of fatigue cracking and the magnitude of rutting, both of which are predicted by the pavement performance prediction models. Thus, for example, if the fatigue cracking model predicted (for a given amount of traffic) that 3 percent of the pavement section under consideration had fatigue cracking and the rutting model predicted (for the same amount of traffic) that the pavement section had 4-mm (0.15-in.) deep ruts in the wheelpaths, then the treatment would be "do nothing." If, on the other hand, the fatigue cracking model predicted the same amount of cracking but the rutting model predicted that the wheelpaths were rutted to a depth of 7 mm (0.3 in.) (for the same amount of traffic), then the treatment would be 50-mm "(2-in.) mill-and-fill."

The cost for the M&R treatment is calculated knowing the type, quantity, and the unit cost of the treatment. Also, knowing the year of service in which the treatment is applied allows the calculation of the net present value of the treatment given certain economic parameters.

Chapter 5 of Part II provides further details regarding M&R decision trees.

3.3 PAY FACTOR RELATIONSHIP NEEDED FOR HOT-MIX ASPHALT SPECIFICATION

If a contractor's final payment will be affected by the individual levels of M&C quality ultimately achieved, then it is essential that information in the construction specification indicate what the effect will be. The approach used in the latest PRS for rigid pavement construction (*7*) involves the preparation of an approximate PF relationship that conveys to the contractor the effect that deviations from the target AQC specifications will have on payment. This relationship, in turn, provides the contractor with a sense of where to place emphasis on M&C quality. The rigid pavement PF relationship is considered to be approximate because it ignores the effect of interactions amongst the AQCs and because it assumes the contractor will achieve the target construction variabilities (standard deviations) for each AQC. The approach used in preparing the preliminary PF relationship for the HMA specification is similar to that for rigid pavements (*7*); however, some steps were taken to help make it less approximate.

3.3.1 Description of the Process

Figure 6 illustrates the basic step-by-step process associated with generating the preliminary PF relationship for the HMA specification. It is important to emphasize that the relationship that results from this process is project-specific, that is, it is applicable only to the construction project under consideration.

Step 1: Supply Required Inputs. To complete the overall process, certain information related to the as-designed pavement must be supplied. This information includes the following:

- Target AQC means and standard deviations for the HMA layer (e.g., layer thickness, initial smoothness, asphalt content, air void content, and aggregate gradation parameters).
- As-designed levels of other M&C factors needed to predict pavement performance (e.g., base/subbase course thicknesses and resilient moduli).
- Design traffic, that is, initial ADT, initial 80-kN (18 kip) ESAL applications, and growth rates.
- Design soil strength/stiffness (e.g., resilient modulus, CBR, and R-value).
- M&R plan (i.e., decision tree and associated distress trigger levels).
- Cost data for M&R actions and the time value of money.

Step 2: Estimate the Mean Life-Cycle Cost for the As-Designed Pavement. The fact that some variability in the construction process is permissible means that the process for estimating the LCC of the as-designed pavement (using the models described above) is not as simple as entering the as-designed (target) values of the AQCs and other design-related factors and calculating the associated LCC. Instead, an iterative process involving hundreds (or thousands) of individual LCC observations is required to determine the mean and standard deviation of the LCC for the as-designed pavement. In each iteration, a single combination is generated by randomly extracting values for the AQCs from their distributions. For each of potentially hundreds (or thousands) of combinations, an LCC is calculated using the models described in Section 3.3 of Part II and the additional data available from step 1. From the resulting list, the mean ($\overline{LCC}_{des}$) and standard deviation (σ_{LCCdes}) of the as-designed LCC are calculated.

Table 2 provides an example of an actual Monte Carlo simulation generated by the *HMA Spec* software. It is evident from this table that different combinations of AQC levels generate different LCCs. The mean LCC and the standard deviation of the LCC distribution are shown at the bottom of Table 2. Also note that the AQC means and standard deviations are very close to the target values using 500 iterations.

Step 3: Calculate the Pay Adjustments for a Simulated Distribution of As-Constructed Pavements. Because the overall goal of this process is to develop a preliminary PF relationship for inclusion in the HMA specification, it is necessary to simulate a distribution of potential as-constructed pavements and calculate the LCC (and associated PA and PF for each). For purposes of creating a database upon which to derive this preliminary PF relationship, it is reasonable to assume that the contractor would achieve the target means and standard deviations for each AQC identified. Accordingly, another iterative process is executed in which possible combinations of the as-constructed pavement are created by randomly sampling from the as-constructed distributions of the different AQCs. For each as-constructed combination, the associated LCC, $(LCC_{con})_i$, is determined using the same models and information as was used for the as-designed pavement combinations. These are the individual LCCs generated during the Monte Carlo simulation process shown in the rightmost column in Table 2. Also during each iteration, the associated pay adjustment (PA_i) and pay factor (PF_i) are calculated using the following equations:

$$PA_i = \overline{LCC}_{des} - (LCC_{con})_i$$

$$PF_i = 1 + (PA_i/BP)$$

where

$\overline{LCC}_{des}$ = mean LCC of the as-designed pavement (from step 2), and
BP = estimated bid price.

Step 4: Determine the "z-value" for Each AQC and Each Combination. The statistic, z, is used in connection with the binomial (or normal) distribution. It represents "a measure of the deviation in terms of the standard deviation or in so-called *standard units*. The expression (given by the relationship below) is also frequently referred to as the *normal deviate*" (*13*):

$$z = (X - m)/\sigma$$

where

X = a given point along the X-axis of the distribution,
m = mean value of the distribution, and
σ = standard deviation of the distribution.

For purposes of its application to PA, z represents a measure of the deviation of an as-constructed AQC from its as-designed mean value:

$$z = (ACQ_{con} - \overline{AQC}_{des})/\sigma_{des}$$

where

AQC_{con} = as-constructed value for a given AQC,
$\overline{AQC}_{des}$ = as-designed mean value for the AQC being considered, and
σ_{des} = standard deviation of the as-designed AQC distribution.

The determination of a z-value is depicted in Figure 7 where the AQC is represented by asphalt content (P_{asp}).

This approach was used to normalize the deviation of the as-constructed AQC from its as-designed (target) value on the basis of the design (target) standard deviation. The z-values will become the independent variables in the preliminary PF relationship. The z-term was selected over two other candidates, percent defective (PD) and percent within limits (PWL), because it indicates whether the AQC_{con} is above or below the as-designed target. (This capability is essential since any particular AQC can have a different effect on LCC, depending on whether it is above or below the target). Table 2 shows the z-values for the four AQCs included in the Monte Carlo simulation example. Note that, for brevity, only the z-values for main effects are shown. That is, although the *HMA Spec* software generates z-values for second order terms and two-factor interactions, these were excluded from the sample data shown in Table 2 due to the large number of these factors.

Step 5: Perform Regression Analysis. With the type and quantity of data generated through simulation in the previous steps, it is possible to develop a statistically valid relationship for PF through multiple regression analysis. The methodology employed here involves the use of the *general linear model* (GLM). Following is the basic form of the GLM in which the independent variables are represented by the z-values for each AQC.

$$\begin{aligned}PF = &+ a_0 + a_1{*}z_{TH} + a_2{*}z_{SM} + a_3{*}z_{P_{asp}} + a_4{*}z_{V_{air}} \\ &+ a_5{*}z_{P200} + a_{11}{*}\,(z_{TH})^2 + a_{12}{*}\,z_{TH}{*}z_{SM} \\ &+ a_{13}{*}\,z_{TH}{*}z_{P_{asp}} + a_{14}{*}\,z_{TH}{*}z_{V_{air}} \\ &+ a_{15}{*}\,z_{TH}{*}z_{\,P200} + a_{22}{*}\,(z_{SM})^2 + a_{23}{*}\,z_{SM}{*}z_{P_{asp}} \\ &+ a_{24}{*}\,z_{SM}{*}z_{V_{air}} + a_{25}{*}\,z_{SM}{*}z_{\,P200} + a_{33}{*}\,(z_{P_{asp}})^2 \\ &+ a_{34}{*}\,z_{P_{asp}}{*}z_{V_{air}} + a_{35}{*}\,z_{P_{asp}}{*}z_{\,P200} + a_{44}{*}\,(z_{V_{air}})^2 \\ &+ a_{45}{*}\,z_{V_{air}}{*}z_{\,P200} + a_{55}{*}\,(z_{P200})^2 + e\end{aligned} \tag{1}$$

where

PF = dependent variable (pay factor),
z_{TH} = z-value representing the deviation in HMA thickness,
z_{SM} = z-value representing the deviation smoothness,
$z_{P_{asp}}$ = z-value representing the deviation in asphalt content,
$z_{V_{air}}$ = z-value representing the deviation in air void content,
z_{P200} = z-value representing the deviation in percent passing the 0.075 mm (#200) sieve,
a_{jk} = coefficients developed from regression analysis, and
e = error or lack-of-fit of the model.

The first line of the equation shows the terms representing all the main effects while the remaining lines show all the possible two-factor interactions (including the squared main effects). Given that the number of combinations will be in the hundreds (possibly thousands), it would even be possible to include higher-order (three-, four-, and five-factor) interactions if it is appears that they can significantly improve predictive accuracy of the model. In general, no more than two-factor interactions are necessary.

It is important to note that not all the terms identified above will end up in the model. Analysis of variance (ANOVA) is used to identify specific terms in the PF equation which explain most of the variability observed in the PF-z data combinations. These are the terms that are included in the regression analysis to calculate the individual a-coefficients and produce the preliminary PF relationship. Equation 2 is an example of one particular *project-specific* relationship that resulted from this process. It was generated from the same Monte Carlo simulation example that generated the data shown in Table 2. However, note that equation 2 includes the second order terms and two-factor interactions.

$$\begin{aligned}PF = 1.0087 &- 0.11877{*}z_{V_{air}} + 0.13160{*}z_{TH} \\ &- 0.00126{*}z_{P200} + 0.34845{*}z_{P_{asp}} - 0.00077{*}(z_{V_{air}})^2 \\ &- 0.00391{*}(z_{TH})^2 + 0.00072{*}(z_{P200})^2 \\ &- 0.00547{*}(z_{P_{asp}})^2 + 0.00573{*}z_{V_{air}}{*}z_{P_{asp}} \\ &+ 0.00340{*}z_{V_{air}}{*}z_{TH} + 0.00305{*}z_{V_{air}}{*}z_{P200} \\ &- 0.00831{*}z_{TH}{*}z_{P_{asp}} - 0.00130{*}z_{TH}{*}z_{P200} \\ &- 0.00395{*}z_{P_{asp}}{*}z_{P200}\end{aligned} \tag{2}$$

It should also be pointed out that the preliminary PF relationship generated by the *HMA Spec* software does not involve the use of ANOVA to identify the most significant terms. The process of ANOVA is considered too complicated to have been included in the software. Accordingly, the *HMA Spec* software includes all independent variables and all two-factor interactions in the model even though an ANOVA might consider some of these as being insignificant. Thus, the software only determines the coefficients of the equation using a standard regression analysis process.

3.3.2 Sensitivity Issues

Because the terms of the preliminary PF relationship were normalized through the use of the z-values, the coefficients on the main effects do give some indication of their relative impact on PF determination. Thus, the larger the absolute value of the coefficient, the greater the effect it will have on PF assessment (and the more attention it should receive during construction). If the relationship includes higher-order interactions between the AQCs, then the issue of understanding sensitivity becomes more complicated.

There are two other ways of examining the sensitivity of the PF to the various AQCs. One is to use the preliminary PF relationship to prepare a large PF table. This can be done with the aid of a computer spreadsheet. By covering the range of all the different AQCs and studying the results, it is possible to determine which AQCs have the greatest influence. A second approach is to use the preliminary PF relationship to prepare a chart or nomograph that graphically relates the effect of the AQCs to the estimated PF. Preparation of the nomograph can be cumbersome (especially when higher-order interactions are involved); however, the ability to examine the effects visually may make it worthwhile.

3.3.3 Consideration of Risk

Whenever a PF is assessed, there is a finite, statistical possibility that a bonus (or penalty) will be applied (or assessed) when one is not deserved. This is a risk that both the client and the contractor accept when entering into an agreement for construction when uncertainty is involved. In the method of PA described here, the goal is to quantify the risk and establish a fair process by which both parties share equal risk. The overall issue of risk is a byproduct of uncertainty in many of the different facets of construction, sampling and testing, prediction model accuracy and economic data quality. Its treatment for purposes of PA from a statistical standpoint is addressed as part of the step-wise process described in the next section.

3.4 POSTCONSTRUCTION PAY FACTOR ASSESSMENT

The purpose of preparing a preliminary PF relationship is to provide information within the specification that would indicate to the contractor how his or her payment would be affected by deviations in the M&C quality from those specified. Although the process allows for the consideration of significant interactions amongst the AQCs, the resulting relationship is still only adequate for its intended purpose. Significantly, the preconstruction process assumes the as-constructed variability will be the same as the as-designed (target) variability. During and after construction, with actual information on variability of M&C gathered as part of quality control/quality assurance (QC/QA) sampling and testing, the assumption that the as-constructed variabilities are equal to the as-designed (target) variabilities is both unnecessary and invalid.

3.4.1 Description of the Process

Figure 8 illustrates the step-by-step process for postconstruction PF assessment for a given lot. Some of the steps are similar, if not identical, to those described in developing the preliminary PF relationship (Section 3.3).

Step 1: Supply Required Inputs. This step is almost the same as the step 1 described in Section 3.3 for the as-designed pavement. The one major difference is that information pertaining to the as-constructed means and standard deviations of the AQCs is also required under this step. These statistics are the key product of the QC/QA sampling and testing procedure.

Step 2: Estimate the Mean Life-Cycle Cost of the As-Constructed Pavement. This step is similar to step 2 for the as-designed pavement (Section 3.3) in that an iterative process is pursued whereby the as-constructed AQC distributions are sampled randomly to create a large number of possible combinations. It is different in that it uses the *as-constructed* means and standard deviations of each AQC to generate the combinations. For each combination, the as-constructed LCC, $(LCC_{con})_i$, is determined in the same manner that the individual LCCs were determined for the as-designed pavement. From the resulting list, the as-constructed LCC mean ($\overline{LCC}_{con}$) and standard deviation (σ_{LCCcon}) are calculated.

Step 3: Estimate the Pay Adjustment and the Pay Factor. Based upon the results of step 2, estimates for the lot PA and associated lot PF are calculated as follows:

$$PA = \overline{LCC}_{des} - \overline{LCC}_{con} \tag{3}$$

$$PF = 1 + (PA / BP) \tag{4}$$

where

$\overline{LCC}_{des}$ and $\overline{LCC}_{con}$ are as previously defined and
BP = bid price (pro-rated based upon the size of the lot as compared with the size of the project).

Step 4: Address Pay Adjustment Uncertainty. The goal of the PF relationship is to relate contractor pay adjustments directly to the quality of the as-constructed pavement. Clearly, there is some uncertainty associated with the PF estimate calculated in the previous step. This uncertainty derives from the uncertainty in the estimation of LCCs which, in turn, derives from design data inaccuracies, uncertainty in field and laboratory measurements, prediction model error, poor unit cost data, and so on.

In this step, a process is carried out whereby one key element of this uncertainty, that is, that related to sampling and testing error, is taken into consideration. It should be noted that although the other factors also contribute to the overall uncertainty, they can reasonably be disregarded for purposes of PA since their effects would apply almost equally to the as-designed and the as-constructed pavements. It should also be noted that if variability or uncertainty were to be ignored, this step would be skipped and the results from step 3 used as input to step 5.

A second feature gained from the process described here is that it provides contractor "relief" for the fact that there are no *acceptable quality levels* (AQLs) in the specification for any of the AQCs. If, for example, the contractor deviates even slightly above or below the target asphalt content specification, the method of PA will result in some penalty being assessed. From the contractor's standpoint, it would have been preferable to have AQLs on each of the AQCs. This would provide some "leeway" or tolerance on every AQC such that the contractor could still receive a PF equal to 1.00. Unfortunately, the problem with setting individual AQLs is that a series of off-target (but acceptable) AQC values can, through compounding, produce an unacceptable LCC_{con}.

Figure 9 provides a graph illustrating the method for addressing uncertainty and tolerance. Shown are two LCC distributions, one for the as-designed pavement and the other for the as-constructed pavement. The premise of this approach is to determine whether the mean LCC of the as-constructed pavement ($\overline{LCC}_{con}$) is significantly different from the mean LCC of the as-designed pavement ($\overline{LCC}_{des}$). If the difference is not significant, then no PA is made. If the difference is significant, then a PA is assessed. In this process, significance is established statistically using the *z Test* (*13*) which provides for a comparison of the means between two large independent unpaired population samples. Note that this should not be confused with the z-value, that is, standard normal deviate described in Section 3.3. The *confidence level* (e.g., 80 percent) represents the difference between 100 percent and the risk (expressed as a percent) of a PA being assessed when one is not in order. This level is established either by mutual agreement between the agency and the contractor (as part of the contracting process) or by agency notification as part of the specification. The confidence level, along with the mean and standard deviation of LCC for the as-designed pavement, define upper and lower *confidence limits* (LCC_{ucl} and LCC_{lcl}). Thus, if the mean LCC for the as-constructed pavement falls within these limits, then it is not significantly different from the mean LCC for the as-designed pavement. Increasing the confidence level (from say 80 to 90 percent) will move both LCC_{ucl} and LCC_{lcl} further from the mean LCC for the as-designed pavement, thereby (1) increasing the likelihood that the mean LCC for the as-constructed pavement will not be significantly different from that of the as-designed pavement and (2) decreasing the likelihood that a PA will be assessed.

It should be noted that both parties will be motivated to agree on a low confidence level (say 10 percent). The reason for this is that a low confidence level increases the likelihood of the contractor being rewarded a bonus (favorable for the contractor) or assessed a penalty (favorable for the agency). In both cases, the contractor is encouraged to achieve a high level of quality control, with particular emphasis on those AQCs which have the most effect on pavement performance.

Another issue associated with this approach to addressing uncertainty is determining the PAs if the estimated mean LCC of the as-constructed pavement is found to be significantly different from the estimated mean LCC of the as-designed pavement. Without any modification, the PA would go from zero at a point where $\overline{LCC}_{con}$ is just inside either of the confidence limits to a sizeable number if $\overline{LCC}_{con}$ is just outside the limit. (In other words, there would be no smooth transition from the region of zero PA). The method selected for incorporation into this version of the PRS for HMA construction is one that basically translates the leeway zone to the entire range of PA. This concept is illustrated in Figure 10.

Following are the two new equations that modify the PA depending on whether a bonus or penalty is being assessed:

For the *bonus* case ($\overline{LCC}_{con}$ is less than $\overline{LCC}_{des}$):

$$\text{PA (new)} = \text{PA (from step 3)} - (\overline{LCC}_{des} - LCC_{lcl}) \tag{5}$$

where LCC_{lcl} = LCC associated with the lower confidence limit (as determined statistically using $\overline{LCC}_{des}$, $\sigma_{LCCdes,}$ and the desired level of confidence).

For the *penalty* case ($\overline{LCC}_{con}$ is greater than $\overline{LCC}_{des}$):

$$\text{PA (new)} = \text{PA (from step 3)} + (LCC_{ucl} - \overline{LCC}_{des}) \tag{6}$$

where LCC_{ucl} = LCC associated with the upper confidence limit (as determined statistically using $\overline{LCC}_{des}$, σ_{LCCdes}, and the desired level of confidence).

Alternatively, the PA may be calculated more directly, that is, without using or modifying the PA calculated in step 3.

$$\text{PA} = LCC_{lcl} - \overline{LCC}_{con} \text{ (for the bonus case)} \tag{7}$$

$$\text{PA} = LCC_{ucl} - \overline{LCC}_{con} \text{ (for the penalty case)} \tag{8}$$

As was the case in step 3, the PF is calculated using the following relationship:

$$\text{PF} = 1 + (\text{PA/BP}) \tag{9}$$

where BP = bid price (pro-rated based on the size of the lot as compared with the size of the project).

Step 5: Check Pay Factor Against Limits. The last step in this process is a comparison of the PF estimate from step 4 with some pre-established limits. As indicated in step 5 (Figure 8), there are two potential PF limits, one which puts a ceiling on the bonus and the other which assures some minimum level of quality. The upper limit defines a maximum bonus (say PF = 1.05) which discourages the practice of "over-construction." The lower limit, on the other hand, defines a specific *rejectable quality* level (say PF = 0.75) below which the entire lot can be rejected. In the latter case, either zero payment would be assessed or the contractor would be required to remove and replace the entire lot.

3.4.2 Development of Simplified Pay Factor Tables and Nomographs

In Section 3.3.2 dealing with sensitivity issues, the prospect of developing *project-specific* PF tables and nomographs as part of the construction specification was discussed. The process was described as being potentially cumbersome but worthwhile if it helped the contractor to better understand the sensitivity of PF to the different AQCs.

Another possible use for such PF tables and nomographs is in a more simplified application setting, such as in agencies which are predisposed to applying modern pavement technology but do not have all the staff and resources to do so. If a PF nomograph, such as that depicted conceptually in Figure 11 were available to an understaffed state agency, it could have a major impact on the contractor's construction practices and the overall performance of the pavement network.

Development of these PF tables, nomographs, or both requires a more rigorous simulation than that associated with any one project. To ensure that a wide range of possible projects is covered, a factorial experiment must be designed in which high, medium, and low levels of the different AQCs are defined and then processed to produce a large database. In fact, separate databases would probably be required to represent different pavement classifications, traffic levels, subgrade soil support and environmental conditions. From these, PF relationships could be statistically derived and used to prepare the PF tables and nomographs. An example of this development process is presented in Chapter 11 of Part II.

3.5 SUMMARY

The purpose of this chapter was to describe the method of PA used in the PRS prepared under this project. Several different aspects of the method were addressed, including the following:

- Why LCC was selected as the basis for PA.
- How the general models are used to estimate LCC.
- How a preliminary PF relationship is developed for use in the construction specification.
- How to calculate the final PA.

A key issue addressed in the process is the treatment of risk and uncertainty, namely, stochastic variability. Overall, the method described in this chapter is rational, defensible, and capable of focusing the attention of contractors and highway agencies on those key M&C factors that have the greatest effect on improving pavement performance.

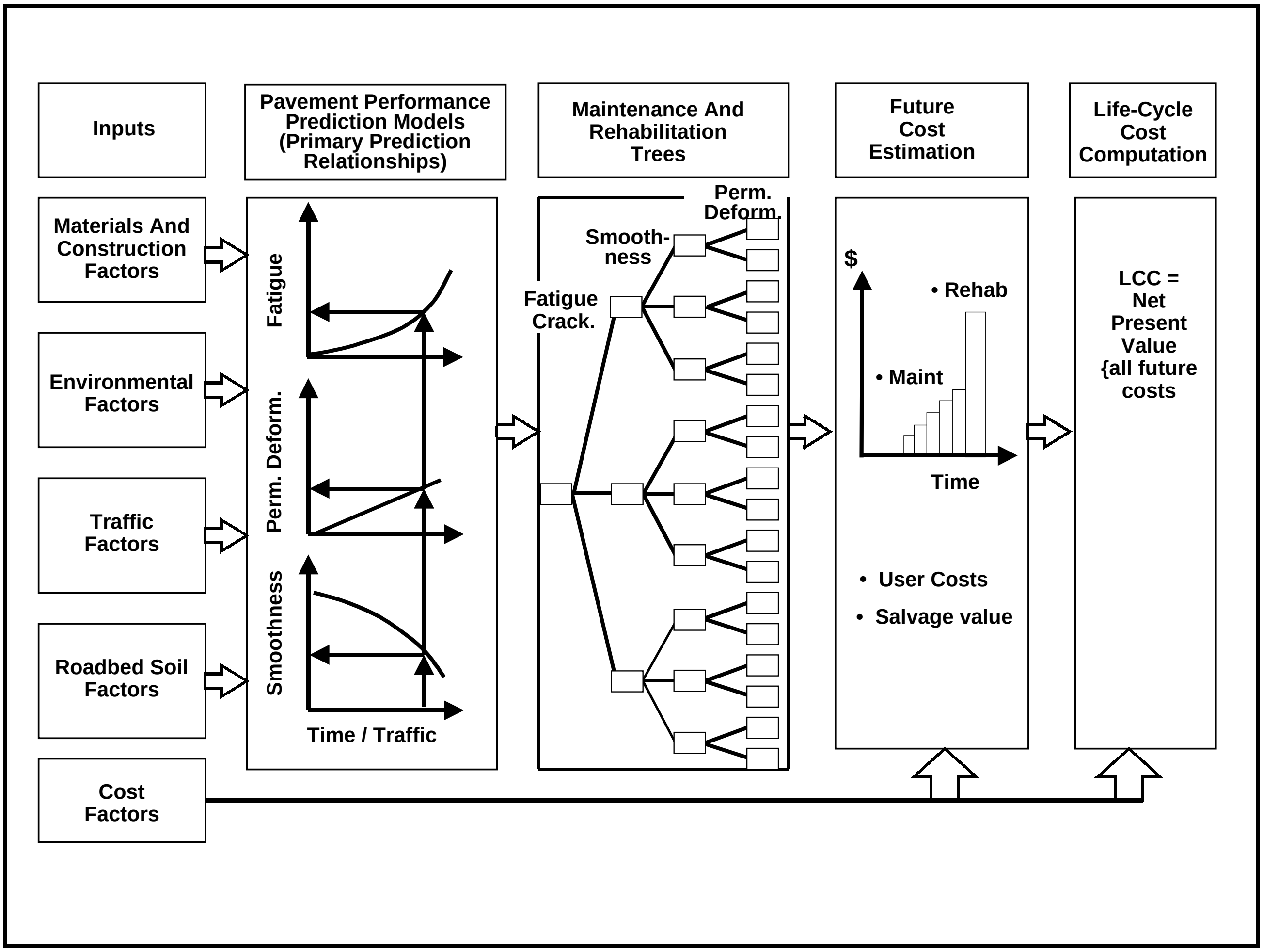
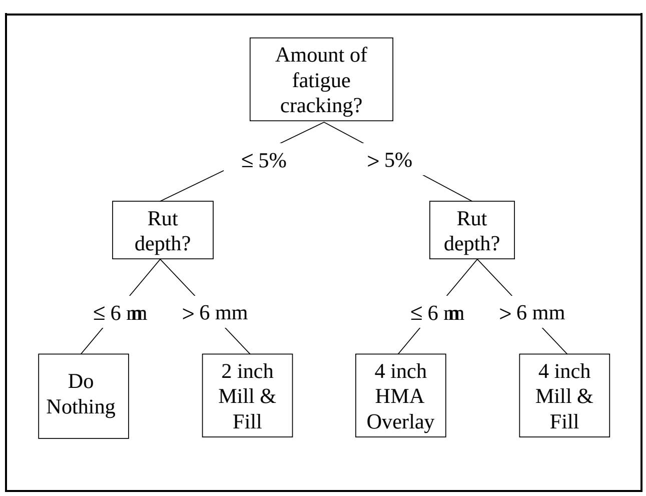


Figure 4. Conceptual framework of the LCC model.



1 in. = 25.4 mm

Figure 5. Example of a simple M&R decision tree.

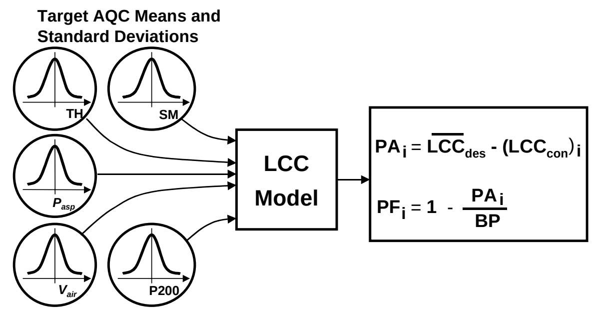
STEP 1

Supply Required Inputs (Characteristics of the As-Designed Pavement)

- Target AQC means and standard deviations for HMA
- As-designed levels of other M&C factors
- Design levels for traffic, soil strength and environmental factors
- Maintenance and rehabilitation plan
- Cost data

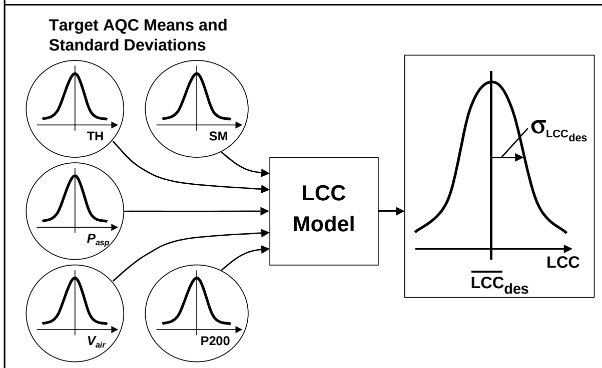
STEP 3

Calculate the Pay Adjustments for a Simulated Distribution of As-Constructed Pavements



STEP 2

Estimate the Mean LCC for the As-Designed Pavement



STEP 4

Determine the "z-value" for Each AQC and Each Combination

$$(z_{TH})_i = [(TH_{con})_i - \overline{TH}_{des}] / (\sigma_{TH})_{des}$$

$$(z_{SM})_i = [(SM_{con})_i - \overline{SM}_{des}] / (\sigma_{SM})_{des}$$

$$(z_{P_{asp}})_i = [(P_{asp_{con}})_i - \overline{P_{asp}}_{des}] / (\sigma_{P_{asp}})_{des}$$

$$(z_{V_{air}})_i = [(V_{air_{con}})_i - \overline{V_{air}}_{des}] / (\sigma_{V_{air}})_{des}$$

$$(z_{P200})_i = [(P200_{con})_i - \overline{P200}_{des}] / (\sigma_{P200})_{des}$$

STEP 5

Perform Regression Analysis

$$PF = f\{ z_{TH}, z_{SM}, z_{P_{asp}}, z_{V_{air}}, z_{P200} \}$$

Figure 6. Basic step-by-step process associated with generating the preliminary PF relationship for the HMA specification.

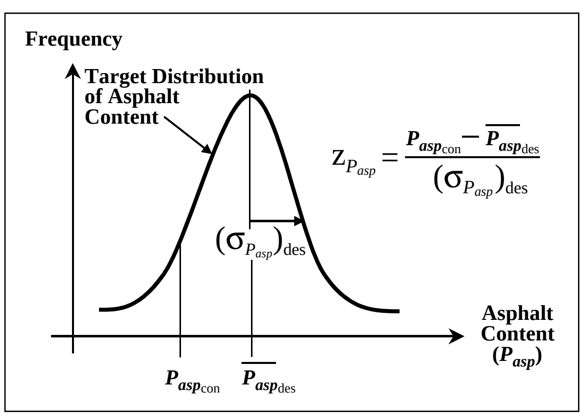


Figure 7. Determination of "z-value" for asphalt content.

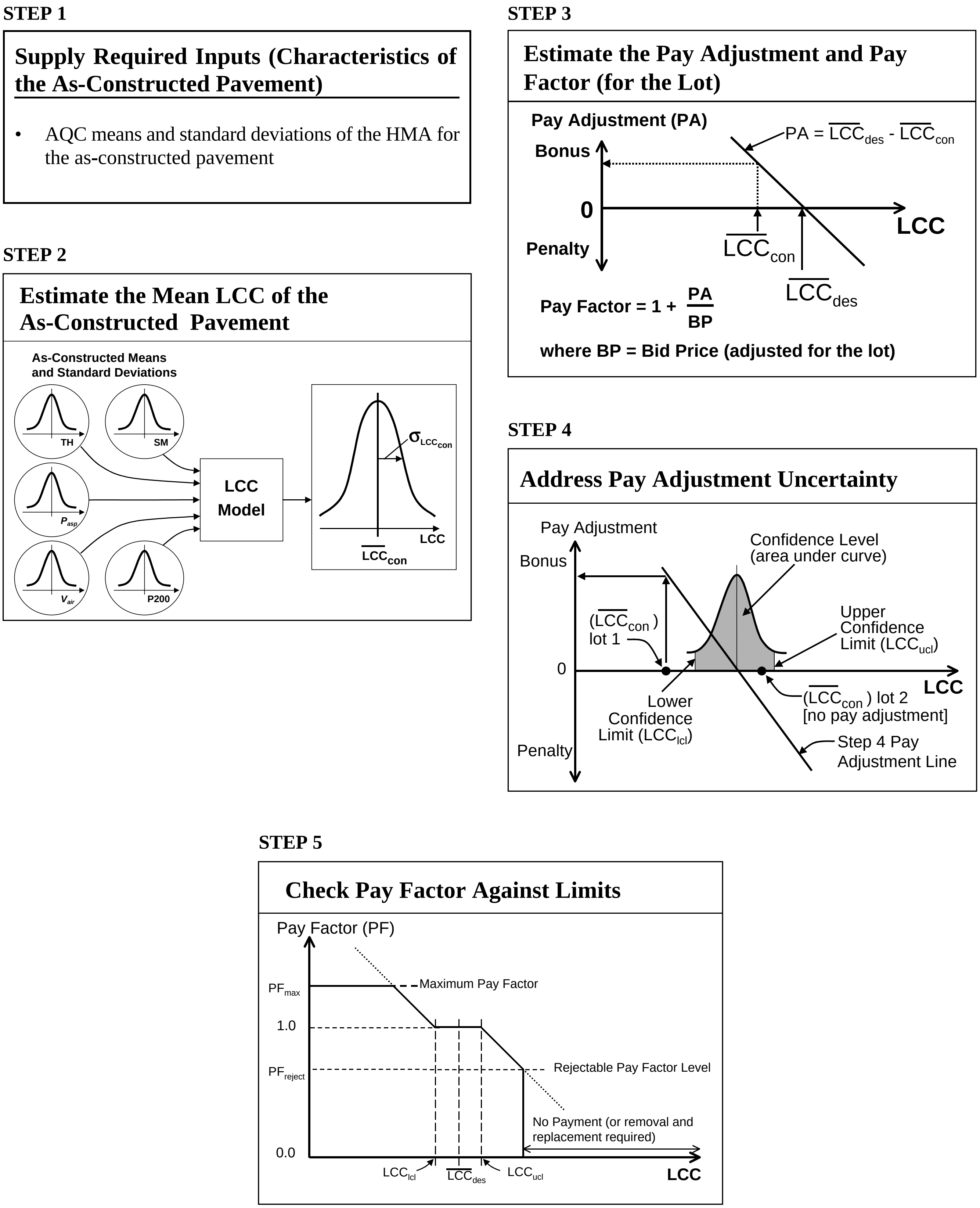


Figure 8. Basic step-by-step process associated with determining the final PA based on as-constructed results.

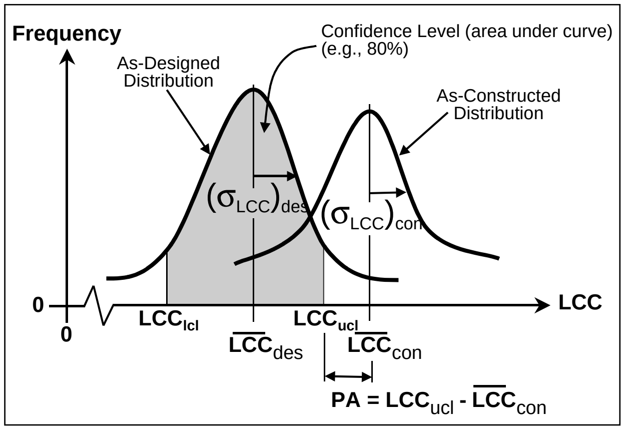


Figure 9. Comparison of as-designed and as-constructed LCC distributions for the purpose of PA (penalty case).

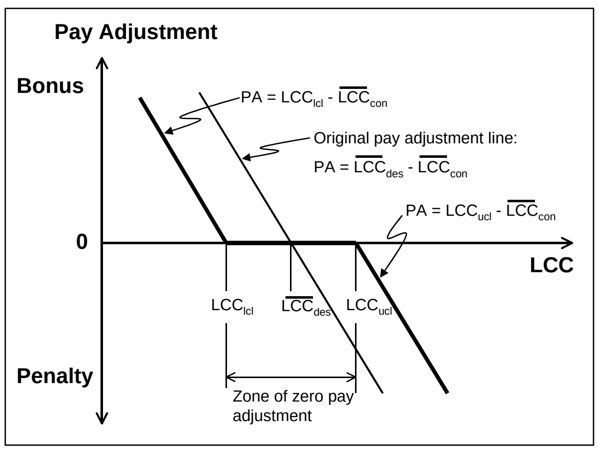


Figure 10. Modified PA concept considering PA uncertainty.

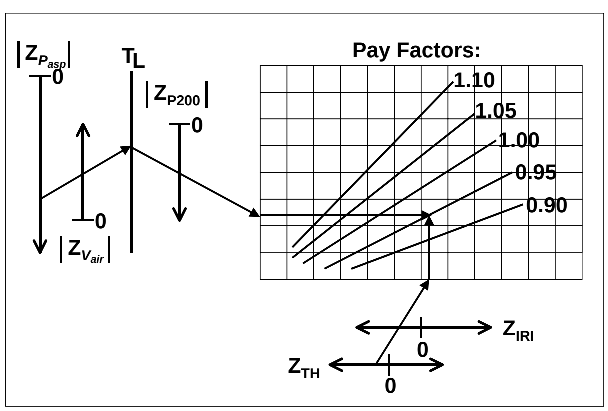


Figure 11. Conceptual PF nomograph.

TABLE 2 Example AQC combinations and associated z-values (main effects only) used to determine the mean and standard deviation of the LCC for the as-designed pavement and the LCC for the simulated as-constructed pavement

Iteration	AQCs and their randomly selected levels				Corresponding z-values (main effects only)				LCC_i
	V_{air}	TH	P200	P_{asp}	Z_V_{air}	Z_TH	Z_P200	Z_P_{asp}	
1	6.044	6.690	5.012	5.250	-1.304	2.300	-1.098	-1.364	3788.47
2	7.954	5.783	6.636	5.770	-0.031	-0.723	0.707	0.213	3788.47
3	8.547	5.596	5.692	6.215	0.365	-1.346	-0.342	1.561	2459.49
4	6.777	5.532	7.623	6.027	-0.815	-1.559	1.804	0.991	2867.53
5	11.298	5.582	4.728	6.201	2.199	-1.394	-1.413	1.518	3542.88
6	8.813	6.391	5.395	5.897	0.542	1.303	-0.673	0.596	2459.49
7	9.116	6.544	5.783	5.064	0.744	1.813	-0.242	-1.929	6265.91
8	8.935	5.959	5.912	4.968	0.623	-0.136	-0.098	-2.218	7979.24
9	9.518	6.347	5.517	5.296	1.012	1.156	-0.536	-1.224	5546.38
10	8.369	5.982	4.946	6.002	0.246	-0.062	-1.171	0.916	2659.91
11	6.028	5.968	6.671	5.770	-1.315	-0.107	0.746	0.213	2867.53
12	5.588	6.519	6.353	5.934	-1.608	1.729	0.392	0.709	681.27
13	9.560	6.284	5.568	5.359	1.040	0.946	-0.480	-1.035	5546.38
14	10.453	6.348	6.090	5.628	1.635	1.160	0.099	-0.219	4315.81
15	8.709	5.526	5.585	5.731	0.473	-1.581	-0.461	0.093	4900.57
16	8.247	6.157	6.812	5.202	0.164	0.524	0.902	-1.510	5896.21
17	10.455	6.454	5.224	5.133	1.637	1.513	-0.863	-1.719	6657.08
18	6.692	6.552	6.223	5.524	-0.872	1.838	0.248	-0.533	3083.30
19	7.346	6.339	7.349	5.290	-0.436	1.131	1.499	-1.242	4900.57
20	6.660	5.978	6.624	5.087	-0.893	-0.075	0.693	-1.857	6265.91
21	10.332	5.572	6.737	6.203	1.555	-1.428	0.819	1.523	3083.30
22	6.223	6.202	5.259	5.650	-1.185	0.673	-0.824	-0.153	3083.30
23	7.942	6.110	5.783	5.558	-0.039	0.367	-0.241	-0.432	4315.81
24	5.168	6.065	7.069	5.399	-1.888	0.217	1.188	-0.913	4045.80
25	8.176	5.459	4.943	6.335	0.118	-1.804	-1.175	1.924	2076.03
↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
491	8.409	6.076	6.693	5.429	0.272	0.254	0.770	-0.820	5214.94
492	8.788	6.143	4.860	5.520	0.526	0.476	-1.266	-0.545	4602.05
493	11.734	6.435	6.026	5.715	2.489	1.451	0.029	0.044	4315.81
494	9.013	6.140	4.020	5.557	0.675	0.468	-2.200	-0.433	4602.05
495	8.847	5.633	4.541	5.681	0.565	-1 .225	-1.621	-0.059	4900.57
496	7.644	5.982	6.088	5.735	-0.237	-0 .060	0.098	0.106	3542.88
497	6.984	6.122	5.121	5.037	-0.677	0.405	-0.977	-2.008	6265.91
498	7.533	6.237	6.613	5.413	-0.311	0.789	0.681	-0.869	4602.05
499	7.640	5.963	6.325	5.332	-0.240	-0.124	0.361	-1.115	5546.38
500	8.919	5.974	7.213	5.798	0.613	-0.086	1.348	0.296	3788.47
Mean	8.136913	5.9885586	5.9420166	5.7132988	0.0912758	-0.038144	-0.064423	0.0402938	3847.89
Std. Dev.	1.4660579	0.3022436	0.9026598	0.3272434	0.9773712	1.007481	1.0029623	0.9916564	1484.19

NOTES:

V_{air} = air void content, percent by volume; target air void content = 8%, target standard deviation = 1.5%
TH = thickness, in.; target thickness = 6 in., target standard deviation = 0.3 in.
P200 = percent passing the #200 sieve; target P200 = 6.0%, target standard deviation = 0.9%
P_{asp} = asphalt content, percent by weight of aggregate; target asphalt content = 5.7%; target standard deviation = 0.33%
Z_V_{air} = z-value for air void content
Z_TH = z-value for thickness
Z_P200 = z-value for percent passing the #200 sieve
Z_P_{asp} = z-value for asphalt content
LCC_i = predicted life-cycle cost, present-worth dollars, utilizing the indicated levels of AQCs
(1 in. = 25.4 mm)

TABLE 3 Parameters used in simulations

Layer	Thickness, in. (mm)	Poisson's Ratio	Moduli psi (MPa)
Surface	6.0 (150)	0.35	20,000-2,000,000 (138-13,800)
Base	12.0 (200)	0.40	5,000-200,000 (34.5-1,380)
Foundation	∞	0.45	2,000-100,000 (13.8-690)

CHAPTER **4**

DEVELOPMENT OF PAVEMENT PERFORMANCE MODELS

4.1 INTRODUCTION

The performance models developed herein are of two general types: (1) those based on direct regressions among the specific performance measure (rut depth or fatigue cracking) and ESALs and mix characteristics and (2) those based on M-E analyses assuming the pavement behaves as a multilayer elastic system. The first category has been termed “level 1” and the second “level 2.” The general framework is illustrated in Figure 12. Details of the research effort summarized in this chapter are contained in WesTrack Technical Report UCB-1 (*15*).

Level 1 Models. For rutting, there are two categories of level 1 models termed level 1A and level 1B. The level 1A models are based on direct regressions between observed rutting or observed cracking as seen in Figure 12. For rutting, the model uses performance data from both the 26 original and the 8 replacement sections while for fatigue, the model uses only the fatigue data for the 26 original sections.

The level 1B model was developed from analyses of the pavements using the M-E analyses. In this instance, rut depth versus ESAL relationships were developed for a 10-year period for 23 sections in which little or no fatigue cracking was observed. The 23 sections included both original and replacement sections. For the analyses, the traffic was uniformly distributed throughout a 24-hour period and the yearly temperature environment was assumed to be the same for each year of the 10-year period. As will be seen, this procedure tended to reduce the impact of early rutting which occurred in some of the sections.

Only a single level 1 model is presented for fatigue; it makes use of a Probit model to define the probability of cracking.

As seen in Figure 12, the level 1 models are based on WesTrack-type mixes. The level 1B model for rutting can, however, be used for other traffic and environmental (temperature) conditions.

Level 2 Models. For rutting, two level 2 models have been developed. The level 2A model can be used for other traffic and temperature environments but is limited to mixes with dense-graded aggregate gradations like those at WesTrack. The level 2B model can be used for other types of mixes so long as they are characterized by means of the repeated simple shear test at constant height (RSST-CH).

The level 2 models for fatigue, a total of three, have not been characterized separately as have the rutting models. Rather, since the procedure is the same for the three models, selection is based on the engineer’s choice of mix fatigue and stiffness characteristics. Three options are available:

- Use of WesTrack mix data.
- Use of stiffness and fatigue data from published information.
- Use of laboratory-determined stiffness and fatigue response data obtained for the specific area in which the mixes are to be used.

These guidelines are also shown in Figure 12.

4.2 MODULUS DETERMINATIONS

4.2.1 Introduction

The purpose of this evaluation was to establish elastic moduli of the pavement components at WesTrack to permit considerations of stresses, strains, and deflections in the test pavement sections. The results of the investigation provided the necessary input for the evaluation of observed pavement performance and the establishment of performance models for the PRSs, a major WesTrack product.

The assumption used is that multilayer elastic analysis can produce sufficiently accurate estimates of stresses, strains, and deflections in the pavement structures. In turn, this forms the basis for establishing performance models for permanent deformation (rutting) and load-associated cracking (fatigue).

The sources from which the moduli were obtained include both field and laboratory measurements. The field data include an extensive series of falling weight deflectometer (FWD) measurements taken at intervals throughout the traffic loading. Laboratory data include data from the following:

- AC—flexural fatigue, RSST-CH, and resilient modulus (indirect tension) tests.
- Untreated base—triaxial compression resilient modulus tests.
- Engineered fill and foundation soil—triaxial compression resilient modulus tests.

This section describes the methodology used to arrive at the various moduli and a summary of the values used in the analyses to establish performance models.

4.2.2 Analyses of Field Data, Original Sections

In the analyses of the FWD data to determine moduli, best estimates of these parameters were considered to be those minimizing the sum of the squared differences between measurements and simulations of FWD surface deflections. In these analyses, simulations were based on regression equations relating the deflection at each of the seven sensor locations of the FWD to the layer moduli. Microsoft Excel's solver routine was used to determine the best-fit moduli.

Regression equations, based on the assumption of a three-layer system for WesTrack, took the following form:

$$\begin{aligned} 1\text{n}\, D = {} & d_0 + d_1\, 1\text{n}\, E_2 + d_3 \ln E_3 + d_4(\ln E_1)^2 \\ & + d_5 (\ln E_2)^2 + d_6 (\ln E_3)^2 + d_7 \ln E_1 \cdot \ln E_2 \\ & + d_8 \ln E_1 \cdot \ln E_3 + d_9 \ln E_2 \cdot \ln E_3\mathbf{)} \end{aligned} \tag{10}$$

where

D = surface deflection, mils
E_1 = modulus of the HMA,
E_2 = modulus of untreated aggregate base,
E_3 = modulus of engineered fill and foundation soil, and
d_0–d_9 = regression coefficients.

Simulations were performed to examine the effects of all possible combinations of five levels of modulus for each of the three layers. Only one load level was simulated, 44.4 kN (10,000 lbf) at a radius of 150 mm (5.9 in.). Parameters used for the simulations are summarized in Table 19.

Examples of the results of these analyses are summarized in Figures 13, 14, and 15; Table 4 summarizes the regression equations (form of equation 10) for the seven sensor locations 0 mm (0 in.), 305 mm (12 in.), 457 mm (18 in.), 610 mm (24 in.), 914 mm (36 in.), 1,219 mm (48 in.), 1,524 mm (60 in.). In the best-fit analyses, assumptions were made as to how the moduli were likely to vary as a function of external influences such as temperature. These included the following:

- The AC modulus, E_1, is related to the average surface temperature, T (based on section 12 data):

$$E_1 = \exp(A_0 + A_1 T) \tag{11}$$

where A_0 and A_1 are regression coefficients.

- The base modulus, E_2, is independent of temperature and season; and
- The foundation-soil moduli, E_3, are sensitive to seasonal but not temperature influences and were investigated as a discreet function of the measurement period and a sinusoidal function as follows:

$$E_3 = D_0 + D_1\left[\frac{\sin(doy - D_2)}{365 \times 2 \times \pi}\right] \tag{12}$$

where

doy = day of the year and
D_0, D_1, D_2 = regression coefficients.

Preprocessing of the FWD's data was accomplished by the following:

- Removing unnecessary information.
- Adjusting surface deflections for each drop to a 44.4 kN (10,000 lbf) load assuming linearity between deflection and load.
- Using the average of 20 sets (five locations and four drops) of FWD deflection measurements for each test section and analysis period.

Figure 16 illustrates the framework used for the simulations; modulus values for the three layers are summarized in Tables 5 and 6. Table 6 also lists the measurement periods used in the analyses.

FWD-determined moduli for the AC as a function of temperature are shown in Figure 17 for the sections evaluated in the south tangent and in Figure 18 for sections from the north tangent. It will be noted in Table 5 that the slope of the ln E_1 versus T relationship was assumed to be constant for all mixes.[1]

Table 7 compares mixes based on moduli at a temperature of 40°C (104°F), examining the effects of asphalt content and air void content. It will be noted that the influence of air void content is significant, with an increase in air void content resulting in a reduction in mix stiffness. It will also be noted that replicate sections have comparable stiffnesses.

The base modulus was assumed to be unaffected by both temperature and season. The values for south and north tangents were estimated to be 104 MPa (15,100 psi) and 93.2 MPa (13,500 psi), respectively. The modulus for the south tangent is about the same as that for the foundation soil, whereas it is less than that for the north tangent. Considering also other investigations which suggest that conventional backcalculation routines may underestimate base moduli, additional investigation appears warranted to determine suitable means for backcalculating untreated base moduli. This was, however, beyond the scope of this project.

The influence of season on foundation soil modulus is illustrated in Figures 19 and 20 for the south and north tangents, respectively. In these figures, it will be noted that the

[1] This simplification seemed reasonable since all the mixes contained the same asphalt binder and aggregate and the gradations were reasonably uniform among the mixes.

modulus varies throughout the year with a high value in mid-December and a low value about mid-May. The minimum value is about 70 percent of the maximum.

There is a significant difference in foundation moduli between the sections of the south and north tangent with the north tangent sections being considerably stiffer than those in the south tangent. Also, the annual range in modulus is larger for the north tangent than for the south tangent sections.

As noted earlier, sinusoidal functions were applied to the foundation-soil moduli to define seasonal variations. Figures 19 and 20 indicate that such functions are reasonably well suited to define the influence of seasonal variations on modulus for the WesTrack foundation.

For the analysis reported subsequently, the decision was made to use the moduli shown in Figures 19 and 20 in the analyses of the permanent deformation and fatigue performance of the test sections.

4.2.3 Comparisons of Field- and Laboratory-Determined Moduli

A limited opportunity was provided to compare laboratory-determined moduli from the flexural fatigue tests at 20°C (68°F) and the RSST-CH tests at 50°C (122°F) for the asphalt mixes with those determined from evaluation of the FWD measurements.

In the laboratory, flexural stiffness measurements were obtained during fatigue testing of mixes from the bottom portion of each of the test sections. Results of a comparison of these stiffness values for a number of the test sections with those determined from the analysis of the FWD results are shown in Figure 21.

Similarly, in the RSST-CH tests to define the permanent deformation response of the various mixes, shear stiffnesses were determined at 50°C (122°F) for mixes from each of the test sections. Comparison of stiffnesses determined from the shear moduli with those estimated from the FWD measurements are shown in Figure 22. For comparisons with the FWD estimates, the laboratory shear stiffnesses were converted according to the following expression:

$$E = G \cdot 2(1 + \nu) \qquad (13)$$

where

G = shear modulus and
ν = Poisson's ratio.

For this conversion, the Poisson's ratio, ν, was assumed to be equal to 0.35 for the AC.

The correlations shown in Figures 21 and 22 were used, as discussed in Section 4.2.4, to determine moduli for use in analyses of response of the replacement sections.

A brief comparison was also made between the FWD- and laboratory-determined moduli for the base and foundation soils. These results are shown in Table 8.

4.2.4 Moduli Used for Permanent Deformation and Fatigue Analyses

The results presented in the previous sections established the basis for selecting moduli for use in developing the performance models for both permanent deformation and fatigue.

For permanent deformation and fatigue, the subgrade moduli for the south and north tangents shown in Figures 19 and 20 have been used to reflect seasonal variations in the subgrade stiffness.

It has been noted that backcalculation procedures result, at times, in estimated moduli for untreated bases which are lower than expected. This was assumed to be the case for the base moduli shown in Table 8. In the Shell pavement design procedure, for example, the moduli of untreated aggregate layers are assumed to be on the order of 2 times the subgrade modulus. Other data suggest that larger stiffness values would be associated with materials similar to the base material used at WesTrack. Accordingly, a modulus value of 138 MPa (20,000 psi) was used in the M-E analyses conducted for permanent deformation response and 172 MPa (25,000 psi) in the M-E analyses for fatigue.

For the AC, moduli at 20°C (68°F) from the flexural fatigue tests and the temperature slope (A_1) from the backcalculations shown in Table 5 were used to represent the stiffness moduli in the permanent deformation analyses to be described in Section 4.3. The values for both A_0 and A_1 for the mixes used in these analyses are summarized in Table 9. Comparison of some of the A_0 values for sections 1–25 with those shown in Table 5 reflect the inclusion of additional flexural stiffness data from the fatigue test program.

The coefficients for sections 35–55 were established from the regressions shown in Figures 21 and 22 using the flexural fatigue test stiffnesses at 20°C (68°F) and the shear stiffnesses at 50°C (122°F) from the RSST-CH test. The flexural stiffnesses were divided by the factor 0.9 and shear stiffnesses were multiplied by the product of a factor of 3 (equation 13 with ν = 0.5) and the regression factor 5.832. When the data were plotted as shown in Figures 17 and 18, the average slope of –0.0610 shown in Table 9 was obtained. This procedure was followed in lieu of FWD analyses of these test sections.

4.2.5 Additional Stiffness Analyses

For the mechanistic analyses used to assess fatigue response, the mix stiffnesses obtained from the flexural fatigue tests were also used. With these data, regression analyses were performed on the results of 186 fatigue tests on the field-mixed/field-compacted (FMFC) mix, primarily at 20°C (68°F) and with limited data at 5°C (41°F) and 30°C (86°F).

Results of the analyses are represented by the following equations:

- Fine and fine plus mixes (127 tests)

$$\ln Stiff = 11.4677 - 0.0827V_{air} - 0.2285P_{asp} - 0.0579T \qquad R^2 = 0.85 \tag{14}$$

- Coarse mixes (59 tests)

$$\ln Stiff = 11.4707 - 0.0576V_{air} - 0.2142P_{asp} - 0.0606T \qquad R^2 = 0.79 \tag{15}$$

where

Stiff = mix stiffness, MPa (1 ksi = 6.9 MPa),
V_{air} = air void content, percent,
P_{asp} = asphalt content, percent, and
T = temperature, °C (°F = 1.8°C + 32).

Comparison of the mix stiffnesses for the original sections in Table 9 with those predicted by the regression equations for specific asphalt and air void contents are shown in Figure 23.

4.2.6 Summary

The moduli used in Sections 4.3 and 4.4 have been developed from laboratory-determined stiffnesses (measured both in flexural and shear) and from backcalculation of FWD data determined over a period of time on the test sections. As seen in Figure 23, the HMA moduli for both types of analyses are compatible over the temperature range which occurred at WesTrack and were used in the distress analyses to develop performance models.

4.3 PERMANENT DEFORMATION

4.3.1 Introduction

The purpose of this phase of the investigation was to develop models which can be incorporated in the PRS program and which define the influence of asphalt content, air void content, and aggregate gradation on the accumulation of permanent deformation (rutting) in the asphalt-bound layer.

Two levels of models have been developed: (1) those based on regressions among measured performance, rut depth[2], and traffic loading and mix variables (termed level 1) and (2) those based on M-E analyses using the same parameters (termed level 2).

Data used to develop these models include the following:

1. Measured downward rut depths and associated traffic (in terms of ESALs).
2. RSST-CH data on FMFC specimens prior to the start of and at the conclusion of traffic (termed post mortem)[3] for the specific tests sections.
3. RSST-CH on laboratory-mixed/laboratory-compacted (LMLC) specimens including:
   a. study of specimens representing sections 4 and 25 to determine effects of temperature and shear stress level on mix performance.
   b. aggregate gradation study at North Carolina State University (NCSU) to define the effects of aggregate grading variations on the performance of specimens over ranges in air void contents, asphalt contents, and aggregate gradations representing reasonable specification tolerances for both the fine and coarse gradings.

To assist in the development of the M-E models, a special study of the effects of traffic wander on rut depth accumulations was also performed. This analytic study was conducted by Dr. S. Weissman of Symplectic Engineering Corporation using finite element simulations for different traffic wander patterns including that used at WesTrack.

4.3.2 Regression Modeling

Two approaches, based on regression, have been followed to develop relationships among rut depth and traffic, environment, and mix variables (including asphalt content, air void content, and aggregate gradation). Although the two methods involve different approaches to arrive at the performance relationship, an expression of the following form has served as the basis for both approaches:

$$\begin{aligned}\ln(rd) = {} & a_0 + a_1 \bullet P_{asp} + a_2 \bullet V_{air} + a_3 \bullet P_{asp}^2 + a_4 \bullet V_{air}^2 \\ & + a_5 \bullet P_{200} + a_6 \bullet fa + a_7 \bullet \ln(ESAL) + a_8 \\ & \bullet T \cdots + \text{(interaction terms among the} \\ & \text{variables)} \cdots + \text{(indicator variables} \\ & \text{representing the three aggregate gradings} \\ & \text{used at WesTrack)} \cdots + \text{(indicator variable} \\ & \text{for aggregate type, replacement sections)}\end{aligned} \tag{16}$$

where

rd = rut depth, mm or in.;
$ESAL$ = number of 80-kN (18,000-lb) equivalent single-axle loads;
P_{200} = percent aggregate finer than 0.075 mm (No. 200) sieve;

[2] All performance models are based on rut depths measured from the original pavement surface, that is, baseline to valley. Those values are somewhat less than those measured from peak to valley in each rut.

[3] For example, sections 7, 9, 13, 21, and 25 were removed from consideration at about 1,460,000 ESALs due to excessive rutting.

fa = percent aggregate passing the 2.36 mm (No. 8) sieve and retained on the 0.075 mm (No. 200) sieve; and
$a_0 \ldots a_n$ = regression constants.

The first approach (level 1A) involved a direct regression among rut depth and traffic, environment (temperature), and mix parameters. Using the individual rut depth measurements up to about 2.0×10^6 ESALs, the ESALs corresponding to each rut depth measurement, and the mix parameters, this approach resulted in the following expressions:

For the fine and fine plus mixes in the original experiment, the expression for rut depth (in inches) is as follows:

$$\ln(rd) = -5.257 + 0.357 \cdot \ln(ESAL) + 0.185P_{asp} + 0.041V_{air} + 0.916P_{200} + 0.005T \quad (R^2 = 0.67) \tag{17*}$$

where T = 90th percentile air temperature during the period for which the rut depth was measured.

For the original *coarse* mix, the expression (in inches) is as follows:

$$\ln(rd) = -4.939 + 0.212 \cdot \ln(ESAL) + 0.439 \cdot P_{asp} + 0.044 \cdot V_{air} + 0.034 \cdot T \qquad (R^2 = 0.80) \tag{18*}$$

For the replacement coarse mix (and for ESALs to 0.6×10^6 rather than 2.0×10^6 as above), the expression (in inches) is as follows:

$$\ln(rd) = -6.204 + 0.190 \cdot \ln(ESAL) + 0.829 \cdot P_{asp} + 0.207 \cdot V_{air} \qquad (R^2 = 0.63) \tag{19}$$

It will be noted that temperature does not appear in equation 19 for the replacement sections because the time interval of measurement was comparatively short for the number of ESALs used to develop the expression.

The second approach (level 1B) used the M-E model described in Section 4.3.3 to develop regressions relating rut depth to traffic (ESALs) and mix properties described earlier. A total of 23 of the 34 test sections, those with no or little fatigue cracking, were used in the analysis; these included 17 original and 6 replacement sections.[4]

For each section, the parameters a and c (see equation 20) were determined to provide the best fit between measured rut depths and traffic for the WesTrack conditions of traffic and temperature. Using these parameters, the accumulation of rut depth in each section was then determined for a period of 10 years with the traffic (WesTrack axle loading) applied at the rate of 60 vehicles per hour continuously throughout the 10 years. The temperature environment was that for WesTrack and was assumed the same for each year of the analysis period.

ESALs were then determined for each of the 23 sections for rut depths of 2.5 mm to 17.5 mm (0.1 in. to 0.7 in.) in increments of 2.5 mm (0.1 in.). Table 10 contains a summary of these results.

Various regressions were then applied to the data to obtain the effects of traffic and mix parameters on rut-depth accumulation. Results of six considered most suitable are summarized in Table 11. Comparisons of rut depths predicted by three of the regressions of Table 11[5] and equations 17, 18, and 19 are shown in Figures 24, 25, and 26 for three sections.[6]

4.3.3 Mechanistic-Empirical Modeling (Level 2)

M-E models were developed to represent the behavior of the pavement sections at WesTrack using the procedure described in this section. These models provide a basis for extending the results of WesTrack to other traffic conditions and environments and for developing regression models for use in the level 1 analyses. Results of the latter were presented in the previous section.

Approach. For the analyses reported herein, the pavement is assumed to behave as a multilayered elastic system. The pavement structure representative of the WesTrack pavement is shown in Figure 27. Moduli for the different AC sections and layers were developed as described in Section 4.2 of Part II. Equation 11 was used to determine the moduli with the coefficients A_0 and A_1 obtained from Table 9; a constant Poisson's ratio of 0.35 was assumed. Moduli for the base courses for the south and north tangents were assumed to be the same, 138 MPa (20,000 psi); a Poisson's ratio of 0.40 was used for this material. Representative values of the subgrade moduli for each month of the year were obtained from equation 12, values for which are shown in Figures 21 and 22. Poisson's ratio for the subgrade was assumed to be 0.45.

The analysis consisted of determining the three parameters τ, γ^e, and ε_v[7] on an hour-by-hour basis. Measured temperature distributions were used to define the moduli of the AC which was subdivided into three layers from top to bottom

* If the temperature term were not included (because the regressions were not significantly improved with it), the following expressions are obtained:

a. for fine and fine plus mixes (~1.8×10^6 ESALs):

$$\ln(rd) = -4.966 + 0.343 \ln(ESAL) + 0.192P_{asp} + 0.042 \cdot V_{air} + 0.196 \cdot P_{200} \quad (R^2 = 0.66)$$

b. for coarse mixes (~0.85×10^6 ESALs):

$$\ln(rd) = -6.852 + 0.449 \ln(ESAL) + 0.504 \cdot P_{asp} + 0.045 \cdot V_{air} \qquad (R^2 = 0.76)$$

[4] Original sections: 1, 4, 7, 9, 11, 12, 13, 14, 15, 18, 19, 20, 21, 22, 23, 24, and 25; replacement sections: 35, 37, 38, 39, 54, and 55.

[5] Defined by their R^2 values.

[6] In Figures 24, 25, and 26, level 1B-1 corresponds to regression 3, level 1B-2 to regression 5, and level 1B-3 to regression 6.

[7] τ, γ^e = elastic shear stress and strain at a depth of 50 mm (2 in.) below outside edge of tire.
ε_v = elastic vertical compressive strain at the subgrade surface.

with thicknesses, respectively, of 25 mm (1 in.), 50 mm (2 in.), and 75 mm (3 in.) to simulate the effects of temperature gradients on mix stiffness.

In this modeling, rutting in the AC was assumed to be controlled by shear deformations. Accordingly, the computed values for τ and γ^e at a depth of 50 mm (2 in.) beneath the edge of the tire were used for the rutting estimates (Figure 27). Densification of the AC was excluded in these estimates because it has a comparatively small influence on surface rutting.

In simple loading, permanent shear strain in the AC was assumed to accumulate according to the following expression:

$$\gamma^i = a \cdot \exp(b\tau\gamma^e n^c) \tag{20}$$

where

γ^i = permanent (inelastic) shear strain at a 50-mm (2-in.) depth,
τ = shear stress determined at this depth using elastic analysis,
γ^e = corresponding elastic shear strain,
n = number of axle load repetitions, and
a, b, c = regression coefficients.

The time-hardening principle was used to estimate the accumulation of inelastic strains in the AC under in situ conditions. The resulting equations are as follows:

$$\mathrm{a}_j = a \cdot \exp(\mathrm{b}\tau\gamma_j^e) \tag{21}$$

$$\gamma_1^i = \mathrm{a}_1[\Delta n_1]^c \tag{22}$$

$$\gamma_j^i = \mathrm{a}_j[(\gamma_{j-1}^i/\mathrm{a}_j)^{(1/c)} + \Delta n_j]^c \tag{23}$$

where

j = jth hour of trafficking,
γ_j^e = elastic shear strain at the jth hour, and
Δn_j = number of axle load repetitions applied during the jth hour.

The concept is illustrated schematically in Figure 28.

Rutting in the AC layer due to the shear deformation was determined from the following:

$$rd_{\mathrm{AC}} = K\gamma_j^i \tag{24}$$

For a 150-mm (6-in.) layer the value of K has been determined to be 5.5, when the rut depth (rd) is expressed in inches (*50*).

To estimate the contribution to rutting from base and subgrade deformations, a modification to the Asphalt Institute subgrade strain criteria (*16*) was used. The equation expressing the criterion for 13 mm (0.5 in.) of surface rutting is as follows:

$$n = 1.05 \times 10^{-9}\ \varepsilon_v^{-4.484} \tag{25}$$

where

n = the allowable number of repetitions and
ε_v = the vertical compressive stain at the top of the subgrade.

Because these criteria do not address rutting accumulation in the pavement structure, rut depth (rd) contributed by the unbound layers was assumed to accumulate as follows:

$$rd = dn^e \tag{26}$$

where d, e = experimentally determined coefficients.

Least squares analyses for the WesTrack data suggest that the value for d in equation 26, using the Asphalt Institute criteria, is as follows:

$$d = f/[1.05 \times 10^{-9}\ \varepsilon_v^{-4.484}]^e \tag{27}$$

where f = 0.14 and e = 0.372.

Using the time-hardening principle, as was used for the AC, rut depth accumulation can be expressed in a form similar to equation 23:

$$rd_j = d_j[(rd_{j-1}/\mathrm{d}_j)^{1/0.372} + \Delta n_j]^{0.372} \tag{28}$$

The framework for rut-depth estimation, using equations 20, 23, and 28, is illustrated in Figure 29. This approach has a distinct advantage over the direct regression approach; it permits prediction of rut depth as a function of traffic and environment as well as a function of the mix parameters.

Initially, 13 sections[8] were used to calibrate the coefficients of equations 20 and 26. While a value of b in equation 20 of 0.0487, based on RSST-CH tests, was used initially, subsequently a value of b = 0.071 (10.28 in SI units) was determined to provide better correspondence between measured and computed rut depths (initially all the computations were in U.S. units). This latter value has been adopted for all subsequent analyses.

Using the procedure illustrated in Figure 29, least squares regression provided values of a and c for each of the 23 sections. These are summarized in Table 12. It will be noted that the average root mean square error (RMSE) for rut depth for the 23 sections is 1.30 mm (0.051 in.). Figures 30, 31, 32, and 33 illustrate comparisons between computed and measured rut depths for WesTrack sections 4 (fine), 19 (fine plus), 7 (coarse), and 38 (replacement, coarse).

Using the values of a and c shown in Table 12, the rut depth versus ESALs relationships for the 23 sections were then determined by applying a uniform traffic load of 60 trucks per hour for a 10-year period and the temperature environment of

[8] Sections 1, 2, 4, 5, 7, 9, 13, 14, 15, 18, 21, 23, and 25.

WesTrack. This temperature distribution was assumed to be the same for each year of the 10-year period.

Regressions using different mix parameters have already been presented in Table 11 and comparisons made of the results determined using direct regression and the regressions based on the M-E analysis. Based on these results, regression 3 of Table 11 should be used:

$$\begin{aligned}\ln(rd) = &-6.1651 + 0.309941\ln(ESAL)\\ &+ 0.00294305V_{air}^2 + 0.0688276P_{asp}^2\\ &- 0.0657803P_{asp}\cdot P_{200} + 0.600498\,(\textit{fine plus})\\ &- 1.59167\,(\textit{coarse}) + 2.35276\,(\textit{replace})\\ &+ 0.21327\ln(ESAL)\,(\textit{coarse})\\ &- 0.140386\ln(ESAL)\,(\textit{replace})\end{aligned} \tag{29}$$

On the one hand, it will be noted that the results of this equation and those of equations 17, 18, and 19 correspond for a number of the sections. On the other hand, the direct regression using these equations may be influenced more by early rutting that occurred in some of the sections at WesTrack than would appear reasonable. This is illustrated by the results for sections 1, 14, and 15 from the original 26 and for the replacement sections (*15*).

To support the recommendation for the use of equation 29, a comparison had been made between the results of the WesTrack loading (e.g., Figures 30 through 33) and those using the extended time period analysis. The comparisons are shown in Figure 34.

Analyses of the data shown in this figure indicate the following:

ESALs for Slower Rates	
0–1,000	Faster estimates exceed slower by average of 147 percent.
1,000–10,000	Faster estimates exceed slower by average of 25 percent.
10,000–100,000	Faster estimates exceed slower by average of 36 percent.
100,000–1,000,000	Slower estimates exceed faster by average of 4 percent.
1,000,000–10,000,000	The two averages are the same.

Rut Depth	
0.1	Faster estimates exceed slower by average of 55 percent.
0.2	Faster estimates exceed slower by average of 38 percent.
0.3	Slower estimates exceed faster by average of 16 percent.
0.4	Slower estimates exceed faster by average of 6 percent.

At the larger ESAL levels and higher rut depths, the rate of trafficking has little influence.

To illustrate the influence of air void and asphalt contents on mix performance, computations have been made using equation 29 for the fine mix with a value of P_{200} of 5.5 percent. The results are shown in Figure 35.

For dense-graded aggregate gradations like those at WesTrack but in different traffic and temperature environments, the level 2 procedure can be followed. This requires the use of values of *a* and *c* (termed field *a* and field *c*) which are dependent on mix properties (e.g., asphalt content, air void content, and aggregate gradation).

To this end, calibrations were performed with the data which resulted from the analyses of the 23 test sections (Table 12) and represented by the results shown in Figures 30 through 33. Results of these calibrations provided the following expressions:

$$\begin{aligned}\ln(\text{field } a) = &-10.0792 + 0.788273P_{asp}\\ &+ 0.0846995V_{air} - 0.358081\,\textit{fine}\\ &+ 0.225354\,\textit{coarse} - 4.52386\ln(\text{field } c)\\ &(R^2 = 0.93)\end{aligned} \tag{30}$$

$$\begin{aligned}\ln(\text{field } c) = &-7.5834 + 1.051941P_{asp}\\ &+ 0.95641\,\textit{fine plus} + 0.66471\,\textit{coarse}\\ &(R^2 = 0.62)\end{aligned} \tag{31}$$

Use of parameters based on these equations to estimate rutting is termed the level 2A methodology.

The approach described in this section thus far allows the designer to use any traffic loading and temperature environment in the PRS (for PFs), so long as the grading of the aggregate used in the mix conforms to one of those used at WesTrack. For wider application, it would be desirable to have relationships which are not limited solely to WesTrack-type mixes.

An approach which is recommended uses the results of the laboratory RSST-CH test and the mix variables—asphalt content and air void content—to determine the field *a* and *c* values. To obtain the requisite parameters, a series of regressions were performed, the results of which are shown in Tables 13 and 14. From these analyses, regression 6 in Tables 13 and 14 is recommended for use to define *a* and *c* for the level 2 procedure if mixes used are not similar to those at WesTrack. This approach is referred to as level 2B.

Effects of Aggregate Gradation. The effects of variations in aggregate gradings on permanent deformation have been difficult to ascertain from only the results of the performance data for the field test sections. Accordingly, a study was conducted to define the influence of aggregate grading on mix rutting performance as measured in the RSST-CH.

For this study, test specimens were prepared by rolling wheel compaction for mixes containing the target gradings for the coarse and fine mixes and variations relative to these gradings. For each mix, that is, coarse and fine, the experiment consisted of (three gradings) × (three asphalt contents) × (three air void contents) × (three replicates) resulting in a total of 162 shear test specimens. The tests were performed at NCSU. Results of this test program as well as the shear tests on cores from the various test sections prior to trafficking have been used in the analyses described in this section.

Using the 23 sections and relating the field *a* to RSST parameters and asphalt content and air void content as necessary, several models were investigated. For this effort, a constant value for field *c* was assumed; values for field *a* are shown in Table 15. Various regressions were performed with these values and the other mix data including the following: asphalt content; air void content; mineral filler (P_{200}) (i.e., percent passing 0.075-mm [No. 200] sieve); and fine-aggregate (*fa*) (i.e., percent passing 2.36-mm [No. 8] sieve and retained on 0.075-mm [No. 200] sieve); and the RSST data. Table 15 lists the data used for these regressions.

Several models including asphalt content, air void content, their squares, and the four RSST measurements [repetitions to 5 percent strain, lab *a* and lab *b* (Figure 36), and G* at 100 repetitions] were examined. Six calibrations were accomplished with this model, each succeeding one recalibrated with one fewer section from the previous by discarding the most offending outlier. Table 16 shows the results of these calibrations. From this evaluation, the most promising model was selected; it included the parameter asphalt content squared (P_{asp}^2), and G*.

The results of the RSST-CH tests shown in Table 16 are based on tests on FMFC cores taken prior to trafficking. To use the NCSU data, it was necessary to recalibrate the models relating RSST results to mix parameters. Four of the models used are shown in Table 17.[9] The G* results were then adjusted as shown in Table 18 and new field *a* values were determined to define the effects of P_{200} and *fa* on mix rutting performance; Table 18 lists these values as well. The model in Table 17 with $R^2 = 0.767$ was used because it showed the largest impact of the independent variables, particularly fine aggregate, *fa*. This is illustrated in Table 19.

With these *a* values and the fixed *c* value, ESALs versus rut depth (*rd*) relationships were generated for the range in P_{200} and *fa* values shown in Table 18 for one asphalt content, 5.5 percent, and one air void content, 6.5 percent. An example of the influence of these parameters on ESALs to a rut depth of 10 mm (0.4 in.) is shown in Figure 37. A regression between ln(*rd*) and the independent variables ln(*ESAL*), P_{200}, *fa*, P_{200}^2, fa^2, $P_{200} \cdot fa$, $P_{200} \cdot \ln(ESAL)$, and $fa \cdot \ln(ESAL)$ was then generated. With an adjusted R^2 of 0.973 and showing only variables statistically significant at the 5 percent level, the following is the resulting equation:

$$\ln(rd) = a_0 + a_1 \ln(ESAL) + a_2 \cdot P_{200} + a_3 \cdot fa \qquad (32)$$

[9] As seen from Table 16, it was only necessary to treat G*.

where a_0, a_1, a_2, a_3 = regression coefficients shown in the following summary.

	Value	Standard Error	P-value
Constant	–2.04253	0.0855	
ln(*ESAL*)	0.362551	0.00373	0.0000
P_{200}, a_2	–0.0891765	0.00918	0.0000
fa^2, a_3	–0.0001491935	0.0000204	0.0000

In this equation, *rd* is the downward rut depth expressed in millimeters. The P_{200} and *fa* contents are expressed as percentages by weight of aggregate. The dataset used for the calibration encompassed rut depths from 2 mm to 18 mm (0.08 in. to 0.71 in.), P_{200} contents from 5 to 7 percent, and *fa* contents from 20 to 36 percent. The equation cannot be used with confidence outside these ranges.

The effects of this model are demonstrated in Figures 38 through 41. The influence of mineral filler and fine aggregate on ESALs to a given rut depth are illustrated by Figures 38 and 39. The effect seems to be significant, at least for the P_{200}. One might envision a contractor missing the P_{200} target by 2 percent and, hence, reducing the rutting life by about 50 percent.

As indicated by Figures 40 and 41, the effect of P_{200} and *fa* on the rut depth at a given level of ESALs is much less dramatic. For trafficking at a level of 10 million ESALs, the rut depth decreases from 25.5 mm (1 in.) at 5 percent P_{200} to 21.3 mm (0.84 in.) at 7 percent mineral filler (both for 28 percent *fa*). For the same level of trafficking, the rut depth decreases from 24.7 mm (0.97 in.) at 20 percent *fa* to 21.6 mm (0.85 in.) at 36 percent *fa* (both for 6 percent P_{200}).

4.3.4 Summary and Recommendations

In this section, a series of models have been presented to define the effects of mix variables on permanent deformation. These models have been divided into two levels: level 1 based on direct regression and level 2 based on a combination of M-E modeling and regression.

For the level 1 analyses, equation 29 is recommended for use. This equation is recommended because it tends to minimize the effects of some of the early rutting observed in some of the WesTrack sections.

The level 2 analyses, using an M-E procedure incorporating layered elastic analysis, permit the use of different temperature regimes and traffic distributions than those occurring at WesTrack.

The level 2A procedure provides a direct use of mix parameters to define the parameters *a* and *c* to be used in the rutting procedure for a specific mix representative of one of the types used at WesTrack. This procedure requires the use of equations 30 and 31 to define the *a* and *c* parameters for the specific mix under consideration so long as it conforms to one of the three general mix types used at WesTrack.

Level 2B requires that the RSST-CH be performed on the specific mix which is intended for use in the pavement system and the use of the regression equations for *a* and *c* shown in Tables 13 and 14 (regression no. 6).

4.4 FATIGUE CRACKING

4.4.1 Introduction

As with permanent deformation, the objective has been to provide models which can be incorporated in the PRS program and which define the influence of asphalt content, air void content, and aggregate gradation on the development of fatigue cracking in the AC layer.

Two levels of models have been developed: those based on regressions between fatigue cracking, traffic loading, and mix variables (level 1); and those based on M-E analyses to predict performance based on laboratory-measured fatigue response and stiffnesses of the pavement layers (level 2).

4.4.2 Regression Modeling

In Section 4.4.3, it will be noted that linear regression is used to relate load repetitions associated with fatigue damage and mix variables in the laboratory tests. For direct comparisons of field performance (cracking) and mix variables, however, linear regression was not considered appropriate because of sample bias (i.e., it only considers mixes which cracked). Accordingly, two models were developed: a Probit model for crack initiation and a continuous model for crack propagation in which the dependent variable is the expected value of wheelpath cracking. For crack initiation, the Probit model was selected because it permits the use of observed field performance data for all 26 original test sections. In the Probit model, the dependent variable is the indication of cracking termed INDCR. For each condition survey, if cracking is observed, INDCR = 1; otherwise it has a zero value.

For a 10 percent probability of cracking, the model for fine and fine plus mixes from the original sections is as follows:

$$\begin{aligned}\text{Prob}(INDCR = 1) = \Phi[&-49.502 + 4.88 \cdot \ln(ESAL)\\ &- 5.245 \cdot P_{asp} + 1.148 \cdot V_{air}\\ &- 2.301P_{200}]\end{aligned} \qquad (33)^{10}$$

where Φ is the cumulative density function of the normal distribution.

For the coarse mixes in the original sections, the model is as follows:

$$\begin{aligned}\text{Prob}(INDCR = 1) = \Phi[&-47.151 + 5.293 \cdot \ln(ESAL)\\ &-5.996 \cdot P_{asp} + 0.450 \cdot V_{air}]\end{aligned} \qquad (34)$$

Examples of the performance predicted by these equations are shown in Figures 42, 43, and 44. It will be noted that both asphalt content and degree of compaction (as measured by air void content) have a significant influence on fatigue performance.

Unfortunately, the PROBIT form of the level 1 fatigue model was not conducive to incorporation within the PRS software. What was required was a model which would calculate the percentage of fatigue cracking directly. Therefore, a series of mathematical adaptations was performed using the PROBIT model to develop models which could directly calculate the percentage of fatigue cracking. These models are presented as equations 35 and 36. A complete discussion of the mathematical steps taken as a part of these adaptations is provided in Appendix A of Part II.

Composite Model for Fine-Graded Mixes

$$\begin{aligned}\text{FC}(\%) = [&1.2313 + 0.071655 * \log(ESAL)\\ &+ 0.2358 * \log(\varepsilon) + 0.061193 * \log(E^*)\\ &- 0.034086 * P_{asp} + 0.0074593 * V_{air}\\ &- 0.014954 * P_{200}]^{154.04}\end{aligned} \qquad (35)$$

Composite Model for Coarse-Graded Mixes

$$\begin{aligned}\text{FC}(\%) = [&1.2850 + 0.07478 * \log(ESAL) + 0.2461\\ &* \log(\varepsilon) + 0.06386 * \log(E^*) - 0.036791\\ &* P_{asp} + 0.002761 * V_{air}]^{147.73}\end{aligned} \qquad (36)$$

where

FC (%) = percent of the wheel path area exhibiting fatigue,
ε = maximum tensile strain in asphalt layer (in./in.), and
E* = asphalt mixture dynamic modulus (psi).

For crack propagation, a continuous regression model was developed in which the dependent variable is the expected value, $|E|$, of wheelpath cracking (CRX):

$$|E| = [\log(\text{CRX}) \mid \text{INDCR} = 1] \qquad (37)$$

where

$|E|$ = expected value of wheelpath cracking,
CRX = extent of cracking (percent of wheelpath), and
INDCR = indicator of cracking present (1) or absent (0).

This equation is a function of the same variables as the model for crack initiation and includes a correction factor for selectivity bias. Parameters for this model for the fine and fine plus and coarse mixes are as follows:

[10] An alternative relation without the P_{200} term is as follows:

$$\text{Prob}(INDCR - 1) = \Phi[-75.832 + 5.234 \cdot \ln(ESAL) - 3.072P_{asp} + 1.050V_{air}]$$

	Fine and Fine Plus		Coarse	
	Value	P-value	Value	P-value
constant	–7.98	0.000	116	0.000
ESALs	+0.176E-05	0.000	+0.105E-5	0.000
P_{asp}	–1.23	0.000	+10.5	0.000
V_{air}	+0.659	0.000	–2.13	0.000
P_{200}	+0.382	0.016	–27.1	0.016
correction, λ	+2.2	0.000	–0.604	0.130
R^2	0.56		0.51	

where

P-value = probability of rejecting a true hypothesis (type I error) and
λ = correction factor to fit expected value to observed values.

4.4.3 Mechanistic-Empirical Modeling

The approach used to predict fatigue cracking within the M-E framework includes flexural fatigue testing of the fine, fine plus, and coarse mixes; and performance predictions based on the models developed during SHRP that have been (a) extended to efficiently treat in situ temperatures and (b) calibrated to the Caltrans flexible-pavement design methodology. The refined models have been used previously in interpreting the results of the California heavy vehicle simulator (HVS) testing of pavement sections in the CAL/APT program (*17*).

As with the permanent deformation analyses, the pavement is treated as a multilayer elastic system. Figure 45 represents the idealization of the WesTrack pavement structure for the analyses performed herein. Modulus values for the base and subgrade are those reported in Chapter 2 of Part II for the north and south tangents of the test track. Stiffness moduli for the AC were represented by equations 14 and 15 for the fine and fine plus and coarse mixes, respectively.

The damage determinant for fatigue which has been used is the principal tensile strain, ε_t, at the underside of the asphalt-bound layer, Figure 45. Results of the laboratory fatigue tests on the three FMFC mixes from the original sections are as follows:

Fine mixes

$$\ln N_f = -27.0265 - 0.1439 V_{air} + 0.4148 P_{asp} - 4.6894 \ln \varepsilon_t \qquad R^2 = 0.88 \tag{38}$$

Fine plus mixes

$$\ln N_f = -27.3409 - 0.1431 V_{air} + 0.4219 P_{asp} + 0.0128 \ln T - 4.6918 \ln \varepsilon_t \qquad R^2 = 0.88 \tag{39}$$

Coarse mixes

$$\ln N_f = -27.6723 - 0.0941 V_{air} + 0.6540 P_{asp} + 0.0331 T - 4.5402 \ln \varepsilon_t \qquad R^2 = 0.92 \tag{40}$$

where

N_f = fatigue life load cycles;
V_{air} = air void content, percent;
P_{asp} = asphalt content, percent;
T = temperature at 150 mm (6 in.), °C (°F = 1.8°C + 32); and
ε_t = maximum tensile strain.

Simulation of performance of the 26 sections covered the time period March 3, 1996, through October 2, 1998. The approach is summarized as follows.

Approach to Performance Simulation. The framework for the analysis of the performance of the test sections is illustrated in Figure 46. The simulations attempted to reflect not only the influence of mix variables on the stiffness and fatigue characteristics of the 26 mixes, but also the following: traffic-hourly variations and wander; temperature-hourly variations with depth; and variations in subgrade stiffness according to Figures 19 and 20.

The *stiffness and fatigue response* for the various WesTrack mixes are represented by equations 14, 15, 38, 39, and 40. It will be noted for the fine and fine plus mixes that only one stiffness relationship has been developed but separate fatigue relationships are used. *Temperature data* for the test site were obtained from the WesTrack database. Temperature at the bottom of the 150-mm (6-in.) AC layer, T_b, was determined on an hourly basis as was the temperature gradient, which is defined in this instance as follows:

$$g = \frac{T_{50} - T_{150}}{100} \tag{41}$$

where

T_{50} = temperature at 50 mm (2 in.) depth, °C and
T_{150} = temperature at 150 mm (6 in.) depth, °C.

Equation 12 was used to determine the *subgrade stiffness* on a daily basis.

To stratify the data in cells, subintervals were identified for each of the variables as follows:

1. for T_b, °C (°F): –10 to 0 (14 to 32), 0 to 10 (32 to 50), 10 to 20 (50 to 68), 20 to 30 (68 to 86), 30 to 40 (86 to 104), 40 to 50 (104 to 122)
2. for g: –0.15 to –0.1, –0.1 to –0.05, –0.05 to 0, 0 to 0.05, 0.05 to 0.1, 0.1 to 0.15
3. for E_{sub}, MPa (ksi): 90 to 95 (13.1 to 13.8), 95 to 100 (13.8 to 14.5), 100 to 105 (14.5 to 15.2), 105 to 110 (15.2 to 16.0), 110 to 115 (16.0 to 16.7), 115 to 120 (16.7 to 17.4)

While a total of 6 × 6 × 6 = 216 cases were identified, not every combination contained observed field data with the result that the number was reduced to 110 cases. Mean values were then developed based on the frequency of occurrence of specific values in each of the cells for the three parameters. It should be noted that E_{sub} would be the same for 24 hours in a single day.

To evaluate the *maximum principal tensile strain* at the underside of the AC, while taking account of the change in stiffness as temperature varies with depth, this layer was subdivided into three sublayers, each with a thickness of 50 mm (2 in.). While the stiffness of the AC changed with temperature and subgrade stiffness varied with season, the modulus of the base was maintained at a constant value of 172.4 MPa (25,000 psi). Strains were calculated using the multilayer elastic analysis program CIRCLY (*19*).

Because the stiffness of the AC mixes varied both with air void content and asphalt content (equations 14 and 15), a range of these parameters was used which encompassed the measured values. This required $110 \times 3 \times 3 = 990$ cases for both the fine and fine plus mixes and the coarse mix. For each case, strains were determined at 51 points on the underside of the AC. Figure 47 illustrates the variation of tensile strain throughout the AC layer and Figure 48 the variation of strain with offset from the centerline of the dual wheels at the underside of this layer.

To simplify the computations of damage, a function was fit to the strain distribution, an example of which is shown in Figure 48. This was accomplished by superimposing the strain distribution for each tire assuming normal distributions of strain resulting from both tires. The equation resulting from this superimposition has the following form:

$$f(m_1, m_2, \sigma) = \frac{A}{\sigma\sqrt{2\pi}} \left[\exp\left(-\frac{(x-m_1)^2}{2\sigma^2}\right) + \exp\left(-\frac{(x-m_2)^2}{2\sigma^2}\right)\right] \tag{42}$$

where

x = offset (m),
A = multiplication factor,
m_1 = mean position of the left tire strain distribution,
m_2 = mean position of the right tire strain distribution, and
σ = standard deviation for both normal distributions.

From Figure 48, it will be noted that an expression based on equation 42 is suitable to represent the tensile strain distribution using the parameters m_1, m_2, A, and σ.

The next step was to relate m_1, m_2, A, and σ to the parameters T_b, g, E_{sub}, V_{air}, and P_{asp} to define a strain function representative of all possible combinations of the material and environmental variables for the WesTrack pavements.

Results of the regressions for the four parameters of equation 42 as a function of the five material and environmental variables are summarized in Table 20 for the coarse mix and Table 21 for the fine and fine plus mixes. Some of the terms shown in these tables are defined as follows:

$T_b \cdot g$ = product of the temperature at the bottom of the AC and the temperature gradient.
$T_b \cdot E_{sub}$ = product of AC temperature (at bottom) and subgrade modulus.
$(T_b \cdot g)^2$ = square of product of bottom AC temperature and temperature gradient.
$(T_b \cdot E_{sub})^2$ = square of product of bottom AC temperature and subgrade modulus.
$T_b \cdot g \cdot V_{air}$ = product of bottom AC temperature, temperature gradient, and air void content.
$T_b \cdot g \cdot P_{asp}$ = product of bottom AC temperature, temperature gradient, and asphalt content.
$T_b \cdot E_{sub} \cdot V_{air}$ = product of bottom AC temperature, subgrade modulus, and air void content.
$T_b \cdot E_{sub} \cdot P_{asp}$ = product of bottom AC temperature, subgrade modulus, and asphalt content.

To illustrate the *fatigue performance* simulation, results for section 8 are used as an example.

The first step is to construct the basic input matrix which contains T_b, g, E_{sub}, V_{air}, and P_{asp} on an hourly basis:

$$\text{Input Matrix} = \begin{bmatrix} T_b & g & E_{sub} & V_{air} & P_{asp} \\ \vdots & \vdots & \vdots & \vdots & \vdots \\ \vdots & \vdots & \vdots & \vdots & \vdots \end{bmatrix}_{21943\times 5} \tag{43}$$

The number 21943 represents the total number of hours during which traffic was applied at WesTrack.

The strain function matrix for each hour can likewise be generated:

$$\text{Strain Function Matrix} = \begin{bmatrix} m_1 & m_2 & \sigma & A \\ \vdots & \vdots & \vdots & \vdots \\ \vdots & \vdots & \vdots & \vdots \end{bmatrix}_{21943\times 4} \tag{44}$$

The four parameters define the shape of the tensile strain versus offset relationship, which is a continuous function for each hour. Thus, the tensile strain can be determined for wander with any offset. The location which incurs the most damage is defined as the local maximum when the applied traffic is in a “no wander” position. To determine the location of the local maximum tensile strain the Newton-Raphson algorithm was used.

For the WesTrack wander pattern, a strain matrix for a particular section, in this case section 8, can thus be formed:

$$strn8 = \begin{bmatrix} 5L & 4L & 3L & 2L & 1L & Center & 1R & 2R & 3R & 4R & 5R \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \end{bmatrix}_{21943\times 11} \tag{45}$$

where $5L \ldots 5R$ represent the 11 positions of the truck sensors.

Once the strain has been calculated for each point for each hour, cycles to failure are obtained from the laboratory fatigue relationship:

$$(N_f)_{\text{Section 8}} = \exp\left\{ -27.6723 - 0.0941 \begin{bmatrix} V_{air} \\ \vdots \\ \vdots \\ \vdots \end{bmatrix} + 0654 \begin{bmatrix} P_{asp} \\ \vdots \\ \vdots \\ \vdots \end{bmatrix} + 0.0331 \begin{bmatrix} T_b \\ \vdots \\ \vdots \\ \vdots \end{bmatrix} - 4.5402 \bullet \ln(\varepsilon_t)_{\text{Section 8}} \right\} \tag{46}$$

This, like the other matrices, has a dimension of 21943×11.

The next step was to form the ESALs matrix with the same dimensions as equation 46:

$$n - ESALs = \begin{bmatrix} E_{5L} & E_{4L} & E_{3L} & E_{2L} & E_{1L} & E_{center} & E_{1R} & E_{2R} & E_{3R} & E_{4R} & E_{5R} \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \end{bmatrix}_{21943 \times 11} \qquad (47)[11]$$

Cumulative damage was then determined using the linear sum of cycle ratios (cumulative damage by Miner's hypothesis):

$$\sum_{i=1}^{n} \frac{n_i}{N_i} = 1 \tag{48}$$

where

n_i = applied ESALs for each entry of the ESALs matrix (i.e., for the *i*th time period) and

N_i = number of load applications to failure for section 8 from the strain matrix (for the *i*th time period).

The matrix for this computation was as follows:

$$\frac{ESALs}{(N_f)_{\text{Section 8}}} = \begin{bmatrix} \frac{E_{5L}}{N_{f5L}} & \frac{E_{4L}}{N_{f4L}} & \frac{E_{3L}}{N_{f3L}} & \frac{E_{2L}}{N_{f2L}} & \frac{E_{1L}}{N_{f2L}} & \frac{E_{center}}{N_{fcenter}} & \frac{E_{1R}}{N_{f1R}} & \frac{E_{2R}}{N_{f2R}} & \frac{E_{3R}}{N_{f3R}} & \frac{E_{4R}}{N_{f4R}} & \frac{E_{5R}}{N_{f5R}} \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \end{bmatrix}_{21943 \times 11} \tag{49}$$

The summation of the n_i / N_i *ratios* for each hour represents the damage caused by the traffic in that hour. The cumulative damage versus cumulative hourly ESALs can then be plotted as shown in Figure 49. In this figure, damage development for the WesTrack wander pattern and the without wander pattern is shown. It will be noted that a considerable difference in ESALs was obtained for the two conditions.

[11] Repetitions were uniformly distributed to the 11 positions when the trucks were operated with drivers.

The question arises as to the definition of fatigue damage as determined by this procedure. To this end, it is necessary to compare the predictions of crack initiation with measurements of surface cracking in the field. Table 22 summarizes predicted ESALs to crack initiation without and with wander and ESALs corresponding to observed fatigue cracking of 10 percent and 50 percent. These results are shown in Figures 50, 51, and 52. Table 23 lists construction variations in mix and structural characteristics.

In Figures 50 through 52, for a given test section, the first diamond point corresponds to predicted crack initiation based on the no-wander condition while the second diamond point corresponds to that with wander. The first x-mark corresponds to 10 percent observed surface cracking while the second x-mark corresponds to 50 percent surface cracking.

An analysis of the data shown in Figures 50 through 52 suggests the following:

For the coarse mixes in which crack initiation was predicted in the first summer, surface cracking appeared during the winter immediately following. This suggests that crack propagation time in the coarse mixes (of the type used at WesTrack) is relatively short, particularly if a cold period immediately follows a warm period leading to crack initiation when the strains are comparatively large.

On the other hand, the time between crack initiation and the appearance of surface cracks is substantially longer for the fine and fine plus mixes.

These data also suggest that the use of a single shift factor to relate laboratory damage to that observed in the field may not be appropriate for the coarse and fine and fine plus mixes, such a factor would be higher for the fine and fine plus mixes than for the coarse mix.

In spite of these differences, however, the test results support earlier studies stressing the importance of proper compaction and controlled asphalt content (*17*). This is particularly important for the coarse mixes. In addition, the results demonstrate the feasibility of the M-E approach and the breadth of its capabilities, including wander effects.

Suggested Procedures for Performance Prediction. To use the method described herein for a specific situation requires a measure of the stiffness characteristics of the asphalt-aggregate mix according to the form of equations 14 and 15. If such an equation is not available, the general stiffness equation developed for AC mixes by Witczak can be used. Stiffness characteristics of the other pavement components are also required to determine tensile strains using multilayer elastic analysis.

A fatigue relationship is required for the mix under consideration. It is recommended that this relation have the form of equations 38, 39, and 40. If such an expression is not available then one of the following expressions can be used (for dense-graded HMA only):

1. Fatigue expression used in the Asphalt Institute mix design methodology, including the correction factor for asphalt content and air void content (*16*).

2. The fatigue expression developed during the SHRP program and representative of mixes containing 16 different asphalt cements (*51*).

With this information, the framework shown in Figure 53 should be followed. Rather than using traffic wander as described in the analysis of the WesTrack pavements, the use of a shift factor is recommended as seen in this figure. Some of the elements of this approach are briefly described in the following paragraphs.

The approach shown in Figure 53 is based on that described in reference 19. This system specifically considers the mean and variance of asphalt content, air void content, and AC layer thickness.[12]

It is recommended that the distribution of fatigue lives be determined by a Monte Carlo simulation for the specific section under consideration. This requires that a random selection be made of the variables under consideration (i.e., asphalt content, air void content, and AC thickness).

Each of the Monte Carlo simulations produces an independent estimate of the laboratory fatigue life, N. The corresponding, simulated in situ fatigue life is determined by applying a shift factor, *SF*, and a temperature conversion factor, *TCF*, as follows:

$$\text{in situ life} = (N \cdot SF)/TCF \qquad (50)$$

Suggested shift factors are as follows:

- For WesTrack-type mixes, that is, mixes having Superpave-type aggregate gradations in the coarse or fine areas of the 0.45 grading chart, approximate values of *SF* are as follows:

  coarse mix 2–3
  fine mix 10–20

- For other types of dense-graded mixes prepared with conventional asphalt cements:

$$SF = 3.2 \times 10^{-5}\ \varepsilon_t^{-1.38} \quad \text{for } \varepsilon_t > 0.000040 \qquad (51)$$

The *TCF* is dependent on region and appears to follow an equation of the form:

$$TCF = a \ln(t_{AC}) + b \qquad (52)$$

where

t_{AC} = asphalt layer thickness, cm and
a, b = coefficients dependent on region.

For three California locations these coefficients are as follows:

[12] Aggregate gradation can also be considered, although it did not appear specifically in equations 38, 39, and 40 for the three mixes used in the experiment.

Location	*a*	*b*
California coastal region	1.754	–2.891
High desert	2.102	–3.884
Mountains (elev. ~1300 m)	1.448	–2.475

From the results of these computations, the effects of the mix parameters can be defined. For example, the effects of as-constructed air void content on pavement performance might take the form shown in Figure 54.

It is recommended, however, that a procedure like that described in reference 51 be followed to determine the *TCF* for a specific locale.

From the results of these computations, the effects of the mix parameters can be defined. For example, the effects of as-constructed air-void content on pavement performance might take the form shown in Figure 54. The results are based on assumptions for construction variations shown in Table 23.

4.5 SUMMARY AND RECOMMENDATIONS

For fatigue, two models have been presented to define the effects of mix and structural (HMA thickness) variables on performance. The level 1 model is based on direct regression using a Probit model rather than linear regression, while the level 2 model is based on M-E analysis.

The level 1 model does not include the effects of thickness because only one thickness of HMA was used in the test pavement. While not recommended for use at this time, a Probit regression model is included in Appendix A that does incorporate HMA thickness as a parameter. This model was developed by combining the results of the WesTrack experiment with a performance model for fatigue developed as a part of the CAL/APT program. Comparisons of the results of the WesTrack model with the Caltrans fatigue model indicate that the WesTrack model shows more effect of asphalt content and less effect of air-void content on mix performance than the Caltrans relationship shows.

For the level 2 model, the user should measure stiffness and fatigue response characteristics in the laboratory to define the effects of mix variables for mixes representative of the area. If this is not feasible, guidelines are provided for data that might be used, depending on the type of mix.

Simulations of mix performance at WesTrack show similar effects of compaction and asphalt content on fatigue performance to those that have been reported elsewhere (e.g., reference 52). The simulations also indicated that cracks, once formed, appeared to propagate more quickly in the coarse mixes at WesTrack than they did in the fine and fine plus mixes.

Finally, the results of the analyses presented in this chapter reinforce the effectiveness of the M-E approach in defining performance models for fatigue in HMA.

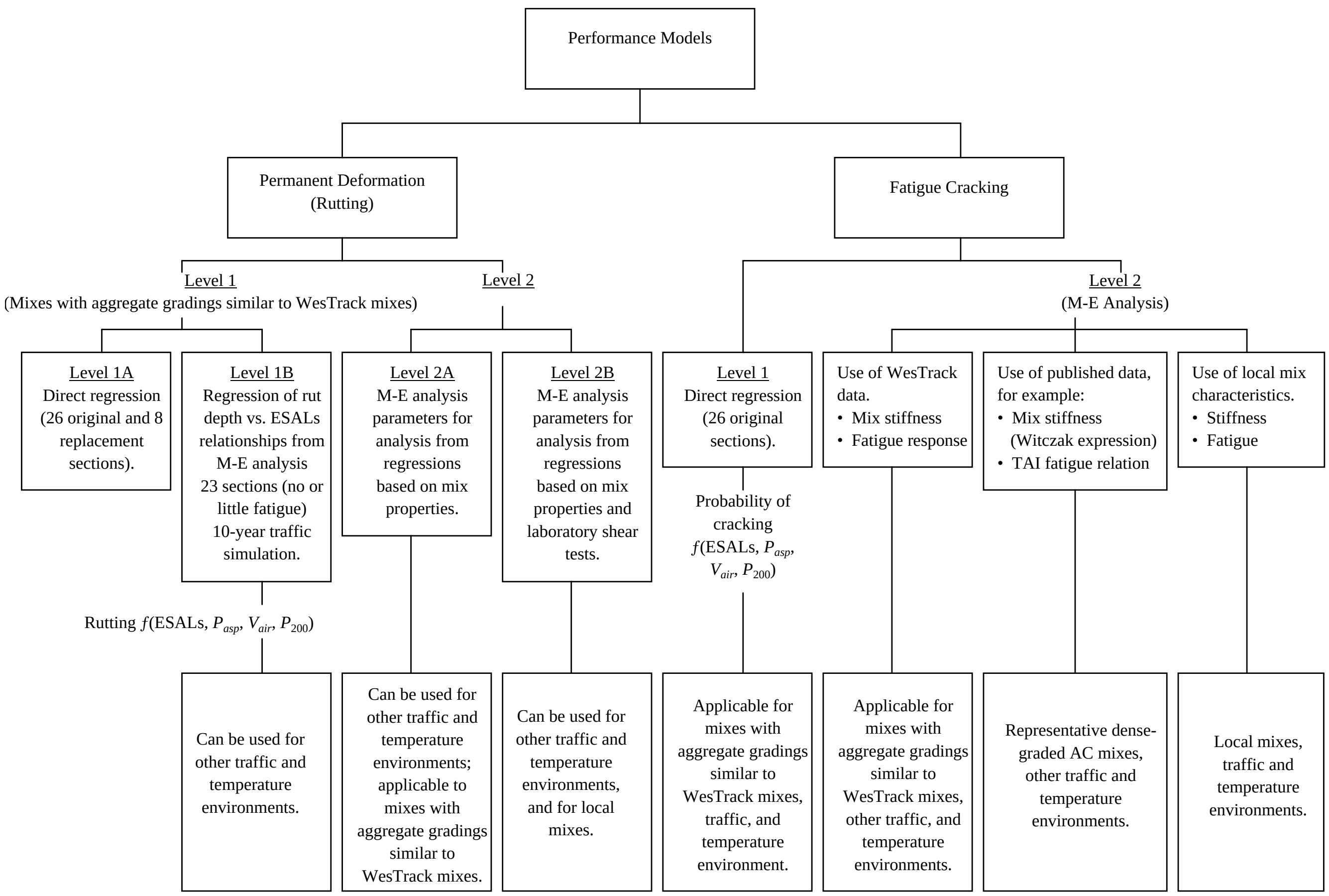


Figure 12. Performance model framework.

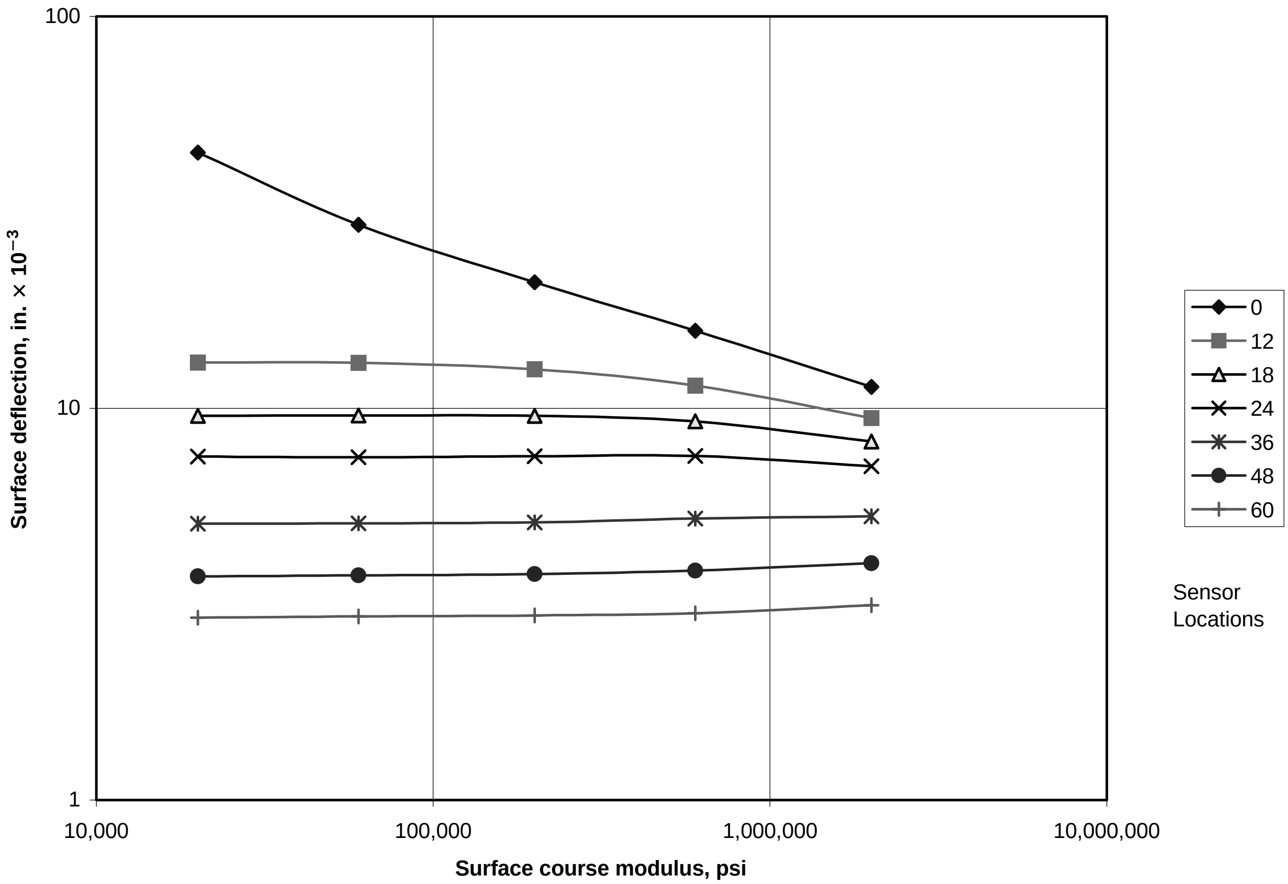


Figure 13. Illustration of effect of surface course modulus on surface deflection; $E_2 = 30{,}000$ psi, $E_3 = 15{,}000$ psi (1 in. = 25.4 mm, 1 psi = 6.9 kPa).

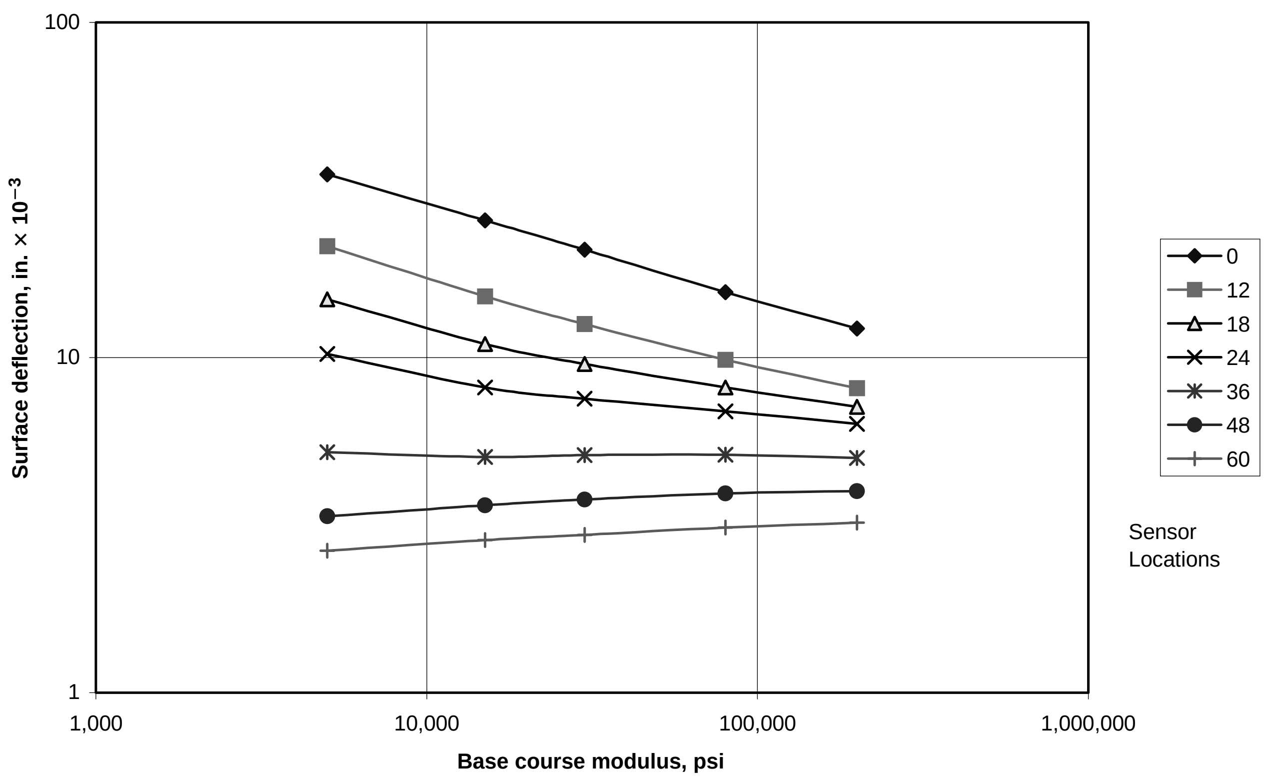


Figure 14. Illustration of effect of base course modulus on surface deflection; $E_1 = 200{,}000$ psi, $E_3 = 15{,}000$ psi (1 in. = 25.4 mm, 1 psi = 6.9 kPa).

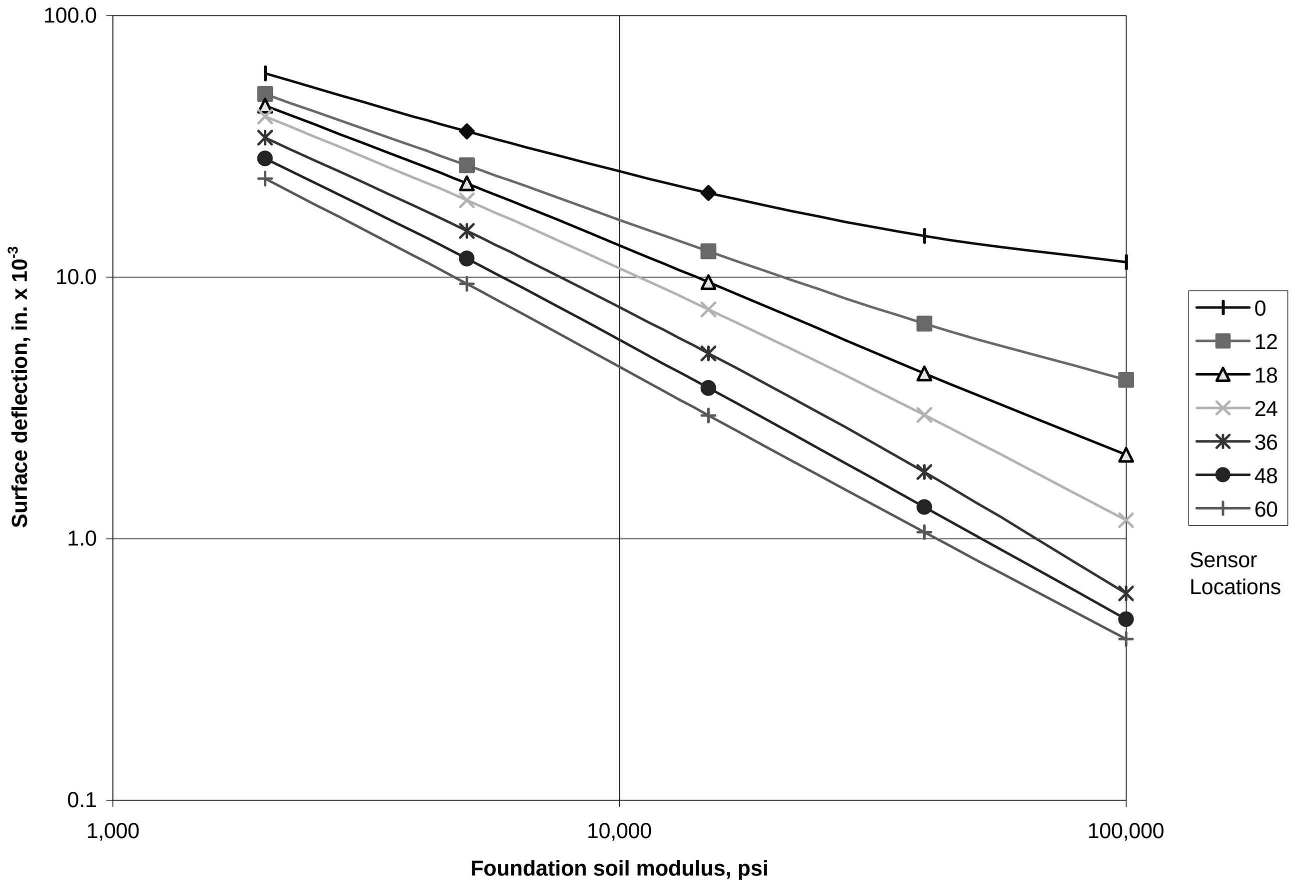


Figure 15. Illustration of effect of foundation soil modulus on surface deflection; E_1 = 200,000 psi, E_2 = 30,000 psi (1 in. = 25.4 mm, 1 psi = 6.9 kPa).

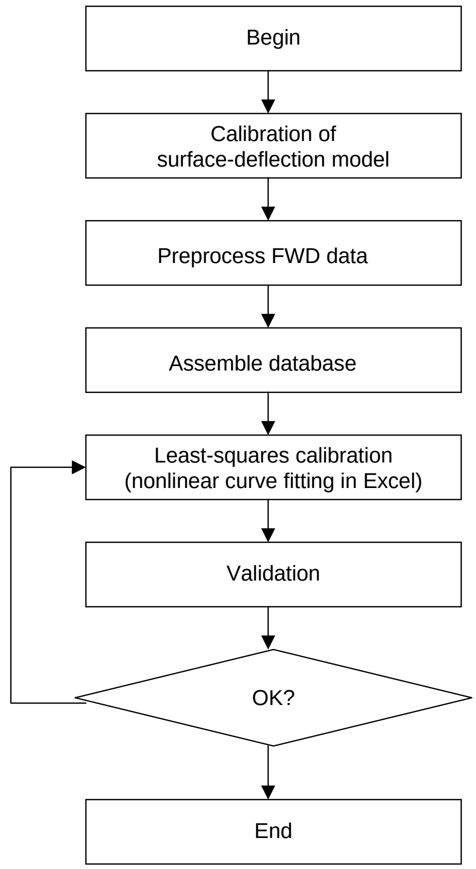


Figure 16. Framework for moduli determination.

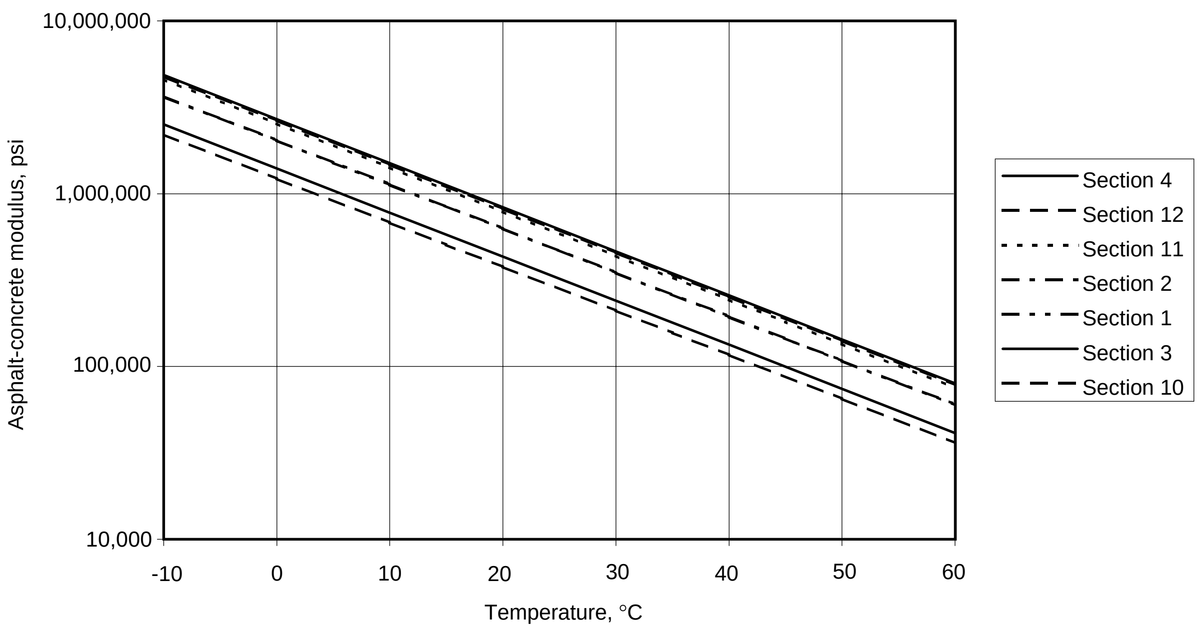


Figure 17. FWD-determined moduli for asphalt mixes, south tangent (1 psi = 6.9 kPa, °F = 1.8°C + 32).

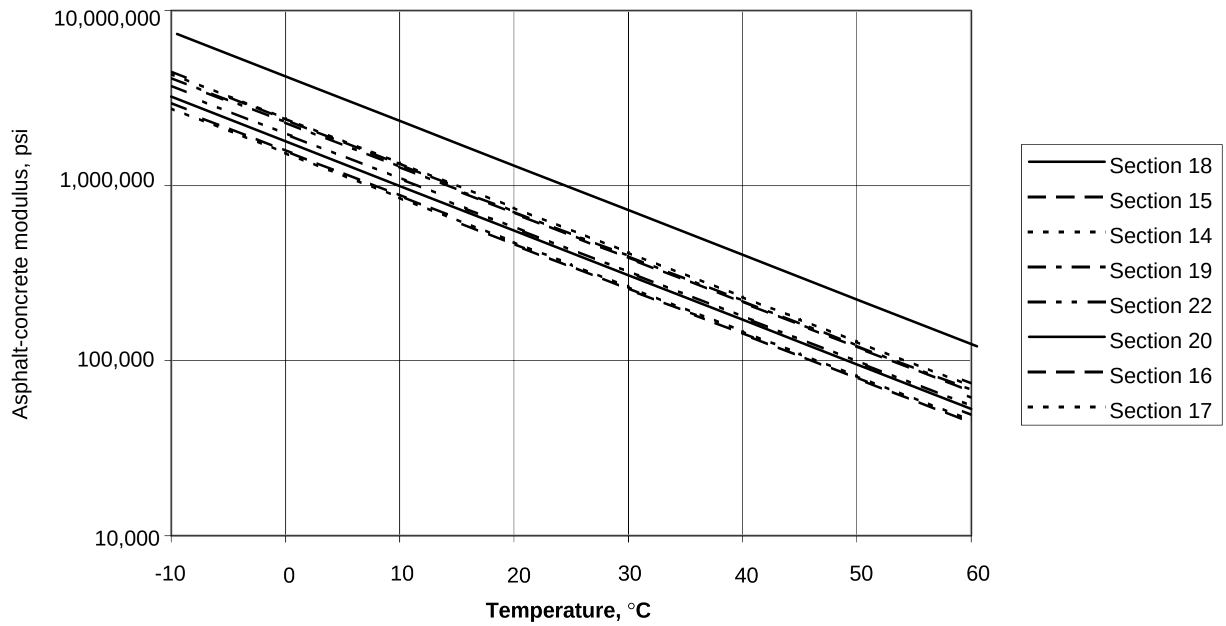


Figure 18. FWD-determined moduli for asphalt mixes, north tangent (1 psi = 6.9 kPa, °F = 1.8°C + 32).

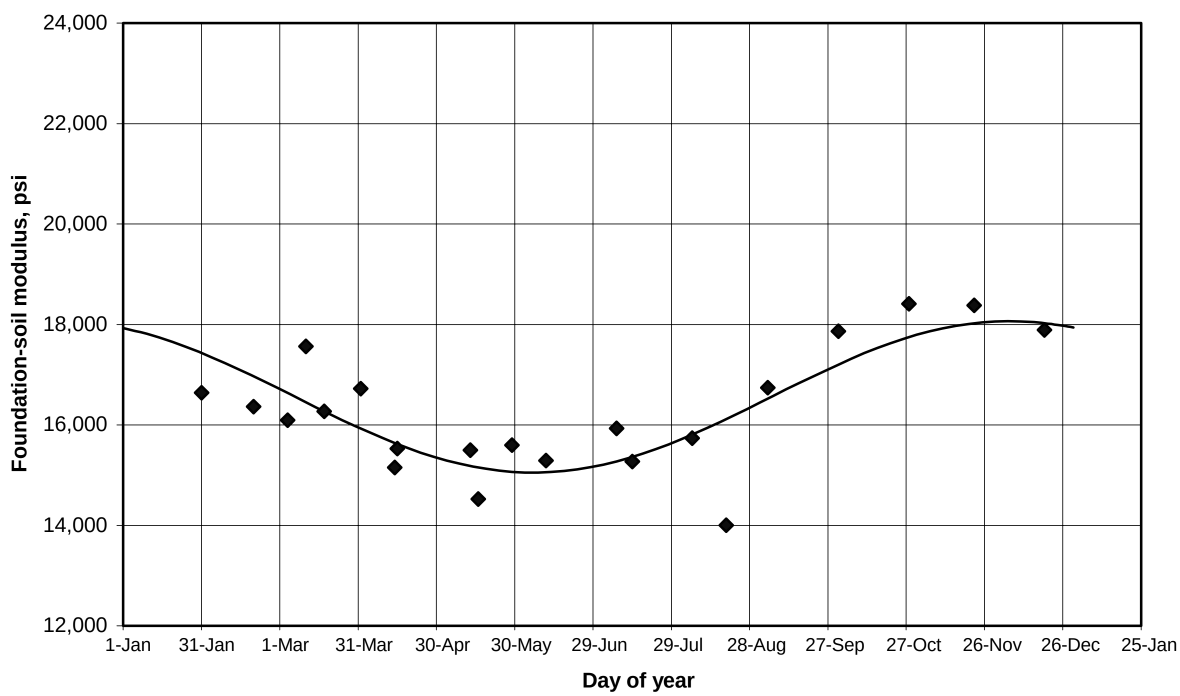


Figure 19. Sinusoidal influence of season on foundation-soil modulus, south tangent (1 psi = 6.9 kPa).

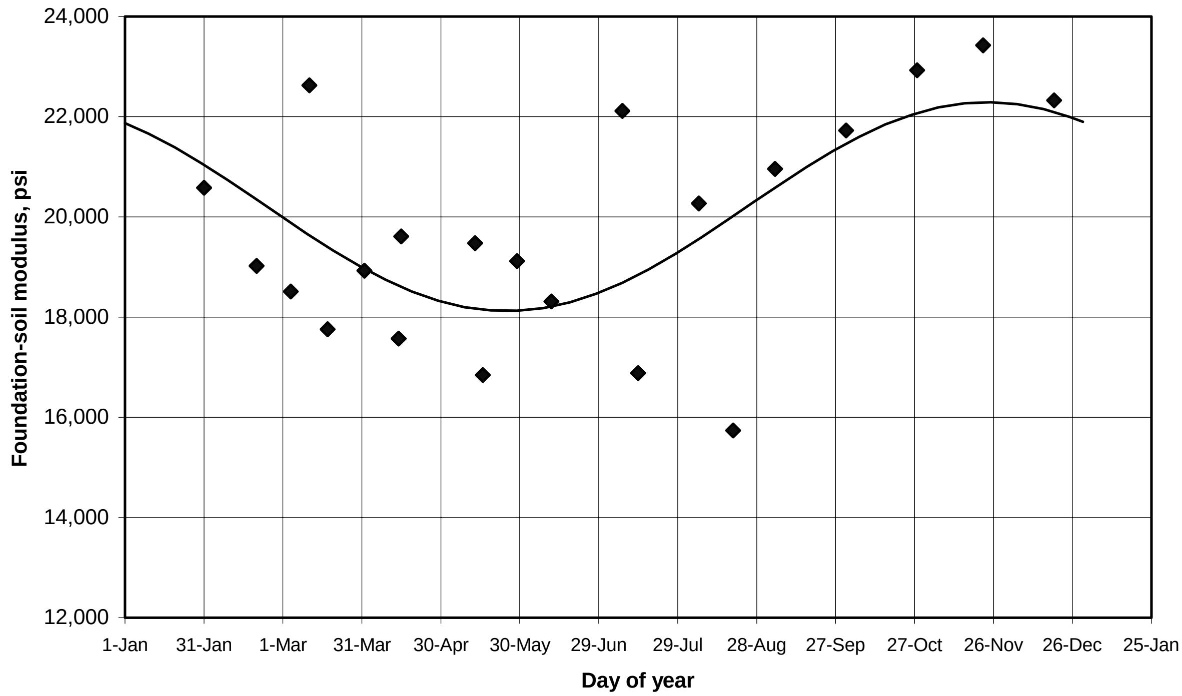


Figure 20. Sinusoidal influence of season on foundation-soil modulus, north tangent (1 psi = 6.9 kPa).

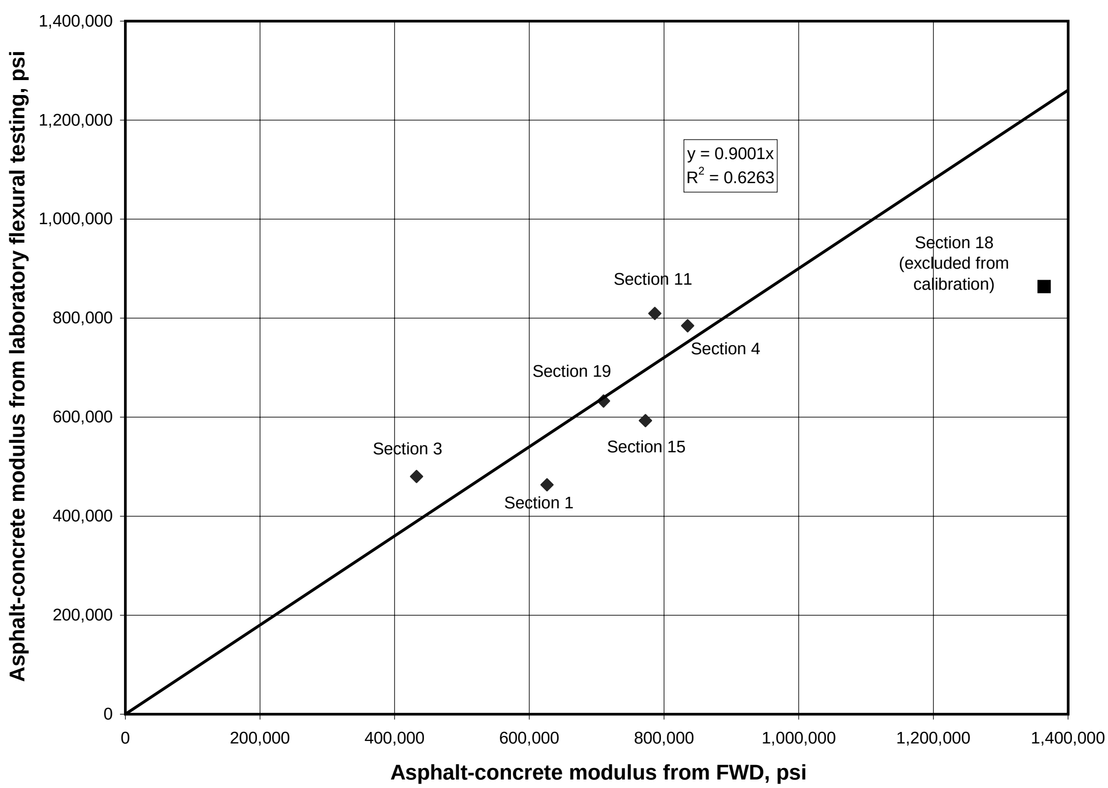


Figure 21. Comparison of laboratory-determined flexural stiffness moduli at 20°C (68°F) and moduli determined from FWD measurements (1 psi = 6.9 kPa).

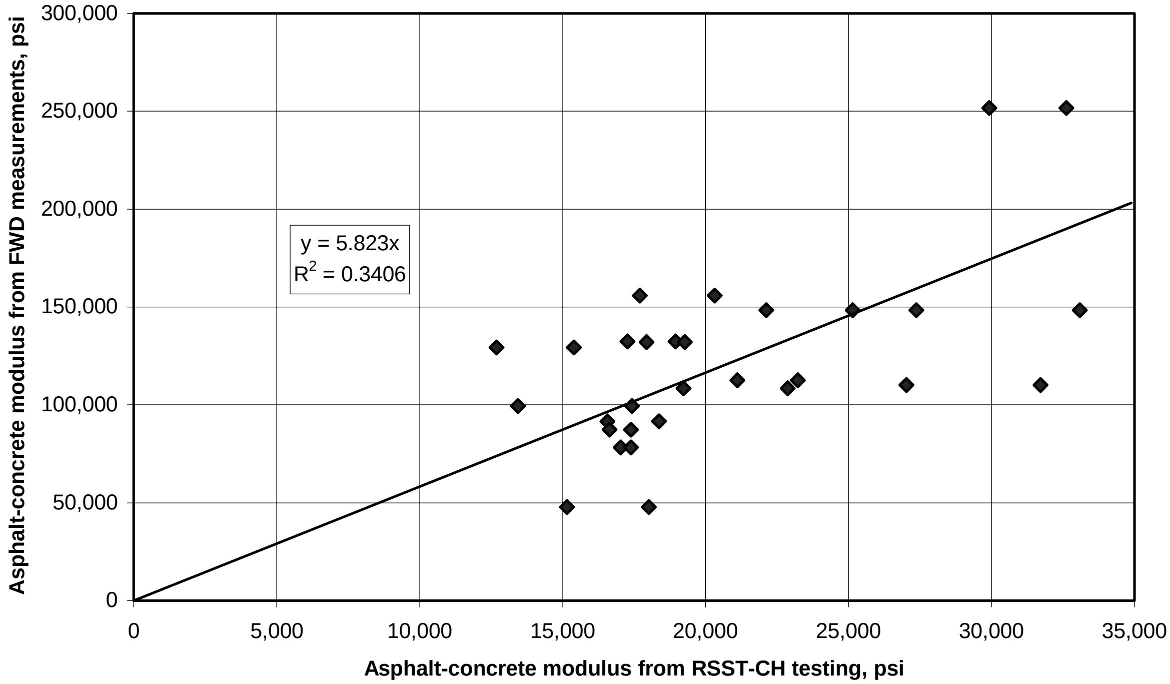
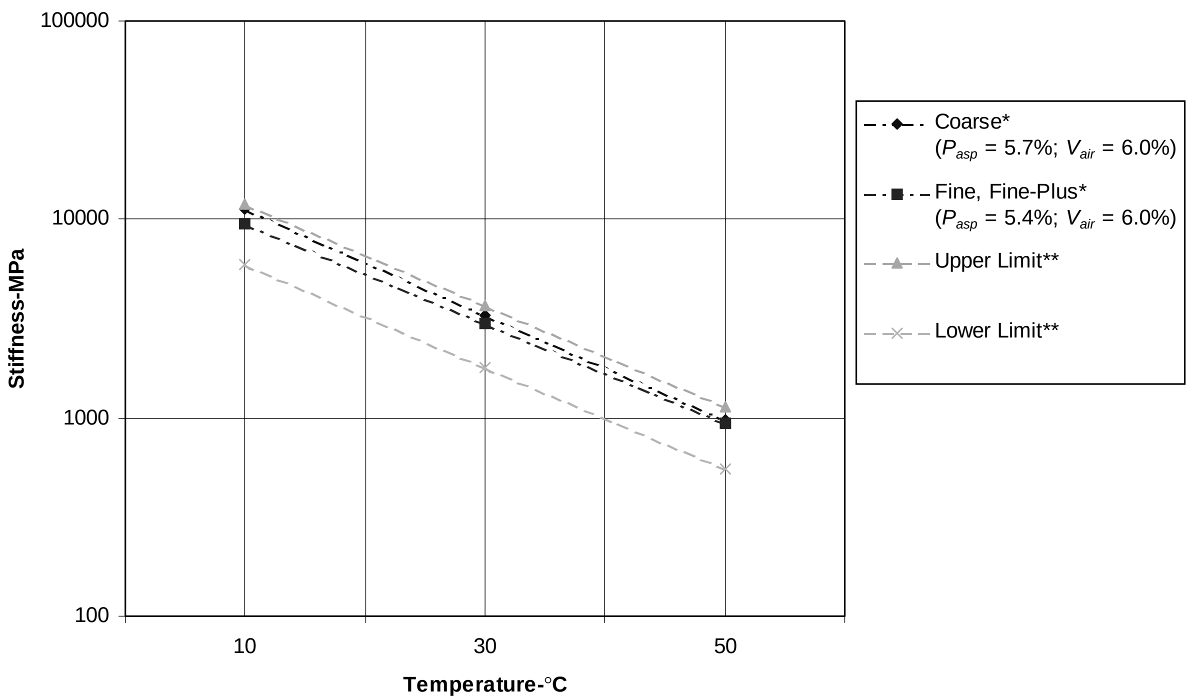


Figure 22. *Comparison of laboratory-determined stiffness moduli determined from RSST-CH tests at 50°C (122°F) and FWD measurements; shear stress = 10 psi (69 kPa), and Poisson's ratio of 0.35 (1 psi = 6.9 kPa).*



*according to equations 14, 15

**Range in stiffness used in permanent deformation analyses; equation 11

Figure 23. *Comparison of moduli used in permanent deformation analyses with those from equations 14 and 15 used in fatigue analyses, original sections (1 ksi = 6.9 MPa, °F = 1.8°C + 32).*

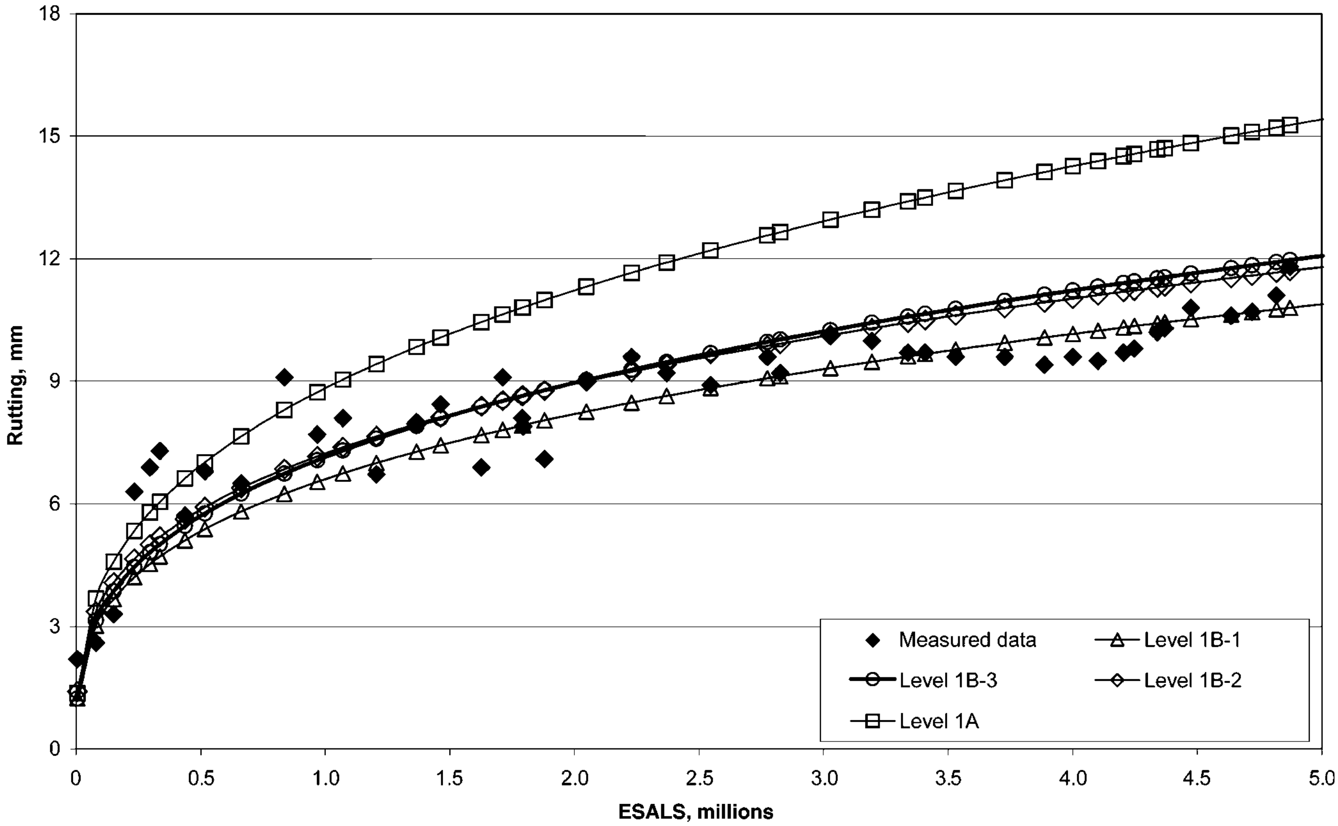


Figure 24. Comparison of rut depth versus ESALs for four regression models and measured rut depths, section 1 (1 in. = 25.4 mm).

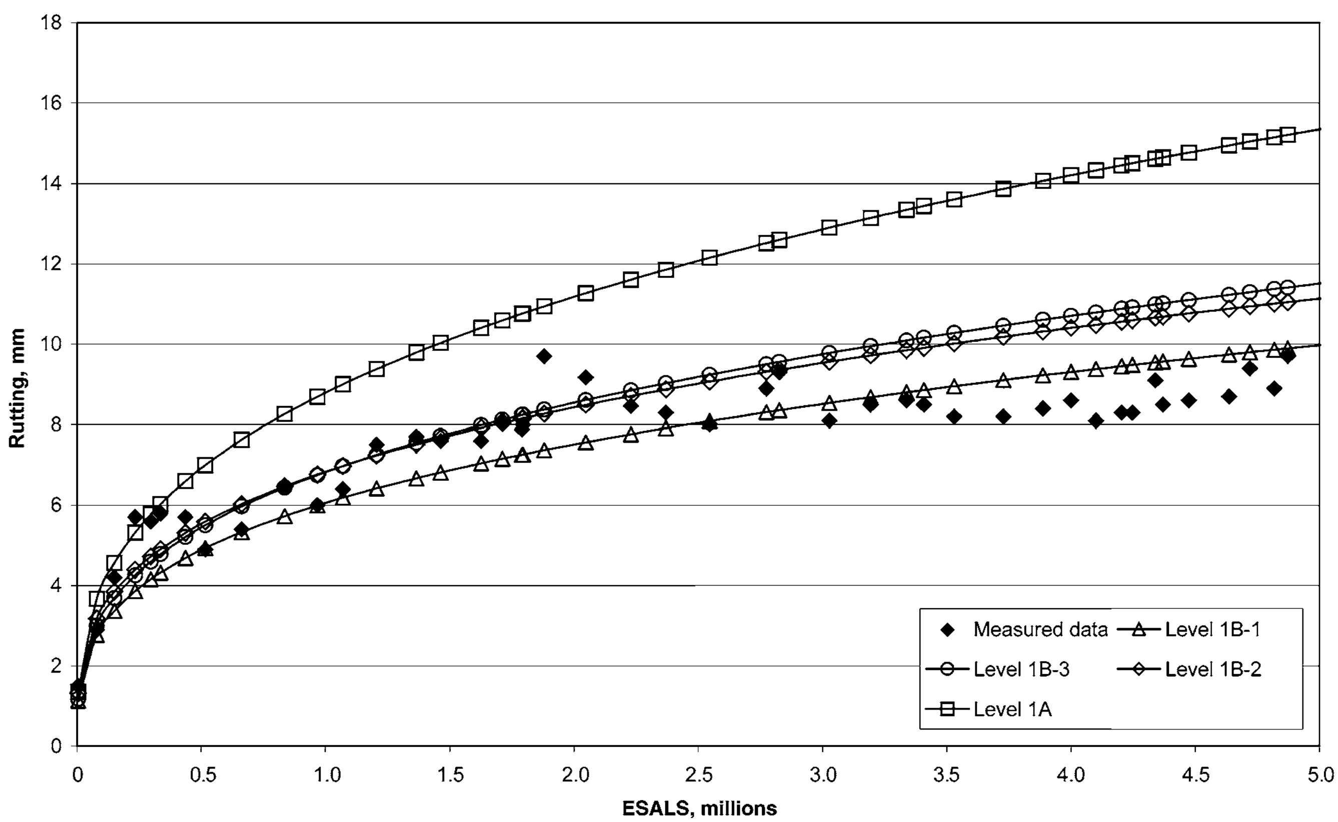


Figure 25. Comparison of rut depth versus ESALs for four regression models and measured rut depths, section 15 (1 in. = 25.4 mm).

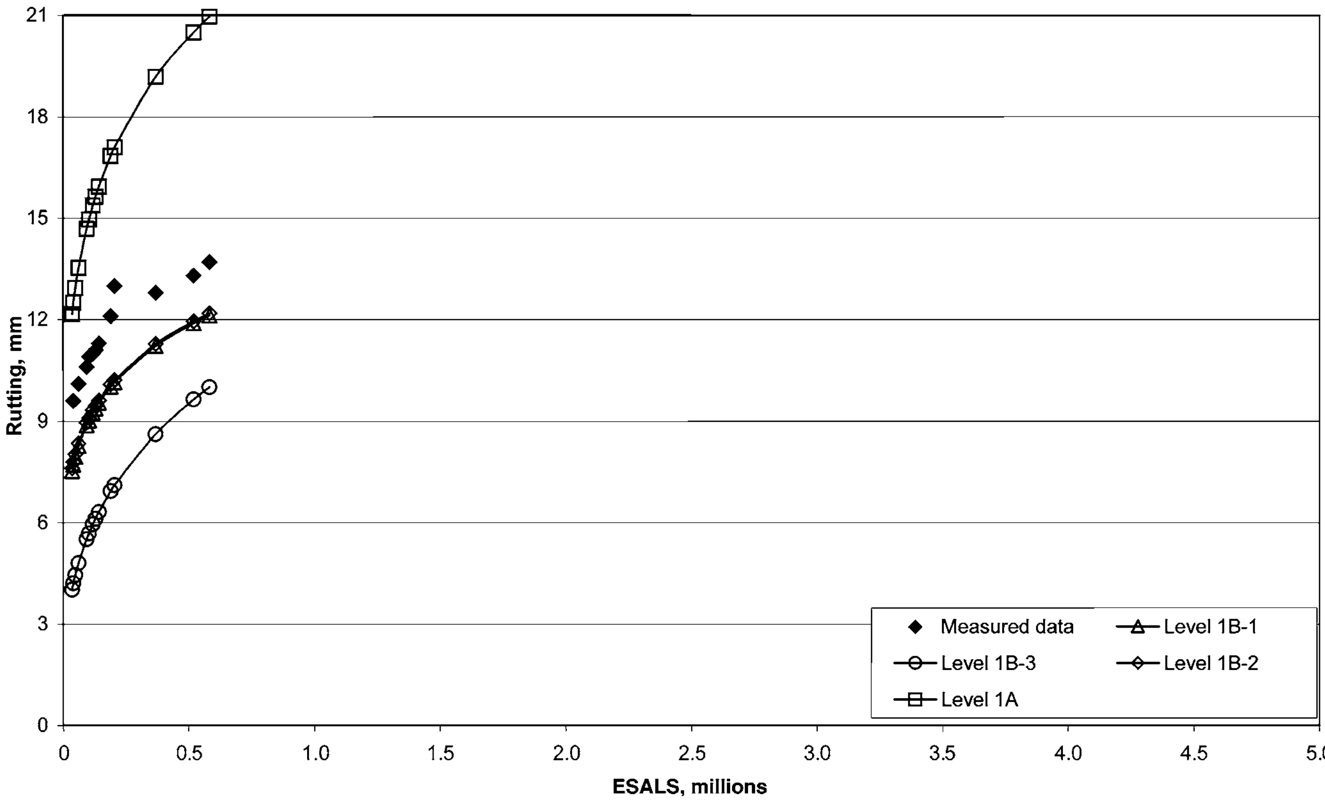


Figure 26. Comparison of rut depth versus ESALs for four regression models and measured rut depths, section 54 (1 in. = 25.4 mm).

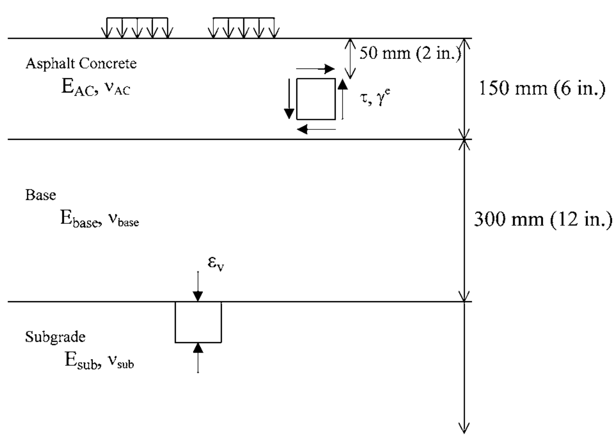


Figure 27. WesTrack pavement representation for M-E modeling for rutting.

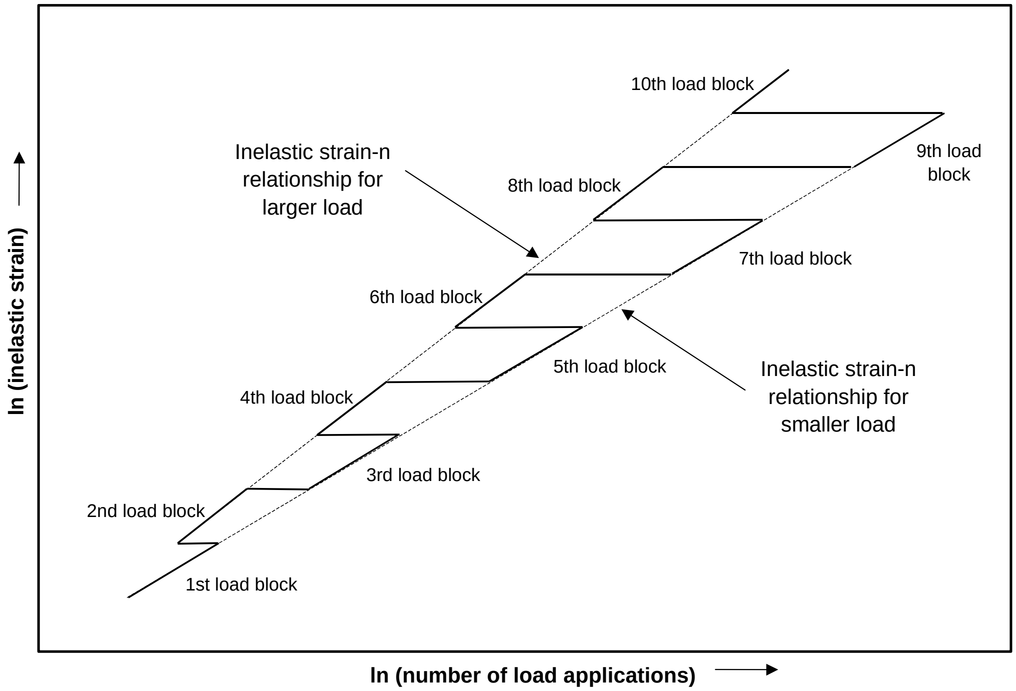


Figure 28. Time-hardening procedure for plastic strain accumulation under stress repetitions of different magnitudes in compound loading.

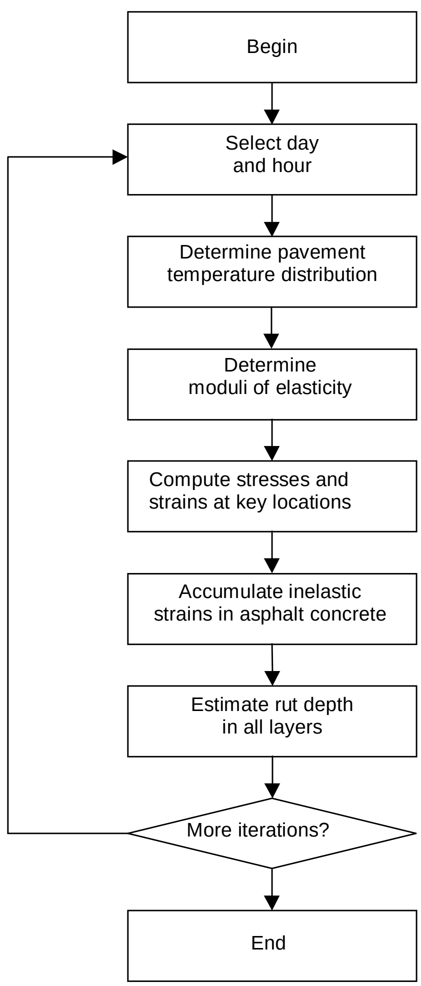


Figure 29. Framework for rut-depth estimates

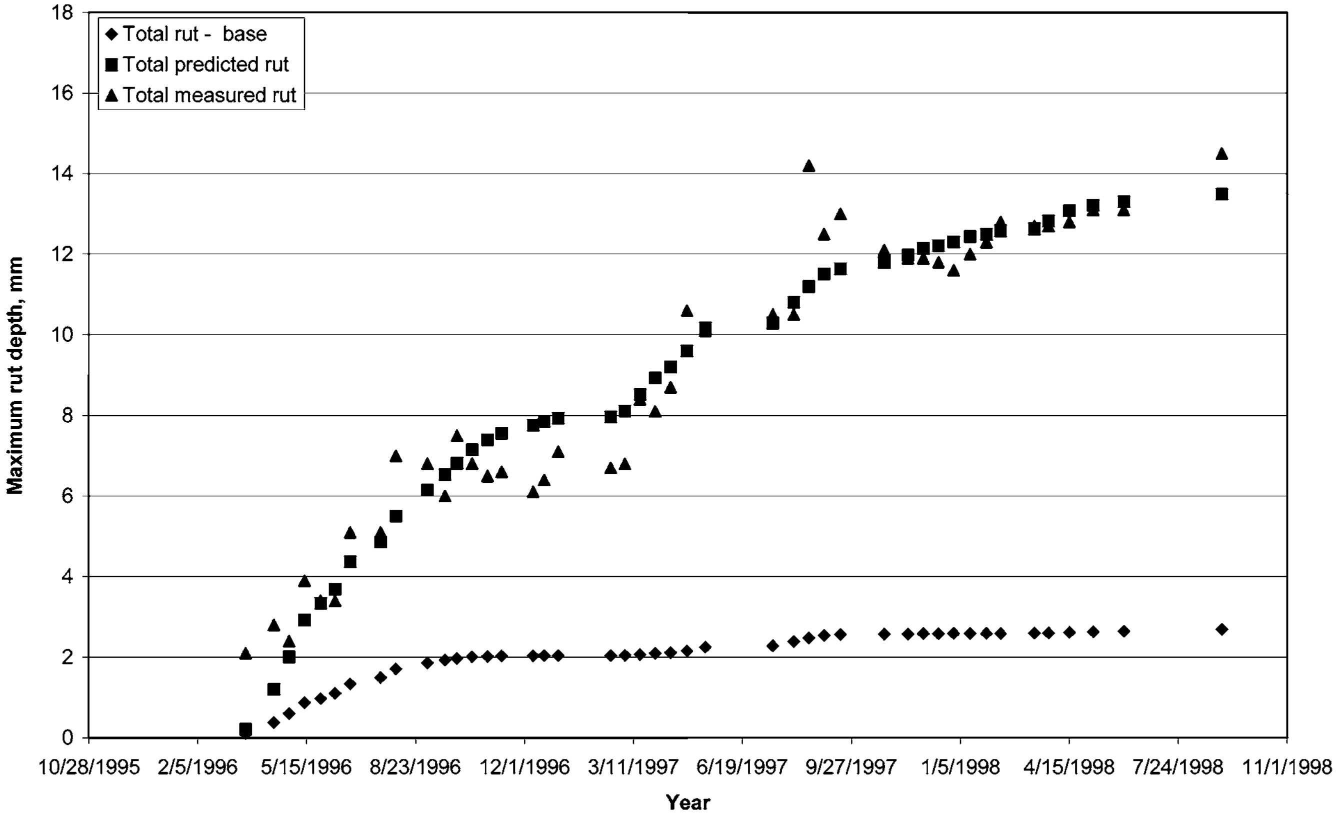


Figure 30. Comparison of computed and measured rut depth versus time, section 4 (1 in. = 25.4 mm).

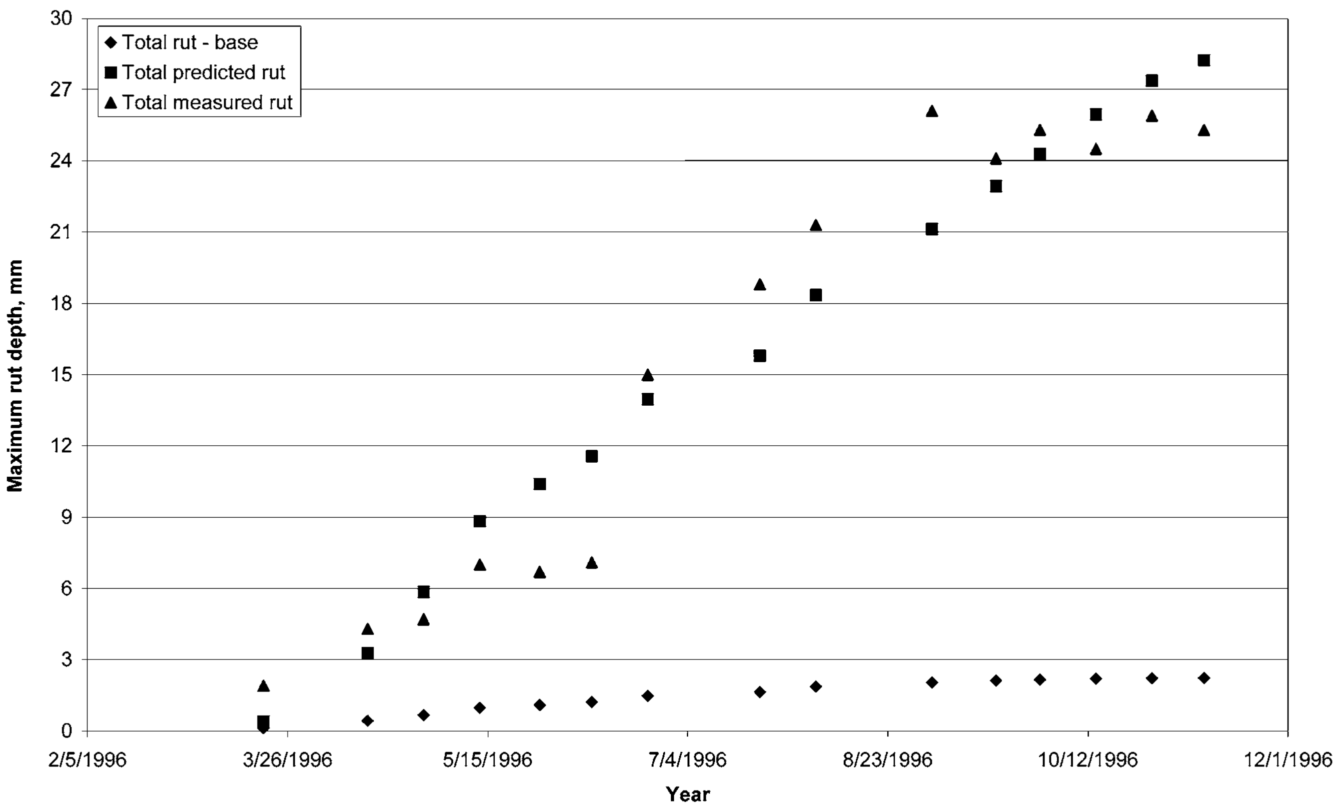


Figure 31. Comparison of computed and measured rut depth versus time, section 7 (1 in. = 25.4 mm).

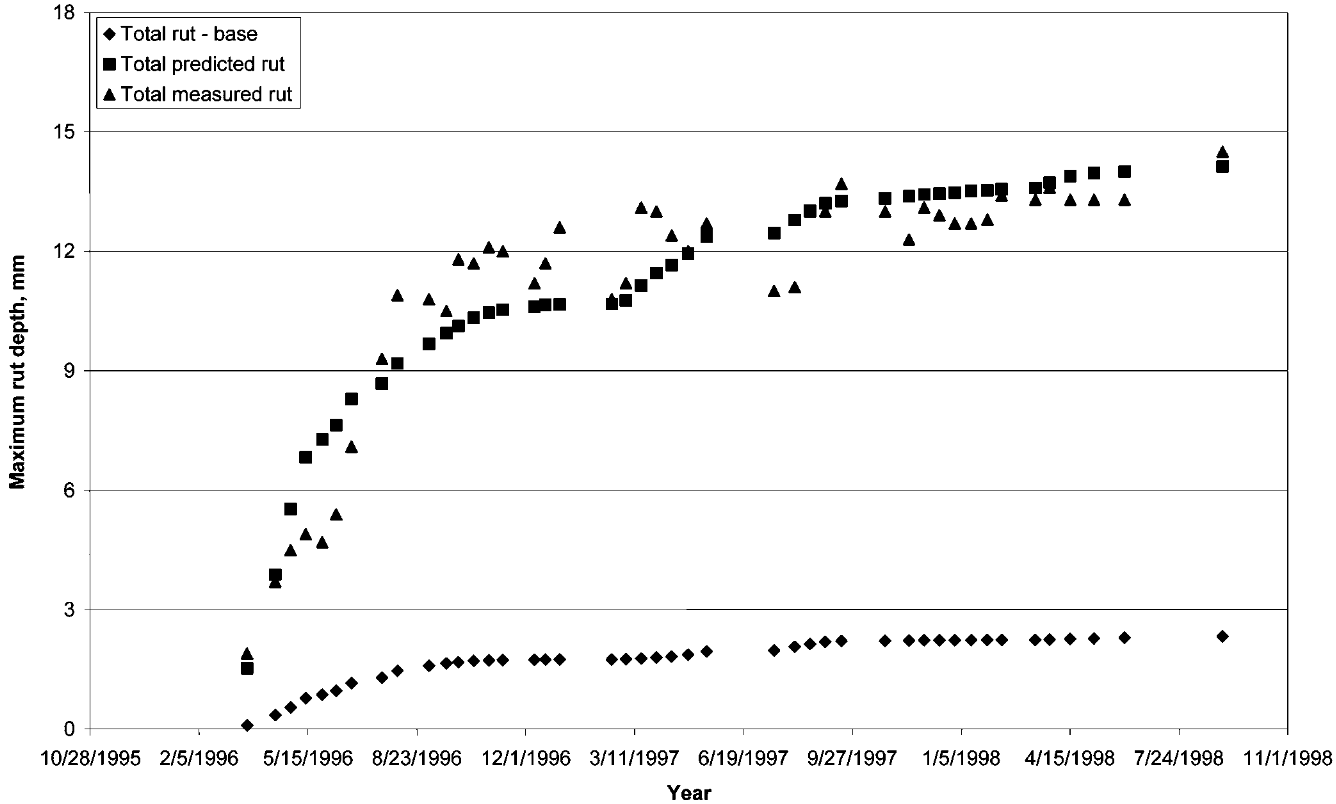


Figure 32. Comparison of computed and measured rut depth versus time, section 19 (1 in. = 25.4 mm).

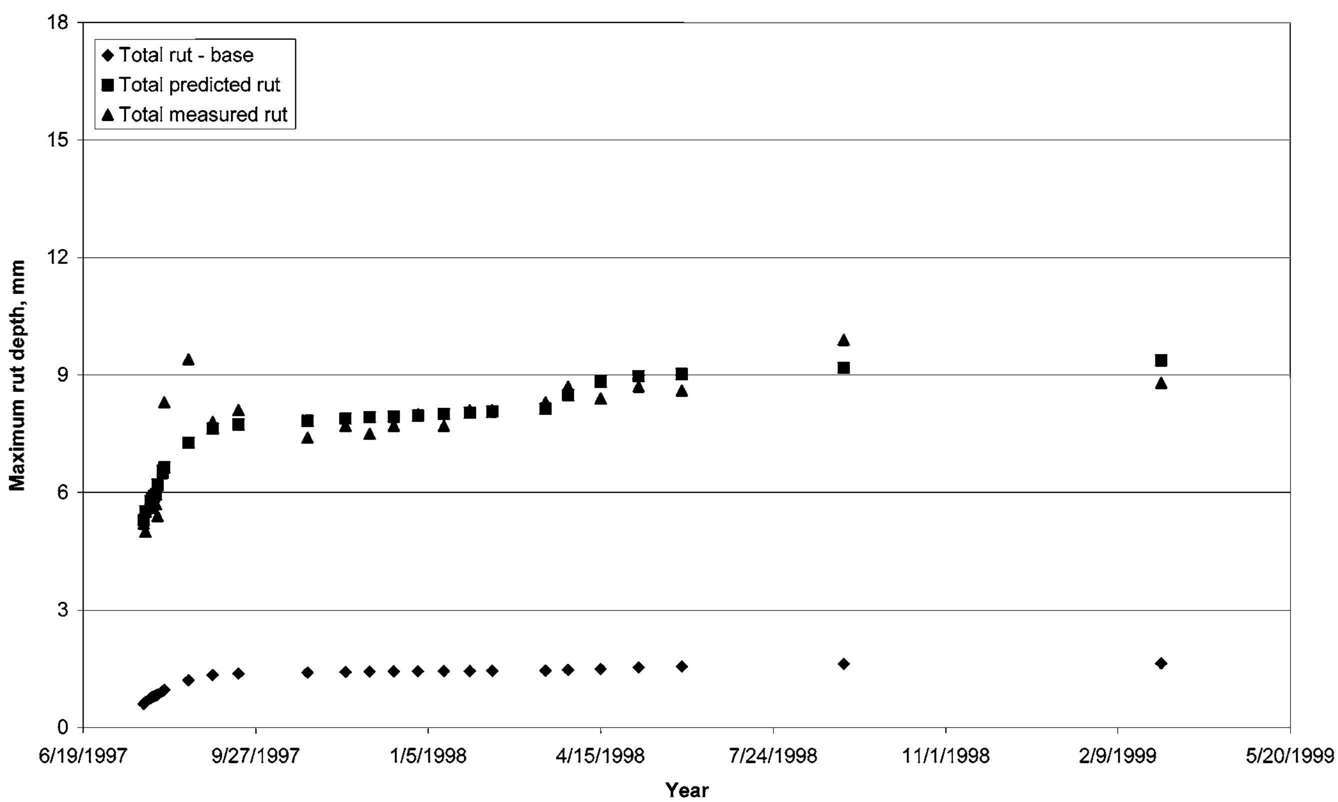


Figure 33. Comparison of computed and measured rut depth versus time, section 38 (1 in. = 25.4 mm).

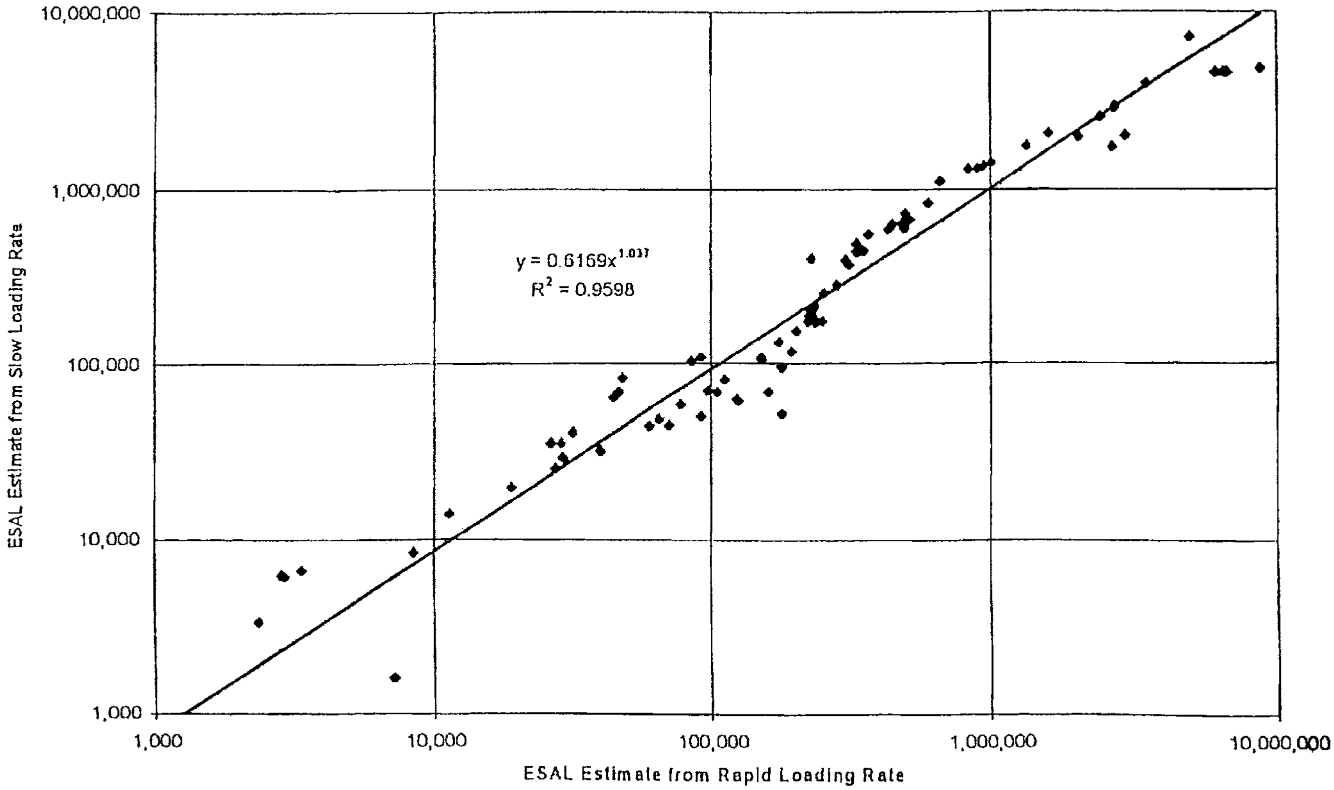


Figure 34. Effect of loading rate on ESAL estimates.

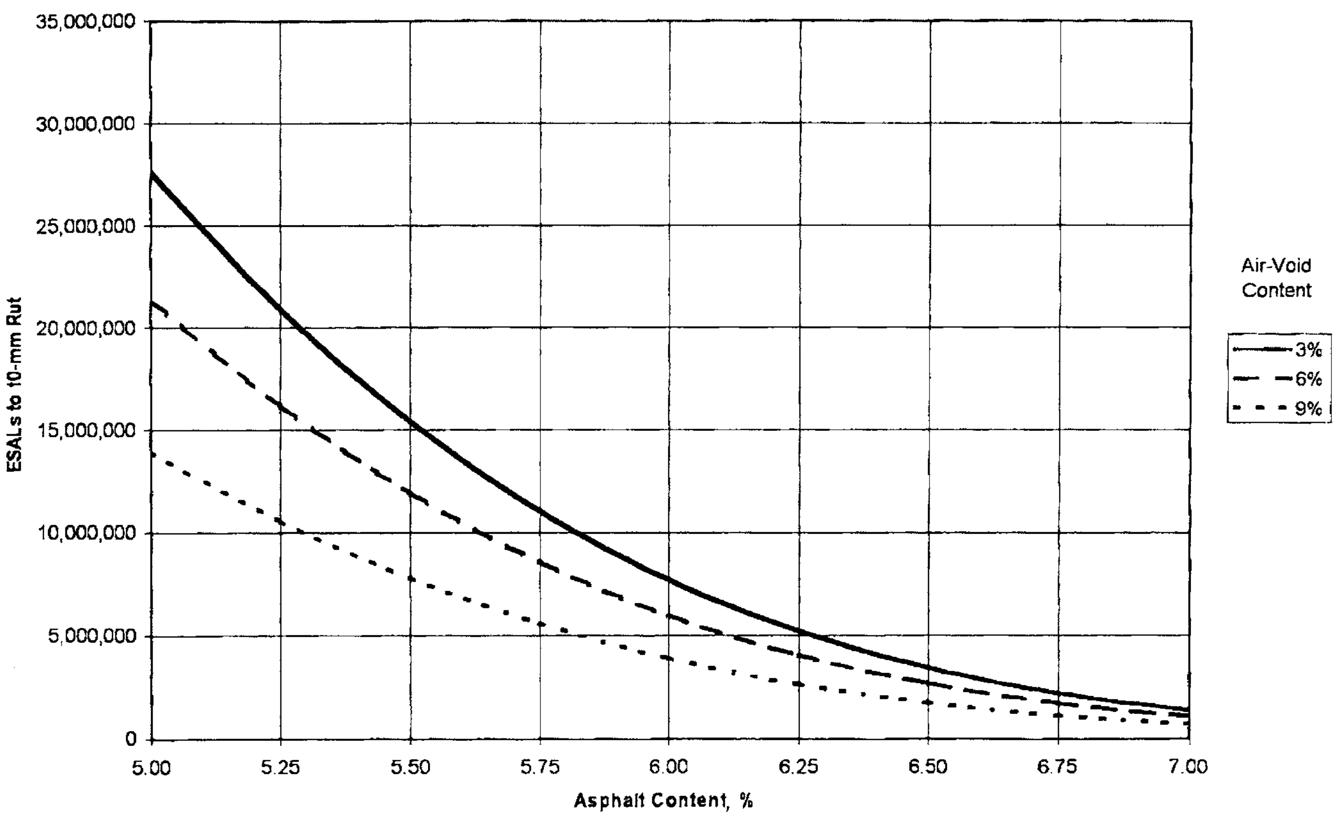


Figure 35. Influence of asphalt content and air void content on computed ESALs to a rut depth of 10 mm (0.4 in.) for P_{200} = 5.5 percent.

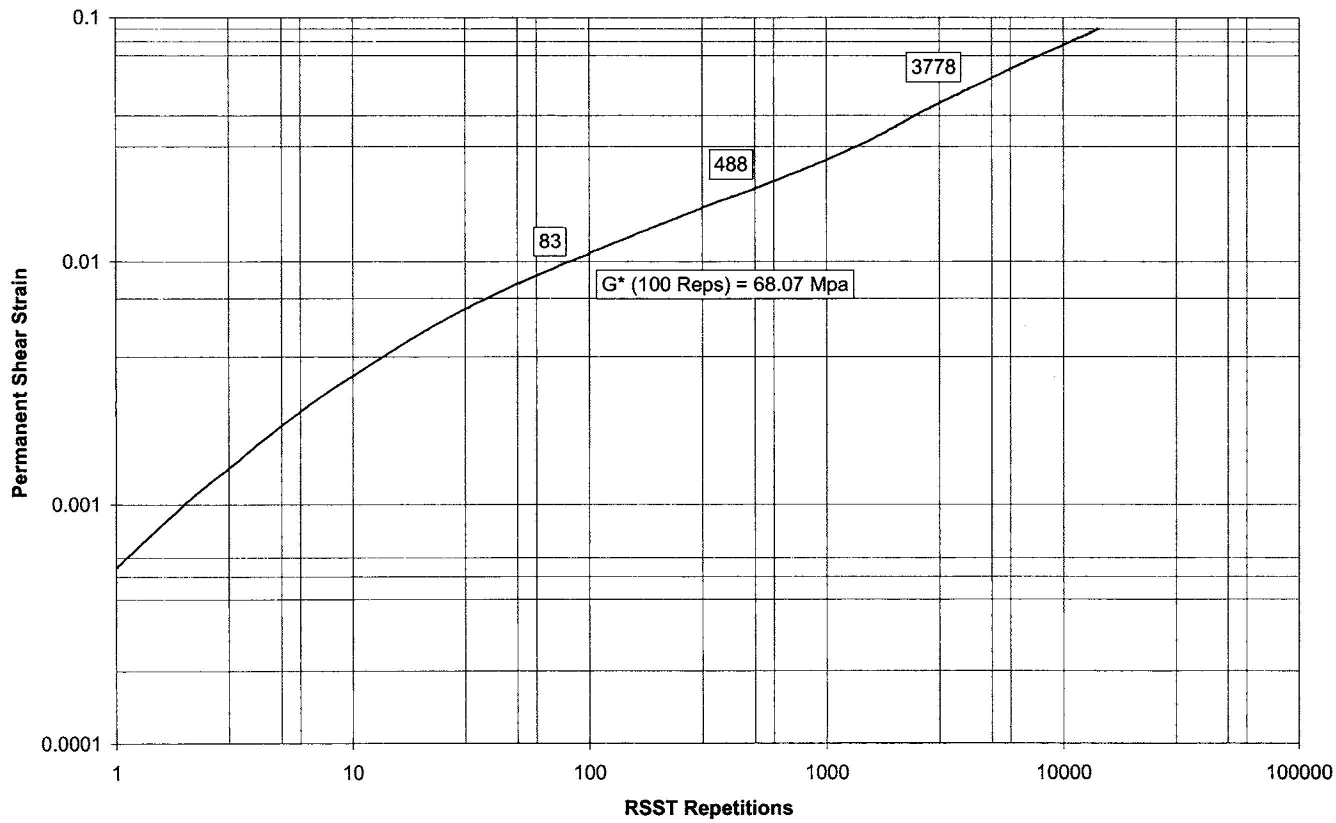


Figure 36. Shear strain, γ_p versus load repetitions, N.

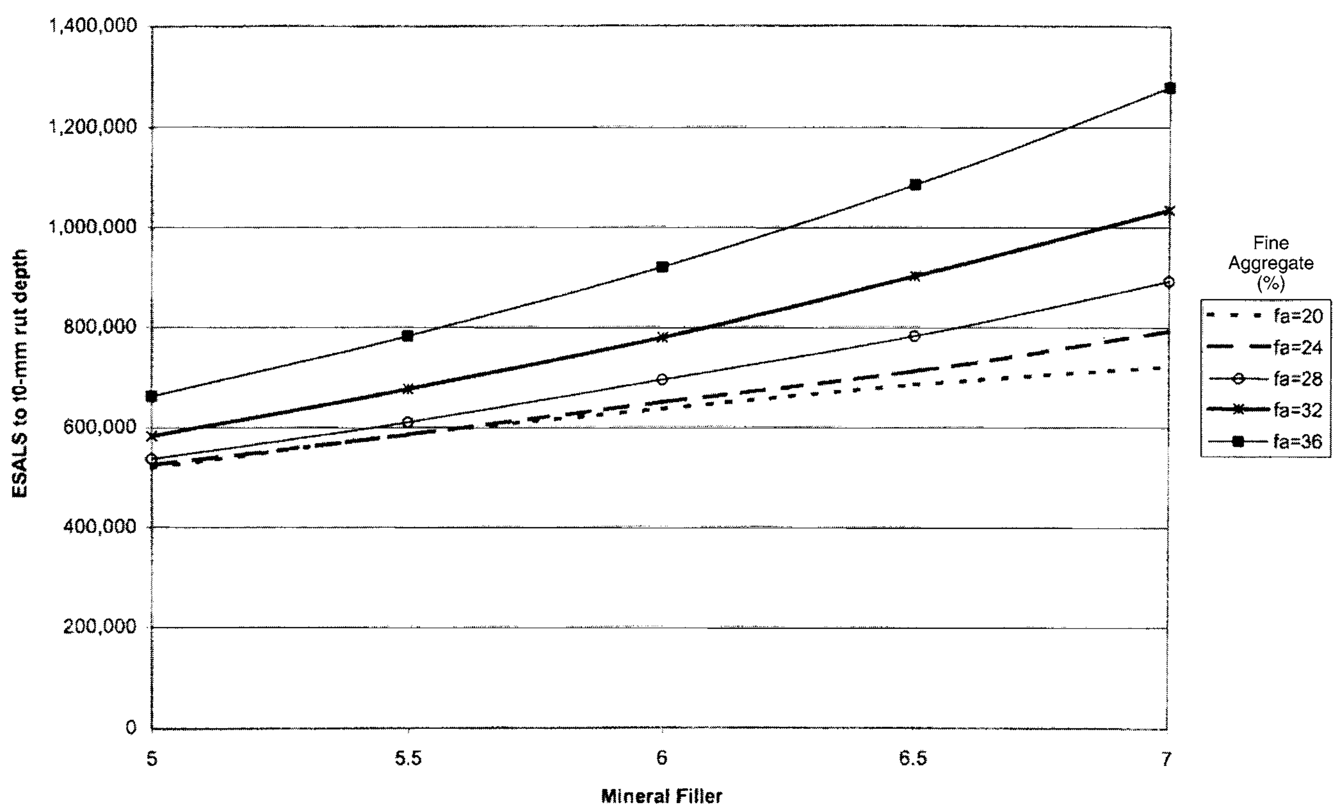


Figure 37. Influence of mineral filler content (P_{200}) and fine aggregate content (fa) on computed ESALs to a rut depth of 10 mm (0.4 in.) for asphalt content = 5.5 percent and air void content = 6.5 percent.

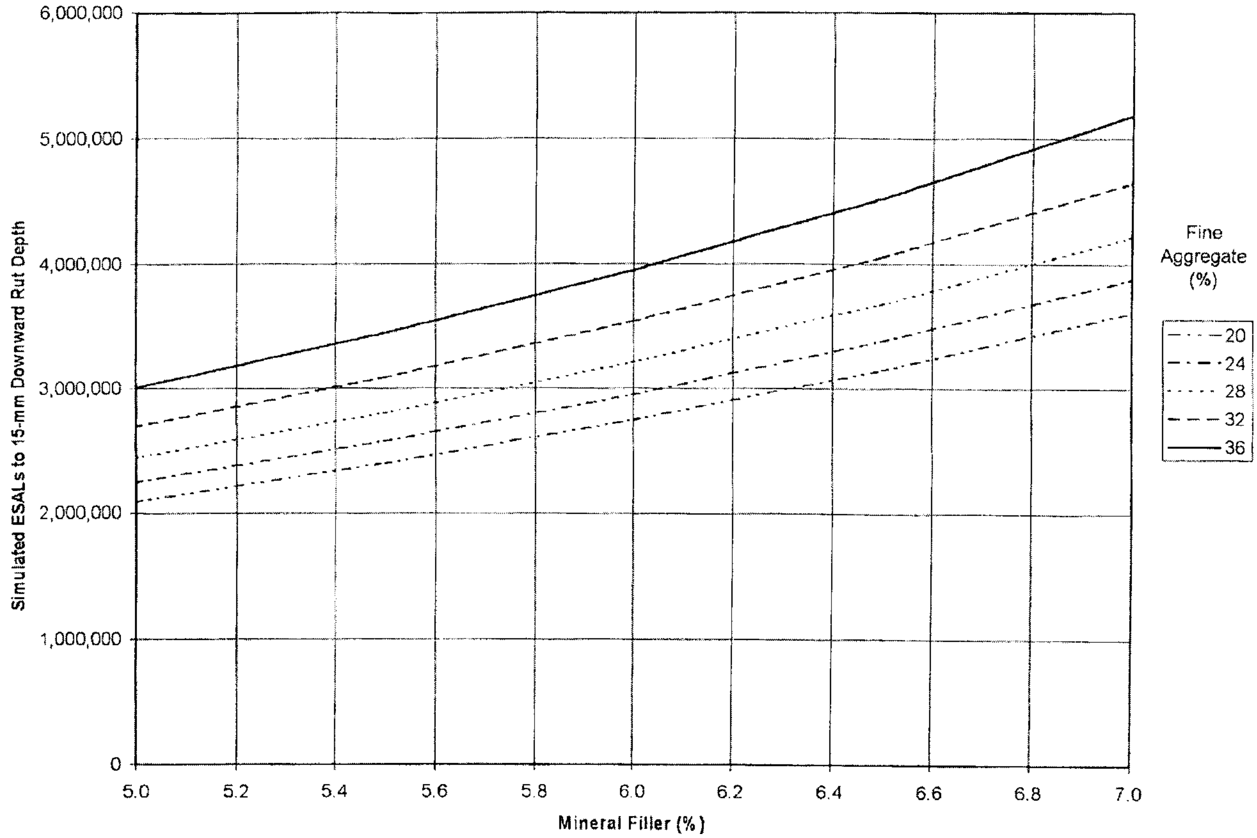


Figure 38. Influence of mineral filler content (P_{200}) and fine aggregate content (fa) on computed ESALs to a rut depth of 15 mm (0.6 in.).

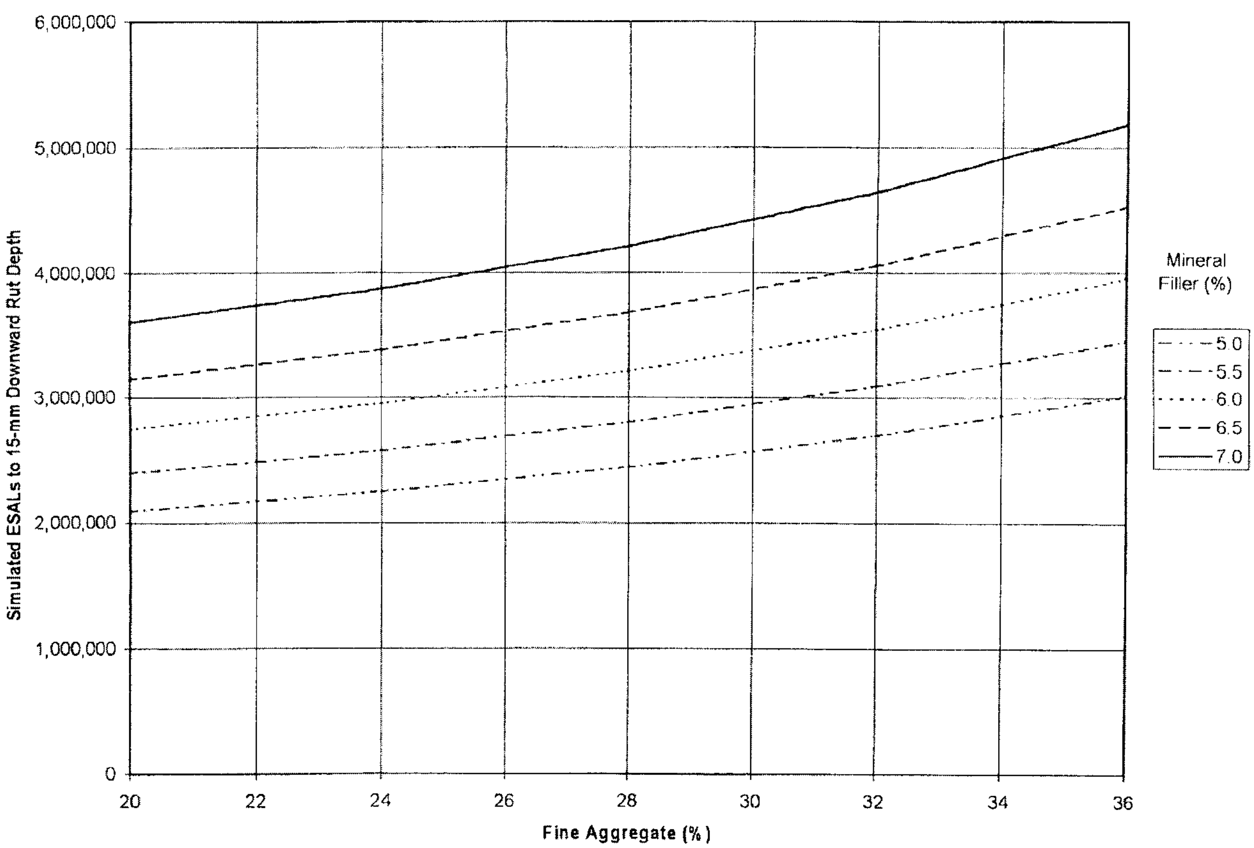


Figure 39. Influence of fine aggregate content (fa) and mineral filler content (P_{200}) on computed ESALs to a rut depth of 15 mm (0.6 in.).

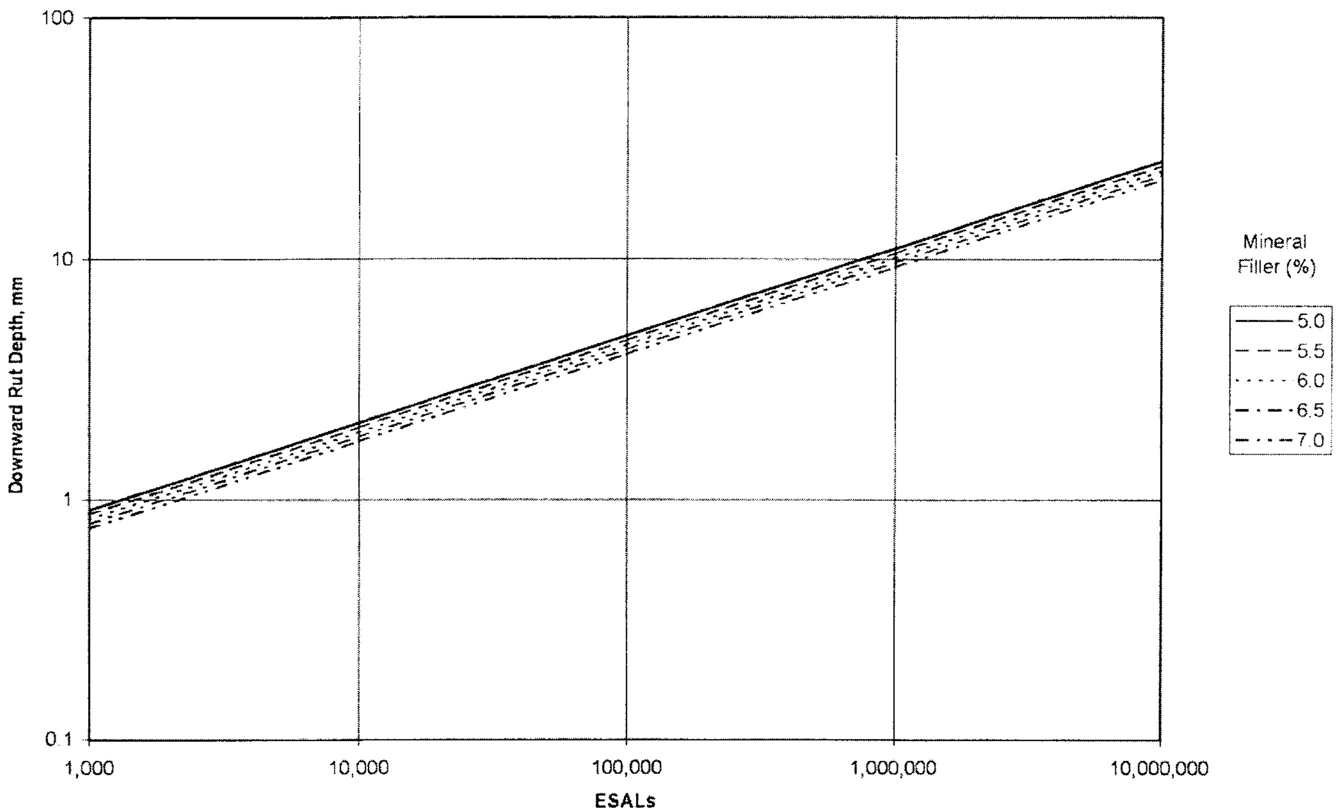


Figure 40. Influence of mineral filler content on rut depth for a range in ESALs (fine aggregate proportion = 28 percent) (1 in. = 25.4 mm).

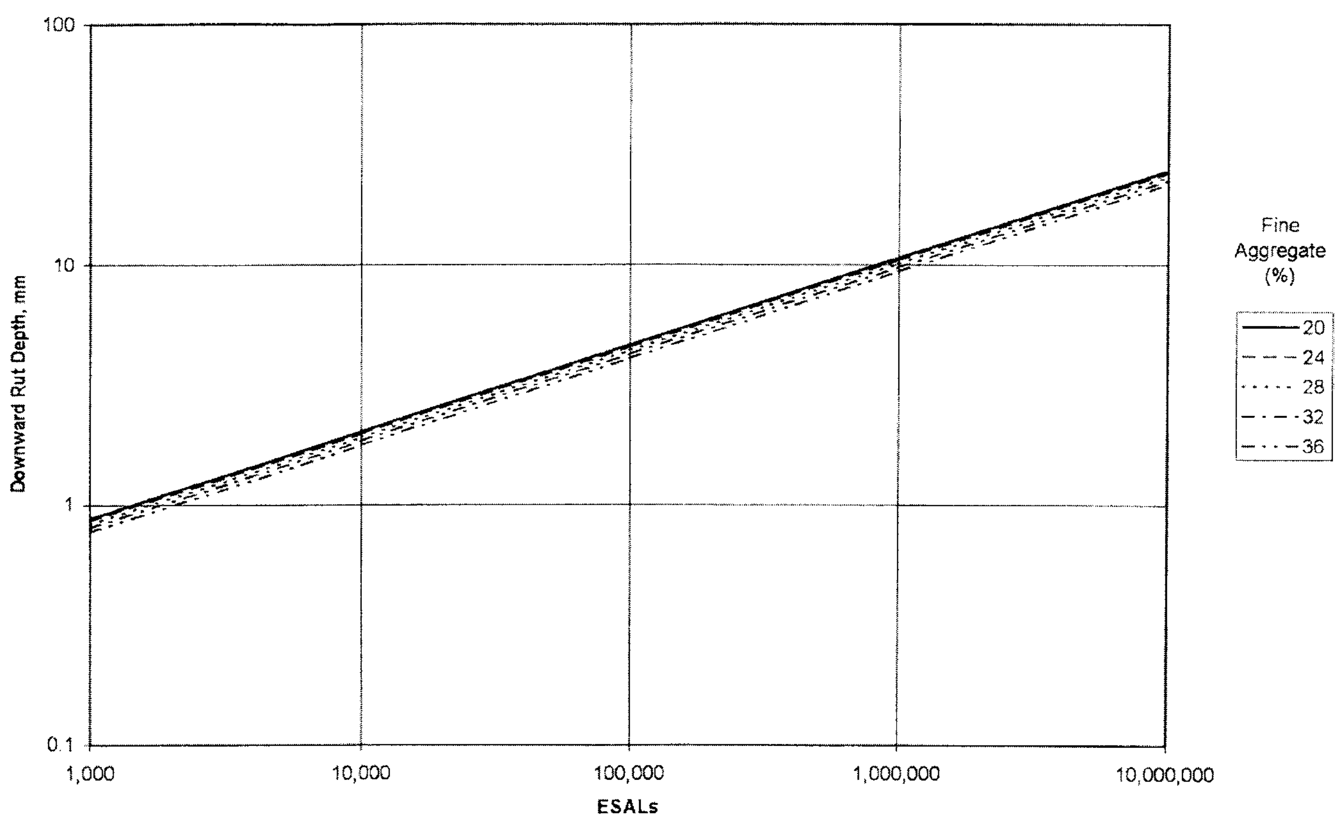


Figure 41. Influence of fine aggregate content on rut depth for a range in ESALs (mineral filler proportion = 6 percent) (1 in. = 25.4 mm).

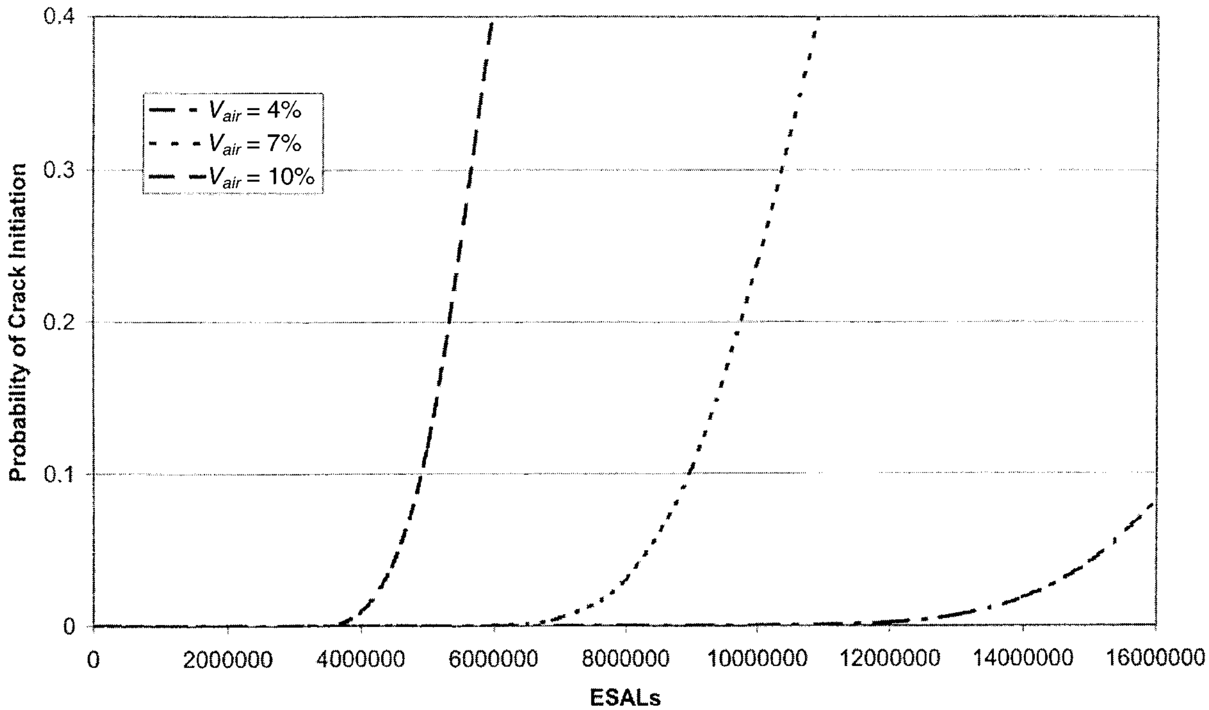


Figure 42. Probability of crack initiation versus ESALs for fine mix for a range in air void contents (asphalt content = 5.4 percent).

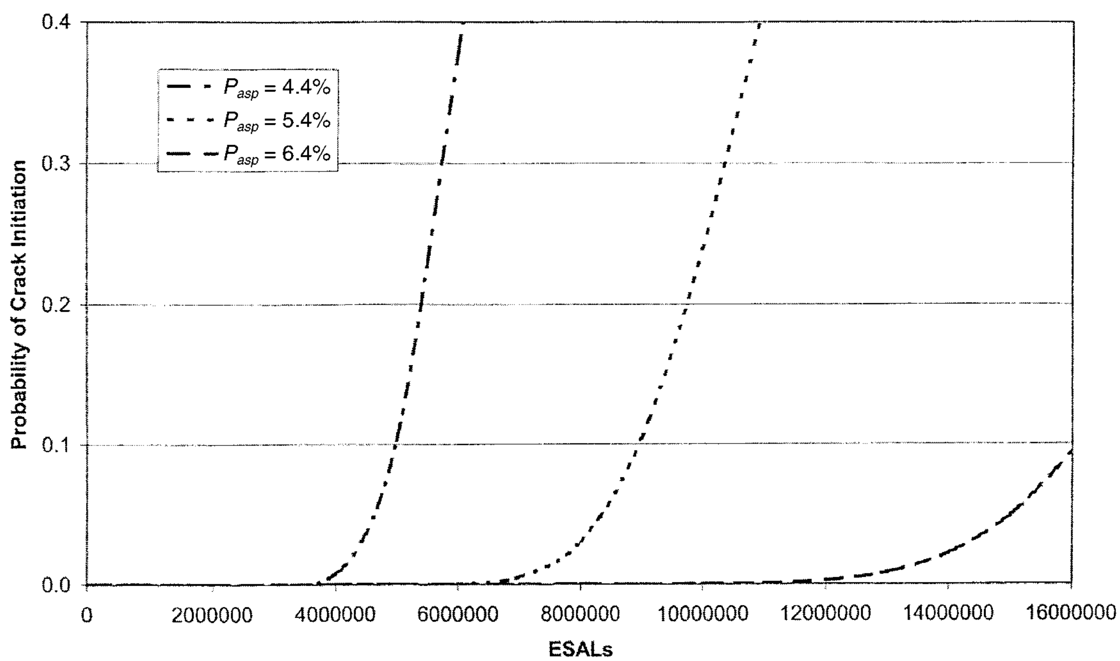


Figure 43. Probability of crack initiation versus ESALs for fine mix for a range in asphalt contents (air void content = 7.0 percent).

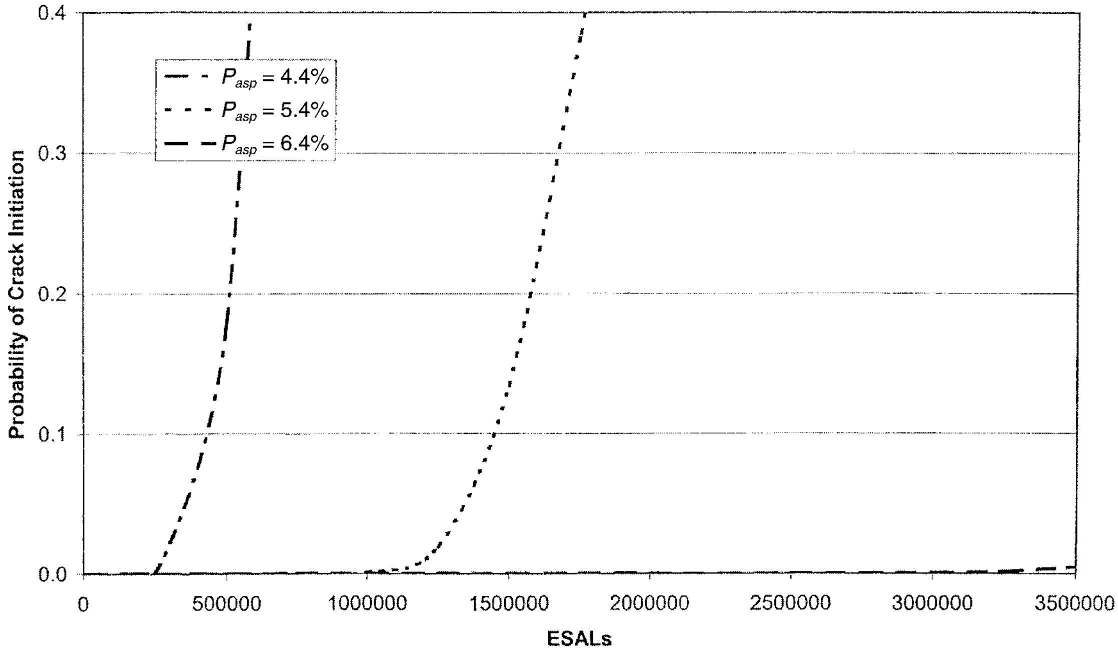


Figure 44. Probability of crack initiation versus ESALs for coarse mix for a range in asphalt contents (air void content = 7.0 percent).

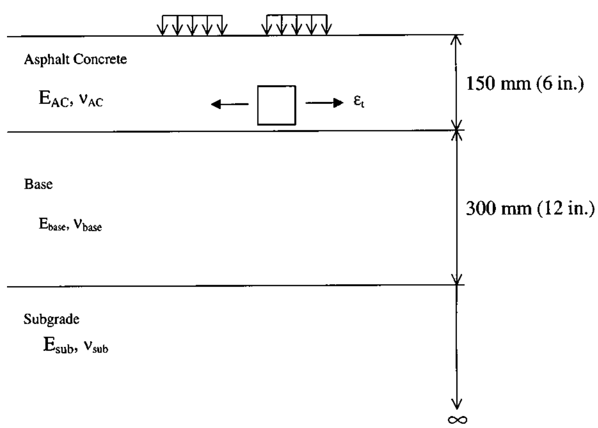


Figure 45. Pavement representation for mechanistic-empirical modeling for fatigue cracking.

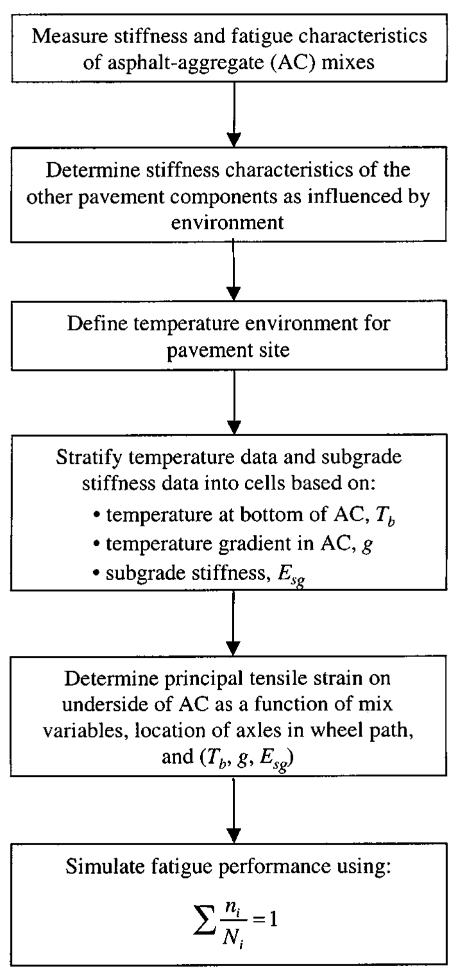


Figure 46. Framework for simulation of fatigue performance of 26 WesTrack sections.

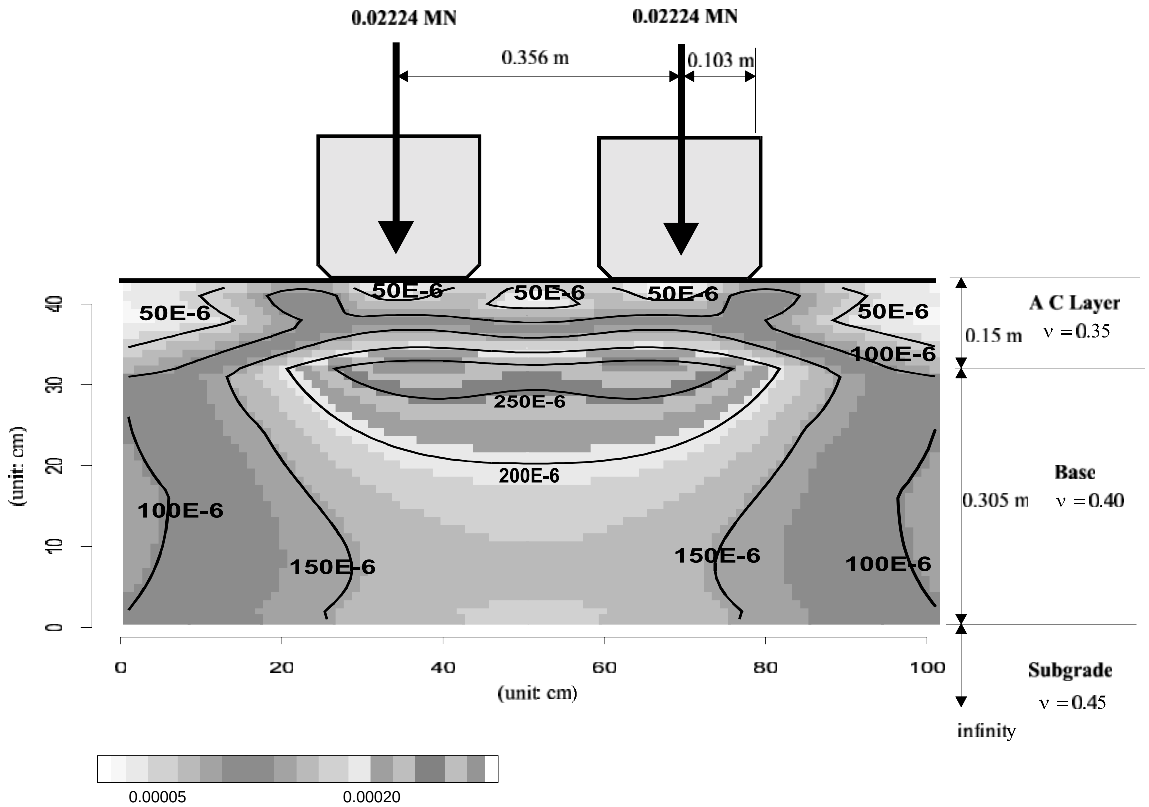


Figure 47. Strain distribution in pavement structure; $T_b = 35°C$ (95°F) and $g = 0.06$ (1 ft = 0.3 m, 1 lb = 4.45×10^{-6} MN).

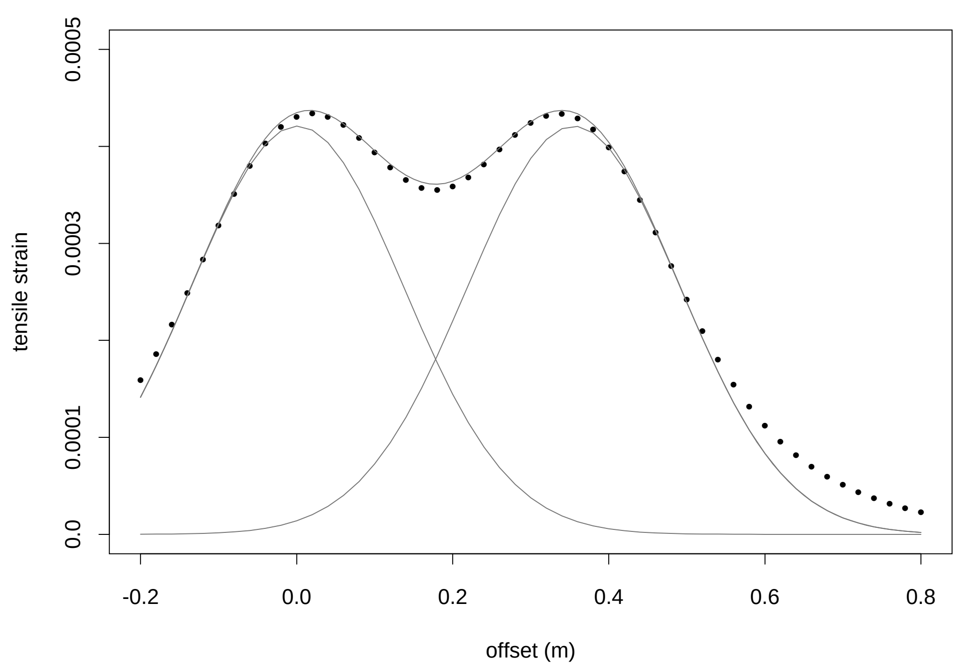


Figure 48. Comparison of the superimposition of two normal distributions with computed tensile strains (individual points) as a function of offset from dual wheel centerline; $T_b = 41.0°C$ (106F), $g = 0.109$, $E_{sub} = 9.58$ MPa (13.9 ksi), $V_{air} = 14.7$; $P_{asp} = 6.22\%$.

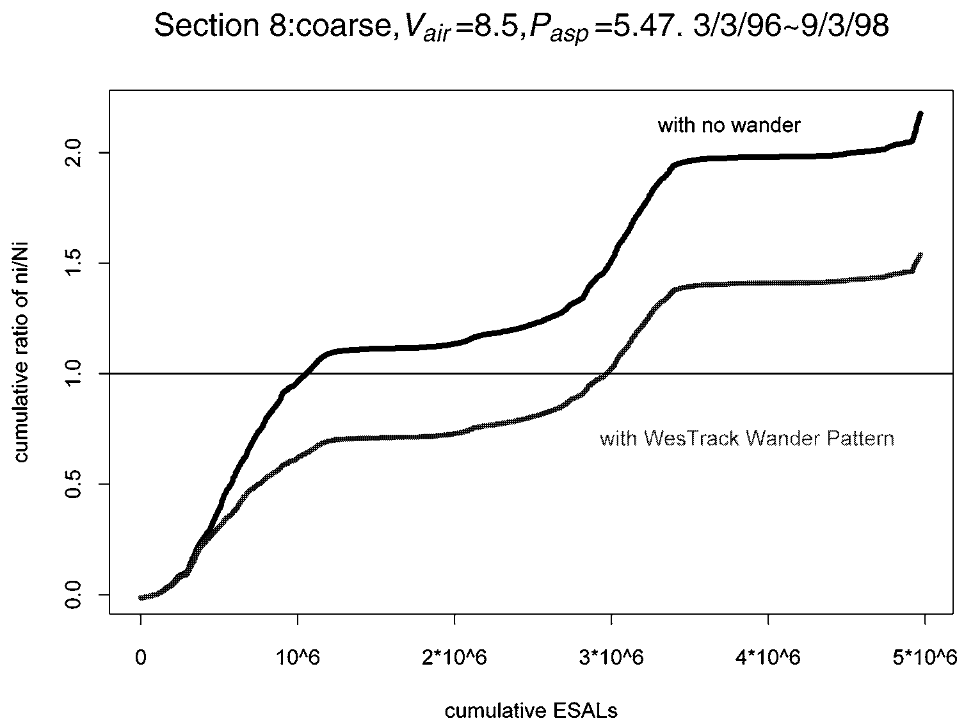


Figure 49. Theoretical development of fatigue damage in section 8 with and without traffic wander.

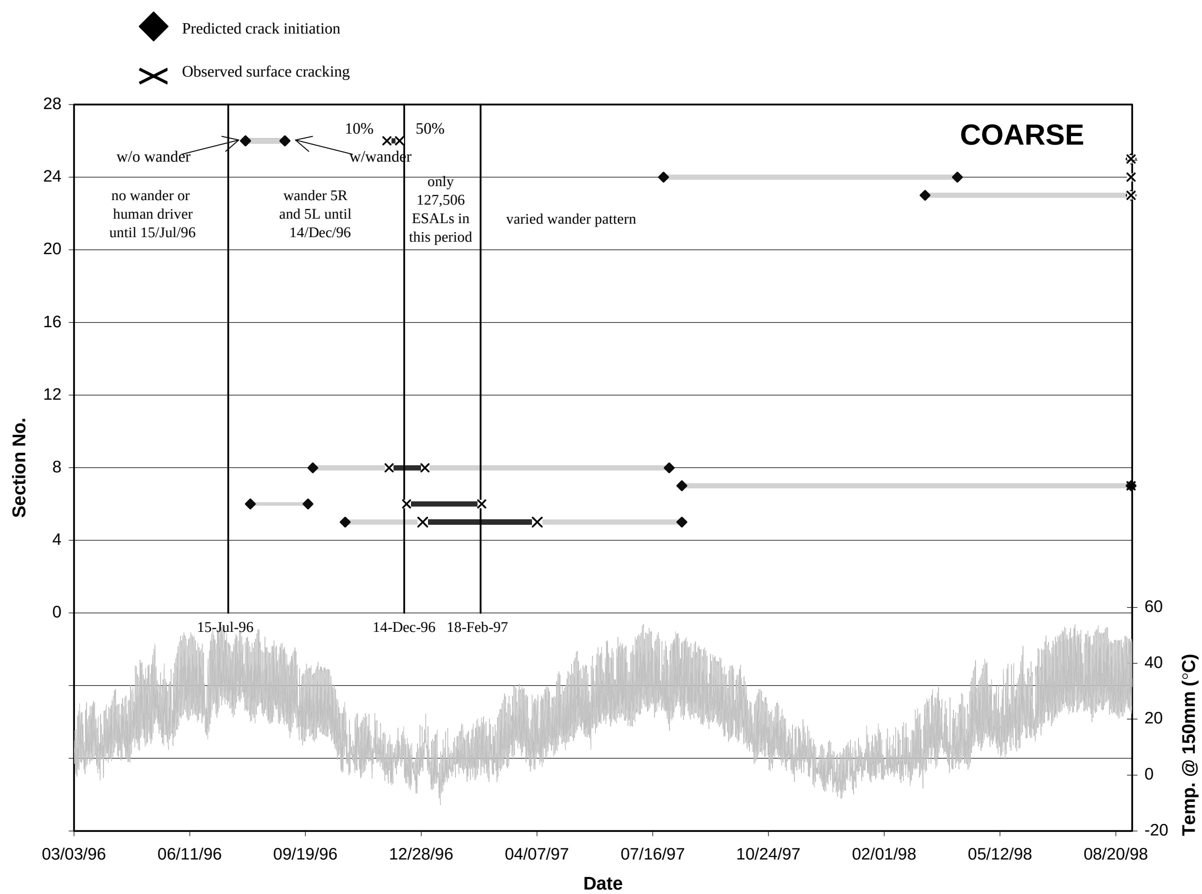


Figure 50. Fatigue performance prediction simulation and condition survey results for the coarse mixes, WesTrack (°F = 1.8°C + 32, 1 in. = 25.4 mm).

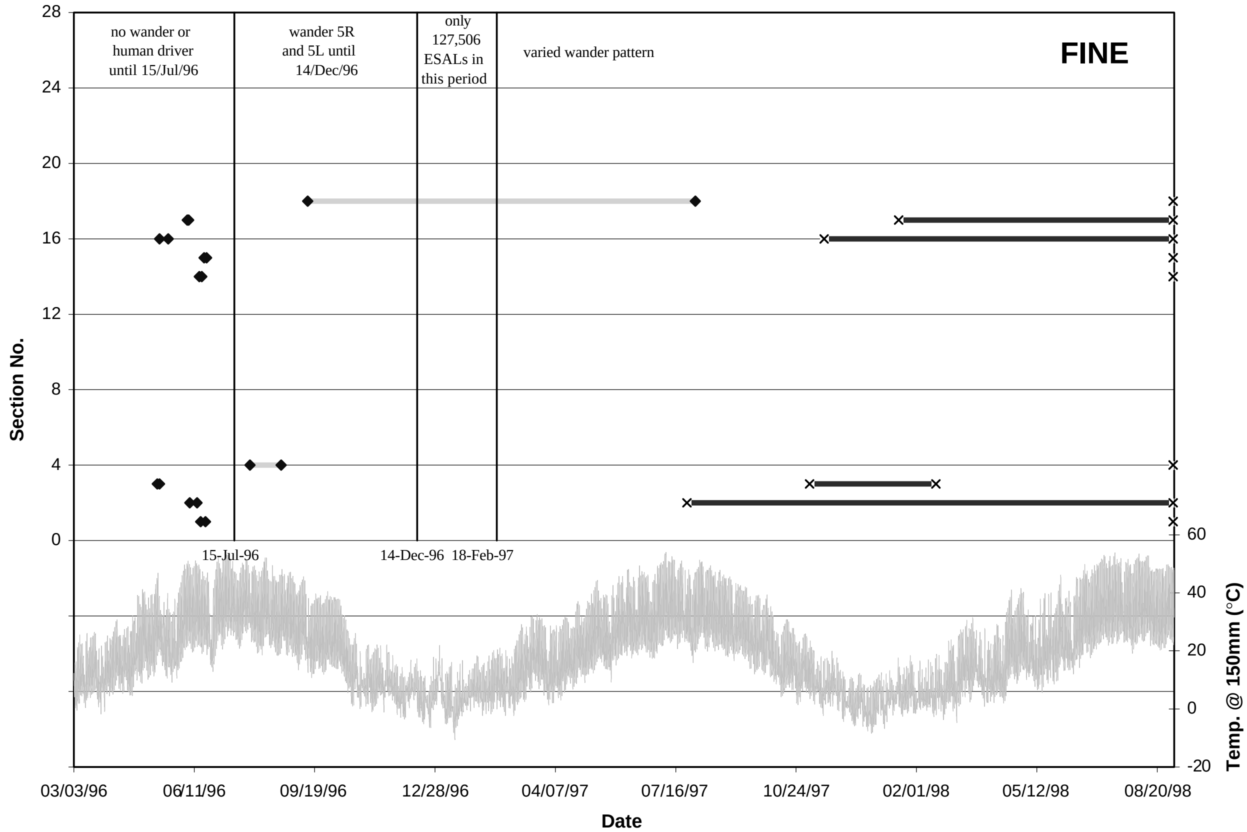


Figure 51. Fatigue performance prediction simulation and condition survey results for the fine mixes, Westrack (°F = 1.8°C + 32, 1 in. = 25.4 mm).

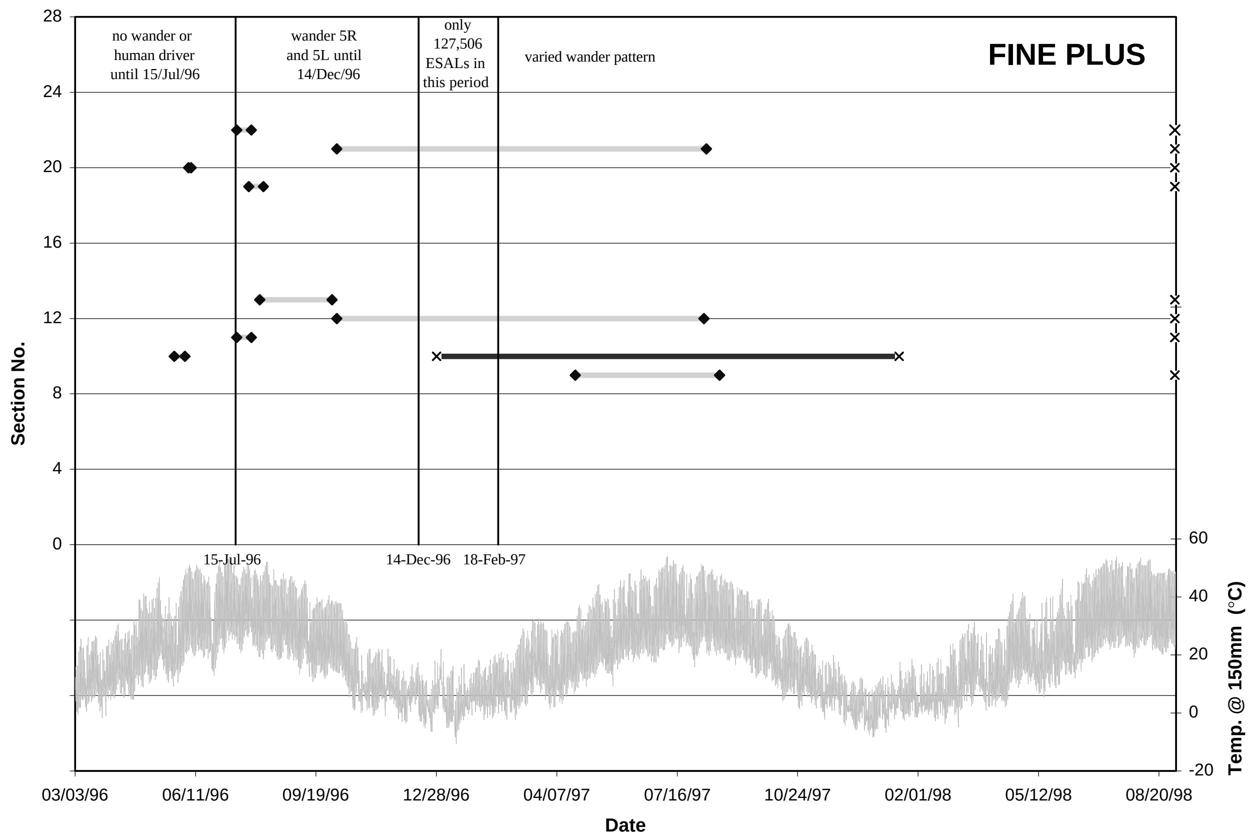


Figure 52. Fatigue performance prediction simulation and condition survey results for the fine plus mixes, WesTrack (1°F = 1.8°C + 32, 1 in. = 25.4 mm).

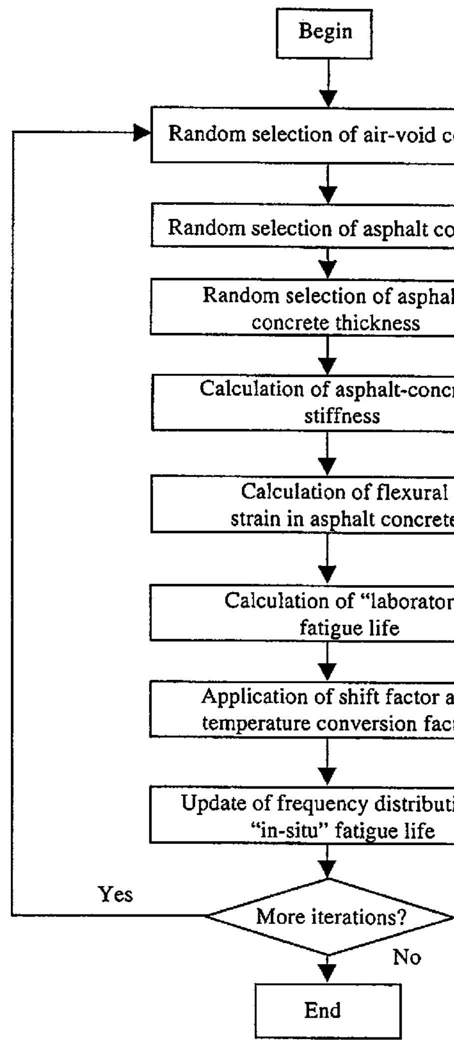


Figure 53. Outline of the pavement performance model simulation.

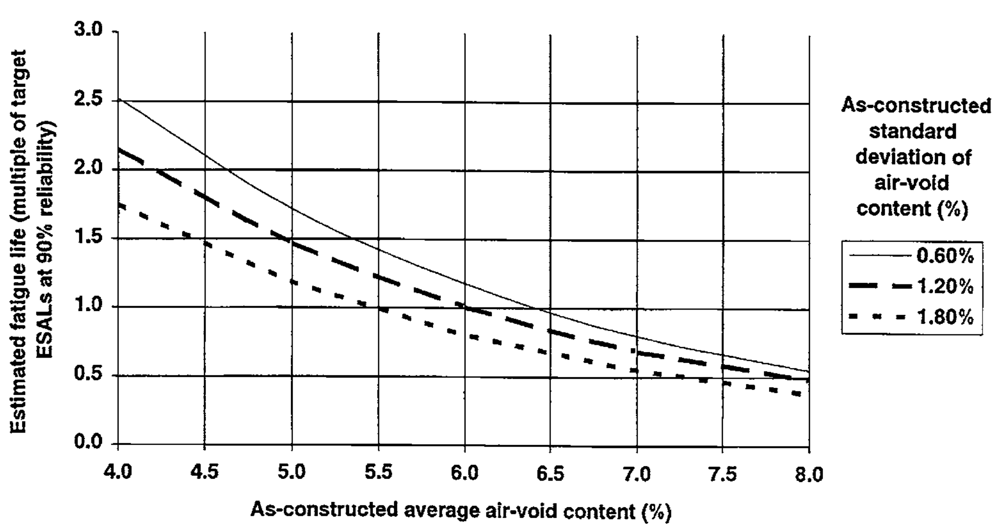


Figure 54. Effects of as-constructed air void content on pavement fatigue performance.

TABLE 4 Regression coefficients for predicting surface deflection in mils (in. $\times 10^{-3}$)

	Location of Deflection Measurement						
	0 in.	**12 in.**	**18 in.**	**24 in.**	**36 in.**	**48 in.**	**60 in.**
d_0	13.8648	13.0839	13.2415	10.5328	10.8222	12.7046	10.6819
d_1	-0.371352			0.319609		-0.262585	
d_2	-0.391083	-0.496013	-0.432364		0.278526		
d_3	-0.266685	-0.604232	-0.803025	-1.20533	-1.24694	-1.04706	-0.980988
d_4	0.018388	-0.022779	-0.017578	-0.018072	0.0044371	0.016013	0.0029934
d_5	0.010939	0.024924	0.031176	0.020782			-0.005306
d_6	0.050517	0.025249		-0.022723	-0.032615	-0.034235	-0.015758
d_7	0.018400	0.029459		-0.044980	-0.049409	-0.029396	-0.006990
d_8	-0.061446	0.016903	0.041682	0.063360	0.045230	0.019796	
d_9	-0.034141	-0.069783	-0.044341		0.032084	0.040621	0.0232921
Adjusted R^2	0.993	0.992	0.991	0.984	0.993	0.991	0.997

1 in. = 25.4 mm

TABLE 5 FWD calibration of asphalt-concrete modulus (psi)

Section	A_0	A_1
1	14.52140	-0.05870
2	14.52709	-0.05870
3	14.151166	-0.05870
4	14.80929	-0.05870
10	14.01662	-0.05870
11	14.74919	-0.05870
12	14.78954	-0.05870
14	14.69831	-0.05870
15	14.73147	-0.05870
16	14.31987	-0.05870
17	14.24529	-0.05870
18	15.30039	-0.05870
19	14.64752	-0.05870
20	14.39871	-0.05870
22	14.54525	-0.05870

1 psi = 6.9 kPa

TABLE 6 FWD calibration of base and foundation-soil moduli (psi)

Period	Parameter	South Tangent	North Tangent
All	E_2	15,125	13,487
1	E_3	17,566	22,625
2		15,529	19,611
4		15,498	19,476
5		15,601	19,121
6		15,293	18,312
8		15,931	22,114
10		15,737	20,269
12		16,743	20,960
14		17,868	21,725
16		18,412	22,929
18		18,381	23,422
20		17,889	22,324
22		16,638	20,583
23		16,365	19,023
24		16,092	18,511
25		16,271	17,755
26		16,720	18,929
27		15,153	17,575
29		14,524	16,843
30		15,270	16,882
32		14,003	15,737
All	D_0	16,560	20,205
	D_1	1,508	2,085
	D_2	249	237

1 psi = 6.9 kPa

TABLE 7 Stiffness ranking of asphalt concrete

Gradation	Section	Asphalt Content	Air Void Content	Modulus at 40°C, psi
South Tangent				
Fine Plus	12	Medium	Low	253,073
	11[a]	Medium	Medium	243,064
	10	Low	High	116,834
Fine	4	Medium	Low	58,119
	2	Low	Medium	194,654
	1[b]	Medium	Medium	193,549
	3[c]	Low	High	133,659
North Tangent				
Fine Plus	19[a]	Medium	Medium	219,565
	22	Low	Medium	198,222
	20	Medium	High	171,201
Fine	18	High	Low	421,795
	15[b]	Medium	Medium	238,794
	14	High	Medium	231,007
	16[c]	Low	High	158,222
	17	Medium	High	146,852

1 psi = 6.9 kPa, °F = 1.8°C + 32
[a]Sections 11 and 19 are replicates.
[b]Sections 1 and 15 are replicates.
[c]Sections 3 and 16 are replicates.

TABLE 8 Comparison of laboratory and FWD moduli, psi, for base and foundation soil (monitoring session 12), (1 psi = 6.9 kPa)

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	

TABLE 9 Asphalt concrete modulus coefficients (units in psi) (1 psi = 6.9 kPa)

Section	A_0	A_1
1	14.21976	-0.0587
4	14,74661	-0.0587
7	14,61647	-0.0587
9	14,89291	-0.0587
11	14.77807	-0.0587
12	14,90155	-0.0587
13	14.53573	-0.0587
14	14.50107	-0.0587
15	14,46691	-0.0587
18	14.83194	-0.0587
19	14.53123	-0.0587
20	14.43511	-0.0587
21	14.81272	-0.0587
22	14.70827	-0.0587
23	14.95398	-0.0587
24	14.66365	-0.0587
25	14.78208	-0.0587
35	14.38958	-0.061
37	14.51254	-0.061
38	14.55094	-0.061
39	14.52325	-0.061
54	14.51352	-0.061
55	14.46777	-0.061

TABLE 10 Calculated ESALs to a range in rut depths for a constant loading of 60 trucks/hour, WesTrack environment

	ESALs to rut depth, mm									
Section	**2.5**	**5**	**7.5**	**10**	**12.5**	**15**	**17.5**	V_{air}	P_{asp}	P_{200}
1	29,074	251,404	831,362	3,550,521	9,204,959	18,393,845		8.6	5.69	5.1
4	237,866	599,311	1,351,031	2,749,240	3,864,101	5,897,509	7,027,092	6.9	5.24	4.4
7	84,864	151,996	228,542	314,463	408,580	514,224	633,857	7.6	6.28	6.4
9	112,102	234,310	354,895	495,961	651,559	856,315	1,82,572	3.2	6.07	5.2
11	125,722	480,784	2,462,380	6,497,720	12,923,651			8	5.5	5.5
12	124,467	432,955	1,621,198	5,010,419	9,009,756	15,410,728		3.8	5.35	6
13	97,741	225,370	356,261	515,817	707,210	949,954	1,267,213	6	6.01	5.7
14	11,326	161,090	355,056	944,083	3,077,436	6,514,499	11,823,166	7.7	6.22	4.9
15	39,936	284,778	2,076,480	6,677,759	16,396,184			8.8	5.55	5.2
18	195,225	662,190	2,761,024	6,111,670	9,973,295	15,985,377		4.6	6.22	5.1
19	19,027	180,022	369,765	1,004,788	2,988,489	5,978,874	9,412,470	6	5.41	5.8
20	8,440	92,110	234,988	496,663	1,380,383	3,223,884	6,329,363	10.4	5.4	5.2
21	77,863	175,415	255,902	345,482	455,783	557,570	687,693	3.6	6.25	5.4
22	7,204	179,666	935,083	8,775,292				6.9	4.76	5.3
23	70,740	224,658	448,097	897,434	2,231,212	3,299,976	5,360,025	5.8	5.78	7
24	27,329	105,115	203,216	305,852	447,490	634,911	927,712	7.5	5.9	6.6
25	60,039	152,845	235,175	333,994	459,260	587,192	765,171	3.1	6.33	6.7
35		911	3,389	47,871	375,276	2,188,005	3,488,886	8.75	6.12	5.6
37		2,944	26,353	85,269	217,944	462,894	1,212,221	9.55	6.14	5.7
38	874	31,622	492,565	3,012,096	7,270,697	17,146,982		7.7	5.55	6.2
39	1,582	46,361	491,444	2,713,246	5,423,741	10,975,692		5.5	5.94	5.7
54		887	2,864	44,521	276,554	2,008,271	2,999,958	7.3	6.11	5.8
55	2,347	28,671	92,441	230,486	454,451	892,584	1,984,093	4.3	6.04	6

1 in. = 25.4 mm

TABLE 11 Regressions relating rut depth to ESALs and mix parameters for simulated 10-year trafficking and WesTrack environment (*16*)

	R^2					
	Regr. 1	**Regr. 2**	**Regr. 3**	**Regr. 4**	**Regr. 5**	**Regr. 6**
Constant	-4.06247	-3.88788	-6.1651	-7.13496	-8.66225	-7.93245
V_{air}					0.0516084	0.038771
P_{asp}					0.487042	0.039324
$V_{air} \cdot P_{asp}$						
V_{air}^2	0.00302745		0.00294305	0.00408994		
P_{asp}^2			0.0688276	0.0423931		
ln(*ESAL*)	0.03019387	0.309198	0.30994	0.304416	0.303358	0.39695
P_{200}	-0.812031	-0.868229				
$V_{air} \cdot P_{200}$		0.00679854				
$P_{asp} \cdot P_{200}$	0.0818242	0.0827899	-0.0657803			
Fine						-0.39695
Fine Plus	0.593592	0.60387	0.600498	0.397368	0.409128	
Coarse	-1.7751	-1.7657	-1.59167	-2.12708	-2.09736	0.08131
Replace	2.72257	2.72868	2.7797	2.35276	2.34783	
Fine ln(*ESAL*)						
Fine plus ln(*ESAL*)						
Coarse ln(*ESAL*)	0.21865	0.218834	0.21327	0.200957	0.19863	
Replace ln(*ESAL*)	-0.13931	-0.139484	-0.140386	-0.135964	-0.135645	
R^2	0.812	0.811	0.809	0.790	0.786	0.740

TABLE 12 Calibration results for 23 sections, conventional analysis

Section	a	c	RMSE (in.)[1]
1	5.41658	.022521	0.027
4	0.01392	0.66306	0.037
7	0.01509	0.77181	0.102
9	0.00410	0.83989	0.076
11	1.64235	0.29677	0.040
12	1.05802	0.33734	0.087
13	0.01186	0.75472	0.078
14	6.15197	0.25614	0.050
15	7.30191	0.20716	0.001
18	0.39160	0.41493	0.035
19	3.86629	0.29245	0.044
20	7.03048	0.26222	0.050
21	0.00973	0.81183	0.118
22	29.32602	0.10116	0.042
23	0.59761	0.43650	0.050
24	0.49708	0.49941	0.084
25	0.05564	0.67400	0.102
35	52.77398	0.12388	0.024
37	12.04868	0.26447	0.032
38	23.14986	0.13996	0.024
39	13.73983	0.18501	0.030
54	51.08506	0.12941	0.012
55	3.22487	0.35783	0.032
Average			**0.051**

[1](1 in. = 25.4 mm)

TABLE 13 Calibration of equations for simulating ln(field *a*) based on mix and RSST variables

	Regr. 1	Regr. 2	Regr. 3	Regr. 4	Regr. 5	Regr. 6
Constant	14.9116	24.7107	24.3317	24.9718	25.3649	20.4844
P_{asp}	-3.67001	-5.02990	-5.04342	-5.23716	-5.71438	-5.12624
V_{air}						0.313875
$P_{asp}\ V_{air}$	0.0823738					
rsst5					6.219E-05	9.699E-05
lab a	1301.81	1622.41	1745.07	1858.91	2472.96	2264.05
R^2	0.611	0.629	0.684	0.752	0.888	0.951
Sect. Del.	None	14	14, 15	1, 14, 15	1, 14, 15, 19	1, 4, 14, 15, 19

TABLE 14 Calibration of equations for simulating field *c* based on mix and RSST variables

	Regr. 1	Regr. 2	Regr. 3	Regr. 4	Regr. 5	Regr. 6
Constant	-0.944102	-1.75309	-1.72144	-1.77798	-1.83917	-1.49931
P_{asp}	0.312598	0.426673	0.427803	0.444915	0.493348	0.452398
V_{air}						-0.0217923
$P_{asp}\ V_{air}$	-0.0064968					
rsst5					-6.216E-06	-8.575E-06
lab a	-87.5258	-113.452	-123.693	-133.748	-190.11	-175.759
R^2	0.556	0.591	0.648	0.728	0.890	0.936
Sect. Del.	None	14	14, 15	1, 14, 15	1, 14, 15, 19	1, 4, 14, 15, 19

TABLE 15 Values of field *a* from the 23 sections using a constant value for *c*

Section	P_{asp}	V_{air}	P_{200}	*fa*	reps 5%	G*	lab "a"	lab "b"	field a	field c
1	5.75	9.6	5.1	33.7	2,113	53.72	0.001588	0.448	1.146	0.34
4	5.63	6.5	4.4	34.8	40,093	70.52	0.000842	0.384	1.280	0.34
7	6.38	8.3	6.4	22.3	2,105	42.35	0.002276	0.407	4.114	0.34
9	5.8	2.5	5.2	32.6	2,764	45.86	0.001263	0.464	2.674	0.34
11	5.16	8.1	5.5	34.6	799	48.54	0.001948	0.486	0.913	0.34
12	5.27	3.7	6.0	32.7	37,572	67.03	0.001332	0.344	1.020	0.34
13	5.76	6.5	5.7	32.9	479	44.89	0.001507	0.570	2.599	0.34
14	6.62	5.9	4.9	35.1	692	46.27	0.002473	0.461	1.979	0.34
15	5.56	8.1	5.2	35.0	6,405	47.49	0.001170	0.428	1.230	0.34
18	6.26	3.5	5.1	34.7	20,266	79.88	0.002750	0.291	1.096	0.34
19	5.48	6.0	5.8	34.5	1,303	35.86	0.000872	0.564	2.020	0.34
20	5.13	10.4	5.2	34.7	340	39.39	0.001766	0.574	2.434	0.34
21	5.84	3.7	5.4	36.1	1,707	44.31	0.001344	0.488	4.419	0.34
22	4.52	6.9	5.3	33.5	2,792	56.57	0.001130	0.477	1.279	0.34
23	5.84	4.3	7.0	22.1	50,918	62.41	0.000990	0.364	2.169	0.34
24	5.78	7.5	6.6	22.1	2,973	59.20	0.003168	0.348	4.143	0.34
25	6.24	2.8	6.7	22.7	10,012	41.09	0.003029	0.308	4.097	0.34
35	5.71	8.7	5.6	18.4	129	34.25	0.004389	0.506	3.739	0.34
37	5.94	9.6	5.7	18.9	184	34.84	0.004065	0.486	5.127	0.34
38	5.43	7.7	6.2	19.4	860	43.24	0.003505	0.397	2.003	0.34
39	5.32	5.5	5.7	18.6	1,388	52.17	0.004003	0.353	2.021	0.34
54	5.78	7.3	5.8	19.9	651	43.24	0.004622	0.373	4.964	0.34
55	5.93	4.3	6.0	19.1	137	40.84	0.004372	0.502	3.956	0.34

TABLE 16 Regressions for ln(*a*) based on mix and laboratory shear test parameters

	Trial 1	Trial 2	Trial 3	Trial 4	Trial 5	Trial 6
Constant	-6.40078E-01	9.28909E-01	4.20514E-01	7.35276E-01	8.72143E-01	8.30836E-01
P_{asp}	4.95130E-01					
P_{asp}^2		4.18146E-02	5.90585E-02	5.05927E-02	4.79060E-02	4.86907E-02
G*	-2.77254E-02	-3.08302E-02	-3.11846E-02	-3.13983E-02	-3.17611E-02	-3.07348E-02
R^2	.0524	0.610	0.697	0.718	0.767	0.823
Sections omitted		24	14, 24	11, 14, 24	11, 14, 15, 24	1, 11, 14, 15, 24

TABLE 17 Models to define G* as a function of mix variables

	R^2 = 0.809	R^2 = 0.804	R^2 = 0.779	R^2 = 0.767
Predicting	ln(G*)	ln(G*)	ln(G*)	G*
constant	8.13378	5.34512	4.79242	256.401
P_{asp}	-0.49722	-0.136602		-35.6794
V_{air}	-0.140085			-15.0318
P_{200}				6.29193
fa	-0.116067		0.0144959	
$(P_{asp})^2$		-0.0312521	-0.030895	
$(V_{air})^2$	0.0034694	0.00355533		0.415377
$(P_{200})^2$	0.024439			
fa^2	0.00337748	0.00128764		0.0343932
$(P_{asp})^2 (V_{air})$	0.0169372	0.0163597		2.15752
$(P_{asp})^2 (P_{200})$	0.0240761	0.0304861	0.0142573	
$(P_{asp})^2 (fa)$		-0.00709602		
$(V_{air}) (P_{200})$				
$(V_{air}) (fa)$	-0.0016362	-0.0016957	-0.00177202	-0.224379
$(P_{200}) (fa)$	-0.010578	-0.00301201		

TABLE 18 Determination of *a* values for mixes containing ranges in P_{200} and fine aggregate *fa* (c = 0.340)

P_{asp}	V_{air}	P_{200}	*fa*	G*			field *a*
				NCSU	% Change	UCB	
5.5	6.5	5.0	20	73.19	-1.55	48.55	2.251714
5.5	6.5	5.0	24	73.41	-1.27	48.68	2.242338
5.5	6.5	5.0	28	74.73	0.53	49.57	2.181931
5.5	6.5	5.0	32	77.15	3.76	51.16	2.077697
5.5	6.5	5.0	36	80.67	8.20	53.35	1.942303
5.5	6.5	5.5	20	76.33	2.47	50.53	2.118575
5.5	6.5	5.5	24	76.55	2.90	50.74	2.104886
5.5	6.5	5.5	58	77.87	4.61	51.59	2.050853
5.5	6.5	5.5	32	80.29	7.65	53.08	1.958803
5.5	6.5	5.5	36	83.81	11.80	55.13	1.839263
5.5	6.5	6.0	20	79.48	6.13	52.33	2.004236
5.5	6.5	6.0	24	79.70	6.74	52.63	1.985780
5.5	6.5	6.0	28	81.02	8.38	53.44	1.937102
5.5	6.5	6.0	32	83.44	11.23	54.85	1.855183
5.5	6.5	6.0	36	86.96	15.13	56.77	1.748862
5.5	6.5	6.5	20	82.62	9.53	54.01	1.903695
5.5	6.5	6.5	24	82.84	10.29	54.39	1.881741
5.5	6.5	6.5	28	84.16	11.86	55.16	1.837606
5.5	6.5	6.5	32	86.58	14.55	56.48	1.764198
5.5	6.5	6.5	36	90.11	18.21	58.29	1.669007
5.5	6.5	7.0	20	85.77	12.69	55.57	1.814708
5.5	6.5	7.0	24	85.99	13.58	56.01	1.790207
5.5	6.5	7.0	28	87.31	15.08	56.75	1.749960
5.5	6.5	7.0	32	89.73	17.63	58.00	1.683767
5.5	6.5	7.0	36	93.25	21.08	59.70	1.598026

TABLE 19 Regression models to define the effects of mix variables on G* measured in LMLC compacted specimens

P_{asp}	V_{air}	P_{200}	*fa*	G*			
				R^2 = 0.809	**R^2 = 0.804**	**R^2 = 0.779**	**R^2 = 0.767**
5.5	6.5	5.5	20	78.38	77.62	77.38	76.33
5.5	6.5	5.5	24	67.79	74.59	78.30	76.55
5.5	6.5	5.5	28	65.33	74.69	79.24	77.87
5.5	6.5	5.5	32	70.14	77.94	80.19	80.29
5.5	6.5	5.5	36	83.90	84.75	81.15	83.81
5.5	6.5	4.5	28	60.27	68.72	73.27	71.58
5.5	6.5	5	28	62.37	71.64	76.20	74.73
5.5	6.5	5.5	28	65.33	74.69	79.24	77.87
5.5	6.5	6	28	69.28	77.87	82.41	81.02
5.5	6.5	6.5	28	74.36	81.18	85.71	84.16
5.5	4.5	5.5	28	72.87	83.38	87.51	87.63
5.5	5.5	5.5	28	68.76	78.63	83.27	82.34
5.5	6.5	5.5	28	65.33	74.69	79.24	77.87
5.5	7.5	5.5	28	62.51	71.45	75.41	74.24
5.5	8.5	5.5	28	60.22	68.84	71.76	71.44
4.5	6.5	5.5	28	84.28	94.69	99.79	99.53
5	6.5	5.5	28	74.20	84.76	89.61	88.70
5.5	6.5	5.5	28	65.33	74.69	79.24	77.87
6	6.5	5.5	28	57.52	64.80	69.00	67.04
6.5	6.5	5.5	28	50.64	55.35	59.16	56.22

TABLE 20A Linear regression fitting results of coefficient m_1 of coarse mix

Gradient	[-0.15,-0.10]	[-0.10,-0.05]	[-0.05,0.0]	[0.0,0.05]	[0.05,0.10]	[0.10,0.15]
Intercept	-0.08175863	-0.03479309	-0.03411011	-0.0431191	-0.03614109	-0.03233883
$T_b \cdot g$	0.01950062			0.03462285		
$T_b \cdot E_{sub}$		6.717861e-6	6.793519e-6		6.942432e-6	7.675834e-6
$(T_b \cdot g)^2$	0.002959697			-0.009660887		
$(T_b \cdot E_{sub})^2$		-3.21376e-11	-1.247114e-10		-2.164686e-10	-3.704025e-10
G		0.0845437	0.03453812		0.05250905	0.04367304
E_{sub}	0.0009615674		-0.00002152473	0.00002656082		-0.00003846102
V_{air}	0.001093299	0.0006746544	0.0006666258	0.0006601759	0.0006720647	0.0006863074
P_{asp}	0.003705114	0.00226696	0.002230678	0.002233238	0.002180534	0.002211977
$T_b \cdot g\, V_{air}$	0.0001268999			-0.0002971607		
$T_b \cdot E_{sub} \cdot V_{air}$		-5.584353e-8	-6.137919e-8		-7.738638e-8	-8.894092e-8
$T_b \cdot g\, P_{asp}$	0.0004663344			-0.001021566		
$T_b \cdot E_{sub} \cdot P_{asp}$		-2.0672e-7	-2.257651e-7		-2.846534e-7	-3.27908e-7
$V_{air} \cdot P_{asp}$	-0.00002792679	-0.00001991557	-0.00002062941	-0.00001568069	-0.00002628364	-0.00002838366
R^2	0.9959	0.9972	0.9971	0.9875	0.9965	0.999

TABLE 20B Linear regression fitting results of coefficient m_2 of coarse mix

Gradient	[-0.15,-0.10]	[-0.10,-0.05]	[-0.05,0.0]	[0.0,0.05]	[0.05,0.10]	[0.10,0.15]
Intercept	0.3562	0.3562	0.3562	0.3561	0.3562	0.3562
m_1	-1.0066	-1.0053	-1.0062	-1.0071	-1.0085	-1.0095
R^2	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000

TABLE 20C Linear regression fitting results of coefficient *A* of coarse mix

Gradient	[-0.15,-0.10]	[-0.10,-0.05]	[-0.05,0.0]	[0.0,0.05]	[0.05,0.10]	[0.10,0.15]
Intercept	0.1431	0.1423	0.1426	0.1445	0.1412	0.1409
m_1	-5.5021	-3.5916	-4.9619	-8.6655	-4.0365	-21.2729
m_1^2	-125.6612	-182.4311	-126.0442	-224.8148	-308.7180	-283.9384
m_1^3		-8698.2050	-668.5057		-22427.9359	15039.9723
m_1^4		-226481.9409			-873964.3969	
m_1^5		-2393516.9861			-13472803.3775	
$\sin(m_1/0.01)$	0.0216		0.0166	0.0453		0.172
R^2	0.9999	0.9998	0.9999	0.9958	0.9999	0.9998

TABLE 20D Linear regression fitting results of coefficient *A* of coarse mix

Gradient	[-0.15,-0.10]	[-0.10,-0.05]	[-0.05,0.0]	[0.0,0.05]	[0.05,0.10]	[0.10,0.15]
Intercept	-13.39914	-11.58667	-11.12271	-11.49544	-12.01581	-11.10854
$T_b \cdot g$	1.53532			2.294153		
$T_b \cdot E_{sub}$		0.0005025015	0.0004804504		0.0004590989	0.0005093146
$(T_b \cdot g)^2$	0.2220458			-0.5428994		
$(T_b \cdot E_{sub})^2$		-1.161947e-9	-5.566515e-9		-1.683449e-9	-1.405131e-8
G		6.384203	0.6557571		5.63907	2.487178
E_{sub}	0.05892822		-0.006828288	-0.002321114		-0.007310587
V_{air}	0.06316996	0.04783163	0.04758049	0.04731206	0.04742552	0.04794235
P_{asp}	0.2173902	0.1641741	0.163119	0.1601871	0.1608133	0.1615919
$T_b \cdot g\, V_{air}$	0.005865289			-0.01427007		
$T_b \cdot E_{sub} \cdot V_{air}$		-3.361553e-6	-3.468244e-6		-3.921476e-6	-4.321448e-6
$T_b \cdot g\, P_{asp}$	0.02175577			-0.0527957		
$T_b \cdot E_{sub} \cdot P_{asp}$		-0.0000124667	-0.00001285962		-0.00001453973	-0.00001601667
$V_{air} \cdot P_{asp}$	-0.00146397	-0.001153217	-0.001164524	-0.001324928	-0.001308766	-0.001402494
R^2	0.9987	0.9923	0.9985	0.9962	0.9917	0.9974

TABLE 21A Linear regression fitting results of coefficient m_1 of fine and fine plus mixes

Gradient	[-0.15,-0.10]	[-0.10,-0.05]	[-0.05,0.0]	[0.0,0.05]	[0.05,0.10]	[0.10,0.15]
Intercept	-0.07988474	-0.03634856	-0.03691352	-0.04270346	-0.03327317	-0.03607125
$T_b \cdot g$	0.0159674			0.03393941		0.006677723
$T_b \cdot E_{sub}$		7.199499e-6	6.842651e-6		7.529676e-6	
$(T_b \cdot g)^2$	0.002579341			-8.99276e-3		-0.0003165719
$(T_b \cdot E_{sub})^2$		-1.351548e-10	-1.361373e-10		-3.466108e-10	
G		0.07178455	0.02041707			
E_{sub}	0.0008506537			2.267549e-5		0.00004793386
V_{air}	0.001564374	0.00100384	0.0009612834	0.001012483	0.0009535552	0.0009445551
P_{asp}	0.003935108	0.002520002	0.002413306	0.002475931	0.002344938	0.002294424
$T_b \cdot g\, V_{air}$	0.0001833901			-0.0004657914		-0.000101136
$T_b \cdot E_{sub} \cdot V_{air}$		-9.289788e-8	-9.387046e-8		-1.124367e-7	
$T_b \cdot g\, P_{asp}$	0.0004900334			-0.001215362		-0.0002752519
$T_b \cdot E_{sub} \cdot P_{asp}$		-2.543658e-7	-2.57598e-7		-3.074625e-7	
$V_{air} \cdot P_{asp}$	-0.000046345	-0.0000360965	-3.451985e-5	-0.0000428518	-0.0000418012	-0.00004531004
R^2	0.9929	0.9971	0.9967	0.9898	0.9943	0.9976

TABLE 21B Linear regression fitting results of coefficient m_2 of fine and fine plus mixes

Gradient	[-0.15,-0.10]	[-0.10,-0.05]	[-0.05,0.0]	[0.0,0.05]	[0.05,0.10]	[0.10,0.15]
Intercept	0.3562	0.3562	0.3562	0.3560	0.3562	0.3561
m_1	-1.0073	-1.0062	-1.0069	-1.0086	-1.0096	-1.0103
R^2	0.9999	1.0000	1.0000	1.0000	1.0000	0.9999

TABLE 21C Linear regression fitting results of coefficient *T* of fine and fine plus mixes

Gradient	[-0.15,-0.10]	[-0.10,-0.05]	[-0.05,0.0]	[0.0,0.05]	[0.05,0.10]	[0.10,0.15]
Intercept	0.1425	0.1423	0.1422	0.1485	0.1411	0.1409
m_1	-6.7044	-6.4943	-6.5630	-3.3726	-4.2418	-4.4155
m_1^2	-162.0980	-157.8808	-160.1644	-57.7235	-379.9716	-503.7375
m_1^3					-31839.2312	-58146.4542
m_1^4					-1396525.8838	-3559716.1828
m_1^5					-23722319.7929	-84038344.7616
$\sin(m_1/0.01)$	0.0310	0.0294	0.0304	0.0073		
R^2	0.9999	0.9998	0.9998	0.9968	0.9998	0.9998

TABLE 21D Linear regression fitting results of coefficient *A* of fine and fine plus mixes

Gradient	[-0.15,-0.10]	[-0.10,-0.05]	[-0.05,0.0]	[0.0,0.05]	[0.05,0.10]	[0.10,0.15]
Intercept	-13.41943	-11.60635	-11.93162	-11.52406	-11.99708	-11.81081
$T_b \cdot g$	1.319236			2.235742		0.4622427
$T_b \cdot E_{sub}$		0.0004821816	0.0004740113		0.0004566367	
$(T_b \cdot g)^2$	0.1982441			-0.5164648		-0.01628367
$(T_b \cdot E_{sub})^2$		1.14947e-9	-2.061125e-9		-3.015393e-9	
g		5.958508	-2.517388		5.119211	
E_{sub}	0.05352491			-0.002119558		0.002269168
V_{air}	0.09092601	0.06912625	0.06805976	0.06760431	0.06782119	0.06745517
P_{asp}	0.2349178	0.1777141	0.1748931	0.1720754	0.1727083	0.1707851
$T_b \cdot g\, V_{air}$	0.008588157			-0.02108253		-0.005128277
$T_b \cdot E_{sub} \cdot V_{air}$		-5.169588e-6	-5.12326e-6		-5.737562e-6	
$T_b \cdot g\, P_{asp}$	0.02376517			-0.05790203		-0.01413554
$T_b \cdot E_{sub} \cdot P_{asp}$		-0.00001428444	-0.00001414514		-0.00001582453	
$V_{air} \cdot P_{asp}$	-0.002397742	-0.001936588	-0.001916482	-0.002129637	-0.002135857	-0.002265763
R^2	0.9975	0.9919	0.9933	0.9967	0.9926	0.9975

TABLE 22 Comparison of the results of the fatigue performance simulations and the condition survey data for WesTrack

section number	mix type	V_{air}	P_{asp}	ESAL@Center		ESAL@Wander		10% fatigue cracking		50% fatigue cracking	
				ESAL	Date	ESAL	Date	ESAL	Date	ESAL	Date
1	fine	8.8	5.55	377,281	06/17/96	407,821	06/21/96	NA	09/03/98	NA	09/03/98
2		10.4	4.92	321,777	06/08/96	339,348	06/14/96	2,899,176	07/26/97	NA	09/03/98
3		12.4	4.97	228,937	05/12/96	237,067	05/14/96	3,585,531	11/05/97	4,340,103	02/18/98
4		6.6	5.12	554,770	07/28/96	722,115	08/23/96	NA	09/03/98	NA	09/03/98
14		9	6.05	359,447	06/16/96	379,046	06/18/96	NA	09/03/98	NA	09/03/98
15		8.7	5.42	389,497	06/20/96	420,307	06/22/96	NA	09/03/98	NA	09/03/98
16		12.2	4.75	237,012	05/14/96	254,878	05/21/96	3,720,800	11/17/97	NA	09/03/98
17		11	5.74	306,473	06/06/96	313,389	06/07/96	4,227,572	01/18/98	NA	09/03/98
18		4.3	6.04	932,825	09/14/96	2,985,674	08/02/97	NA	09/03/98	NA	09/03/98
19	fine plus	7.2	5.89	522,507	07/26/96	671,603	08/07/96	NA	09/03/98	NA	09/03/98
20		10.9	5.88	311,190	06/06/96	321,934	06/08/96	NA	09/03/98	NA	09/03/98
21		4.2	6.75	1,120,796	10/07/96	3,077,371	08/10/97	NA	09/03/98	NA	09/03/98
22		8.1	5.23	477,540	07/16/96	557,757	07/28/96	NA	09/03/98	NA	09/03/98
9		3.9	6.56	2,473,892	04/23/97	3,207,338	08/21/97	NA	09/03/98	NA	09/03/98
10		11.8	5.28	281,464	05/25/96	301,208	06/03/96	1,750,322	12/29/96	4,215,885	01/17/98
11		7.9	5.99	475,939	07/16/96	564,704	07/28/96	NA	09/03/98	NA	09/03/98
12		4.6	5.84	1,120,257	10/07/96	3,051,019	08/08/97	NA	09/03/98	NA	09/03/98
13		5.9	6.51	657,852	08/04/96	1,092,379	10/03/96	NA	09/03/98	NA	09/03/98
5	coarse	8.1	5.63	1,332,219	10/24/96	3,090,234	08/11/97	1,757,238	12/30/96	2,311,472	04/08/97
6		10.8	5.71	639,137	08/03/96	994,875	09/22/96	1,668,205	12/16/96	1,794,207	02/19/97
7		6.9	6.49	3,096,162	08/11/97	NA	09/03/98	NA	09/03/98	NA	09/03/98
8		8.5	5.47	1,032,261	09/26/96	2,954,219	07/31/97	1,515,380	12/01/96	1,789,150	01/01/97
23		4.9	5.79	NA	03/09/98	NA	09/03/98	NA	09/03/98	NA	09/03/98
24		7.2	5.94	2,897,595	07/26/97	4,538,003	04/06/98	NA	09/03/98	NA	09/03/98
25		3.7	6.55	NA	09/03/98	NA	09/03/98	NA	09/03/98	NA	09/03/98
26		11	5.31	594,392	07/30/96	829,604	09/02/96	1,487,207	11/29/96	1,626,490	12/10/96

TABLE 23 Construction variations in mix and structural characteristics

Property	**Measure of variance**	**Value or range, WesTrack**	**Suggested value***
asphalt content	standard deviation	0.1-0.4%	0.19%
air void content	standard deviation	0.4-1.5%	1.2%
thickness, AC	standard deviation	0-5 mm***	$0.173(t_{AC})^{0.69}$ **

* Reflects materials/construction part of standard deviation.
** Although there was no range in thicknesses for WesTrack, this value is recommended for the thickness of AC used in the project under consideration.
***1 in. = 25.4 mm

CHAPTER 5

LIFE-CYCLE COST MODEL

5.1 OVERVIEW OF LIFE-CYCLE COST ANALYSIS

This chapter provides an overview of the LCC methodology used in the PRS for HMA pavements. It draws heavily on FHWA's recently completed, *Life-Cycle Cost Analysis in Pavement Design–Interim Technical Bulletin* (*20*). The *Interim Technical Bulletin* serves as a more thorough exposition of the information provided in this chapter.

According to the *Interim Technical Bulletin*, LCC analysis is defined as the following:

> . . . *an analysis technique that builds on well-founded principles of economic analysis to evaluate the overall long-term economic efficiency of competing alternative investment options. It does not address equity issues. It incorporates initial and discounted future agency, user, and other relevant costs over the life of alternative investments. It attempts to identify the best value (the lowest long-term cost that satisfies the performance objective being sought) for investment expenditures.*

In short, LCC analysis is a decision support tool that includes a logical and analytical evaluation framework for the purpose of comparing alternative investment options (e.g., an HMA pavement versus a portland cement concrete [PCC] pavement).

5.2 LIFE-CYCLE COST MODEL USED IN THE HOT-MIX ASPHALT PERFORMANCE-RELATED SPECIFICATION

The conceptual framework for the LCC model used in the PRS for HMA pavements was previously shown in Figure 4 of Part II. A more detailed depiction of the LCC model is shown in Figure 55, which represents the logic of the analysis process. For any given pavement, a simulated LCC distribution is required as part of the process for considering variability. The model employs a Monte Carlo simulation process to generate the LCC distribution by randomly varying the levels of the selected acceptance quality characteristics (AQCs). The LCC for a given iteration depends primarily on the predicted performance and triggered rehabilitation needs. In this version of the HMA PRS, the rehabilitation treatment will either be an HMA overlay to repair fatigue cracking or a milling operation followed by an HMA overlay to repair a rutting problem. In either case, the thickness of the overlay to last the remainder of the analysis period is determined using the overlay design procedure presented in the 1993 *AASHTO Guide for Design of Pavement Structures* (*21*). The cost to construct the overlay is used in conjunction with the year the treatment was triggered to calculate the net present value of the treatment cost. The process is applied for both the as-designed and the as-constructed pavement so that, ultimately, a contractor PA can be established based on the difference between the means of the two LCC distributions. The principal elements of the LCC model are described in subsequent sections of this chapter.

5.2.1 Inputs to the Life-Cycle Cost Model

Inputs to the model include those factors that influence pavement performance and the computation of the net present value of future costs associated with the rehabilitation activity performed during the duration of the analysis period. Inputs that influence pavement performance include the following:

- M&C factors (presently air void content, asphalt content, thickness, and percent of aggregate passing the 0.075-mm (No. 200) sieve).
- Environmental factors (presently limited to pavement temperature).
- Cumulative traffic (expressed in ESALs and forecasted at yearly intervals throughout the analysis period).
- Base course and roadbed soil characteristics (thickness and modulus value for each layer).

Inputs that influence the net present value of future costs associated with rehabilitation treatments include the following:

- The actual cost for the rehabilitation treatment (presently the future cost of the treatment expressed in real, constant dollars).
- A factor to account for the time value of money (presently a real, constant discount rate).

It must be emphasized that the inputs into the LCC model that cannot be directly controlled by the contractor *are not* *varied* during the Monte Carlo simulation. Stated another way, only those factors under direct control of the contractor are varied in the Monte Carlo simulation. These are as follows:

- Air void content.
- Asphalt content.
- Thickness.
- Percent of aggregate passing the 0.075-mm (No. 200) sieve.

All other inputs are held constant throughout the simulation process, except that ESALs are incremented through each year of the analysis period.

Chapter 6 of Part II provides guidelines for determining the required inputs for the LCC model.

5.2.2 Definition and Selection of Performance Prediction Models

The prediction of pavement performance is a key element in the LCC model because the distresses predicted are used in conjunction with the decision tree to trigger a rehabilitation activity which, in turn, generates a cost. Thus, the definition and selection of an appropriate model directly and significantly influences LCC analyses. Chapter 4 of Part II discussed the pavement performance prediction models included in the HMA PRS.

It should be noted that the *HMA Spec* software allows the incorporation of user-defined models. Thus, customized models more suitable to local or regional conditions than the default models can be used in the *HMA Spec* software.

5.2.3 Definition and Selection of Decision Trees

The decision tree in the LCC model is another key element. It provides the logic for determining if a treatment should be applied based on the magnitude of distress predicted by the pavement performance models. Presently, only the following treatments are employed in the default decision tree:

- *Do nothing* when the magnitude of predicted distress is less than a specified threshold.
- *HMA overlay* when the magnitude of predicted fatigue cracking is greater than a specified threshold.
- *Mill and HMA overlay* when the magnitude of predicted rut depth is greater than a specified threshold.

A portion of the default decision tree for coarse mixtures (for metric units) programmed into the *HMA Spec* software is shown in Figure 56. Note that it expects as input the output from the fatigue cracking and rut depth models developed for coarse mixtures and designated as Fatigue_Cracking_Coarse_1 and Rut_Depth_Coarse_SI_1, respectively. Note also that there is a companion default decision tree for fine mixtures. As indicated, the default decision tree only contains provision for triggering treatments based on fatigue cracking (expressed as a percent of the wheelpath) and rut depth (expressed in millimeters or inches). The criteria in the default decision tree for triggering treatments are based on the values used in the FHWA's original Nationwide Pavement Cost Model (NAPCOM) (*22*).

The decision tree is designed to branch on the first expression that evaluates to "true." Thus, if the road for which a specification is being developed is a rural interstate, the decision tree shown in Figure 56 will branch on the very first expression. Assuming the road is a rural interstate and that the fatigue cracking model predicted a magnitude of 3 percent and that the rut depth model predicted a magnitude of 5 mm (0.2 in.), the next expression (Fatigue_ Cracking_Coarse_1 ≥ 5 OR Rut_Depth_Coarse_SI_1 ≥ 6.4) would evaluate to "false." Given these levels of distress, the next expression (Fatigue_Cracking_Coarse_1 < 5 AND Rut_Depth_Coarse_SI_1 < 6.4) would evaluate to "true" and the decision tree would select the "do nothing" treatment.

It should be noted that the *HMA Spec* software permits incorporation of user-defined decision trees. Thus, custom decision trees can be developed that may be more suitable to local or regional policies and preferences.

5.2.4 Hot-Mix Asphalt Overlay Thickness Design

If the decision tree triggers a treatment that is either an HMA overlay or a mill and HMA overlay operation, the *HMA Spec* software will calculate the overlay thickness to last the remainder of the analysis period. This process is depicted in Figure 57 for a hypothetical problem involving both the as-constructed and as-designed pavement. The difference in thickness of the overlay (as well as in the time during the life of the pavement at which it is constructed) between the predicted life-cycle of the as-designed and as-constructed pavement, then, represents key determinants in the calculation of contractor PA. The principal reasons for adopting this approach are twofold:

- This approach eliminates the need to consider additional treatments beyond the first treatment—the overlay—assuming, of course, the overlay does in fact last the remainder of the analysis period.
- This approach eliminates the need to consider salvage value because the two alternatives under consideration—the as-designed pavement lot versus the as-constructed pavement lot—will have the same salvage value and, therefore, cancel out in the comparison.

The method for determining the overlay thickness is the "remaining life" approach from Part III, Chapter 5 of the 1993 *AASHTO Guide for Design of Pavement Structures* (*21*). The methodology is applicable to both the as-designed and the as-constructed pavements. Appendix B of Part II provides a description of the step-by-step procedures used for determining the HMA overlay thickness for either case.

5.2.5 User Costs

Although it is recognized that user costs can have a significant (if not overwhelming) impact on LCCs of competing pavement design alternatives and, therefore, should be included in the LCC model, the WesTrack team could not reach a consensus on how to incorporate such costs in this initial version of the HMA PRS. Thus, the ability to consider user costs must await possible future research to enhance the present version.

5.2.6 Net Present Value of Future Costs

The method for calculating the net present value (NPV) of the future cost(s) associated with future rehabilitation needs in this version of the PRS is restricted to the use of real, constant dollars and a real discount rate. The method for discounting future cost(s) is as follows:

$$\text{NPV} = \sum_{j=1}^{n} (\text{COST}_j \times (1+i)^{-j}) \tag{53}$$

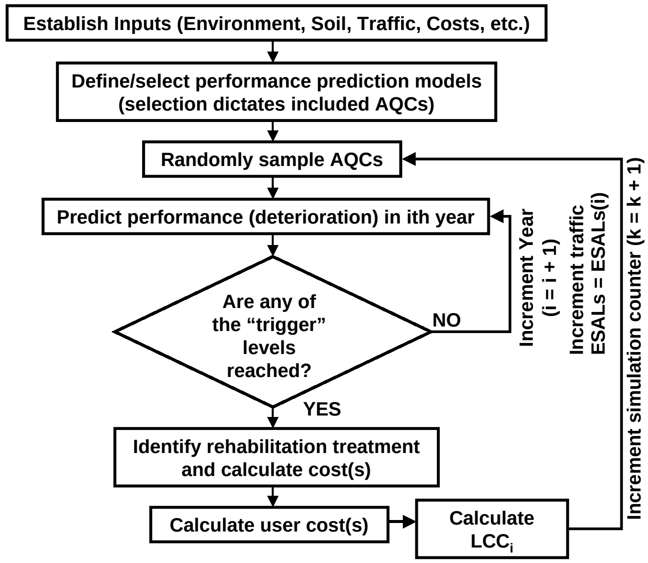
where

NPV = net present value of all future costs,
i = the *real* discount rate,
j = the year in which the future cost is incurred,
COST_j = magnitude of the future cost in real, constant dollars, and
n = last year of the analysis period.

5.3 SUMMARY

As discussed in Chapter 3 of Part II, the basis for contractor PA in this PRS is LCC. If the predicted LCC for the as-constructed pavement is significantly different from the specified (as-designed) pavement, then the contractor payment should be adjusted upwards (a bonus) if the LCC for the as-constructed pavement is less than the LCC for the as-designed pavement. Similarly, the contractor's payment should be adjusted downwards (a penalty) if the LCC for the as-constructed pavement is greater than the LCC for the as-designed pavement.

In describing the LCC analysis approach, this chapter provides a rationale for its use as a basis for PA. This chapter also describes the various components of the overall LCC model including required inputs, the selection of performance prediction models and rehabilitation decision trees, and the calculation of the net present value for any given existing pavement/rehabilitation combination. Included in this is a description of the rehabilitation needs model used for determining the HMA overlay thickness requirement for a given pavement combination.



$$\text{Mean LCC} = (\sum LCC_i) / k$$

Figure 55. Determination of LCCs using a Monte Carlo simulation process.

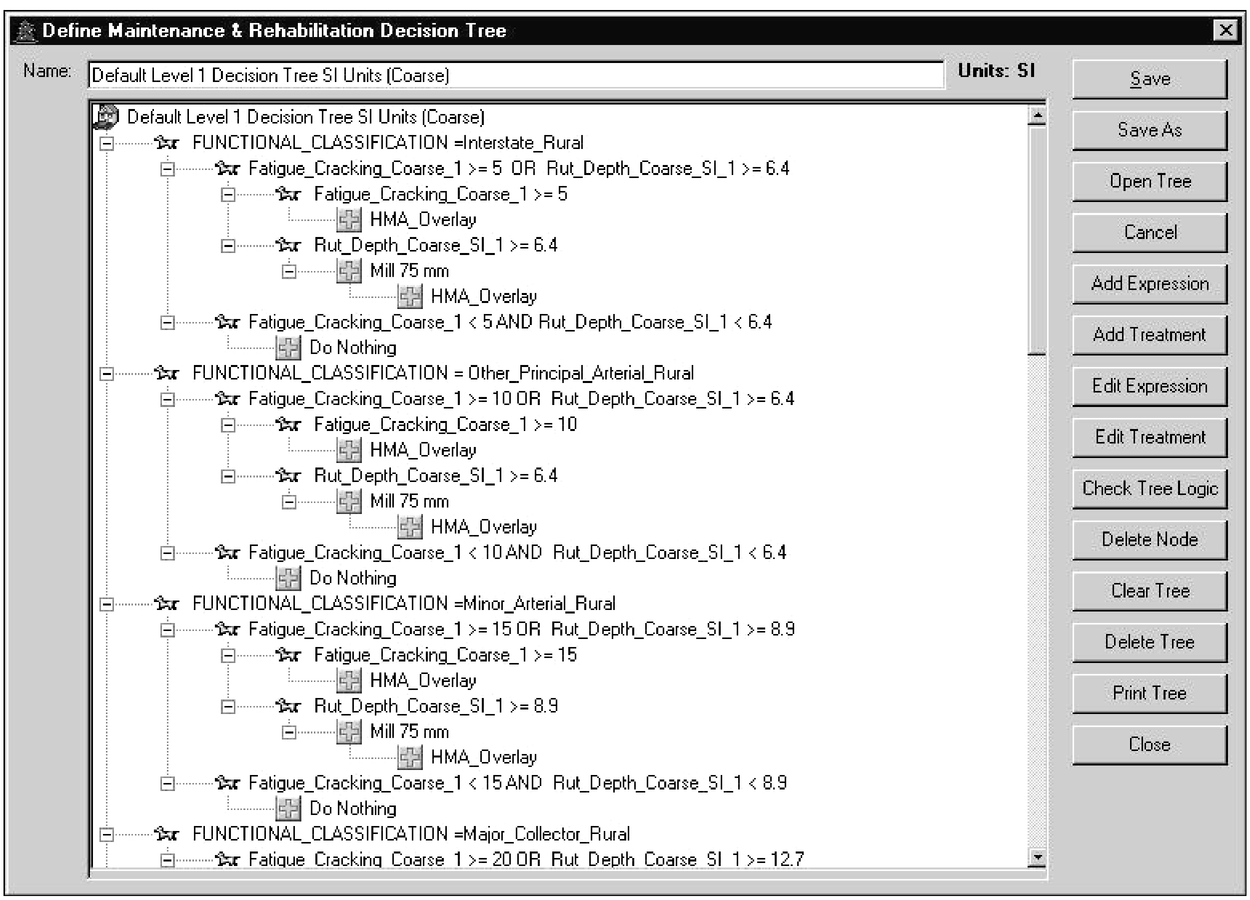


Figure 56. Default decision tree for coarse mixtures (level 1 models, SI units).

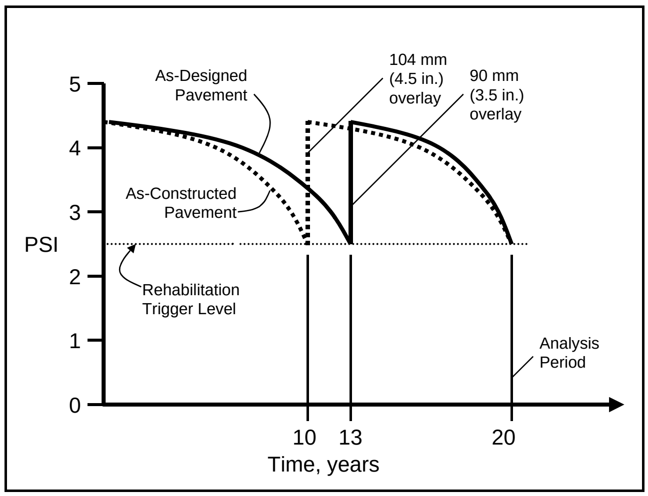


Figure 57. Illustration of approach to estimating HMA overlay thickness requirement for both as-designed and as-constructed pavements.

CHAPTER **6**

GUIDELINES FOR DETERMINING REQUIRED PERFORMANCE-RELATED SPECIFICATION INPUTS

6.1 INTRODUCTION

To use the HMA PRS software (*HMA Spec*) for the purpose of generating a PRS, the user must understand the implications of decisions made in its application. These include decisions for (1) defining appropriate levels for fixed inputs, (2) selecting appropriate models for predicting pavement performance, (3) selecting suitable parameters for the decision tree, and (4) selecting reasonable cost parameters. This chapter provides guidelines for determining the required PRS inputs while a user's guide for the *HMA Spec* software is provided in WesTrack Technical Report NCE-9 (*11*).

6.2 DEFINING PAVEMENT PERFORMANCE

As stated in Chapter 3 of Part II, the fundamental basis for PA in the PRS for HMA pavements is the difference in *predicted* LCCs between the as-designed (target) pavement and the as-constructed (delivered) pavement. Chapter 3 also detailed the process for predicting LCCs of the pavement lots and indicated that this was accomplished through prediction of pavement performance (in the form of distresses) in combination with the decision tree for triggering repair activities based on the predicted distresses. Obviously, defining future pavement performance requires performance prediction models. Chapter 4 details the development of pavement performance prediction models based on the results of the WesTrack project. These include the following:

- Empirical regression models for predicting fatigue cracking in the wheelpaths for both coarse and fine mixtures.
- Empirical regression models for predicting rut depth in the wheelpaths for both coarse and fine mixtures.
- M-E models for predicting fatigue cracking for both coarse and fine mixtures.
- M-E models for predicting rut depth for both coarse and fine mixtures.

Note that the regression models are referred to as level 1 models whereas the M-E models are referred to as level 2 models.

It is important to note that these models were developed with both fixed and variable inputs. Fixed inputs are those that the contractor has no control over such as traffic and environment. Variable inputs are those under the direct control of the contractor such as air void content, asphalt content, and thickness. Development of the models in this way allows prediction of pavement distress (performance) based on factors under the direct control of the contractor. The variable inputs to the models are the AQCs for the pavement lot.

The agency must consider which AQCs it needs to include in the specification. This, in turn, dictates selection of the performance models and their inputs. For example, inputs to the level 1 fatigue cracking model for coarse mixtures includes air void content, asphalt content, and traffic. Thus, the required AQCs would be air void content and asphalt content with traffic being considered a fixed input because the contractor has no control over it.

In development of a specification using the *HMA Spec* software, the agency must include at least one pavement performance prediction model and the selected model must have at least one associated AQC. In addition, because the output from the pavement performance prediction model(s) is used with a decision tree to decide if a repair activity is warranted, the agency must select a decision tree that includes decisions based on the output of the selected model. Further discussion of selection of decision trees is provided in Section 6.4.

The agency should select a pavement performance prediction model that best represents the conditions that exist in the location where the pavement lot is being placed. Arguably, selection of a level 2 M-E model is likely to be better than selection of a level 1 regression model. Regression models are generally limited to the conditions under which the original data were collected and used in the development of the model. Stated another way, regression models may not accurately represent conditions different from those under which the original data were collected. M-E models use engineering mechanics in an attempt to better represent actual conditions independent of location but still rely on empirical data in their development. Thus, although better suited for extrapolation beyond the range of the original data than regression models, M-E models have limitations as well.

With this understanding, selection of a level 1 regression model may be entirely adequate for many situations. For example, if the pavement lot for which the specification is being developed is on a low-volume road such as a two-lane rural highway, employing the use of a level 1 regression model may be appropriate. On the other hand, if the pavement lot is on a high-type facility such as an urban interstate, selection of a level 2 model may be more appropriate.

6.3 SELECTION OF INCLUDED ACCEPTANCE QUALITY CHARACTERISTICS

Selection of a pavement performance prediction model dictates the AQCs required as inputs into the model. Thus, to determine a PA based on the difference in predicted LCCs of the as-designed and as-constructed pavement lots, the required AQCs will need to be sampled and tested. For example, if the level 1 fatigue cracking model for coarse mixtures is selected as the only model used in the development of the specification, then the contractor, or agency, or both would need to sample the as-constructed pavement lot and test these samples to determine values for all AQCs required by the model. In the case of the level 1 fatigue cracking model for coarse mixtures, the AQCs are air void content and asphalt content. Thus, samples would need to be obtained from the as-constructed pavement lot to determine air void content and asphalt content such that the LCC of the as-constructed pavement lot could be predicted and compared with the predicted LCC of the as-designed pavement lot. Section 6.7 provides further details regarding AQC sampling and testing plans.

6.4 IDENTIFICATION OF FIXED INPUTS

The methodology for predicting LCCs includes, in addition to pavement performance prediction models (which contain fixed inputs), other required inputs including pavement design parameters, traffic parameters, decision tree criteria, and cost parameters. All of these required inputs are also termed "fixed inputs" because they are not under the contractor's control and are therefore equally applied to the as-designed and as-constructed pavement lots. For example, the number of ESALs applied to the pavement lot over the analysis period in the prediction of LCCs is not under the contractor's control. Thus, traffic is handled as a fixed input (even though it increases over time) and it is equally applied to the as-designed and as-constructed pavement lots. Further discussion regarding the selection of values for the fixed inputs is provided in this section.

6.4.1 Pavement Design Parameters

Pavement design parameters are required for both modeling purposes and overlay design purposes. These are equally applied to the as-designed and as-constructed pavement lots and include the following:

- The pavement design life.
- Initial and terminal smoothness expressed in terms of the International Roughness Index (IRI).
- The structural layer coefficient for HMA overlays.
- The thickness and modulus value for each layer (up to four) of the pavement structure.
- Design reliability expressed in percent as a function of the functional classification of the facility.
- The functional classification of the facility.
- The dimensions of the pavement lot.

Each of these is discussed in further detail as follows.

Pavement Design Life

This is the total expected amount of time in years for which the chosen pavement design is expected to carry traffic loads without the application of a global rehabilitation treatment (e.g., HMA overlay). It is the design life used in the procedure used for pavement design.

Initial and Terminal Smoothness

These are the initial and terminal IRI values corresponding to the initial design serviceability index (p_0) and the terminal design serviceability index (p_t), respectively, used in the AASHTO procedure for pavement design (*21*). These are used in the *HMA Spec* software to calculate HMA overlay thickness. Converting IRI to PSI may be accomplished using the following relationships (*23*):

$$PSI = 5 * e^{-0.18 * IRI} \quad \text{(where IRI is in units of m/km)} \qquad (54)$$

and

$$PSI = 5 * e^{-0.0028 * IRI} \quad \text{(where IRI is in units of in./mi)} \qquad (55)$$

Layer Coefficient for HMA Overlays

This is the structural layer coefficient needed for the design of an HMA overlay thickness using the "remaining life" procedure in Chapter 5 of Part III of the 1993 *AASHTO Guide for Design of Pavement Structures* (*21*). Figure 2.5 in Chapter 2 of Part II of the 1993 *AASHTO Guide for Design of Pavement Structures* is a chart for estimating this layer coefficient.

Pavement Layer Properties

These are the design thickness and design modulus values of each layer in the pavement structure including the roadbed soil. Section 2.3.5 in Chapter 2 of Part II of the 1993 *AASHTO Guide for Design of Pavement Structures* provides guidance for estimating the modulus values if these are not readily known.

Design Reliability

This is the level of reliability expressed in percent that is to be used by the *HMA Spec* software for overlay design. Generally, higher levels of reliability are chosen for higher-type facilities. For example, a typical level of reliability for an urban interstate might be 99.9 percent whereas that for a local road in a rural setting might be only 50 percent. Table 2.2 in Chapter 2 of Part II of the 1993 *AASHTO Guide for Design of Pavement Structures* provides suggested levels of reliability for various functional classifications. The values from this table have been entered as the default values within the *HMA Spec* software. However, it is important to note that the *HMA Spec* software allows the agency to define the level of design reliability for all of the functional classifications included in the software.

Functional Classification of the Facility

The functional classification identifies the type of facility according to its intended use or function, its economic importance, or both. Roadway facilities are generally classified as interstates, freeways/expressways, arterials, collectors, or local roads. These classifications can be further divided into urban or rural, major or minor, and so forth. The functional classification of a particular facility can usually be obtained from the agency's planning section. Within the *HMA Spec* software, the following classifications are available for identifying the type of facility:

- Urban interstate.
- Other urban freeway/expressway.
- Other urban principal arterial.
- Minor urban arterial.
- Urban collector.
- Rural interstate.
- Other rural principal arterial.
- Minor rural arterial.
- Major rural collector.
- Minor rural collector.

It should be noted that selection of the functional classification of the facility is how the agency indicates to the *HMA Spec* software the level of design reliability to be used for overlay design. Also, the *HMA Spec* software allows the user to define the level of reliability for each of the functional classifications listed above.

Dimensions of the Pavement Lot

These are the length and width of the pavement lot and are used in conjunction with treatment (e.g., HMA overlay, unit cost(s) to determine the overall cost of the treatment). This is the future cost that is discounted to the present for the purpose of determining the predicted LCC of a pavement alternative, that is, an as-designed pavement lot.

6.4.2 Traffic Parameters

Traffic parameters are required for both performance modeling and overlay design. These are equally applied to the as-designed and as-constructed pavement lots and include the following:

- Cumulative ESALs applied in any year of the LCC analysis period.
- Traffic growth rate.
- Traffic growth type.

Each of these is discussed in further detail in the following paragraphs.

Cumulative ESALs

To model pavement performance for the purpose of predicting LCCs, knowledge of the cumulative ESALs in each year of the analysis period is required. This same information is required for the purpose of determining HMA overlay thickness. Two methods of calculating the cumulative ESALs for each year of the analysis period are provided within the *HMA Spec* software as described below. In addition, the *HMA Spec* software allows calculation of these ESAL values based on average daily traffic.

Traffic Growth Rate

The traffic growth rate defines the annual increase in traffic expressed as a percentage. Growth in truck traffic, that is, ESALs, is assumed to be proportional to the growth in traffic. The expected growth of traffic on a particular facility can generally be obtained from the agency's planning section. The growth rate is used in conjunction with the growth type discussed in the next section to calculate cumulative ESALs over the analysis period.

Traffic Growth Type

The traffic growth type determines how the traffic growth rate is applied to calculate cumulative ESALs in any year of the analysis period. Selection of the traffic growth type dictates which method is used to calculate cumulative ESALs. The two traffic growth types included in the *HMA Spec* software are "simple" and "compound." These are defined as follows.

Simple Traffic Growth Type. Simple traffic growth type should be selected if traffic growth is assumed to follow a linear relationship over time. The following relationship is used to calculate the ESALs based on simple growth in any future year given the cumulative ESALs in the first year and the traffic growth rate:

$$(ESALs)_i = (ESALs)_1 + (i - 1) \times [(ESALs)_1 \times (Rate/100)] \quad (56)$$

where

$(ESALs)_i$ = number of cumulative ESALs in any future year, i,
$(ESALs)_1$ = number of cumulative ESALs applied in the first year of the pavements service life,
i = year for which cumulative ESALs are being calculated, and
Rate = traffic growth rate, expressed as a percentage (e.g., if 3.5 percent is the selected traffic growth rate, then rate = 3.5).

Compound Traffic Growth. Compound traffic growth type should be selected if traffic growth is assumed to increase based on a constant percentage of the previous year's traffic, that is, compounded over time. The following relationship is used to calculate the ESALs based on compound growth in any future year given the cumulative ESALs in the first year and the traffic growth rate:

$$(ESALs)_i = (ESALs)_1 \times [1 + (Rate/100)]^{(i-1)} \quad (57)$$

where

$(ESALs)_i$ = number of cumulative ESALs in any future year, i,
$(ESALs)_1$ = number of cumulative ESALs applied in the first year of the pavements service life,
i = year for which cumulative ESALs are being calculated, and
Rate = traffic growth rate, expressed as a percentage (e.g., if 3.5 percent is the selected traffic growth rate, then rate = 3.5).

6.4.3 Maintenance and Rehabilitation Decision Tree

The outputs from pavement performance prediction models are used in conjunction with the M&R decision tree to determine if a treatment is warranted in any particular year of the analysis period. Hence, the models selected to predict pavement performance drive the selection of a particular decision tree. For example, a decision tree containing criteria and treatments based on the magnitude of rutting is needed if a rutting model is selected to predict pavement performance.

The *HMA Spec* software contains several default decision trees as follows:

- Default decision trees for level 1 fatigue cracking and rut depth models for coarse mixtures for both U.S. customary and SI units.
- Default decision trees for level 1 fatigue cracking and rut depth models for fine mixtures for both U.S. customary and SI units.
- Default decision trees for level 2 fatigue cracking and rut depth models for coarse mixtures for both U.S. customary and SI units.
- Default decision trees for level 2 fatigue cracking and rut depth models for fine mixtures for both U.S. customary and SI units.

Selection of a particular *default* decision tree should be done only after it has been reviewed in the *HMA Spec* software because the decision criteria may or may not be appropriate to the agency. If the decision criteria is not appropriate, the *HMA Spec* software allows incorporation of user-defined decision trees. Thus, custom decision trees can be developed that may be more suitable to local or regional policies and preferences. It is important to note, however, that because the LCC model requires an overlay, user-defined decision trees must include an HMA overlay as a rehabilitation treatment (see Section 5.2).

6.4.4 Cost Parameters

In order to calculate the cost of a treatment triggered by the decision tree, information regarding the unit cost of the treatment is required. For example, the unit cost for an HMA overlay is needed to determine the total cost of the treatment. In this version of the PRS, only global rehabilitation treatments are considered (i.e., only treatments that are applied to the entire lot are included in the *HMA Spec* software). Moreover, this version of the PRS limits the global rehabilitation activities to the following:

- HMA overlays.
- A milling operation (prior to an HMA overlay).

Thus, unit cost information for these two global treatments are required for determining the total cost of the treatment applied to the entire lot. Unit cost data for various rehabilitation treatments is provided in the WesTrack Technical Report UNR-28 (*24*). Tables 1 and 2 of UNR-28 indicate that the typical average representative unit cost for HMA overlays is $0.094/m^2-mm ($2.00/yd^2-in.) whereas Table 3 of UNR-28 indicates the typical average representative cost for cold milling (including cleanup and haulage) is $0.0136/m^2-mm ($0.85/ yd^2-in.).

In addition to treatment unit costs, a discount rate is required to determine the net present value of the future treatment cost. The FHWA *Interim Technical Bulletin* recommends use of real, constant dollars and a real discount rate (*20*). This document further recommends a real discount rate of 4 percent, but states that a real discount rate in the range of 3 to 5 percent is acceptable.

6.5 SELECTION OF TARGET ACCEPTANCE QUALITY CHARACTERISTIC VALUES

The target mean and standard deviation for each AQC selected for inclusion in the specification for the as-designed pavement lot identifies the desired level of quality the agency expects from the contractor. In addition, the level of quality desired by the agency, and so specified by the AQC target values (means and standard deviations), identifies the quality for which the agency is willing to pay 100 percent of the contractor's bid price for the pavement lot. If the contractor provides an as-constructed pavement lot that exceeds the agency's expectations, then (on average) the contractor will receive an incentive (bonus) payment that exceeds the submitted bid price for the pavement lot. If, on the other hand, the contractor provides a pavement lot that falls short of agency expectations, then (on average) the contractor will receive a disincentive (penalty) payment that is less than the submitted bid price for the payment lot.

This version of the PRS assumes that the quality of the as-constructed pavement is determined solely from measured values of each included AQC because these are used to predict the LCC of the as-constructed pavement. However, exceeding agency expectations for a single included AQC, disregarding all other included AQCs, will not necessarily result in a bonus because of the potential effect of interaction amongst included AQCs. As an example, assume that only a fatigue cracking model is used for predicting pavement performance and that this model is dependent only on asphalt content and air void content (both AQCs) and applied traffic (a fixed input). Exceeding the target (agency specified value) for asphalt content would benefit resistance to fatigue cracking if the target thickness were provided, but an asphalt content greater than the target content may not benefit fatigue cracking if the provided thickness was less than the target thickness.

Target means for each included AQC should be the same as those derived from the pavement design and mixture design processes. That is, target HMA thickness should come from the pavement design process whereas asphalt content, air void content, and percent passing the 0.075-mm (No. 200) sieve should come from the job mix formula (JMF). WesTrack Technical Report UNR-29 (*25*) provides typical standard deviations for each of these AQCs. Representative values taken from this report are summarized in Table 24.

6.6 DEFINITION OF LOTS AND SUBLOTS

A basic premise within the HMA PRS is that a specification is developed for each pavement lot. Thus, a project which would typically comprise several pavement lots, would have several specifications associated with it. The default definition of a lot is provided in the Guide Specification for Hot-Mix Asphalt Pavement Material (see Chapter 10). It defines a placement (pavement) lot as an area of asphalt placed in a production lot, excluding miscellaneous areas and a placement (pavement) sublot as one-fifth the area of the placement lot. The Guide Specification further defines a production lot as 1,800 Mg (2,000 tons) of HMA regardless of the number of days required to produce the HMA. Thus, a sublot would consist of 360 Mg (400 tons) of HMA.

It should be emphasized that the *HMA Spec* software allows the user to define a lot and the number of sublots differently from the above definitions. For example, the lot could be defined as the entire project with each day's production comprising the sublots.

6.7 SPECIFYING AN ACCEPTANCE QUALITY CHARACTERISTIC SAMPLING AND TESTING PLAN

To determine a PA based on as-constructed pavement quality versus as-designed pavement quality, the included AQCs will need to be sampled and tested. The *HMA Spec* software assumes (in preconstruction use of the software) that sampling and testing is based on a per sublot basis and that the number of samples to obtain and test for each included AQC for each sublot are as specified in the Guide Specification (see Chapter 10). In postconstruction use of the software, the *HMA Spec* software allows input of the actual number of samples obtained and tested (which could be different from that specified).

The *HMA Spec* software also assumes that the test methods to be used for determining the AQC values are as specified in the Guide Specification (see Chapter 10). Regardless of the test method, a sufficient number of samples must be obtained and tested for each type of test to ensure reasonable confidence in the results. In general, the greater the number of tests, the greater the confidence in the results. Given that sampling and testing is relatively inexpensive compared with the capital cost of a typical HMA pavement project, agencies should obtain and test a minimum of one sample per sublot for each AQC and a pavement lot should contain a minimum of five sublots.

6.8 SELECTING AN APPROPRIATE BID PRICE FOR DEVELOPING PRECONSTRUCTION OUTPUT

As discussed in Chapter 3 of Part II, an appropriate bid price for the pavement lot for which a specification is to be generated by the *HMA Spec* software is required for preconstruction use of the software. In particular, a bid price is required for the development of the PF relationship for the pavement lot. As noted in Chapter 3, this relationship provides a tool for assessing the sensitivity of the PF (and PA) to the various AQCs and AQC combinations, that is, the interaction amongst AQCs. Thus, it is important that a representative bid price be provided by the agency.

Unit bid prices from previous paving projects of similar magnitude should be used to estimate an appropriate bid price. For example, an appropriate bid price could be the average of unit bid prices from similar paving projects from the previous year's construction season. Note that unit bid prices from other past years should be updated to include the effects of inflation.

6.9 SELECTING SIMULATION PARAMETERS

The *HMA Spec* software uses a Monte Carlo simulation process to predict the mean LCC of the as-designed pavement lot as well as to develop the PF relationship. The mean LCC of the as-designed pavement lot is the sum of the individual LCCs generated during each iteration of the Monte Carlo simulation process divided by the total number of iterations. Typically, Monte Carlo simulations involve hundreds, sometimes thousands, of iterations. Although the minimum number of iterations required to obtain stability in the predicted LCC of the as-designed pavement lot using the *HMA Spec* software has not been determined, the developers of PaveSpec (the rigid pavement PRS software) indicate stability is achieved at around 200 to 250 iterations using PaveSpec (*7*). They further indicate that the mean LCC does not significantly change with 500 iterations or more.

In the *HMA Spec* software, the total number of iterations for a given pavement lot is agency-defined. Based on this information, a minimum of 500 iterations is recommended in the Monte Carlo simulation process.

TABLE 24 Typical standard deviations for various acceptance quality characteristics

Acceptance Quality Characteristic	Representative Value
In-place air void content[1]	1.5%
Asphalt content[1]	0.3%
Percent passing #200 sieve[1]	0.9%
HMA thickness[2]	8 mm (0.3 in.)

[1] See Table 14 in reference 25.
[2] See Table 12 in reference 25.

CHAPTER 7

GUIDELINES FOR MAKING DECISIONS REGARDING PAY ADJUSTMENT

In the process of developing a PRS for any given project, the highway agency must make certain decisions regarding PA within the framework of the system. These decisions fall within the following five categories:

- Selecting an appropriate operating level.
- Selecting an appropriate application level.
- Selecting a suitable confidence level.
- Establishing rejectable quality levels.
- Placing constraints on PFs.

This chapter provides some general guidelines to the agency for making these decisions.

7.1 SELECTING AN APPROPRIATE OPERATING LEVEL

As discussed in Chapter 2 of Part II, there are two primary operating levels within this PRS system: basic and advanced. Each has its own strengths and weaknesses. Selection of the appropriate operating level depends on the needs and best interests of the highway agency.

The advanced operating level refers to the use of the *HMA Spec* software to generate the specification for any given project. With all the major components of the analytical process built into the computer code, the software represents a powerful tool to the engineer for generating a site-specific preconstructing specification. As part of the specification, the software produces a unique PF relationship which, in turn, focuses the emphasis of HMA pavement construction on the most significant M&C factors. Using postconstruction data, that is, field sampling and testing results, as input, the software is also capable of calculating the individual PAs for each lot. As with any tool, however, proper operation requires a certain level of understanding and expertise. Given the number of inputs and the inherent complexity of the advanced PRS, this is especially true for the *HMA Spec* software. Training, analysis of sensitivity, experimentation, and trial runs are all important to its successful operation.

The basic operating level represents the second option for using the PRS and is one that several SHAs (including New Jersey, California, and Illinois) have examined or adopted. Although some expertise is required to develop the standard PF relationships or PF tables associated with this level, actual operation is relatively simple. The user prepares many aspects of the specification using the more conventional approach, but then inserts a PF relationship or PF table (along with other suitable PRS language) that, along with the needed field sampling and testing, provide the basis for contractor payment. The advantage of the basic approach is its relative simplicity. The disadvantages, with regard to this PRS, are as follows:

a. Up-front work is required by someone with appropriate expertise to produce the matrix of candidate PF relationships or PF tables that can be selected for use in any given specification. (The *HMA Spec* software can be used for this.)
b. The ability to treat uncertainty by establishing a range of acceptable performance is lost. (Thus, *any* deviation in the predicted performance of the as-constructed pavement from the as-designed pavement becomes a basis for PA).
c. The effort required to calculate lot-by-lot PFs becomes a separate, manual spreadsheet operation for the user.
d. Some (albeit small) error will be introduced by the fact that the PF relationship or PF table will represent a category of sections, rather than a specific section.

For an agency making (or considering) the transition from more conventional methods of generating specifications, the basic level is probably the best place to start. If the agency already has experience with PRS and the use of computer software, then the advanced level is probably the next step.

7.2 SELECTING AN APPROPRIATE APPLICATION LEVEL

The current PRS system offers two levels of application: state-of-the-practice and state-of-the-art. Both levels require some evaluation and experimentation to comprehend. However, the advanced testing associated with the state-of-the-art level makes it likely to require the most effort. Consequently, it is recommended that the agency first become familiar with the PRS at the state-of-the-practice level before moving on to the state-of-the-art level.

7.3 SELECTING A SUITABLE CONFIDENCE LEVEL

One of the capabilities of the PRS system incorporated into the *HMA Spec* software is the ability to treat the effect of uncertainty in performance prediction as part of the method of PA (see Chapter 3). The primary sources of this uncertainty are the error associated with sampling and testing and the lack-of-fit inherent in the performance prediction models. The fact that this uncertainty exists makes it possible that a PA can be made (either bonus or penalty) when the difference in the LCC between the as-designed and as-constructed pavement is not significant.

The diagram shown in step 4 of Figure 8 (Chapter 3 of Part II) provides a conceptual diagram indicating how uncertainty of the LCC associated with the as-designed pavement (LCC_{des}) is considered in assessing a PA. Depending on the variance (or standard deviation) of LCC_{des} and desired confidence level, there is a zone on the LCC axis (defined by LCC_{lcl} and LCC_{ucl}) within which the actual LCC_{des} may exist. The confidence level (area under the curve) represents the probability of the contractor being assessed a PA when one is deserved. That being the case, it would not be statistically valid to penalize (or reward) the contractor if the predicted mean LCC of the as-constructed pavement is within that zone. (In the example of Figure 8, payment for lot 1 would result in a bonus because it is outside the zone, while payment for lot 2 would not be adjusted because it is inside the zone).

The advantage of a high confidence level is that it minimizes the risk of adjusting the contractor's payment when an adjustment is not deserved. The disadvantage is that a high confidence level reduces the motivation for the contractor to achieve quality. For example, Figure 58 illustrates the determination of the confidence limits for a relatively high confidence of 80 percent. Assuming in this case that the distribution of LCC is normal, the following equations would be used to calculate the lower and upper confidence limits, $(LCC)_{lcl}$ and $(LCC)_{ucl}$:

$$(LCC)_{lcl} = \overline{LCC}_{des} - Z_{CL} * (\sigma_{LCC})_{des} \quad (58)$$

$$(LCC)_{ucl} = \overline{LCC}_{des} + Z_{CL} * (\sigma_{LCC})_{des} \quad (59)$$

where

$\overline{LCC}_{des}$ = predicted mean LCC for the as-designed pavement,
Z_{CL} = standard normal deviate for the selected confidence level, and
$(\sigma_{LCC})_{des}$ = standard deviation of LCC for the as-designed pavement.

As can be seen from the example in Figure 58, the confidence limits associated with the 80 percent confidence level are relatively wide and less likely to encourage the contractor to strive for high quality.

Ultimately, the determination of the most suitable confidence level is going to depend on a thorough understanding of the magnitude of the variability of predicted LCC and the impact that confidence level has on PA. For the sake of achieving quality, the choice of a low confidence level (say 10 to 20 percent) is recommended, accepting the increased likelihood that a penalty or bonus may be assessed when one is not in order. In the example shown in Figure 58, the upper and lower confidence limits for a confidence level of 20 percent would be \$7.39/m^2 (\$6.22/sq yd) and \$8.11/m^2 (\$6.78/sq yd), respectively. (These compare to the \$6.10/m^2 (\$5.10/sq yd) and \$9.46/m^2 (\$7.91/sq yd) for the 80 percent confidence level shown in Figure 58).

7.4 ESTABLISHING REJECTABLE QUALITY LEVELS

The PRS system is designed to calculate a PA (or PF) based on the combined effect of the deviations of the as-constructed AQCs from their specified or as-designed levels. Although it can, the system is not intended to calculate PFs for individual AQCs. Consequently, it is important for the agency to establish a rejectable quality level (RQL) for each AQC. The RQL defines a level for a given AQC outside of which the lot would be rejected. For example, if the specified HMA thickness for a given project is 102 mm (4.0 in.), an agency may define an absolute minimum (or rejectable) thickness level of 91 mm (3.6 in.). If the mean thickness of a given lot is below this level, it would be rejected (leading to either a zero payment or a requirement for removal and replacement). Any thickness above the rejectable level, on the other hand, becomes a contributing factor, along with other surviving AQC levels, in the determination of an overall PA.

Table 25 provides some typical RQLs for the five original AQCs considered in this project. Note that three of the AQCs, that is, asphalt content, air void content, and percent passing the 0.075-mm (No. 200) sieve, have RQLs on both sides of the specified level.

7.5 PLACING CONSTRAINTS ON PAY FACTORS

Any given pavement lot which is not rejected (because of the criteria described in Section 7.4) becomes a candidate for PA (penalty or bonus). Step 5 of the flowchart shown in Figure 8 provides a diagram illustrating the constraints that can be placed on contractor's PFs. As indicated, constraints can be placed to cap the maximum PF, that is, maximum bonus, as well as to define a minimum PF level, below which zero payment or removal and replacement would be required.

In the first case, the agency must choose a PF level that provides some incentive to the contractor to achieve quality construction while, at the same time, limiting its maximum potential payout. The range of maximum PF is between 1.02 and 1.15. Typical values are in the range of 1.02 to 1.07.

In the second case, the agency must choose a PF level that behaves like an RQL. The reason for this is that, although each AQC has satisfied its individual RQL, a compound interaction between some (e.g., high asphalt content and low air void content) could result in an unacceptable performance level. Thus, by selecting a rejectable PF level, say between 0.5 and 0.8, the agency can exercise some additional control on the minimum quality level.

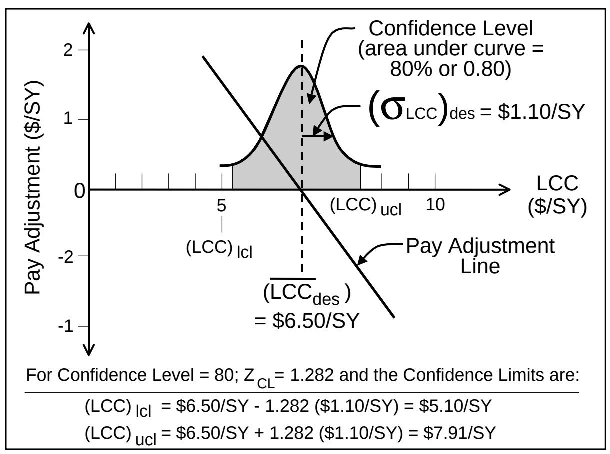


Figure 58. Determination of confidence limits for a given confidence level (1 SY = 0.84 m^2).

TABLE 25 Typical rejectable quality levels for five acceptance quality characteristics

Acceptance Quality Characteristic	Amount Below Target	Amount Above Target
HMA Thickness	8 to 13 mm (0.3 to 0.5 in.)	None
Initial Smoothness (California Profilograph)	None	48 to 80 mm/km (3 to 5 in./mi)
Asphalt Content	0.4 to 0.6 percent	0.3 to 0.5 percent
Air Void Content	1.0 to 2.0 percent	1.0 to 1.5 percent
Material Passing the 75 µm (No. 200) Sieve	1.0 to 1.5 percent	1.0 to 1.5 percent

CHAPTER **8**

STEP-BY-STEP GUIDE FOR GENERATING PERFORMANCE-RELATED SPECIFICATION PRECONSTRUCTION OUTPUT

The previous chapters of this report focused primarily on describing the PRS, summarizing the inherent method of PA and providing information on the selection of appropriate inputs. This chapter is to provide a step-by-step guide to generating preconstruction output using the advanced operating level of the PRS system, that is, the *HMA Spec* software. The step-wise part of this process is designed to be similar to the level 1 specification used in the FHWA's PRS for PCC pavements (*7*). The first ten steps deal with identifying the required inputs while the last three deal with generating the PF relationship(s) and producing the specification.

8.1 IDENTIFYING REQUIRED INPUTS

Step 1: Define the General Project Information. The information required as part of this first step is specific to a given project. It includes such items as project location, lane configuration, starting and ending stations, and functional classification of the facility. Guidelines for selection of these inputs are provided in Section 6.4 of this report.

Step 2: Define Pavement Performance. In this step, the agency must identify the distress indicators and the most appropriate mathematical models for predicting their progression. Alternative models developed, in part, from analysis of WesTrack performance are currently incorporated into the software for the two different application levels. For the state-of-the-practice level, level 1 models are available for permanent deformation and fatigue cracking for fine and coarse mixtures. Similarly, for the state-of-the-art level, level 2 candidate models exist for permanent deformation and fatigue cracking. Section 6.2 provides guidance on the selection of appropriate models, while Chapter 4 of Part II describes the actual models developed from analysis of WesTrack data.

Step 3: Select the Acceptance Quality Characteristics. AQCs refer to those AQCs that have the greatest impact on pavement performance and, so, receive special attention within the PRS system by way of potential for contractor PA. Five AQCs were targeted in the WesTrack experiment. These include HMA surface layer thickness, initial smoothness, asphalt content, air void content, and an aggregate gradation parameter (percent passing the 0.075-mm [No. 200] sieve). All but initial smoothness are provided for in the version of the *HMA Spec* software developed under this project. Agencies may choose to incorporate from one to all four for their specification. Guidance for their selection is provided in Section 6.5.

Step 4: Define the Required Fixed Input Values. Besides the levels associated with each AQC, a number of input values related to design, climate, and traffic are required by the various pavement performance prediction models. These values are equally used for predicting the performance of both as-designed and as-constructed pavements. Guidelines for their selection are provided in Section 6.4.

Step 5: Define the Acceptance Quality Characteristics Sampling and Testing Plan. The *HMA Spec* software was originally designed to allow the agency to define the sampling and testing procedures required for measuring the AQCs in the field. This included not only the sampling and testing methods, but also the number of samples per sublot. However, as indicated in Section 6.7, the *HMA Spec* software assumes (in preconstruction use of the software) that sampling and testing is based on a per sublot basis and that the number of samples to obtain and test for each included AQC for each sublot are as specified in the Guide Specification (see Chapter 10). In addition, the *HMA Spec* software also assumes that the test methods to be used for determining the AQC values are as specified in the Guide Specification (see Chapter 10).

Step 6: Define the As-Designed Acceptance Quality Characteristics Target Values. These are the target values included by the agency in the specification for the key AQCs. They include both the target mean and standard deviation for each AQC and are dependent on the selected sampling and testing plan set forth in the Guide Specification (see Chapter 10). Guidelines for selecting appropriate target values are presented in Section 6.5.

Step 7: Define Lots and Sublots. Lots and sublots must be clearly defined for each project. In the case of HMA pavements, they are defined geometrically by either length or area (and usually depend on the quantity of material that can be placed in one day). Section 6.6 provides more detailed guidance on this subject.

Step 8: Define the Maintenance and Rehabilitation Decision Tree. To estimate the costs associated with future M&R for both the as-designed and the as-constructed pavement, the agency must establish an appropriate M&R decision tree for triggering treatments based on the magnitude of predicted distress. M&R decision trees within the *HMA Spec* software are very similar to typical decision trees for pavement management systems. Guidance on selecting default decision trees is presented in Section 6.4 whereas guidance for establishing custom decision trees is provided in the user's guide for the *HMA Spec* software (*11*).

Step 9: Define the Unit Costs and Time Value of Money. Unit costs for all M&R treatments contained in the M&R decision tree (step 8) must be defined by the agency. In addition, to account for the differences in timing between the treatments applied to the as-designed and as-constructed pavements, an appropriate discount rate must be defined. Guidance on these inputs is provided in Section 6.4.

Step 10: Define the Simulation Parameters. A Monte Carlo simulation approach is used to generate an appropriate number of performance simulations for both the as-designed and the as-constructed pavements. From these simulations, the program calculates valid estimates of the mean and standard deviation of the LCC of the as-designed and as-constructed pavements. The simulations require certain agency-defined parameters, as discussed in Section 6.9, to perform these simulations.

8.2 GENERATING THE SPECIFICATION

Step 11: Develop/Modify the Specification. In this step, the agency must prepare the text (or language) of the HMA construction specification. The Guide Specification (provided in Appendix C and included as the default specification in the *HMA Spec* software) may be used in total or modified by the agency. The agency also has the option of incorporating its own specification, after it has been modified to accommodate the new method of contractor PA. Although the *HMA Spec* software was designed and developed to accommodate specifications different from the default specification, this must be done manually within the *HMA Spec* database.

Step 12: Execute the Software (in Preconstruction Mode). One of the easiest steps of the entire process is executing the *HMA Spec* software to generate the specification. With the inputs from steps 1 through 10, the software will carry out the pavement performance simulations, estimate the LCCs for all treatment combinations and, then, generate a single PF relationship. This PF relationship will account for the performance-weighted effects of all the AQCs and their significant interactions. When completed, the relationship will be incorporated directly into the electronic version of the construction specification and printed. Although the software does not yet incorporate a feature for producing OC curves that permit the agency and the contractor to assess their respective risk, the PF relationship does provide some indication of the relative sensitivity of PF to each AQC. (Since the z-values for each AQC represent normalized variations with respect to their as-designed standard deviations, the coefficients for each AQC term indicate the relative effect on contractor payment). However, the preconstruction specification component of the *HMA Spec* software does not have the ability to consider the effect of the agency and contractor agreeing to some confidence level within which no PA would be made.

Step 13: Proof the Specification. The PRS system is a tool available to the agency to help prepare specifications for HMA pavement construction that encourage the contractor to achieve better quality on those M&C factors that have the greatest effect on performance. Like most computer software, the PRS system is neither bug-free nor foolproof. Accordingly, it is the ultimate responsibility of the agency to thoroughly review the specification and the accompanying PF relationship to ensure that no text errors have been made and that the terms for contractor payment are reasonable.

CHAPTER **9**

STEP-BY-STEP GUIDE FOR DETERMINING PAY ADJUSTMENTS FOR AS-CONSTRUCTED PAVEMENT LOTS

9.1 INTRODUCTION

The PA for a pavement lot can be determined after the lot has been constructed and sampled and tested for each included AQC. This chapter provides guidance on determining the PA for a given as-constructed pavement lot.

9.2 PROCESS

Step 1: Divide the As-Constructed Pavement Lot into Sublots. As stated in Chapter 6, the *HMA Spec* software assumes that sampling and testing of each included AQC is conducted on a per sublot basis. That is, the number of samples to obtain for testing is specified on a per sublot basis. The *HMA Spec* software further assumes the as-constructed pavement lot will be divided into sublots as defined in the Guide Specification (see Chapter 10). The Guide Specification indicates the lot should be divided into five sublots with each sublot having approximately equal areas.

Because it is likely that loose-mix samples will be taken during the placement of the HMA pavement lot, determination of the sublot size must be accomplished prior to the paving operation. Further, the starting and ending locations of each sublot should be determined and identified on site to facilitate easy identification of sampling locations. These lengths become the target sublot lengths.

Step 2: Determine the As-Constructed Acceptance Sampling Locations. Once the lot is divided into the appropriate number of sublots and the termini of each sublot identified, the procedure outlined in Attachment A of the Guide Specification (see Chapter 10) should be used to identify exact sampling locations. This attachment outlines procedures for stratified random sampling.

Step 3: Conduct Lot Sampling and Testing. When paving begins for a given lot, the actual sublot length should equal the target sublot length. If, however, a problem occurs during the paving of this first sublot (requiring the start of a new lot) and the pavement length is less than the target sublot length, then the sublot should be assumed to represent the entire lot and should be accepted by another method mutually agreed on between the contractor and agency.

If paving of the first sublot is successful, additional sublots (each equal in length to the target sublot) are defined consecutively until paving operations are complete for the lot. Following this approach, each sublot will have a length equal to the target sublot length except for the final sublot. If the final sublot has a length less than half the target sublot length, then the final sublot area should be included with the previous sublot. If, on the other hand, the final sublot length is greater than or equal to half the target sublot length, then the final sublot should be sampled and tested in the same manner as all other sublots in the lot.

Once the actual number of sublots is determined in the field, each sublot will need to be sampled according to the sampling and testing plan in the specification and at locations previously determined in step 2. The results of these tests will then be used to determine the PF and PA for the as-constructed pavement lot.

Step 4: Determine the Overall Lot Pay Factor. Once the sampling and testing program for the lot is complete, the PF for the as-constructed pavement lot is determined through use of the *HMA Spec* software or through simplified PF relationships as described in Chapter 11. If simplified PF relationships are used, the results from the testing program must be reduced to mean values for each AQC tested and the z-value for the AQC must be determined (see step 4 in Section 3.3.1). Once the z-values of all AQCs are determined, they are entered into the PF relationship to determine the PF. Although stochastic variability was accounted for in the development of the PF relationship, use of the relationship does not account for stochastic variability.

If the *HMA Spec* software is used, the individual results from the testing program are entered into the program and reduced or reduced data are entered. In either case, the program utilizes the mean and standard deviation of each AQC in a Monte Carlo simulation process to predict the LCC of the as-constructed pavement lot in the same manner as the as-designed pavement lot. Once the predicted LCC of the as-constructed pavement lot is determined, the PF is determined by the program according to equation 4 in step 4 of Section 3.4.1.

Step 5: Compute the Payment for the Lot. Once the PF is calculated, the payment for the lot can be determined from equation 60 as follows:

$$\text{Contractor's Lot Payment} = (\text{Bid Price}) \times (\text{Pay Factor}) \times (\text{Lot Size}) \quad (60)$$

where

Contractor's Lot Payment = adjusted payment made to the contractor for the as-constructed pavement lot,
Bid Price = contractor's actual bid price in \$/sq m (or \$/sq yd),
Pay Factor = computed PF from step 4 above, and
Lot Size = area of lot in sq m (or sq yd).

CHAPTER **10**

GUIDE PERFORMANCE SPECIFICATION FOR WESTRACK

10.1 INTRODUCTION

This chapter provides a summary of the information used to develop the "WesTrack Guide Specification" that is contained within the HMA performance-related software developed in the WesTrack project. The WesTrack Guide Specification was developed with background provided by references 8, 29 through 49, and the NCHRP Project 9-20 panel's specific guidance to use the guide specification resulting from NCHRP Project 9-7 (*38*). The principal investigator's experience as a team member in the development of three state QC/QA-type specifications was also used.

10.2 BACKGROUND

A review of existing QC/QA specifications (*26–37*) indicates that a wide variety of methods are used to describe their workings. The Guide Specification described here attempts to simplify the QC/QA specification formats currently used while providing the basic elements of this type of specification.

This Guide Specification was developed based on several assumptions and decisions made by the WesTrack team:

- AASHTO test methods and specifications are used wherever possible.
- ASTM test methods and specifications are used when AASHTO methods are not available.
- A QC plan is required with minimum sampling and testing requirements.
- All personnel performing the sampling, testing, and inspection must be certified or qualified.
- All testing must be performed in an accredited or qualified laboratory.
- Recycled asphalt pavement (RAP) is allowed in the HMA.
- Baghouse fines can be reintroduced into the HMA.
- A construction section is included in the Guide Specification.
- Superpave mixture design methods are used.
- Multiple mixture designs can be used on a project.
- The laboratory mixture design is approved based on a paper review of information.
- The field trial section information provides the mixture design for the project.
- Multiple job mix formulae can be used on a project.
- QC/QA sampling and testing and pay are based on lots/sublots.
- An HMA lot is 1,800 Mg (2,000 tons) regardless of the number of days required to produce the quantity.
- An HMA sublot is 360 Mg (400 tons) (5 sublots per lot).
- Sampling is performed by the contractor and witnessed by the agency engineer.
- The contractor is responsible for QC testing.
- The agency is responsible for QA testing and testing for pay.
- QC/QA sampling and testing are included for the asphalt binder, aggregate, HMA, and ride quality.
- Statistical QC/QA techniques are used for sampling, QC charts, comparison of data sets, and determination of PWL.
- HMA production is separated from HMA placement for QC testing, QA testing, and acceptance.
- Acceptance is based on QC and QA tests.
- PA factors are used for asphalt binder, HMA, and ride quality.
- PA factors for HMA are based on WesTrack performance models for rutting and fatigue.

10.3 KEY SECTIONS

The sections and first order subsections of the Guide Specification are shown in Figure 59 (Figure C.1 of the Guide Specification). The following are the section titles:

- 1.0 Introduction.
- 2.0 Materials.
- 3.0 Construction.
- 4.0 Mixture Design.
- 5.0 Job Mix Formula.
- 6.0 Lot and Sublot.
- 7.0 Quality Control.
- 8.0 Quality Assurance.
- 9.0 Acceptance.
- 10.0 Measurement.
- 11.0 Pay Adjustment Factor and Payment.

The sections of the Guide Specification and their interaction that control the acceptance activities are shown in Figure 60 (Figure C.2 of the Guide Specification). Both QC and QA testing are used to control the quality of the project. The Guide Specification will not allow for more than 3 consecutive days of PA factors below 1.00.

10.3.1 Introduction

The "Introduction" section defines the material, its use, and the scope of the Guide Specification. Relevant documents are identified, as well as terminology associated with the Guide Specification. Requirements for personnel (certification) and laboratories (accreditation) are defined. Sampling and testing standards and the required QC plan are defined in this section of the Guide Specification.

10.3.2 Materials

The "Materials" section of the Guide Specification defines the requirements for the asphalt binder, aggregate, mineral filler, lime, baghouse fines, and RAP.

10.3.3 Construction

The "Construction" section of the Guide Specification describes the materials handling requirements, mixing plant requirements, hauling equipment, and laydown and compaction operations. General restrictions on placement of the HMA are also included in the Guide Specification. The equipment section is that typically included in current state highway agency specifications.

10.3.4 Mixture Design

The "Mixture Design" section of the Guide Specification defines the material and mixture design requirements. The HMA mixture design is performed by the contractor and approved by the agency engineer. The approval process includes the engineer performing tests on the asphalt binder and aggregate and a paper review of the submitted LMLC mix design submitted by the contractor.

The engineer does a complete set of tests on the FMLC samples obtained from the trial field section placed by the contractor. New mixture designs can be developed as frequently as necessary to produce a quality HMA pavement. New mixture designs are required with a change in asphalt binder source or grade and with a change in aggregate source or with aggregate variability.

10.3.5 Job Mix Formula

The "Job Mix Formula" section of the Guide Specification defines the process of developing the JMF to be used on the project. Multiple job mix formulae can be developed for a project to ensure that quality HMA is produced. JMF adjustments can be made during the construction of the project.

10.3.6 Lot and Sublot

The "Lot and Sublot" section of the Guide Specification defines the lot and sublot size for asphalt binder sampling and testing, aggregate sampling and testing, HMA sampling and testing for production, and placement and ride quality determination. The lot size is a production day of HMA. Five production days form a lot for the asphalt binder and aggregate.

An HMA lot is defined as 1,800 Mg (2,000 tons) regardless of the number of days required to produce the HMA. Five sublots of 360 Mg (400 tons) each form a lot.

10.3.7 Quality Control

The "Quality Control" section of the Guide Specification defines the QC sampling and testing requirements for the contractor. Sampling and testing is required for the asphalt binder, aggregate, HMA production, HMA placement, and ride quality, as shown in Figure 60. Statistical QC charts are used as well as statistical techniques for comparison of QC results with QA results.

10.3.8 Quality Assurance

The "Quality Assurance" section of the Guide Specification defines the QA sampling and testing requirements for the engineer. Testing is required for the asphalt binder, aggregate, HMA production, HMA placement, and ride quality as shown in Figure 60. Referee testing is also identified in this section of the Guide Specification.

10.3.9 Acceptance

The "Acceptance" section of the Guide Specification defines the acceptance procedure for the engineer. Both QC and QA test results are required for acceptance. Mat irregularities and individual loads of HMA are addressed in this section of the Guide Specification. Acceptance programs are described for asphalt binders, aggregate, HMA, and ride quality.

10.3.10 Measurement, Pay Adjustment Factors, and Payment

The "Measurement, Pay Adjustment Factors and Payment" sections of the Guide Specification define the procedure to be used by the engineer to determine payment for the HMA. Separate PAs are made for asphalt binder, HMA, and ride quality.

The HMA PA factors are based on performance of fine- and coarse-graded Superpave mixtures at WesTrack. Different PA factors are used for fine- and coarse-graded mixtures. Fine-graded mixtures are those with gradations passing above the lower limits of the restricted zone. Coarse-graded mixtures are those with gradations passing below the lower limits of the restricted zone.

The HMA is paid on a weight basis. If lightweight aggregates are used, payment programs on a volume basis must be developed by the SHA.

10.4 COMMENTS

10.4.1 Acceptance Tolerances for Mix Design and Quality Control and Quality Assurance Testing

The requirements for a "field trial section" mixture acceptance based on JMF 1 and the tolerances shown in Tables 26 and 27 (Tables C.15 and C.16 of the Guide Specification) are based on NCHRP Project 9-7 recommendations (*38*). These same tolerances are used in the plans for QC and acceptance. Based on field construction variability information presented in reference 31, these limits are very restrictive and may be modified by the SHA and the contractors to fit local practices.

10.4.2 Superpave Asphalt Binders and Mixture Design

The latest approved specification for Superpave Performance-Graded Binders and the Superpave Volumetric Mixture Design have been used in this Guide Specification. Future changes to these specifications should be incorporated as part of the Guide Specification.

10.4.3 Quality Control/Quality Assurance Sampling and Testing

Tables 28 and 29 (Tables C.18 and C.20 of the Guide Specification) contain a summary of the QC/QA sampling and testing requirements for asphalt binders and aggregates. Tables 30 and 31 (Tables C.22 and C.23 of the Guide Specification) contain a summary of the QC/QA sampling and testing requirements for HMA production and placement. In general, the frequency of testing is reduced for the QA testing as compared with the QC testing. The test results used for determining PA factors (with the exception of gyratory-compacted air voids) are those used for QA purposes. Public agencies should consider reducing the frequency of testing based on statistical comparisons of the results of QC and QA testing as well as the uniformity achieved in production and placement.

In-place air voids determined from cores are used for QA and PA factor determination for HMA placement. Nuclear or other approved methods may be used for QC purposes.

10.4.4 Asphalt Binders

Table 32 (Table C.19 of the Guide Specification) defines the acceptable difference between contractor and engineer test results for asphalt binders. This table is based on Utah Department of Transportation (UDOT) requirements and was to be revised by the department prior to the 2000 construction season.

Table 33 (Table C.24 of the Guide Specification), which defines PA factors for asphalt binders, was also obtained from the UDOT. It was to be revised by the department prior to the 2000 construction season.

10.4.5 Aggregates

Table 34 (Table C.21 of the Guide Specification), which defines the acceptable difference between contractor and engineer test results for aggregates, was developed from AASHTO and ASTM precision and bias statements for the test methods listed in the table. The acceptable difference for the flat and elongated particles was estimated by the authors.

10.4.6 Guide Specification

The Guide Specification appears in Appendix C.

1.0 INTRODUCTION	2.0 MATERIALS	3.0 CONSTRUCTION	4.0 MIXTURE DESIGN	5.0 JOB MIX FORMULA
1.1 DESCRIPTION 1.2 USE 1.3 SCOPE 1.4 DOCUMENTS 1.5 TERMINOLOGY 1.6 LAB, EQUIPMENT & PERSONNEL 1.7 SAMPLING & TESTING 1.8 QC PLAN	2.1 ASPHALT BINDER 2.2 AGGREGATE 2.3 MINERAL FILLER 2.4 LIME 2.5 BAGHOUSE FINES 2.6 RAP	3.1 GENERAL 3.2 STOCKPILING, STORAGE, FEEDING & DRYING 3.3 MIXING PLANTS 3.4 SURGE-STORAGE SYSTEMS 3.5 HAULING EQUIPMENT 3.6 SURFACE PREPARATION 3.7 PLACING 3.8 JOINTS 3.9 COMPACTION 3.10 RESTRICTIONS	4.1 GENERAL 4.2 REQUIREMENTS 4.3 SUBMITTALS 4.4 APPROVAL 4.5 NEW LAB DESIGN	5.1 DESCRIPTION 5.2 JOB MIX FORMULA 1 5.3 FIELD TRIAL SECTIONS 5.4 JOB MIX FORMULA 2 5.5 SECOND DAY PRODUCTION 5.6 JOB MIX FORMULA ADJUSTMENTS 5.7 COMPACTION ROLLING PATTERN

6.0 LOT AND SUBLOT	7.0 QUALITY CONTROL	8.0 QUALITY ASSURANCE	9.0 ACCEPTANCE	10.0 MEASUREMENT
6.1 GENERAL 6.2 ASPHALT BINDER 6.3 AGGREGATE 6.4 HMA PRODUCTION 6.5 HMA PLACEMENT 6.6 RIDE QUALITY	7.1 ASPHALT BINDERS 7.2 AGGREGATES 7.3 HMA PRODUCTION 7.4 HMA PLACEMENT 7.5 RIDE QUALITY	8.1 ASPHALT BINDERS 8.2 AGGREGATES 8.3 HMA PRODUCTION 8.4 HMA PLACEMENT 8.5 RIDE QUALITY	9.1 GENERAL 9.2 ASPHALT BINDER 9.3 AGGREGATE 9.4 HMA PRODUCTION 9.5 HMA PLACEMENT 9.6 RIDE QUALITY 9.7 IRREGULARITIES & SEGREGATION 9.8 INDIVIDUAL LOADS	10.1 GENERAL **11.0 PAY ADJUSTMENT FACTOR & PAYMENT** 11.1 GENERAL 11.2 ASPHALT BINDER 11.3 AGGREGATE 11.4 HMA 11.5 RIDE QUALITY 11.6 PAY FACTOR ACCEPTANCE 11.7 PAY FACTORS BELOW 1.00

Figure 59. Hot-mix asphalt quality control/quality assurance guide specification section designations.

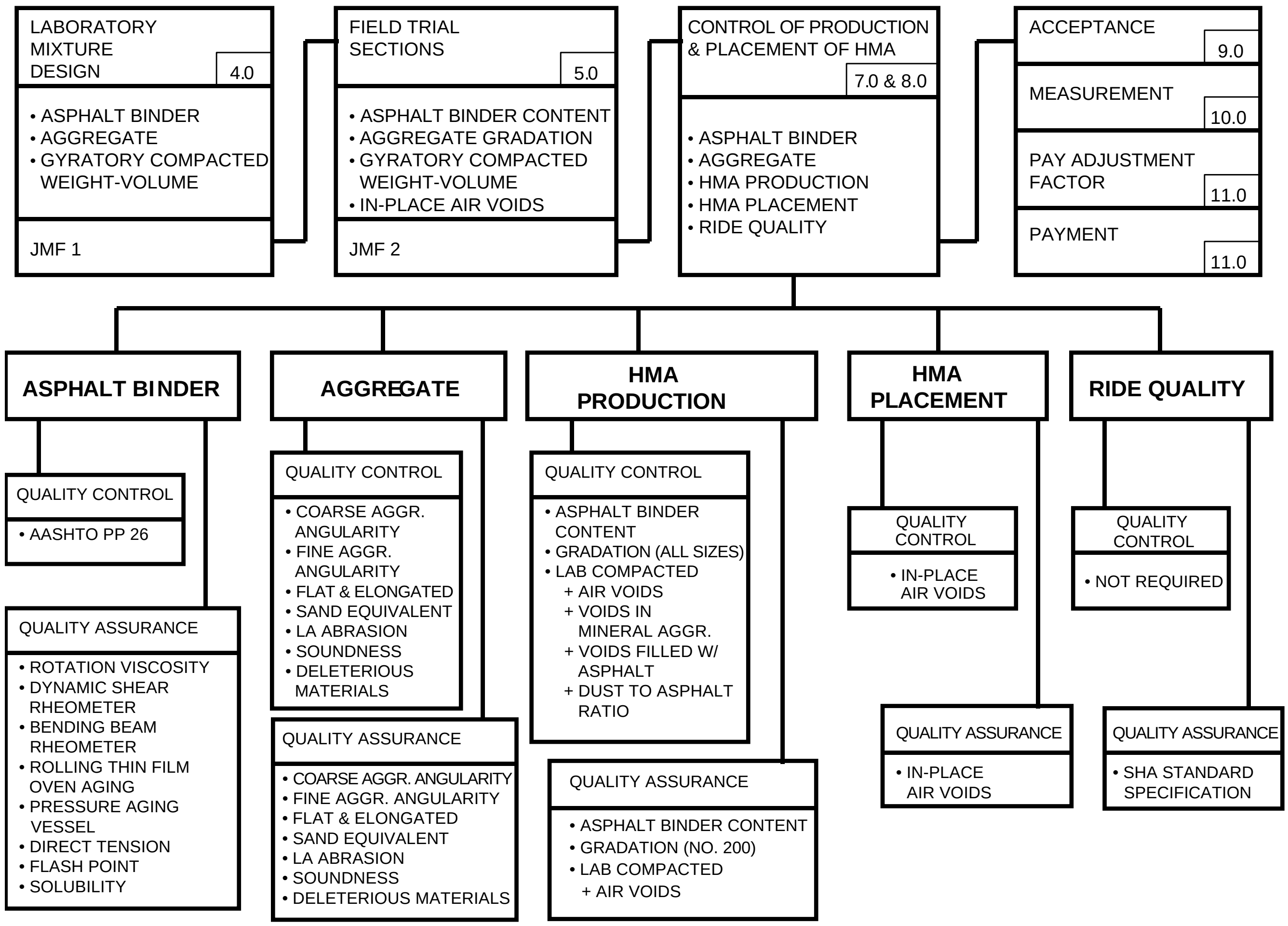


Figure 60. Hot-mix asphalt quality control/quality assurance guide specification tests.

TABLE 26 Aggregate gradation and asphalt binder content tolerances for field trial section acceptance

Sieve Size*		Tolerance, Mass
Metric, mm	U.S. Customary	
50	2.0-in.	± 3.0
37.5	1.5-in.	± 3.0
25	1-in.	± 3.0
19	3/4-in.	± 3.0
12.5	1/2-in.	± 3.0
9.5	3/8-in.	± 3.0
4.75	No. 4	± 3.0
2.36	No. 8	± 2.0
1.18	No. 16	± 2.0
0.600	No. 30	± 2.0
0.300	No. 50	± 2.0
0.150	No. 100	± 2.0
0.075	No. 200	± 0.7
Asphalt binder content,** percent by mass		± 0.13

*The gradation (AASHTO T27) shall be determined after the asphalt content is determined by the Ignition Test (ASTM D6307).
**Asphalt content determined by ASTM D6307 (Ignition Test).

Note: Tolerances based on JMF 1.

TABLE 27 Volumetric tolerances for field trial section acceptance

Test Method			Tolerances
Description	Number		
	AASHTO	ASTM	
A. Gyratory-compacted sample properties at N_{design}	TP4		
1. Air voids (V_{air})	T269	D3203	± 1
2. Voids in mineral aggregate (VMA)	PP28		± 1
3. Voids filled with asphalt (VFA)	PP28		± 5
4. Bulk specific gravity (Gmb)	T166	D2726	± 0.022
5. Dust-to-binder ratio	PP28		0.6 to 1.6
6. Theoretical maximum specific gravity (Gmm)	T209	D2041	± 0.015
B. In-place air voids	T269	D3203	2 to 7

Note: Tolerances based on JMF 1.

TABLE 28 Asphalt binder sampling and testing

Test Method			Contractor's Quality Control Testing*		Engineer's Verification Testing		Engineer's Quality Assurance Testing		Engineer's Pay Factor Testing	
Description	Number AASHTO	Number ASTM	Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency
Rotational viscometer (original asphalt)		D4402					Feed line**	1 per lot***		
Dynamic shear rheometer (original asphalt)	TP5						Feed line	1 per lot		
Dynamic shear rheometer (RTFO-aged)	TP5 T240						Feed line	1 per lot		
Dynamic shear rheometer (PAV-aged)	TP5 PP1						Feed line	1 per lot		
Bending beam rheometer (PAV-aged)	TP1 PP1						Feed line	1 per lot		
Direct tension (PAV-aged)	TP3 PP1						Feed line	1 per lot		
Flash point	T48	D92					Feed line	1 per lot		
Solubility	T44	D2042					Feed line	1 per lot		
Specific gravity	T228	D70								

*Meet requirements of PP26.
**Asphalt binder feed line between contractor's storage tank and plant mixing chamber.
***Five sublots per lot.

TABLE 29 Aggregate sampling and testing

Test Method			Contractor's Quality Control Testing		Engineer's Verification Testing		Engineer's Quality Assurance Testing		Engineer's Pay Factor Testing	
Description	Number AASHTO	Number ASTM	Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency
Coarse aggregate angularity		D5821	Combined cold feed	1 sample per sublot*			Combined cold feed	1 sample per lot		
Fine aggregate angularity	T304		Combined cold feed	1 sample per sublot			Combined cold feed	1 sample per lot		
Flat and elongated particles		D4791	Combined cold feed	1 sample per sublot			Combined cold feed	1 sample per lot		
Sand equivalent	T176	D2419	Combined cold feed	1 sample per sublot			Combined cold feed	1 sample per lot		
Los Angeles abrasion	T96	C131, C535	Combined cold feed	1 sample per lot			Combined cold feed	1 sample per 3 lots		
Soundness	T104	C88	Combined cold feed	1 sample per lot			Combined cold feed	1 sample per 3 lots		
Deleterious materials	T112	C142	Combined cold feed	1 sample per sublot			Combined cold feed	1 sample per lot		

*Five sublots per lot.

TABLE 30 Quality control, quality assurance, and pay factor tests for HMA production and placement

Test Method			Contractor's Quality Control Testing		Engineer's Verification Testing		Engineer's Quality Assurance Testing		Engineer's Pay Factor Testing	
Description	Number		Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency
	AASHTO	ASTM								
A. Gradation	T27	C136	Behind paver	1 per sublot				1 per sublot	Behind paver	1 per sublot
B. Asphalt binder content		D6307	Behind paver	1 per sublot			Behind paver	1 per sublot	Behind paver	1 per sublot
C. Gyrator-compacted sample properties at N_{design}	TP4		Behind paver	1 per sublot			Behind paver	1 per sublot		
1. Air voids (V_{air})	T269	D3203		1 per sublot				1 per sublot		
2. Voids in mineral aggregates (VMA)	PP28			1 per sublot						
3. Voids filled with asphalt (VFA)	PP28			1 per sublot						
4. Bulk specific gravity (Gmb)	T166	D2726		1 per sublot				1 per sublot		
5. Dust to binder ratio	PP28			1 per sublot						
6. Theoretical maximum specific gravity (Gmm)	T209	D2041		1 per sublot				1 per sublot		1 per sublot
D. In-place air voids	T269	D3203 D2950		2 per sublot			Cores from pavement	2 per sublot	Cores from pavement	2 per sublot

TABLE 31 Aggregate gradation determination requirements for quality control, quality assurance, and pay factor testing

Sieve Size		Contractor Quality Control Testing	Engineer's Quality Assurance Testing	Engineer's Pay Factor Testing
Metric, mm	U.S. Customary			
50	2.0-in.			
37.5	1.5-in.			
25	1-in.	X*		
19	3/4-in.	X		
12.5	1/2-in.	X		
9.5	3/8-in.			
4.75	No. 4			
2.36	No. 8	X		X
1.18	No. 16	X		
0.600	No. 30	X		
0.300	No. 50	X		
0.150	No. 100			
0.075	No. 200	X	X	X

*Use gradation control sieves for nominal maximum aggregate specified. Requirements for 19-mm (3/4-in.) nominal maximum size aggregate are shown in the table.

TABLE 32 Acceptable difference between contractor's and engineer's test results for asphalt binders

Test Method			
Designation	Number		Acceptable Difference*
	AASHTO	ASTM	
Dynamic shear rheometer on original asphalt, G*/sin δ	TP5		20 percent**
Dynamic shear rheometer on RTFOT-aged binder, G*/sin δ	TP5 TP240	D2872	20 percent**
Dynamic shear rheometer on PAV-aged binder, G* sin δ	TP5 PP1		20 percent**
Bending beam rheometer on PAV-aged binder, S-value	TP1 PP1		10 percent**
Bending beam rheometer on PAV-aged binder, *m*-value	TP1 PP1		0.015
Fracture strain on PAV-aged binder, fracture strain	TP3 PP1		30 percent

*Based on UDOT specification.
**Percent of average value of two test results.
RTFOT = rolling thin film oven test.

TABLE 33 Pay adjustment factor for asphalt binders*

Property	Compliance Limit for Price Adjustment of 1.00	Rejection Limit for Price Adjustment of 0.75
G*/sin δ of the original binder at high grade temperature, kPa	0.84 Min.	0.70 Min.
G*/sin δ of RTFO residue at high grade temperature, kPa	1.74 Min.	1.40 Min.
Stiffness of the PAV residue at low grade temperature + 10°C, MPa	311 Max.	355 Max.
Slope (*m*-value) of the creep curve at low grade temperature +10°C	0.294 Min.	0.265 Min.
Failure strain of PAV residue in direct tension at low grade temperature + 10°C[1]	1.04 Min.	0.78 Min.
[1]Use only for binders for which the test temperature of the low temperature properties is -18°C or colder.		

*Based on UDOT draft specification.

TABLE 34 Acceptable difference between contractor's and engineer's test results for aggregates

Test Method			Acceptable Difference*
Designation	Number		
	AASHTO	ASTM	
Coarse aggregate angularity		D5821	28
Fine aggregate angularity	T304		1
Flat and elongated particles		D4791	3**
Sand equivalent	T176	D2419	9
Los Angeles abrasion	T96	C131, C535	13
Soundness	T104	C88	70 percent***
Deleterious materials	T112	C142	1.7

*Represent multi-laboratory precision for AASHTO or ASTM test methods.
**Estimated.
***Magnesium sulfate, percent of average value of test results.

CHAPTER **11**

DEVELOPMENT OF PAY FACTOR RELATIONSHIPS FOR USE AT THE BASIC OPERATING LEVEL

11.1 INTRODUCTION

The *HMA Spec* software represents a powerful and convenient tool that agencies can use to generate an HMA pavement construction specification in which attention is focused on those M&C factors that have a significant effect on performance. The software represents the advanced PRS operating level. The methodology relates contractor payment to the level of quality delivered on one or more key performance-related M&C factors through the development and incorporation of a project-specific PF relationship into the specification. Despite its power and convenience, the *HMA Spec* software still relies on the engineer's judgment to ensure a valid specification. Thus, it is important for the user to possess an understanding of statistics as well as the ability to interpret the sensitivity and meaningfulness of the PF relationship. The only way for an agency to avoid the sophistication and level of understanding associated with the use of *HMA Spec* software is to develop a standard (but more general) matrix of PF relationships that covers the expected range of project characteristics, that is, HMA layer thickness, structural support conditions, traffic level, environmental conditions, and so on. Then, for any given project, the appropriate PF relationship would be selected and incorporated into the specification based on the characteristics of the project. This simplified approach constitutes the basic operating level of the PRS system and is similar to the approach used by New Jersey DOT (*10*).

This chapter provides an example of the development of a basic PF relationship derived through statistical analysis of the results of the Monte Carlo simulations performed by the *HMA Spec* software. Depending on its environment, traffic levels, and pavement structural capacities, an individual agency can develop a matrix of similar basic PF relationships to represent its own conditions.

11.2 DEVELOPMENT AND USE

As part of the process of generating a specification, the *HMA Spec* program performs a Monte Carlo simulation involving hundreds of iterations; this procedure simulates the variability of the selected AQCs to determine the mean LCC of the as-designed (as-specified) pavement. The program also uses data from these iterations to develop the preconstruction PF relationship that goes into the specification. Because the inputs for a given project are site-specific, the output PF relationship is considered to be limited to the specific project. However, if the user selects inputs to simulate a broader range of conditions, then the resultant PF relationship may be considered applicable to those conditions.

The example below is a basic PF relationship developed for the same conditions represented by one of the HMA mixes at the WesTrack project site, that is, coarse-graded mix, dry climate with freeze-thaw cycling in winter and hot days/cool nights in summer, moderate HMA thickness (150 mm or [6 in.]), strong subgrade soil and base support, and moderate traffic level (5 million ESAL applications during the first life-cycle). The design (or target) values for mean and standard deviation for each AQC used in the simulation are as follows:

AQC	Code	Units	Mean	Std. Deviation
HMA Thickness	TH	in.[1]	6	0.3
Asphalt Binder Content	P_{asp}	%	5.7	0.33
Air Void Content	V_{air}	%	8	1.5

[1] 1 in. = 25.4 mm

Development of the basic PF relationship shown in equation 61 used only the level 2 fatigue cracking model so that the effects of deviations in the AQCs could be emphasized without the compounding effects of rutting or any other performance model. It should be emphasized here, however, that the *HMA Spec* software allows development of PF relationships using a single or combination of pavement performance models, including fatigue cracking, rutting, and roughness. Under the advanced level of operation, this PF relationship would be incorporated directly into the preconstruction specification to allow the agency and contractor to determine the effects of deviations in the AQCs on the PF.

$$PF = 1.0118 + 0.11795 \times z_{TH} - 0.07173 \times z_{V_{air}} + 0.10982 \times z_{P_{asp}} - 0.00289 \times z_{TH} \times z_{P_{asp}} + 0.00274 \times z_{TH} \times z_{V_{air}} + 0.00208 \times z_{V_{air}} \times z_{P_{asp}} - 0.00247 \times z_{TH}^2 - 0.00043 \times z_{V_{air}}^2 - 0.00158 \times z_{P_{asp}}^2 \quad (61)$$

where

z_{TH} = normalized deviation of HMA thickness from its specification,
$z_{P_{asp}}$ = normalized deviation of asphalt binder content from its target specification, and
$z_{V_{air}}$ = normalized deviation of air void content from its target specification.

Recall that the z-value for any given AQC is calculated using the following basic equation:

$$z_{AQC} = (AQC_{con} - \overline{AQC}_{des})/\sigma_{des} \quad (62)$$

where

AQC_{con} = as-constructed value for a given AQC,
$\overline{AQC}$ = as-designed mean value for the AQC being considered, and
σ_{des} = standard deviation of the as-designed AQC distribution.

A note of caution should be expressed here regarding the potential for misuse of any given PF relationship. The use of the z-value may give the impression that PF is a true function of the "standardized" deviation of any AQC from its mean. This is *not* the case. The PF relationship is a function of the actual deviation of the AQC from its mean. Thus, any basic PF relationship developed using the approach described above will be limited in application to the range of target AQC means and standard deviations used in its development. Use outside that range can lead to error in PF assessment.

Because of the two-factor interactions in the example PF relationship, an accurate assessment of the effect of deviations in each AQC on the PF calls for a more statistically rigorous sensitivity analysis. However, it is possible to make some general observations:

1. The fact that the initial coefficient (PF axis intercept) is 1.0118 means that if the contractor were to achieve the target means and standard deviations of the project, a bonus of 1.18 percent of the bid price would be awarded. This is an anomaly associated with the combination of the Monte Carlo simulation and statistical analysis approaches used to develop the model. (The initial coefficient could just as easily have been 0.99 and resulted in a 1 percent penalty). The recommended solution to this is to assign an initial coefficient of 1.00.
2. Examination of the coefficients of the main effects in equation 61 suggests that deviation in HMA thickness likely has the largest effect on the PF, with asphalt content being a close second and air void content being a not-too-distant third. A sensitivity analysis of the complete model, including interactions, might show a rearrangement of the relative effects of these three AQCs on PF.
3. A positive coefficient indicates that positive deviations in thickness and air void content from their target value would increase the PF. In contrast, the negative coefficient for $z_{V_{air}}$ indicates that positive deviations in air void content would reduce the PF. These effects are all reasonable recognizing that the PF relationship was developed considering fatigue cracking only.

It should be emphasized that the PF relationship shown in equation 61 was developed by selecting inputs that simulated a coarse-graded mix under WesTrack conditions. At this point, it is unknown whether the relationship would be valid for any other mix, environment, structure, or traffic level. It should also be noted that, unlike the feature in the *HMA Spec* software, use of the simplified PF relationship precludes the direct consideration of uncertainty in PF determination, that is, the establishment of range in predicted LCC of the as-constructed pavement where no PA would be assessed.

Assuming that PF relationships are sensitive to other factors besides the AQCs, it would be necessary to develop a matrix of PF relationships to cover the range of possible conditions that might be encountered. These might include the following, for example:

- Five to ten mix types or gradations.
- Five to ten environments.
- Three HMA thickness levels.
- Three subgrade soil and subbase support levels.
- Three ESAL traffic levels.
- Three levels for failure criteria associated with fatigue cracking and rut depth.

Alternatively, simplified PF tables for these different conditions could be developed. Table 35 provides an example of a PF table (derived from equation 61 using a spreadsheet) that illustrates this approach. It should be emphasized that the values shown in the table are direct outputs of the PF relationship and therefore have not been restricted by maximum and minimum limits for the PFs. It must be further emphasized that the PFs apply only to those conditions for which they were developed and are therefore not applicable to other conditions.

TABLE 35 Example pay factor tale derived from the pay factor relationship in equation 61

Normalized Deviation in Thickness from the Target Value, z_{TH}	Normalized Deviation in Air Void Content from the Target, $z_{V_{air}}$								
	-1			0			1		
	Normalized Deviation in Asphalt Content from the Target, $z_{P_{asp}}$								
	-1	0	1	-1	0	1	-1	0	1
-1	0.85	0.96	1.07	0.78	0.89	1.00	0.70	0.82	0.93
0	0.97	1.08	1.19	0.90	1.01	1.12	0.83	0.94	1.05
1	1.09	1.20	1.30	1.02	1.13	1.23	0.95	1.06	1.16

CHAPTER **12**

CONCLUSIONS AND RECOMMENDATIONS

Part II of this report, documenting the research results of the WesTrack project, focuses on the development of a PRS for HMA pavement construction. The following conclusions and recommendations relate only to this key aspect of the study.

12.1 CONCLUSIONS

With the completion of the WesTrack study and the associated PRS system for HMA pavement construction, the following conclusions may be drawn:

1. Analysis of the laboratory and field data showed conclusively that asphalt content, air void content, and gradation each have a significant effect on the performance of HMA pavements.
   - For asphalt content and air void content, the effects are clear and quantifiable. Higher asphalt content and lower air void content translated into accelerated rutting and reduced fatigue cracking in all mixes. Lower asphalt contents and higher air void contents, on the other hand, produced mixes that experienced less permanent deformation, but more fatigue cracking.
   - From the gradation perspective, the two fine-graded mixes (those that have fines contents that go through or above the Superpave restricted zone) outperformed the coarse-graded mix. The fine mixes exhibited less distress (in terms of both fatigue cracking and permanent deformation) and were also less sensitive to deviations in asphalt content and air void content from their targets.
   - In terms of a particular gradation parameter, that is, percent passing the 0.075-mm (No. 200) sieve (i.e., P_{200}), the experiment showed that it had a small but significant effect on the performance of the fine-graded mixes. The experiment was not set up to measure its effect on the coarse-graded mixes.
2. Statistical analyses of the data showed that prediction models could be developed for the state-of-the-practice application level (level 1). These prediction models encompass both fatigue cracking and permanent deformation and took into consideration mix type, asphalt content, air void content, and aggregate gradation. The fatigue cracking model was adapted (using the results of past research) to also consider the effect of HMA thickness.
3. Statistical and mechanistic analyses of the data showed that prediction models could be developed for the state-of-the-art application level (level 2). These prediction models encompass both fatigue cracking and permanent deformation and took into consideration mix type, asphalt content, air void content and aggregate gradation. The fatigue cracking model was adapted to make it operational within the HMA PRS system.
4. With the PRS for PCC pavements developed under FHWA sponsorship serving as a template and the WesTrack pavement performance prediction models providing a rational analysis engine, an alpha version of a PRS system for HMA pavement construction was developed. The alpha version is a sophisticated Microsoft Windows-based software tool (called *HMA Spec*) that is designed to (1) produce a construction specification that focuses attention on those AQCs that have the greatest effect on pavement performance and (2) determine a rational contractor PA based on the quality of the as-constructed pavement.

12.2 RECOMMENDATIONS

Based on the results reported in this part of the study, the following recommendations are offered:

1. Evaluate the Performance Prediction Models. The pavement performance prediction models were developed very near the end of the 5-year research project. During the implementation of the PRS, the following evaluations should be conducted:
   - Verify that the models actually "fit" the data from which they were derived and confirm that the models produce practical, logical results.
   - Carry out sensitivity analyses to establish the relative influence of each AQC on the predicted deterioration of the pavement.
   - Validate the output of the models with field data and the output of other models that may be available in the literature. Re-calibrate the models as needed.
2. Evaluate Other Models. The statistical and mechanistic approaches used to develop the new models may not apply to all environments or M&C practices. Con-

sequently, other models in the literature or under development should be evaluated for different circumstances.

3. Enhance the *HMA Spec* Software. The first stage of development of *HMA Spec* is complete. The alpha version will require the following:
   - Beta testing. Other individuals or agencies need to test the software to identify software weaknesses or flaws that the developer may have missed.
   - Incorporation of User Cost Model. The PRS system should incorporate the ability to estimate user costs for both as-designed and as-constructed pavement.
   - Development of OC Curves. The PRS system should also have the capability to automatically generate OC curves which help the agency and contractor assess their individual risks.
4. Conduct Field Trials. The PRS system should be implemented on actual construction projects, both as a "shadow specification" and as a trial specification. As with the development of the PRS for PCC pavements, this step will assist adoption by state agencies.
5. Develop Additional Simplified Pay Factor Relationships. For the basic PRS operating level, more PF relationships are needed to complete the combinations of frost and moisture environments, traffic levels, and underlying pavement/soil support conditions that are likely to be encountered by state agencies.
6. Conduct More Laboratory and Accelerated Pavement Testing. Additional laboratory and accelerated pavement testing will identify effects of other asphalt binders (including modifiers), aggregate types, and gradations. These will enhance the wide applicability of the PRS.

ABBREVIATIONS

AAP	AASHTO Accreditation Program
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Concrete
ANOVA	Analysis of Variance
AQC	Acceptance Quality Characteristics
AQL	Acceptable Quality Levels
ASTM	American Society for Testing and Materials
AV	Air Void Content
BBR	Bending Beam Rheometer
BP	Bid Price
CBR	California Bearing Ratio
CF	Condition Factor
DSR	Dynamic Shear Rheometer
DTT	Direct Tension Test
ESAL	Equivalent Single-Axle Load
FA	Fine Aggregate
FC	Fatigue Cracking
FHWA	Federal Highway Administration
FMFC	Field-Mixed/Field-Compacted
FWD	Falling Weight Deflectometer
GLM	General Linear Model
GUI	Graphical User Interface
HMA	Hot-Mix Asphalt
HVS	Heavy Vehicle Simulator
IRI	International Roughness Index
JMF	Job Mix Formula
LMLC	Lab-Mixed/Lab-Compacted
LCC	Life-Cycle Cost
LCCA	Life-Cycle Cost Analysis
LTL	Lower Tolerance Limit
LTPP	Long-Term Pavement Performance
M&C	Materials and Construction
M-E	Mechanistic-Empirical
M&R	Maintenance and Rehabilitation
NAPCOM	Nationwide Pavement Cost Model
NCHRP	National Cooperative Highway Research Program
NCSU	North Carolina State University
NJDOT	New Jersey Department of Transportation
NPV	Net Present Value
OC	Operating Characteristic
PA	Pay Adjustment
PAV	Pressure Aging Vessel
PBS	Performance-Based Specification
PCC	Portland Cement Concrete
PD	Percent Defective
PF	Pay Factor
PFT	Pay Factor Table
PFR	Pay Factor Relationship
PGAB	Performance-Graded Asphalt Binder
PRS	Performance-Related Specification
PWL	Percent Within Limits
QA	Quality Assurance
QC	Quality Control
QC/QA	Quality Control/Quality Assurance
QCP	Quality Control Plan
RAP	Recycled Asphalt Pavement
RMSE	Root Mean Square Error
RQL	Rejectable Quality Level
RSST-CH	Repeated Simple Shear Test at Constant Height
RTFO	Rolling Thin Film Oven
RV	Rotational Viscometer
SF	Shift Factor
SHA	State Highway Agency
SHRP	Strategic Highway Research Program
SN	Structural Number
TCF	Temperature Conversion Factor
TEA	Transportation Efficiency Act
UTL	Upper Tolerance Limit
VFA	Voids Filled with Asphalt
VMA	Voids in Mineral Aggregate

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APPENDIX A

ADAPTATION AND CONVERSION OF PREDICTION MODELS

Chapter 4 of Part II describes the development of pavement performance prediction models using WesTrack data. Included in the description are models for two pavement distress types (fatigue cracking and permanent deformation) and two levels of PRS application: level 1 (state-of-the-practice) and level 2 (state-of-the-art). Although a rigorous analytical approach was used in their development, several of the models were not amenable for direct use within the PRS framework of the *HMA Spec* software. This appendix identifies the weaknesses or difficulties of each model in terms of their use within the software and describes the adaptation (or conversion) process used to overcome them.

A.1 LEVEL 1A FATIGUE CRACKING MODELS

Because many WesTrack sections exhibited little or no fatigue cracking, a probabilistic approach was employed in the development of the level 1A fatigue cracking models. Use of the probabilistic approach helps to minimize the bias associated with neglecting the test sections that "survived" the test. The PROBIT models (as they are also referred to) were developed for the two WesTrack mix gradations (fine and coarse) and for three distress levels (2, 5, and 10 percent fatigue cracking). Variability in the initial progression of fatigue cracking in the WesTrack sections led to concern about the viability of the PROBIT models for the two lower distress levels. Consequently, emphasis (as far as the adaptation of the models for use in the PRS) was put on the fatigue cracking models for the highest distress level, that is, 10 percent fatigue cracking. Following are the two models selected for further development as PRS level 1A fatigue cracking models. Both of these equations are designed to predict the probability of the occurrence of 10 percent or greater fatigue cracking, that is, Prob (>10 percent FC), and, therefore, require the use of the cumulative density function (Φ). Since these equations do not attempt to account for seasonal temperature fluctuations or mix stiffness, they are considered site-specific.

PROBIT Model for Fine-Graded Mixes

$$\text{Prob}(>10\%\ \text{FC}) = \Phi[-49.502 + 4.788 \cdot \ln(ESAL) - 5.245 \cdot P_{asp} + 1.148 \cdot V_{air} - 2.301 \cdot P_{200}] \tag{A.1}$$

PROBIT Model for Coarse-Graded Mixes

$$\text{Prob}(>10\%\ \text{FC}) = \Phi[-47.151 + 5.293 \cdot \ln(ESAL) - 5.996 \cdot P_{asp} + 0.450 \cdot V_{air}] \tag{A.2}$$

The independent variables in equations A.1 and A.2 are as follows:

ESAL = cumulative 80-kN (18-kip) ESAL applications.
P_{asp} = asphalt content in mix (percent).
V_{air} = air void content in mix (percent).
P_{200} = percent of aggregate passing the 0.075-mm (No. 200) sieve.

The ln(*ESAL*) term in both equations refers to the natural log of ESAL. Recall that ESALs for WesTrack were based on the conversion of the eight-axle WesTrack loading vehicle to ESAL applications using AASHTO load equivalence factors. This conversion is not technically accurate because of the less-than-average wander (lateral distribution) of the vehicles as well as the higher tire pressures.

The P_{200} term does not appear in the model for coarse-graded mixes because (unlike the fine-graded mix) none of the WesTrack coarse-graded mix sections were constructed with a fines content that was different than the target. In other words, the WesTrack experiment was not designed to measure the effect. Accordingly, the user should not interpret the lack of a P_{200} term in the coarse-graded mix model as an indication that a difference in fines content (from the target) will have no effect on pavement performance.

Because of (1) their probabilistic nature, that is, they predict the probability of a certain level of fatigue cracking, (2) the fact that they do not account for the effects of HMA thickness and stiffness, and (3) the fact that they are geared to only one level of fatigue cracking, the equations were not considered amenable to direct incorporation into the *HMA Spec* software. Accordingly, three adaptations were made to make the equations deterministic, incorporate an HMA tensile strain (layer thickness) effect, and provide a means for the prediction of fatigue cracking on a continuous basis.

A.1.1 First Adaptation

The first adaptation was to convert equations A.1 and A.2 into new equations that predict the cumulative 80-kN (18-kip) ESAL applications to 10 percent fatigue cracking as a function of the mix characteristics. This was accomplished by solving the equations for the number of ESAL applications required to achieve the 50 percent probability level of 10 percent fatigue cracking for a factorial of mix characteristics. Tables A.1 and A.2 show the data and results of this process for the fine and coarse mixes, respectively.

Column 2 indicates that the model is for the 10 percent fatigue cracking (FC) level. Columns 3 through 5 show the combinations of asphalt content (P_{asp}), air void content (V_{air}) and percent aggregate passing the 0.075-mm (No. 200) sieve (P_{200}) used in the factorial. Column 6 shows the level of ESAL applications required to produce a 50 percent probability of 10 percent fatigue cracking, while column 7 represents the log (base 10) of column 6. The data in columns 3 through 7 provide the basis for simple regression analyses that resulted in the following models:

50 Percent Probability Model for Fine-Graded Mixes

$$\log_{10}(\text{ESAL}) = 4.490 + 0.4757 \cdot P_{asp} - 0.1041 \cdot V_{air} + 0.2087 \cdot P_{200} \quad (\text{A.3})$$

50 Percent Probability Model for Coarse-Graded Mixes

$$\log_{10}(\text{ESAL}) = 3.8686 + 0.4920 \cdot P_{asp} - 0.03692 \cdot V_{air} \quad (\text{A.4})$$

All variables are as previously defined and, again, the P_{200} term does not appear in the model for coarse-graded mixes because the effect of increased fines in coarse mixes was not included in the WesTrack experiment. Both equations are still site-specific to WesTrack.

The nature of the original PROBIT models made it easy to develop regression models with excellent fit. In fact, the coefficient of determination (r^2) for equations A.3 and A.4 was essentially 1.0 in both cases. The standard error of estimate [on $\log_{10}$(ESAL)] for each model was also very good (0.0000544 and 0.0000629, respectively). Columns 9, 10, and 11 of Tables A.1 and A.2 provide an indication of how well the modified models "fit" the factorial data. Column 9 may be compared with column 7 and, likewise, column 11 may be compared with column 6. Column 10 shows the residual (error) for each individual treatment combination.

A.1.2 Second Adaptation

The second adaptation transformed the models so that they could account for the effects of HMA layer thickness and stiffness on fatigue cracking. This adaptation required several steps and was accomplished with the aid of data from the original 26 WesTrack sections and the NCHRP Project 1-10B fatigue cracking models (*A.1*). Table A.3 is used to help describe the step-by-step process.

Step 1: Estimate Dynamic Modulus for WesTrack Sections. One of the independent variables in the NCHRP 1-10B fatigue relationship(s) is the dynamic (or complex) modulus of the HMA layer. It was incorporated into the model to account for the fact that stiff mixes (despite having lower load-related tensile strains) tend to fatigue faster than soft mixes. To help provide for portability of the final WesTrack level 1 fatigue models to other environments, the relative effect of dynamic modulus from the NCHRP Project 1-10B relationship was selected for inclusion in the adaptation process. Although resilient modulus testing was performed on all layers of the WesTrack structure, testing for dynamic modulus was not part of the WesTrack laboratory test plan. Therefore, it was estimated using a predictive relationship derived by Fonseca and Witczak (*A.2*) modified slightly for consistency:

$$\begin{aligned} E^* = {} & -0.261 + 0.008225 \cdot P_{200} - 0.00000101 \\ & \cdot (P_{200})^2 + 0.00196 \cdot P_4 - 0.03157 \cdot V_{air} - 0.415 \\ & \cdot V_{beff}/(V_{beff} + V_{air}) + [1.87 + 0.002808 \cdot P_4 \\ & + 0.0000404 \cdot P_{38} - 0.0001786 \cdot (P_{38})^2 \\ & + 0.0164 \cdot P_{34}]/[1 + e^{(-0.716 \cdot \ln f - 0.7425 \cdot \ln \eta)}] \end{aligned} \quad (\text{A.5})$$

where

- E^* = asphalt mix dynamic modulus (10^5 psi) (1 psi = 6.9 kPa),
- η = bitumen viscosity (poise, at any temperature and degree of aging),
- f = load frequency (Hz),
- V_{air} = air void content in mix (percent by volume),
- V_{beff} = effective bitumen content (percent by volume),
- P_{34} = percent retained on the 19-mm (3/4 in.) sieve, by total aggregate weight (cumulative),
- P_{38} = percent retained on the 9.5-mm (3/8 in.) sieve, by total aggregate weight (cumulative),
- P_4 = percent retained on the 4.75-mm (No. 4) sieve, by total aggregate weight (cumulative), and
- P_{200} = percent passing the 0.075-mm (No. 200) sieve, by total aggregate weight.

Columns 3 through 12 show the data from the original 26 WesTrack sections used to estimate the initial dynamic modulus for each mix (column 13) at a standard test temperature of 20°C (68°F). A more refined model would take into consideration the variability of the dynamic modulus with time and temperature during loading (to provide better portability to other environments). However, that approach was beyond the scope of this project.

Step 2: Calculate Maximum Tensile Strain for WesTrack Sections. The maximum critical tensile strain for each of the 26 original WesTrack sections was calculated using the U.S. Army Corps of Engineers' WESLEA elastic layer program, the known HMA layer thicknesses (column 14), the known subsurface layer structure, and estimated modulus values for each layer. These strains are shown in column 15.

For incorporation into the PRS software, an extensive statistical analysis was performed on a factorial of results from the WESLEA program in an effort to develop a tensile strain (G) equation. The factorial used is illustrated in

Table A.4 and the tensile strain equation is included as equation A.6.

$$\log_{10}(\varepsilon) = +2.979343598 + 2.455798724 \cdot \log(TH1) - 0.488054165 \cdot \log(TH2) - 0.475116528 \cdot \log(TH3) - 0.502900343 \cdot \log(E4) - 1.049789136 \cdot \log(E3) - 0.721885714 \cdot E23 - 0.119641587 \cdot \log(E1) - 0.737169201 \cdot \log(TH1)^2 + 0.088049851 \cdot \log(TH2) \cdot \log(TH3) + 0.115648268 \cdot \log(TH2) \cdot \log(E4) + 0.094517328 \cdot \log(TH3) \cdot \log(E4) + 0.059291906 \cdot \log(E4) \cdot \log(E3) - 0.238641065 \cdot \log(E3)^2 + 0.128477256 \cdot \log(TH1) \cdot E23 - 0.078371929 \cdot \log(TH2) \cdot E23 + 0.007672458 \cdot \log(E4) \cdot E23 + 0.010962106 \cdot E23^2 - 0.445345749 \cdot \log(TH1) \cdot \log(E1) + 0.415729220 \cdot \log(E3) \cdot \log(E1) + 0.095260104 \cdot E23 \cdot \log(E1) - 0.173593980 \cdot \log(E1)^2 \quad (A.6)$$

In this equation, T1, T2, and T3 represent the thicknesses (in inches) of the HMA surface, aggregate base, and subbase courses, respectively; E1, E3, and E4 represent the elastic moduli (in psi) of the HMA surface, subbase, and natural subgrade soil, respectively; and E23 represents the ratio of the elastic modulus of the aggregate base course to that of the subbase course.

This model has an r^2 of 0.9992 and a standard error of estimate of 0.005908 on the base 10 logarithm of the tensile strain.

Step 3: Determine the (NCHRP 1-10B) Fatigue Life. The estimated fatigue life (in ESAL applications) according to the NCHRP 1-10B study (*A.1*) was calculated using equation A.7. This equation requires the critical tensile strain in the HMA surface layer and the HMA dynamic modulus as inputs. The results are presented in column 16 of Table A.3.

$$\log_{10}(N_f) = A_0 - 3.291 \cdot \log_{10}(\varepsilon/10^{-6}) - 0.854 \cdot \log_{10}(E^*/1000) \quad (A.7)$$

where

- N_f = allowable cumulative load applications to a given level of fatigue cracking;
- ε = critical tensile strain at the bottom of the HMA surface layer, in./in.;
- E^* = complex (or dynamic) modulus of the HMA surface layer (psi); and
- A_0 = regression constant depending on the fatigue cracking level:
  15.947 for 10 percent fatigue cracking (in the wheelpaths)
  16.086 for 45 percent fatigue cracking (in the wheelpaths).

Step 4: Use 50 Percent Probability Models to Estimate Fatigue Life of WesTrack Sections. Since not all of the WesTrack sections exhibited fatigue cracking at the time the test was completed, the 50 percent probability models represented by equations A.3 and A.4 provide a statistically sound basis for estimating what their fatigue lives would have been. The results of these calculations are shown in column 17.

The NCHRP 1-10B relationship was derived using data from AASHO Road Test sections that were optimally constructed. Consequently, the only valid comparisons between the ESAL values in columns 17 and 16 are the optimally-constructed WesTrack sections, that is, fine-mix replicate sections 1 and 15 and coarse-mix replicate sections 5 and 24. Overall, these comparisons indicate the following:

- The 50 percent probability model for coarse mixes predicts slightly higher ESAL values than the NCHRP 1-10B model.
- The 50 percent probability model for fine mixes predicts ESAL values that are about an order of magnitude greater than the NCHRP 1-10B model.
- A reasonable basis for equation adaptation exists, although the model for fine mixes seems to produce overly high estimates of fatigue life.

Step 5: Apply Shift Factor(s) to Existing Models. Recognizing that all WesTrack sections have a fixed HMA surface thickness (and, therefore, little variation in tensile strain), the best approach to adapting the 50 percent probability equations was to "shift" the NCHRP Project 1-10B equations (and their accompanying tensile strain and dynamic modulus effects) to the location of the 50 percent probability models (and their corresponding air void content, asphalt content, and gradation effects). This was accomplished by determining a new A_0-intercept for the NCHRP Project 1-10B equation that shifts it to the performance of the four key WesTrack mixes. Column 19 shows the four A_0 coefficients required to make the conversions for just the sections with the optimal mixes. The final A_0 coefficient for the fine mix is represented by the average of the values for sections 1 and 15. Similarly, the final A_0 coefficient for the coarse mix is represented by the average of the values for sections 5 and 24. The end-products of this step are equations A.8 and A.9. Again, these are for a level of fatigue cracking equal to 10 percent. Column 18 shows the effect of the shift from NCHRP Project 1-10B (AASHO Road Test) to WesTrack performance.

Composite Model for Fine-Graded Mixes (10 Percent Fatigue Cracking Level)

$$\log_{10}(ESAL) = 14.166 - 3.291 \cdot \log_{10}(\varepsilon/10^{-6}) - 0.854 \cdot \log_{10}(E^*/1000) + 0.4757 \cdot P_{asp} - 0.1041 \cdot V_{air} + 0.2087 \cdot P_{200} \quad (A.8)$$

Composite Model for Coarse-Graded Mixes (10 Percent Fatigue Cracking Level)

$$\log_{10}(ESAL) = 13.583 - 3.291 \cdot \log_{10}(\varepsilon/10^{-6}) - 0.854 \cdot \log_{10}(E^*/1000) + 0.4920 \cdot P_{asp} - 0.03692 \cdot V_{air} \quad (A.9)$$

For these equations, all the variables are as previously defined.

A.1.3 Third Adaptation

The third and final adaptation was to convert equations A.8 and A.9 into formulas that could be used to predict the extent of fatigue cracking (in the wheelpaths) as a function of all the key independent variables, including cumulative axle load applications. Average A_0 values for the 45 percent fatigue cracking level were determined in the same manner as step 5 above (14.305 for the fine-graded mixes and 13.722 for the coarse-graded mixes). The end-results were the following equations for fine- and coarse-graded mixes, respectively.

$$\log_{10}(A_0) = 1.1448 + 0.006492 \cdot \log_{10}(FC) \quad (A.10)$$

$$\log_{10}(A_0) = 1.1262 + 0.006769 \cdot \log_{10}(FC) \quad (A.11)$$

Substituting these relationships for the A_0-terms in equations A.8 and A.9 and then rearranging the equation to solve for fatigue cracking level results in the following two recommended level 1 models.

Composite Model for Fine-Graded Mixes

$$FC\ (\%) = [1.2313 + 0.071655 \cdot \log_{10}(ESAL) + 0.2358 \cdot \log_{10}(\varepsilon) + 0.061193 \cdot \log_{10}(E^*) - 0.034086 \cdot P_{asp} + 0.0074593 \cdot V_{air} - 0.014954 \cdot P_{200}]^{154.04} \quad (A.12)$$

Composite Model for Coarse-Graded Mixes

$$FC\ (\%) = [1.2850 + 0.07478 \cdot \log_{10}(ESAL) + 0.2461 \cdot \log_{10}(\varepsilon) + 0.06386 \cdot \log_{10}(E^*) - 0.036791 \cdot P_{asp} + 0.002761 \cdot V_{air}]^{147.73} \quad (A.13)$$

In both of these equations, FC (%) represents the percent of the wheelpath area exhibiting fatigue cracking and all other variables are as previously defined.

Figure A.1 illustrates the predictive output of the two equations for the two primary WesTrack mixes. The example shown is for a pavement having a 150-mm (6-in.) HMA surface layer thickness, a bitumen stiffness of 1,929 poise (4.03 lb-sec/ft^2), a load frequency of 10 Hz, the WesTrack subsurface structure, and other needed HMA mix properties provided in Table A.5.

A.2 LEVEL 2 FATIGUE CRACKING MODELS

Included within Chapter 4 of Part II is a discussion of the development of the WesTrack level 2 fatigue cracking models. Unlike the WesTrack level 1A fatigue cracking models, which were derived from a probabilistic analysis of fatigue cracking observed in the field, the level 2 models were derived through an M-E analysis of fatigue test results on WesTrack mix samples gathered in the field and tested in the lab. The primary reason for using the laboratory fatigue results was the fact that many of the field test sections exhibited little or no fatigue cracking. The problem with models developed based on laboratory test results is that they still need to be calibrated to match field performance.

The major advantage of the level 2 models is that they incorporate the use of state-of-the-art testing and modeling approaches to better simulate loading conditions, environmental effects and materials behavior, and more accurately predict performance. The level 2 fatigue cracking models consider pavement response to wheel load and temperature; however, during their development, consideration was also given to the lateral distribution of wheel loads and cyclic changes in temperature. The *HMA Spec* software was not intended to accommodate this level of sophistication. Consequently, simplifying assumptions were made that resulted in models that were easier to incorporate.

A separate laboratory-based fatigue cracking model was developed for all three WesTrack mix types:

Level 2 Model for Fine-Graded Mixes

$$\ln(N_f) = -27.0265 - 0.1439 \cdot V_{air} + 0.4148 \cdot P_{asp} - 4.6894 \cdot \ln(\varepsilon_t) \quad (A.14)$$

Level 2 Model for Fine-Plus-Graded Mixes

$$\ln(N_f) = -27.3409 - 0.1431 \cdot V_{air} + 0.4219 \cdot P_{asp} + 0.0128 \cdot \ln(T) - 4.6918 \cdot \ln(\varepsilon_t) \quad (A.15)$$

Level 2 Model for Coarse-Graded Mixes

$$\ln(N_f) = -27.0723 - 0.0941 \cdot V_{air} + 0.6540 \cdot P_{asp} + 0.0331 \cdot T - 4.5402 \cdot \ln(\varepsilon_t) \quad (A.16)$$

Definitions for each of the terms in these models are as follows:

N_f = laboratory fatigue life, load cycles to failure,
V_{air} = air void content (percent),
P_{asp} = asphalt content (percent),
T = HMA temperature at 150 mm (6 in.) depth (°C) (°F = 1.8°C + 32), and
ε_t = maximum HMA tensile strain (in./in.).

Summarizing the process described in Chapter 4 of Part II, pavement damage is computed hourly based on the estimated hourly load applications (n_i) and the estimated fatigue life (from the appropriate level 2 equation for N_f), that is, damage $= n_i/(N_f)_i$. Because traffic and pavement temperature and support conditions change with time, the hourly damage can be different from hour to hour, day to day, and season to season, hence the reason for more rigorous level 2 models. As the damage is computed for each hour, it, along with load applications, is accumulated. Once the cumulative damage reaches 1.0, the corresponding cumulative load applications represent laboratory fatigue life (N_f). This value is then translated into an estimate of the pavement fatigue life (ESAL) using the following relationship:

$$ESAL_{\text{in situ}} = (N \cdot SF)/TCF \tag{A.17}$$

where SF represents the shift factor and TCF represents the temperature conversion factor. Chapter 4 provides general criteria for SF ranging from 2 to 3 for coarse mixes and 10 to 20 for fine mixes. It also provides the following relationship for estimating TCF:

$$TCF = a \cdot \ln(t_{AC}) + b \tag{A.18}$$

where

t_{AC} = HMA layer thickness (cm) (1 in. = 2.54 cm) and
a, b = coefficients depending on climatic region (see Chapter 4 of Part II).

To simplify the model for use in the PRS, it is recommended that the iterative process associated with computing and accumulating the hourly damage be replaced by a process involving the estimation of an effective year-round pavement temperature (T) and an effective year-round HMA tensile strain (ε_t) for use in the above equations.

Table A.6 summarizes the results of the analysis of WesTrack data involving the application of a simplified process to estimate shift factors (SF values) for the different WesTrack mixes. The steps corresponding to the development of the data in each column of Table A.6 are described as follows. Note that one of the steps demonstrates an approach to predicting the progression of fatigue cracking.

1. Columns 1 through 8 represent the characteristics of all the original 26 WesTrack sections. These columns should be self-explanatory.
2. Column 9 (average pavement temperature) represents the average pavement surface temperature for each section up to the time it had developed 10 percent fatigue cracking. If the section failed earlier in rutting, the average pavement temperature is the average up to that time. If the section survived the test, the average pavement temperature is the best estimate of the mean pavement temperature up to the time the pavement was expected to fail. These average pavement temperatures were calculated using temperature data from the WesTrack database.
3. Column 10 represents the HMA stiffness (in MPa) as a function of P_{asp}, V_{air}, and the average pavement surface temperature. The WesTrack HMA stiffness model described in Chapter 4 of Part II was used to calculate this value. Column 11 represents the conversion of the HMA stiffness from SI to U.S. customary units (MPa to psi) for each pavement section.
4. Column 12 represents the as-constructed HMA surface thickness for each WesTrack pavement section.
5. Column 13 represents the maximum HMA tensile strain calculated using an early two-layer strain model. (The data had no effect on the analysis).
6. Column 14 represents the maximum HMA tensile strain calculated using the four-layer strain model. Resilient modulus values for the underlying WesTrack layers were selected based on the values used to develop the level 2 fatigue model. This included fixed modulus values of 172 MPa (25,000 psi) for the 305 mm (12-in.) base course and 110 MPa (16,000 psi) for the engineered fill and subgrade soil. (The 110 MPa for the engineered fill/subgrade soil represents an estimate of the effective year-round resilient modulus for those layers).
7. Column 15 represents the predicted load applications (N_f) from the NCHRP Project 1-10B fatigue equation for 10 percent cracking. It is included for purposes of comparison and has no effect on the analysis.
8. Column 16 represents the predicted load applications (N_f) for 10 percent cracking from the three level 2 laboratory-based fatigue models identified in Chapter 4 of Part II. The three models are for the fine, fine plus and coarse mixes.
9. Columns 17 represents the TCF calculated for each WesTrack section. It is based on the "high-desert" coefficients provided in Chapter 4 of Part II, that is, a = 2.102 and b = −3.884, as well as the actual HMA layer thickness.
10. Column 18 represents a visual estimate of the ESAL applications ($ESAL_{\text{in situ}}$) required to achieve 10 percent fatigue cracking in most of the WesTrack pavement sections. The estimates for sections denoted by a star (*) are based on an extrapolation beyond the range of axle applications at WesTrack.
11. Column 19 represents the effective shift factor required to adjust the laboratory-based load applications to the performance observed at WesTrack (column 16). The effective shift factor was calculated using the following formula:

$$SF = ESAL_{\text{in situ}} \cdot TCF/N_f \tag{A.19}$$

12. Based on the effective shift factors calculated in column 19, the shift factors (required to match WesTrack performance) are provided in the table below.

Because WesTrack performance is representative of channelized traffic, these shift factors would be higher for typical highway pavements.

Mix Type	Range in Shift Factor	Average Shift Factor
Fine or Fine Plus	0.7 to 5	3.0
Coarse	0.3 to 0.8	0.5

13. Application of the WesTrack PRS software requires a fatigue model capable of predicting the progression of fatigue cracking with increasing ESAL applications. Thus, the following model was derived to satisfy this requirement:

$$\ln(\mathrm{FC}) = A_1 \cdot [\, \ln(ESAL_{\mathrm{t}}) - \ln(ESAL_{\mathrm{in\ situ}})\,] + \ln(10) \qquad \text{(A.20)}$$

where

FC = fatigue cracking (percent of wheelpath area),
A_1 = regression coefficient that accounts for the rate of fatigue crack progression,
$ESAL_{\mathrm{t}}$ = cumulative ESAL applications at some time, t, during the life of the pavement,
$ESAL_{\mathrm{in\ situ}}$ = predicted fatigue life (to 10 percent fatigue cracking) of the in situ pavement in ESAL applications, and calculated from equation A.19 as $N_f \cdot SF/TCF$.

Analysis of the cracking progression rates in WesTrack sections (six fine mixes and five coarse mixes) provides the following criteria for selection of A_1:

Mix Type	Range in A_1	Suggested A_1
Fine or Fine Plus	2 to 8	5
Coarse	4 to 56	25

It should be noted that high A_1 values result in accelerated crack progression rates. For example, a value of 25 means that a section can go from zero to 100 percent fatigue cracking in less than 100,000 ESAL applications.

A.3 LEVEL 2 PERMANENT DEFORMATION MODEL

As indicated in Chapter 4 of Part II, the level 2 permanent deformation (rutting) model was developed using hourly traffic estimates and varying layer moduli for the HMA layer and the subgrade layer. This allowed the principle of time hardening to be used in the analyses. Use of the level 2 rutting model in the *HMA Spec* software differs from the way the model was developed in three important ways: (1) traffic is estimated on an annual basis, (2) average moduli are used for the pavement layers, and (3) the principle of time hardening is not included. These simplifications were adopted to maintain consistency with the way the level 1 models and the level 2 fatigue cracking models are used in the software.

The level 2 rutting model used in the *HMA Spec* software is shown in equation A.21. As indicated, it includes both rutting in the HMA layer ($\mathrm{RD_{AC}}$) and rutting in the roadbed soil ($\mathrm{RD_{SG}}$).

$$\mathrm{RD_{Total}} = \mathrm{RD_{AC}} + \mathrm{RD_{SG}} \qquad \text{(A.21)}$$

where

$\mathrm{RD_{Total}}$ = total estimated rut depth, mm (in.),
$\mathrm{RD_{AC}}$ = estimated rut depth in the HMA layer, mm (in.), and
$\mathrm{RD_{SG}}$ = estimated rut depth in the roadbed soil (subgrade), mm (in.).

A.3.1 Permanent Deformation in the HMA Layer

The estimated rut depth in the HMA layer ($\mathrm{RD_{AC}}$) is determined from the relationship shown in equation A.22.

$$\mathrm{RD_{AC}} = \mathrm{K} \cdot \gamma^i \qquad \text{(A.22)}$$

where

$\mathrm{RD_{AC}}$ = estimated rut depth in the HMA layer, mm (in.),
K = a factor to account for HMA thickness, and
γ^i = permanent (inelastic) shear strain at a depth of 50 mm (2 in.) in the HMA layer.

Suggested values for K as shown in the following table (*A.3*).

Thickness of HMA layer, in.[1]	K
≥12	10
9–12	8.5
7–9	7.0
5–7	5.5

[1] 1 in. = 25.4 mm

The inelastic shear strain (γ^i) is determined from equation A.23.

$$\gamma^i = a \cdot \exp(b\tau) \cdot \gamma^e \cdot \mathrm{ESAL}^c \qquad \text{(A.23)}$$

where

γ^i = permanent (inelastic) shear strain at a depth of 50 mm (2 in.) in the HMA layer,

τ = shear stress at a depth of 50 mm (2 in.) in the HMA layer, psi or MPa,
γ^e = elastic shear strain at a depth of 50 mm (2 in.) in the HMA layer,
ESAL = number of axle load repetitions, and
a, b, c = regression coefficients.

The shear stress (τ) and elastic shear strain (γ^e) are determined from equations A.24 and A.25, respectively. These are regression equations developed from layered elastic analyses using the CIRCLY layered elastic analysis software. Figure A.2 indicates the axle load and configuration used in these analyses whereas Figure A.3 indicates the pavement structure and experiment design.

$$\tau = \exp(-0.53883 + 0.036073 \cdot \ln(\gamma^e) - 0.00509 \cdot \mathrm{TH}_1 + 0.0000632 \cdot E_1 - 0.00072 \cdot E_2) \quad \text{(A.24)}$$

$$\gamma^e = \exp(-7.52845 - 0.00528 \cdot \mathrm{TH}_1 - 0.00022 \cdot E_1 - 0.00075 \cdot E_2) \quad \text{(A.25)}$$

where

τ = shear stress at a depth of 50 mm (2 in.) in the HMA layer, MPa,
γ^e = elastic shear strain at a depth of 50 mm (2 in.) in the HMA layer,
TH_1 = thickness of HMA layer, mm (1 in. = 25.4 mm),
E_1 = elastic modulus of HMA layer, MPa, and
E_2 = elastic modulus of the base course layer, MPa.

Values for E_1 and E_2 are user input values and are considered constant throughout the analysis period within *HMA Spec*. The value for TH_1 is varied in the Monte Carlo simulation process. These simplifications result in constant values for τ and γ^e for a given iteration in the process.

Chapter 4 of Part II indicates that a value for b of 10.28 for SI units (0.071 for in.-lb units) was used in the analyses during model development. This value was adopted for use in the software.

The values for a and c in equation A.23 are determined from equations A.26 and A.27, respectively. Eliminating the indicator variables for gradation and substituting equation A.27 into equation A.26 gives equations A.28 and A.29 for coarse-graded mixtures and equations A.30 and A.31 for fine-graded mixtures.

$$\ln(\mathit{field\ a}) = -10.0792 - 0.788273 \cdot P_{asp} + 0.0846995 \cdot V_{air} - 0.358081 \cdot \mathit{fine} + 0.225354 \cdot \mathit{coarse} - 4.52386 \cdot \ln(\mathit{field\ c}) \quad \text{(A.26)}$$

$$\ln(\mathit{field\ c}) = -7.5834 + 1.051941 \cdot P_{asp} + 0.95641 \cdot \mathit{fine\ plus} + 0.66471 \cdot \mathit{coarse} \quad \text{(A.27)}$$

where

field a = calculated coefficient accounting for field conditions,
field c = calculated coefficient accounting for field conditions,
P_{asp} = percent asphalt content by weight,
V_{air} = percent air void content,
fine = indicator variable for fine mixtures (=1 for fine, =0 for other mixture types),
fine plus = indicator variable for fine plus mixtures (=1 for fine plus, =0 for other mixture types), and
coarse = indicator variable for coarse mixtures (=1 for coarse, =0 for other mixture types).

Coarse-graded mixtures

$$\mathit{field\ a} = \exp[21.44552 - 5.547107 \cdot P_{asp} + 0.0846995 \cdot V_{air}] \quad \text{(A.28)}$$

$$\mathit{field\ c} = \exp[-6.91873 + 1.051941 \cdot P_{asp}] \quad \text{(A.29)}$$

Fine-graded mixtures

$$\mathit{field\ a} = \exp[23.86914 - 5.547107 \cdot P_{asp} + 0.0846995 \cdot V_{air}] \quad \text{(A.30)}$$

$$\mathit{field\ c} = \exp[-7.58344 + 1.051941 \cdot P_{asp}] \quad \text{(A.31)}$$

where

field a = calculated coefficient accounting for field conditions,
field c = calculated coefficient accounting for field conditions,
P_{asp} = percent asphalt content by weight, and
V_{air} = percent air void content.

The value for axle load repetitions (n) is calculated based on user input of initial ESALs (or initial average daily traffic (ADT) with percent trucks and an ADT-to-ESAL conversion factor), traffic growth rate, and growth type (see equations 56 and 57 in Chapter 6 of Part II). These are calculated on an annual basis. In developing the model, traffic (ESAL) was calculated on an hourly basis and the time-hardening principle was used to estimate accumulation of permanent strain (see Chapter 4 of Part II). *HMA Spec* does not incorporate the time-hardening principle.

A.3.2 Permanent Deformation in the Roadbed Soil

The contribution of subgrade rutting ($\mathrm{RD_{SG}}$) in equation A.21 is determined using equations 25 and 26 (see Chapter 4 of Part II). However, as with $\mathrm{RD_{AC}}$, the principle of time hardening is not incorporated in *HMA Spec*. The vertical strain at the surface of the roadbed soil is determined using equation A.32. This equation was developed during the analyses used to develop equations A.24 and A.25 above.

$$\varepsilon_v = \exp(0.175553 + 0.734386 \cdot \ln(\gamma^e) + 0.0000913 \cdot E_1 - 0.00767 \cdot (E_2/E_3) - 0.01088 \cdot E_{SG} - 0.0018 \cdot TH_{Total}) \quad (A.32)$$

where

ε_v = vertical compressive strain at the top of the subgrade;
γ^e = elastic shear strain at a depth of 50 mm (2 in.) in the HMA layer;
E_1 = elastic modulus of the HMA layer, MPa (1 ksi = 6.9 MPa);
E_2 = elastic modulus of the base layer, MPa;
E_3 = elastic modulus of the subbase layer, MPa;
E_{SG} = elastic modulus of the subgrade soil, MPa; and
TH_{Total} = total thickness of the pavement structure, ($TH_1 + TH_2 + TH_3$) mm (1 in. = 25.4 mm).

A.3.3 Summary

HMA Spec uses traffic calculated on an annual basis and average values for the moduli of the pavement layers to estimate total rut depth. This is a departure from the way the models were developed but results in a reasonable approximation of rut depth accumulation. Figure A.4 indicates the difference, for Section 24 (original coarse-graded mixture), between calculation of total rut depth based on hourly traffic, varying moduli, and time hardening and that based on the methodology incorporated in *HMA Spec*. It must be emphasized here that the values for *a* and *c* were obtained from Table 12 and not from equations A.28 and A.29 above. As indicated, the methodology using hourly traffic, varying moduli, and time hardening better estimates early rutting, but, for practical purposes, the two methods are essentially the same.

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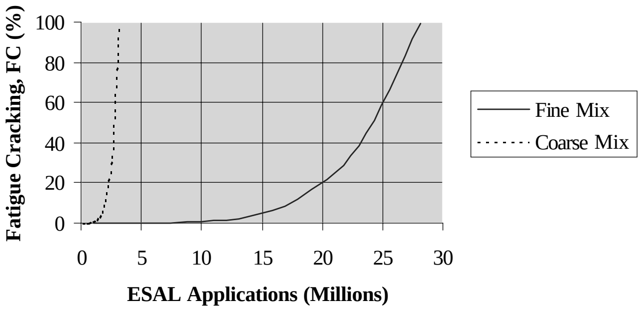


Figure A.1. Illustration of the WesTrack level 1 (alternative 3) composite models for the prediction of fatigue cracking.

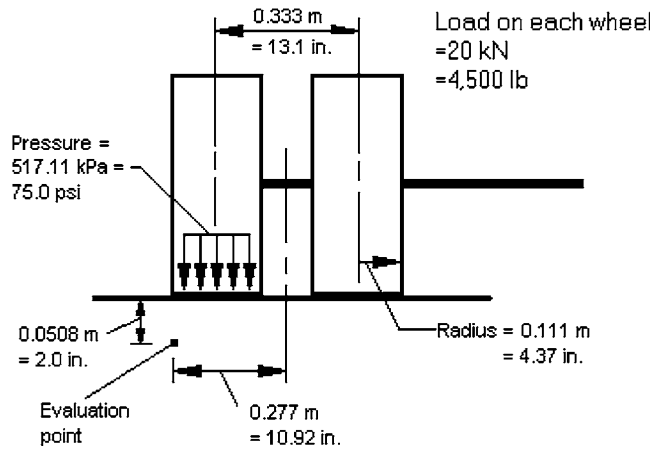
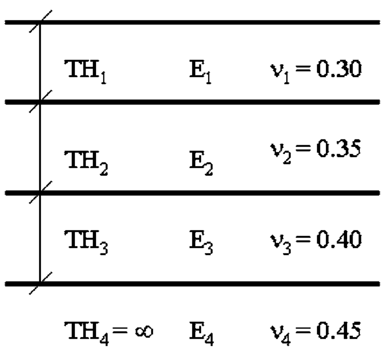


Figure A.2. Axle load and configuration.



	Low	Med	High
TH_1	4	5.66	8
TH_2	6	8.49	12
TH_3	6	8.49	12
E_1	200,000	447,213	1,000,000
E_2/E_3	1.0	2.0	3.0
E_3	10,000	14,142	20,000
E_4	3,000	6,000	12,000

Thicknesses in inches, moduli in psi

Notes:
- **TH = thickness**
- **E = elastic modulus**
- **ν = Poisson's ratio**
- **1 psi = 6.895 kPa**
- **1 in. = 25.4 mm**

Figure A.3. Pavement structure and experiment design.

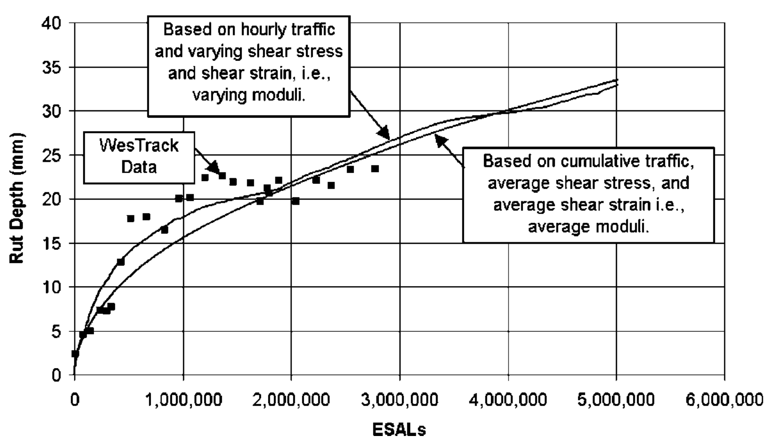


Figure A.4. Comparison of the methodology used to develop the level 2 permanent deformation model with that used in the HMA Spec software (1 in. = 25.4 mm) for Section 24 using a and c from Table 12.

TABLE A.1 Development of fatigue crack prediction model for fine-graded asphalt mixes

1	2	3	4	5	6	7	8	9	10	11
Obs. No.	FC (%)	P_{asp}	V_{air}	P_{200} (%)	PROBIT Model ESALs	PROBIT Model log (ESALs)	Fatigue Crack Probability[1]	Predicted log(ESALs)	Residuals	Predicted ESALs
1	10	4	4	3	4,005,000	6.603	50.0%	6.6025	0.00010	4,004,055
2	10	4	4	5	10,475,000	7.020	50.0%	7.0199	0.00025	10,468,875
3	10	4	4	7	27,390,000	7.438	50.0%	7.4373	0.00029	27,371,588
4	10	4	7	3	1,951,000	6.290	50.0%	6.2902	0.00006	1,950,743
5	10	4	7	5	5,102,000	6.708	50.0%	6.7076	0.00014	5,100,350
6	10	4	7	7	13,340,000	7.125	50.0%	7.1250	0.00016	13,335,214
7	10	4	10	3	950,500	5.978	50.0%	5.9779	0.00005	950,386
8	10	4	10	5	2,485,000	6.395	50.0%	6.3953	0.00003	2,484,849
9	10	4	10	7	6,497,000	6.813	50.0%	6.8127	0.00001	6,496,808
10	10	5	4	3	11,980,000	7.078	50.0%	7.0782	0.00026	11,972,918
11	10	5	4	5	31,320,000	7.496	50.0%	7.4956	0.00022	31,304,012
12	10	5	4	7	81,900,000	7.913	50.0%	7.9130	0.00028	81,846,479
13	10	5	7	3	5,834,000	6.766	50.0%	6.7659	0.00007	5,833,108
14	10	5	7	5	15,260,000	7.184	50.0%	7.1833	0.00025	15,251,059
15	10	5	7	7	39,900,000	7.601	50.0%	7.6007	0.00027	39,874,936
16	10	5	10	3	2,842,000	6.454	50.0%	6.4536	0.00002	2,841,842
17	10	5	10	5	7,430,000	6.871	50.0%	6.8710	-0.00001	7,430,191
18	10	5	10	7	19,430,000	7.288	50.0%	7.2884	0.00007	19,426,743
19	10	6	4	3	35,820,000	7.554	50.0%	7.5539	0.00023	35,801,399
20	10	6	4	5	93,650,000	7.972	50.0%	7.9713	0.00021	93,605,205
21	10	6	4	7	244,900,000	8.389	50.0%	8.3887	0.00029	244,737,207
22	10	6	7	3	17,450,000	7.242	50.0%	7.2416	0.00020	17,442,149
23	10	6	7	5	45,620,000	7.659	50.0%	7.6590	0.00016	45,603,692
24	10	6	7	7	119,300,000	8.077	50.0%	8.0764	0.00024	119,233,969
25	10	6	10	3	8,500,000	6.929	50.0%	6.9293	0.00012	8,497,673
26	10	6	10	5	22,220,000	7.347	50.0%	7.3467	0.00004	22,217,746
27	10	6	10	7	58,120,000	7.764	50.0%	7.7641	0.00023	58,089,816

[1] Probability of 10 percent fatigue cracking.

TABLE A.2 Development of fatigue crack prediction model for coarse-graded asphalt mixes

1	2	3	4	6	7	8	9	10	11
Obs. No.	FC (%)	P_{asp}	V_{air}	PROBIT Model ESALs	PROBIT Model log (ESALs)	Fatigue Crack Probability[1]	Predicted log(ESALs)	Residuals	Predicted ESALs
1	10	4	4	488,700	5.689	50.0%	5.689	0.00012	488,562
2	10	4	7	378,600	5.578	50.0%	5.578	0.00002	378,582
3	10	4	10	293,400	5.467	50.0%	5.467	0.00006	293,359
4	10	5	4	1,516,600	6.181	50.0%	6.181	-0.00005	1,516,771
5	10	5	7	1,175,500	6.070	50.0%	6.070	0.00006	1,175,330
6	10	5	10	911,000	5.960	50.0%	5.959	0.00012	910,752
7	10	6	4	4,710,000	6.673	50.0%	6.673	0.00010	4,708,906
8	10	6	7	3,649,000	6.562	50.0%	6.562	0.00001	3,648,884
9	10	6	10	2,828,000	6.451	50.0%	6.451	0.00008	2,827,483

[1] Probability of 10 percent fatigue cracking.

TABLE A.3 Summary of data used in development of composite fatigue cracking models from WesTrack and NCHRP 1-10B projects

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
WesTrack Section	Description	P_{200} (% pass)	P_4 (% retain)	P_{38} (% retain)	P_{34} (% retain)	V_{air}	P_{asp}	Absorp-tion (%)	(Vb)eff (%)	Load Freq (Hz)	Visco-sity (poise)	Witczak Formula E* (psi)	HMA Thick. (in.)	HMA Strain (in./in.)	NCHRP 1-10B 10% FC ESALs	UCB/WesTrack Level 1(a)[1] ESALs	UCB/WesTrack w/ NCHRP 1-10B ESALs	Effective Intercept (10% FC)
1	Fine (MM)	4.7	26.6	11.8	0	8.6	5.7	1.0	4.7	10	1,929	378,322	6.30	0.0001837	1,970,899	19,369,571	20,454,284	14.1423
2	Fine (LM)	4.7	26.6	11.8	0	10.0	4.9	1.0	3.9	10	1,929	373,068	6.20	0.0001881	1,844,645	5,765,275	5,698,135	
3	Fine (LH)	4.7	26.6	11.8	0	11.8	5.1	1.0	4.1	10	1,929	355,798	6.40	0.0001862	1,985,567	4,662,083	4,959,804	
4	Fine (ML)	4.7	26.6	11.8	0	6.6	5.4	1.0	4.4	10	1,929	395,258	6.10	0.0001859	1,825,069	22,522,158	22,023,646	
14	Fine (HM)	4.7	26.6	11.8	0	8.1	6.6	1.0	5.6	10	1,929	375,609	6.10	0.0001906	1,754,173	58,519,418	55,001,225	
15	Fine (MM)	4.7	26.6	11.8	0	8.7	5.6	1.0	4.6	10	1,929	378,308	6.10	0.0001900	1,763,958	16,948,841	16,018,737	14.1905
16	Fine (LH)	4.7	26.6	11.8	0	11.8	4.8	1.0	3.8	10	1,929	357,911	6.30	0.0001888	1,887,770	3,356,371	3,394,839	
17	Fine (MH)	4.7	26.6	11.8	0	11.0	5.7	1.0	4.7	10	1,929	358,667	6.30	0.0001886	1,890,868	10,896,324	11,039,291	
18	Fine (HL)	4.7	26.6	11.8	0	4.8	6.2	1.0	5.2	10	1,929	398,045	6.10	0.0001852	1,835,060	83,281,780	81,884,214	
9	Fine+ (HL)	5.5	25.7	12.1	0	3.4	5.9	1.0	4.9	10	1,929	404,470	6.10	0.0001838	1,858,035	123,179,943	122,629,167	
10	Fine+ (LH)	5.5	25.7	12.1	0	12.4	4.6	1.0	3.6	10	1,929	354,275	5.80	0.0002065	1,417,034	3,429,494	2,603,815	
11	Fine+ (MM)	5.5	25.7	12.1	0	7.6	5.3	1.0	4.3	10	1,929	389,563	6.10	0.0001872	1,804,603	23,329,208	22,557,017	
12	Fine+ (ML)	5.5	25.7	12.1	0	5.0	5.3	1.0	4.3	10	1,929	405,526	5.90	0.0001899	1,664,893	43,507,086	38,810,244	
13	Fine+ (HM)	5.5	25.7	12.1	0	6.6	5.9	1.0	4.9	10	1,929	391,399	6.20	0.0001837	1,914,692	57,203,157	58,683,894	
19	Fine+ (MM)	5.5	25.7	12.1	0	8.1	5.4	1.0	4.4	10	1,929	384,952	6.20	0.0001852	1,890,141	23,089,791	23,383,750	
20	Fine+ (MH)	5.5	25.7	12.1	0	10.3	5.3	1.0	4.3	10	1,929	367,792	6.20	0.0001894	1,824,342	12,213,215	11,938,127	
21	Fine+ (HL)	5.5	25.7	12.1	0	4.3	6.0	1.0	5.0	10	1,929	401,837	6.30	0.0001782	2,065,444	110,769,450	122,583,886	
22	Fine+ (LM)	5.5	25.7	12.1	0	8.5	4.6	1.0	3.6	10	1,929	388,813	6.00	0.0001906	1,704,140	8,734,137		
5	Coarse (MM)	6.05	23.7	19.5	0	8.4	5.6	0.7	4.9	10	1,929	361,543	6.00	0.0001976	1,610,521	2,059,074	1,941,057	13.6086
6	Coarse (MH)	6.05	23.7	19.5	0	11.3	5.8	0.7	5.1	10	1,929	337,873	5.90	0.0002078	1,444,760	2,018,385	1,706,867	
7	Coarse (HM)	6.05	23.7	19.5	0	7.5	6.3	0.7	5.6	10	1,929	362,957	6.10	0.0001939	1,708,087	4,912,471	4,911,453	
8	Coarse (LM)	6.05	23.7	19.5	0	8.5	5.5	0.7	4.8	10	1,929	361,530	6.00	0.0001976	1,610,475	1,822,972	1,718,439	
23	Coarse (ML)	6.05	23.7	19.5	0	5.8	5.8	0.7	5.1	10	1,929	376,600	5.80	0.0002004	1,485,945	3,221,544	2,801,991	
24	Coarse (MM)	6.05	23.7	19.5	0	7.6	5.8	0.7	5.1	10	1,929	365,724	6.20	0.0001899	1,816,365	2,764,445	2,939,079	13.5564
25	Coarse (HL)	6.05	23.7	19.5	0	3.8	6.3	0.7	5.6	10	1,929	380,416	6.10	0.0001894	1,771,591	6,728,279	6,976,980	
26	Coarse (LH)	6.05	23.7	19.5	0	10.1	5.4	0.7	4.7	10	1,929	349,959	6.20	0.0001941	1,755,231	1,420,718	1,459,628	

[1]50 percent probability 10 percent fatigue cracking
1 psi = 6.9 kPa
1 in. = 25.4 mm

TABLE A.4 Factorial of predictor variables for the development of a tensile strain equation

Layer 1 Thickness, TH1 (mm)	102, 144, and 203
Layer 2 Thickness, TH2 (mm)	102, 216, and 305
Layer 3 Thickness, TH3 (mm)	152, 216, and 305
Layer 1 Elastic Modulus, E1 (MPa)	1.38, 3.09, and 6.90
Layer 3 Elastic Modulus, E3 (MPa)	0.07, 0.10, and 0.14
Layer 4 Elastic Modulus, E4 (MPa)	0.02, 0.04, and 0.08
Ratio of Layer 2 to Layer 3 Modulus, E23	1, 2, and 3

TABLE A.5 HMA mix properties used to illustrate composite fatigue equations

Independent Variable	Fine-Graded Mix	Coarse-Graded Mix
HMA tensile strain (mm/mm)	0.0001932	0.0001976
HMA dynamic modulus (MPa)	2.61	2.49
Percent passing (0.075-mm sieve)	4.7	6.05
Percent retained (4.75-mm sieve)	26.6	23.7
Percent retained (9.5-mm sieve)	11.8	19.5
Percent retained (19-mm sieve)	0.0	0.0
Asphalt content (%)	5.7	5.6
Absorption (%)	1.0	0.7
Air void content (%)	8.6	8.4

TABLE A.6 Summary of data used in development of simplified level 2 fatigue cracking models

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
								HMA Stiffness				HMA Strain (in./in.)		NCHRP	UCB/	Temp.	WesTrack	Effective
		P_{200}	P_4	P_{38}	P_{34}				UCB/ WesTrack		HMA	2-Layer	4-Layer	1-10B	WesTrack	Conv.	ESALs[1] to	Shift
WesTrack Section	Description	(% pass)	(% retain)	(% retain)	(% retain)	V_{air}	P_{asp}	Temp. (°C)	E (MPa)	E (psi)	Thick. (in.)	Level 1 Model	Level 2 Model	10% FC ESALs	Level 2 (Lab) Nf	Factor[2] TCF	FC = 10 % (eye est.)	Factor SF
1	Fine (MM)	4.7	26.6	11.8	0	8.6	5.7	18.5	4,372	633,879	6.30	0.0001422	0.0001484	2,557,649	5,075,857	1.9442	6 *	2.30
2	Fine (LM)	4.7	26.6	11.8	0	10.0	4.9	20.3	4,212	610,732	6.20	0.0001473	0.0001544	2,318,557	2,474,605	1.9106	3.6	2.78
3	Fine (LH)	4.7	26.6	11.8	0	11.8	5.1	20.1	3,508	508,608	6.40	0.0001560	0.0001657	2,147,898	1,489,461	1.9773	3.1	4.12
4	Fine (ML)	4.7	26.6	11.8	0	6.6	5.4	17.7	5,786	838,931	6.10	0.0001280	0.0001296	3,140,987	11,264,508	1.8764	6.5 *	1.08
14	Fine (HM)	4.7	26.6	11.8	0	8.1	6.6	18.8	3,645	528,577	6.10	0.0001609	0.0001706	1,886,985	4,117,994	1.8764	5.5 *	2.51
15	Fine (MM)	4.7	26.6	11.8	0	8.7	5.6	18.5	4,436	643,189	6.10	0.0001460	0.0001524	2,312,947	4,233,669	1.8764	7 *	3.10
16	Fine (LH)	4.7	26.6	11.8	0	11.8	4.8	18.7	4,074	590,688	6.30	0.0001473	0.0001547	2,369,685	1,814,943	1.9442	4.2	4.50
17	Fine (MH)	4.7	26.6	11.8	0	11.0	5.7	17.7	3,755	544,408	6.30	0.0001533	0.0001621	2,176,213	2,372,271	1.9442	5.1	4.18
18	Fine (HL)	4.7	26.6	11.8	0	4.8	6.2	18.5	5,340	774,230	6.10	0.0001332	0.0001363	2,853,092	16,084,680	1.8764	6 *	0.70
9	Fine+ (HL)	5.5	25.7	12.1	0	3.4	5.9	23.4	4,834	700,986	6.10	0.0001399	0.0001448	2,544,151	10,616,730	1.8764	Rut Fail	- -
10	Fine+ (LH)	5.5	25.7	12.1	0	12.4	4.6	20.4	3,677	533,222	5.80	0.0001686	0.0001789	1,602,750	626,567	1.7704	1.8	5.09
11	Fine+ (MM)	5.5	25.7	12.1	0	7.6	5.3	19.1	5,025	728,668	6.10	0.0001373	0.0001415	2,658,809	5,031,257	1.8764	5.7	2.13
12	Fine+ (ML)	5.5	25.7	12.1	0	5.0	5.3	18.5	6,451	935,404	5.90	0.0001255	0.0001257	3,170,361	12,708,507	1.8064	6	0.85
13	Fine+ (HM)	5.5	25.7	12.1	0	6.6	5.9	23.4	3,710	537,995	6.20	0.0001569	0.0001661	2,032,332	3,532,807	1.9106	Rut Fail	- -
19	Fine+ (MM)	5.5	25.7	12.1	0	8.1	5.4	17.7	5,111	741,058	6.20	0.0001338	0.0001375	2,878,545	5,579,327	1.9106	6.5 *	2.23
20	Fine+ (MH)	5.5	25.7	12.1	0	10.3	5.3	18.8	4,090	593,062	6.20	0.0001495	0.0001570	2,247,298	2,093,145	1.9106	5.5 *	5.02
21	Fine+ (HL)	5.5	25.7	12.1	0	4.3	6.0	23.4	4,386	636,006	6.30	0.0001420	0.0001481	2,567,092	8,759,946	1.9442	Rut Fail	- -
22	Fine+ (LM)	5.5	25.7	12.1	0	8.5	4.6	18.5	5,667	821,785	6.00	0.0001315	0.0001338	2,880,617	4,270,751	1.8417	7 *	3.02
5	Coarse (MM)	6.05	23.7	19.5	0	8.4	5.6	20.4	5,172	750,004	6.00	0.0001376	0.0001416	2,587,464	9,953,983	1.8417	1.8	0.33
6	Coarse (MH)	6.05	23.7	19.5	0	11.3	5.8	21.4	3,947	572,263	5.90	0.0001601	0.0001690	1,820,824	3,997,001	1.8064	1.67	0.75
7	Coarse (HM)	6.05	23.7	19.5	0	7.5	6.3	23.4	3,910	566,897	6.10	0.0001555	0.0001640	2,024,613	9,694,751	1.8764	Rut Fail	- -
8	Coarse (LM)	6.05	23.7	19.5	0	8.5	5.5	22.3	4,683	678,985	6.00	0.0001446	0.0001503	2,313,069	7,494,688	1.8417	1.54	0.38
23	Coarse (ML)	6.05	23.7	19.5	0	5.8	5.8	18.7	6,381	925,208	5.80	0.0001283	0.0001290	2,934,378	20,865,173	1.7704	Rut Fail	- -
24	Coarse (MM)	6.05	23.7	19.5	0	7.6	5.8	17.9	6,038	875,520	6.20	0.0001232	0.0001238	3,520,783	20,666,827	1.9106	Rut Fail	- -
25	Coarse (HL)	6.05	23.7	19.5	0	3.8	6.3	23.4	4,838	701,554	6.10	0.0001399	0.0001447	2,546,479	24,221,189	1.8764	Rut Fail	- -
26	Coarse (LH)	6.05	23.7	19.5	0	10.1	5.4	22.8	4,233	613,732	6.20	0.0001470	0.0001539	2,330,765	5,508,304	1.9106	1.46	0.51

1 in. = 25.4 mm
[1] in millions
[2] Based on the “high-deviant” coefficients, a = 2.102 and b = -3.884. See Chapter, Section 4.4.3

* Estimate based on extrapolation.

APPENDIX B

HOT-MIX ASPHALT OVERLAY THICKNESS DESIGN MODEL

B.1 INTRODUCTION

For the LCC analysis approach described in Chapters 3 and 5 of Part II to work successfully in generating a contractor PA, it is important that a rational process be used to determine the structural need for both the as-designed and the as-constructed pavement when rehabilitation is triggered. The most widely accepted overlay design methodology is the one developed for and presented in the 1993 *AASHTO Guide for Design of Pavement Structures* (*B.1*). The methodology is rational and defensible, and it takes into account the effect of a pre-overlay milling operation. Consequently, it was selected for use in the HMA PRS.

This appendix describes the step-by-step processes used for determining the overlay thickness requirement for the as-designed and as-constructed pavement. The processes for both are very similar. The primary difference is that the design life of the as-designed pavement is given, while the life of the as-constructed pavement is unknown. Both processes rely on the "remaining life" approach described in the 1993 *AASHTO Guide for Design of Pavement Structures*.

B.2 OVERLAY DESIGN FOR AS-DESIGNED PAVEMENT

Five steps are required to develop the overlay thickness design for the as-designed pavement. To help demonstrate the process, an example problem is started in the first step and carried through the fifth.

Step 1: Determine the Associated AASHTO Structural Number. For purposes of the HMA PRS, the effective AASHTO structural number associated with the as-designed pavement is backcalculated using the AASHTO design equation and information supplied by the user:

- The year in the life of the as-designed pavement at which the rehabilitation is triggered. This should be the same as the user-defined estimate of the mean life (in years) of the as-designed pavement. It identifies the cumulative ESAL applications through use of the traffic/ESAL cumulative growth formula. *[Example: For rehabilitation triggered in year 12, assume ESAL = 1,000,000.]*
- The user-defined target smoothness of the as-designed pavement, $(IRI_o)_{des}$. This is converted into $(p_o)_{des}$, the initial PSI, using the following PSI-IRI relationship (*B.2*):

$$PSI = 5 * e^{-0.24 * IRI} \qquad (B.1)$$

where IRI is in units of m/km. *[Example: $(IRI_o)_{des} = 0.44$ m/km, $(p_o)_{des} = 4.5$.]*

- The user-defined terminal (trigger) roughness level, IRI_t. This value is converted to p_t, the terminal PSI, using equation B.1. *[Example: $IRI_t = 2.13$, $p_t = 3.0$.]*
- The user-defined roadbed soil resilient modulus, M_R (psi). *[Example: $M_R = 5{,}000$ psi.]*

The basic equation from the 1993 *AASHTO Guide for Design of Pavement Structures* (*B.1*) for the design of flexible pavements, that is, equation 1.2.1 in Chapter 1 of Part I provides the basis for determining the required structural number (SN), which is referred to as $(SN_0)_{des}$. Since SN appears in two locations, it must be solved for through an iterative process using the following equation(s):

$$SN = 10^{PWR} - 1.0 \qquad (B.2)$$

where

$$\begin{aligned} PWR = {} & 0.1068*\log_{10}(ESAL) - 0.1068 \\ & * \log_{10}(\Delta PSI/2.7)/[0.40 + 1094/(SN + 1)^{5.19}] \\ & - 0.2479 * \log_{10}(M_R) + 0.8835 - 0.1068 \\ & * Z_R * S_o \end{aligned} \qquad (B.3)$$

$$\Delta PSI = p_o - p_t \qquad (B.4)$$

where Z_R = standard normal deviate and S_o = combined standard error of the traffic and performance predictions.

The iterative process is started by assuming an SN of 4.0, entering it into the formula for PWR (equation B.3), and solving for a new SN in equation B.2. The difference between the assumed SN and the new SN is checked and if the difference is less than 0.01, the iterative process is stopped. If the difference is greater than 0.01, an SN equal to the new SN is assumed and another iteration is completed. Once convergence is achieved, $(SN_0)_{des}$ is set equal to SN.

Example of Iterative Process

1. *$\Delta PSI = (p_o)_{des} - p_t = 4.5–3.0 = 1.5$*
2. *SN = 4.00, PWR = 0.6487, SN(new) = 3.45*
3. *SN = 3.45, PWR = 0.6386, SN(new) = 3.35*
4. *SN = 3.35, PWR = 0.6366, SN(new) = 3.33*
5. *SN = 3.33, PWR = 0.6362, SN(new) = 3.33 (convergence)*
6. *$(SN_0)_{des} = 3.33$*

It should be noted that there is one other term with two variables in the basic AASHTO design equation which was not included in the above derivation. This $Z_R * S_o$ term accounts for design reliability and becomes zero when, as is recommended in this instance, design reliability, R, is established at 50 percent.

Step 2: Estimate the Cumulative ESAL Applications to "Ultimate" Pavement Failure. The cumulative ESAL applications discussed in step 1 referred to the level of axle load traffic that could be carried by the pavement until it reached a certain terminal serviceability (trigger) level, p_t. Typically, this trigger level is significantly greater than the serviceability level associated with ultimate pavement failure, that is, $p_t = 1.5$. The purpose of this step is to estimate the cumulative ESAL traffic, $(N_{1.5})_{des}$, corresponding to this ultimate failure level for the as-designed pavement. This is accomplished using the basic AASHTO design equation where $p_o = (p_o)_{des}$, $p_t = 1.5$, $Z_R * S_o = 0$, $SN = (SN_0)_{des}$, and all other variables are the same as in step 1.

The value of ESAL arising from this calculation then becomes $(N_{1.5})_{des}$.

[Example: For $p_o = 4.5$, $p_t = 1.5$, SN = 3.33, and M_R = 5,000 psi; $(N_{1.5})_{des}$ = 2,100,000.]

Step 3: Determine the Effective Structural Number (when rehabilitation is triggered). With time and cumulative ESAL applications, the structural number of the pavement will deteriorate to a lesser value that would be representative of the support the pavement would provide to a subsequent overlay or rehabilitation option. In the 1993 *AASHTO Guide for Design of Pavement Structures*, this is referred to as the effective structural number, SN_{eff}, and may be calculated any one of three ways: (1) through backcalculation based on nondestructive testing, (2) through interpretation/analysis of pavement condition, or (3) through remaining life analysis. The third method was selected in this process because it is related directly to ESAL applications, which are readily and accurately available.

The information required for the determination of the effective structural number of the as-designed pavement, $(SN_{eff})_{des}$, include the following:

- The cumulative ESAL applications, $(N_p)_{des}$, in the year when the rehabilitation is triggered. This is obtained from the traffic/ESAL growth formula for the year when the rehabilitation is triggered. It corresponds to the value of ESAL determined in step 1(a) above.
- The cumulative ESAL applications, $(N_{1.5})_{des}$, when the pavement reaches ultimate failure (from step 2).

With the values of $(N_p)_{des}$ and $(N_{1.5})_{des}$ from above, the remaining life of the as-designed pavement (expressed as a decimal fraction) is determined using the following equation:

$$(RL)_{des} = [1 - (N_p)_{des}/(N_{1.5})_{des}] \quad \text{(B.5)}$$

If the value of $(RL)_{des}$ is determined to be negative, it should be reassigned a value of 0.0.

[Example: $(RL)_{des}$ = [1 – 1000000/ 2100000] = 0.524.]

There are some instances where the estimated remaining life of the pavement is controlled by the extent of fatigue cracking. Accordingly, the following equation is used to calculate the maximum level of remaining life, RL_{max} (expressed as a decimal fraction), associated with the extent fatigue cracking, FC (expressed as a percentage of the wheelpath area exhibiting cracking):

$$RL_{max} = 1 - 0.087 * FC + 0.00144 * FC^2 \quad \text{(B.6)}$$

Then, if the remaining life of the as-designed pavement, $(RL)_{des}$, determined based on AASHTO serviceability, is found to be greater than RL_{max}, it should be reassigned a value equal to RL_{max}.

[Example: Assuming the predicted fatigue cracking (FC) at year 12 is 5 percent, then RL_{max} = 0.60. Since the $(RL)_{des}$ calculated using equation B.5 is not greater that 0.60, it is not corrected.)

Next, the pavement condition factor, $(CF)_{des}$, is determined as a function of the remaining life:

$$(CF)_{des} = (RL)_{des}^{0.165} \quad \text{(B.7)}$$

[Example: $(CF)_{des}$ = 0.90.]

In this case, if the value of $(CF)_{des}$ is determined to be less that 0.5, it should be reassigned a value equal to 0.50. *[In the example, $(CF)_{des}$ is greater than 0.50, so no correction is necessary.]*

Finally, the effective structural number of the as-designed pavement is calculated using the following relationship:

$$(SN_{eff})_{des} = (CF)_{des} * (SN_0)_{des} - (a_1' * D_{mill}) \quad \text{(B.8)}$$

where

a_1' = The (reduced) layer coefficient of the HMA surface layer at the time the rehabilitation operation is triggered. For programming purposes within the *HMA Spec* software, a value of 0.20 was assumed and

D_{mill} = The depth (inches) of the pre-overlay milling operation specified by the M&R decision tree (1 in. = 25.4 mm).

The remaining variables are as previously defined (or calculated).

[Example: Assuming values for a_1' and D_{mill} of 0.20 and 1 in., respectively, $(SN_{eff})_{des}$ = 2.80.]

Step 4: Determine the Structural Number Required for a New Pavement to Last the Rest of the Analysis Period. The purpose of this step is to determine the structural number, $(SN_f)_{des}$, that would be required if a new pavement were to be constructed to last the remaining years of the analysis period. The calculation is made using either the basic AASHTO design equation or, for programming purposes within the *HMA Spec* software, the set of relationships represented by equations B.2, B.3, and B.4 that require an iterative process (see description in step 1) for proper determination. In either case, the following inputs are required:

- The cumulative ESAL applications expected from the time of the rehabilitation to the end of the analysis period. This is calculated using the ESAL/traffic growth formula assuming that traffic starts to accumulate at the time the rehabilitation is complete. *[Example: Assume ESAL traffic between year 12 when the rehabilitation is triggered and the end of the analysis period (year 20) is 800,000. Thus, the total ESAL traffic over the 20-year analysis period is 1,800,000.]*
- The user-defined roadbed soil modulus, M_R. This is the same value that was used in step 1. *[Example: M_R = 5,000 psi.]*
- The initial pavement serviceability, p_o, after the rehabilitation is complete. For programming purposes within the *HMA Spec* software, it was assumed that this value is the same as that determined for the as-designed pavement, $(p_o)_{des}$. *[Example: $p_o = 4.5$.]*
- The terminal pavement serviceability, p_t, which triggers a rehabilitation operation. This value is the same as that used in the step 1 calculation process. *[Example: $p_t = 3.0$.]*
- The predefined values associated with the treatment of life uncertainty, that is, design reliability and overall standard deviation, R and S_o, respectively. S_o is a value typically in the range of 0.39 to 0.49 for flexible pavements. A default value of 0.45 is used in the *HMA Spec* software. R is a value in the range of 50 to 99.99 percent. Table 4.1 (from Part I of the 1993 *AASHTO Guide for Design of Pavement Structures*) provides the standard normal deviate, Z_R, corresponding to the range of reliability values typically used by pavement designers. The R values required for rehabilitation design in the *HMA Spec* software have the default values specified in Table 4.1 of Part I of the 1993 *AASHTO Guide for Design of Pavement Structures* (*B.1*). *[Example: $S_o = 0.45$, R = 90 percent and $Z_R = -1.282$.]*

In this instance, the iterative process is started by assuming an SN equal to $(SN_0)_{des}$ (from step 1), entering it into the formula for PWR (equation B.3) and solving for a new SN in equation B.2. The difference between the assumed SN and the new SN is checked and, if the difference is less than 0.01, the iterative process is stopped. If the difference is greater than 0.01, an SN equal to the new SN is assumed and another iteration is completed. Once convergence is achieved, $(SN_f)_{des}$ is assigned equal to SN.

Example of Iterative Process

1. $\Delta PSI = (p_o)_{des} - p_t = 4.5–3.0 = 1.5$
2. *SN = 3.33, PWR = 0.6875, SN(new) = 3.87*
3. *SN = 3.87, PWR = 0.6978, SN(new) = 3.99*
4. *SN = 3.99, PWR = 0.7000, SN(new) = 4.01*
5. *SN = 4.01, PWR = 0.7002, SN(new) = 4.01 (convergence)*
6. $(SN_f)_{des} = 4.01$

Step 5: Determine the Required HMA Overlay Thickness. The following equation is used to calculate the thickness, $(D_{ol})_{des}$ (inches), of the new layer of HMA surface associated with the prescribed rehabilitation operation:

$$(D_{ol})_{des} = [(SN_f)_{des} - (SN_{eff})_{des}]/a_{ol} \quad \text{(B.9)}$$

where a_{ol} = The user-defined layer coefficient for the HMA overlay material.

The remaining variables are as calculated in previous steps.

[Example: Assuming a typical HMA layer coefficient of 0.44, the design HMA overlay thickness required for the as-designed pavement (in year 12) is 1.5 in.]

B.3 OVERLAY DESIGN FOR AS-CONSTRUCTED PAVEMENT

Following is a description of the steps required to determine the overlay thickness design for the as-constructed pavement. They are very similar to the steps described for the as-designed pavement. Again, an example is followed from start to finish to help demonstrate the process.

Step 1: Determine the Associated AASHTO Structural Number. The following information is required to determine the AASHTO structural number associated with the as-constructed pavement:

- The year in the life of the as-constructed pavement at which the rehabilitation is being triggered. This is converted into cumulative ESAL applications using the traffic/ESAL cumulative growth formula. *[Example: Assume that rehabilitation is triggered in year 9, when the cumulative ESAL applications is 720,000.]*
- The user-defined initial measured smoothness of the as-constructed pavement, IRI_o. This is converted into p_o using the PSI-IRI relationship identified earlier as equation B.1. *[Example: $(IRI_o)_{des}$ = 0.44 m/km, $(p_o)_{des} = 4.5$.]*
- The user-defined terminal (trigger) roughness level, IRI_t. This is the same as for the as-designed pavement and is

also converted into p_t using the PSI-IRI correlation in equation B.1. *[Example: $IRI_t = 2.13$, $p_t = 3.0$.]*

- The user-defined roadbed soil resilient modulus, M_R (psi). This, too, is the same as the value defined under step 1 of the overlay design process for the as-designed pavement. *[Example: $M_R = 5{,}000$ psi.]*
- As was the case with the as-designed pavement, reliability does not play a role in estimating the mean structural number of the as-constructed pavement. Therefore, reliability is set to 50 percent and the $Z_R * S_o$ term becomes zero and there is no effect on the calculations.

The same series of relationships used in step 1 of the as-designed pavement rehabilitation process (equations B.2, B.3, and B.4) are used for the as-constructed pavement. Some of the inputs are different, as described previously, and the result is the estimated mean structural number of the as-constructed pavement, $(SN_0)_{con}$.

The iterative process is started by assuming an SN of 4.0, entering it into the formula for PWR (equation B.3), and solving for a new SN in equation B.2. The difference between the assumed SN and the new SN is checked and if the difference is less than 0.01, the iterative process is stopped. If the difference is greater than 0.01, an SN equal to the new SN is assumed and another iteration is completed. Once convergence is achieved, $(SN_0)_{con}$ is assigned equal to SN.

Example of Iterative Process

1. *$PSI = (p_o)_{des} - p_t = 4.5–3.0 = 1.5$*
2. *$SN = 4.00$, $PWR = 0.6335$, $SN(new) = 3.30$*
3. *$SN = 3.30$, $PWR = 0.6204$, $SN(new) = 3.17$*
4. *$SN = 3.17$, $PWR = 0.6178$, $SN(new) = 3.15$*
5. *$SN = 3.15$, $PWR = 0.6174$, $SN(new) = 3.14$ (convergence)*
5. *$(SN_0)_{con} = 3.14$*

Step 2: Estimate the Cumulative ESAL Applications to Ultimate Pavement Failure. This step is very similar to step 2 of the as-designed pavement rehabilitation process. The only input difference(s) now are $p_o = (p_o)_{con}$ and $SN = (SN_0)_{con}$.

The value of ESAL arising from the calculation becomes $(N_{1.5})_{con}$.

[Example: For $p_o = 4.5$, $p_t = 1.5$, $SN = 3.14$, and $M_R = 5{,}000$ psi; $(N_{1.5})_{con} = 1{,}360{,}000$.]

Step 3: Determine the Effective Structural Number (when rehabilitation is triggered). The information required for the determination of the effective structural number of the as-constructed pavement, $(SN_{eff})_{con}$, includes the following:

- The cumulative ESAL applications, $(N_p)_{con}$, in the year when the rehabilitation is triggered. This corresponds to the value of ESAL determined in step 1 above. *[Example: $(N_p)_{con} = 720{,}000$.]*
- The cumulative ESAL applications, $(N_{1.5})_{con}$, when the pavement reaches ultimate failure (from step 2). *[Example: $(N_{1.5})_{con} = 1{,}360{,}000$.]*

With the values of $(N_p)_{con}$ and $(N_{1.5})_{con}$ from above, the remaining life of the as-constructed pavement (expressed as a decimal fraction) is determined using the following equation:

$$(RL)_{con} = [1 - (N_p)_{con}/(N_{1.5})_{con}] \quad \text{(B.10)}$$

If the value of $(RL)_{con}$ is determined to be negative, it should be reassigned a value of 0.0.

[Example: $(RL)_{con} = [1 - 720000/1360000) = 0.471$.]

As was the case for the as-designed pavement, this value of remaining life must be checked against the maximum value associated with the extent of fatigue cracking. If $(RL)_{con}$ is greater than RL_{max} (determined from equation B.6 for the predicted extent of fatigue cracking in the as-constructed pavement), then $(RL)_{con}$ must be set equal to RL_{max}.

[Example: Assuming the predicted fatigue cracking (FC) at year 9 is 8 percent, then $RL_{max} = 0.396$. Since the $(RL)_{des}$ calculated using equation B.5 is greater than 0.396, it must be corrected to the lower value, that is, $(RL)_{con} = 0.396$.]

Next, the pavement condition factor, $(CF)_{con}$, is determined as a function of the remaining life:

$$(CF)_{con} = (RL)_{con}^{0.165} \quad \text{(B.11)}$$

[Example: $(CF)_{con} = 0.858$.]

In this case, if the value of $(CF)_{con}$ is determined to be less that 0.5, it should be reassigned a value equal to 0.5. *[In the example, $(CF)_{con}$ is greater than 0.5, so no correction is necessary.]*

Finally, the effective structural number of the as-designed pavement is calculated using the following relationship:

$$(SN_{eff})_{con} = (CF)_{con} * (SN_0)_{con} - (a_1' * D_{mill}) \quad \text{(B.12)}$$

where the remaining variables are as previously defined (or calculated).

[Example: Assuming values for a_1' and D_{mill} of 0.20 and 1 in., respectively, $(SN_{eff})_{con} = 2.49$.]

Step 4: Determine the Structural Number Required for a New Pavement to Last the Rest of the Analysis Period. The purpose of this step is to determine the structural number, $(SN_f)_{con}$, that would be required if a new pavement were to be constructed to last the remaining years of the analysis period. This step is almost identical to step 4 of the rehabilitation design process for the as-designed pavement. The calculation is the set of relationships represented by equations B.2, B.3, and B.4 that require an iterative process for proper determination. The following inputs are required:

- The cumulative ESAL applications expected from the time of the rehabilitation to the end of the analysis period. This is calculated using the ESAL/traffic growth formula assuming that traffic starts to accumulate at the time the rehabilitation is complete. *[Example: Assume ESAL traffic between year 9 when the rehabilitation is triggered and the end of the analysis period (year 20) is 1,080,000. This means that the total ESAL traffic over the 20-year analysis period, that is, 1,800,000, is the same as that used in the example for the as-designed pavement.]*
- The user-defined roadbed soil modulus, M_R. This is the same value that was used for the as-designed pavement. *[Example: $M_R = 5{,}000$ psi.]*
- The initial pavement serviceability, p_o, after the rehabilitation is complete. For purposes of the *HMA Spec* software, this value is assumed to be the same as that determined for the as-designed pavement, $(p_o)_{des}$. *[Example: $p_o = 4.5$.]*
- The terminal pavement serviceability, p_t, which triggers a rehabilitation operation. This value is the same as that used in the step 1 calculation process and for the as-designed pavement. *[Example: $p_t = 3.0$.]*
- The predefined values associated with the treatment of life uncertainty, that is, design reliability and overall standard deviation, R and S_o, respectively. The values used for rehabilitation design of the as-constructed pavement are the same as for the as-designed pavement. *[Example: $S_o = 0.45$, R = 90 percent, and $Z_R = -1.282$.]*

The same iterative process used for the as-designed pavement is followed for the as-constructed pavement. Once convergence is achieved, $(SN_f)_{con}$ is set equal to SN.

Example of Iterative Process

1. *PSI* $= (p_o)_{des} - p_t = 4.5–3.0 = 1.5$
2. *SN = 3.14, PWR = 0.6976, SN(new) = 3.98*
3. *SN = 3.98, PWR = 0.7136, SN(new) = 4.17*
4. *SN = 4.17, PWR = 0.7167, SN(new) = 4.21*
5. *SN = 4.21, PWR = 0.7173, SN(new) = 4.22 (convergence)*
6. $(SN_f)_{con} = 4.22$

Step 5: Determine the Required HMA Overlay Thickness. The following equation is used to calculate the thickness, $(D_{ol})_{con}$ (inches), of the new layer of HMA surface associated with the prescribed rehabilitation operation:

$$(D_{ol})_{con} = [(SN_f)_{con} - (SN_{eff})_{con}]/a_{ol} \qquad \text{(B.13)}$$

where all variables are as previously defined or calculated.

[Example: Assuming a typical HMA layer coefficient of 0.44, the design HMA overlay thickness required for the as-constructed pavement (in year 9) is 3.9 in. This compares with the 1.5-in. overlay thickness required for the as-designed pavement in year 12.]

REFERENCES

B.1 American Association of State Highway and Transportation Officials, *AASHTO Guide for Design of Pavement Structures*, Washington, D.C., 1993.

B.2 Al-Omari, B. and Darter, M.I., "Relationships Between International Roughness Index and Present Serviceability Rating," In *Transportation Research Record 1435*, Transportation Research Board, National Research Council, Washington, D.C., 1994.

APPENDIX C

GUIDE SPECIFICATION FOR HOT-MIX ASPHALT PAVEMENT MATERIAL

GUIDE SPECIFICATION FOR HOT-MIX ASPHALT PAVEMENT MATERIAL AASHTO PP 400

CHAPTER 1—INTRODUCTION

1.1 DESCRIPTION

A mixture composed of aggregate, asphalt cement with or without mineral filler, modifiers and/or additives, recycled asphalt and baghouse fines which has been designed, mixed at an elevated temperature at a central plant, transported, laid, and compacted in compliance with the lines, grades, thickness, and typical cross sections shown on the plans.

1.2 USE

The mixture shall be used as a base course, leveling course, or surface course, or any combination of these courses as shown on the plans.

1.3 SCOPE

This specification provides the framework (Figures C.1 and C.2) for a quality control/quality assurance specification for hot-mix asphalt (HMA) pavement material. Included in the specification are requirements for the following:

- Laboratories facilities.
- Laboratory equipment.
- Sampling and testing personnel.
- Sampling, testing methods, and testing frequencies for quality control and quality assurance.
- Quality Control Plans.
- Materials.
- Construction practices.
- Mixture design method.
- Acceptance plan.
- Measurement.
- Pay adjustment factors.
- Payment.

This standard may involve hazardous materials, operations, and equipment. It does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this practice to consult and establish appropriate safety and health practices and determine the applicability of regulatory requirements prior to use.

1.4 REFERENCED DOCUMENTS

Procedures, guides, sampling methods, test methods, and specifications are referenced in this specification.

1.4.1 Quality Control/Quality Assurance

Table C.1 contains procedures, guides, specifications, and general references associated with quality control and quality assurance types of specifications.

1.4.2 Asphalt Binder Tests

Table C.2 contains sampling methods, test methods, and specifications for asphalt binders.

1.4.3 Aggregate Tests

Table C.3 contains sampling methods, test methods and specifications for aggregates used in HMA.

1.4.4 Hot-Mix Asphalt Tests

Table C.4 contains sampling methods, test methods, specifications, and mixture design methods for HMA.

1.4.5 Pavement Roughness Measurement

Table C.5 contains test methods for measuring pavement roughness.

1.5 TERMINOLOGY

1.5.1 Standard Terminology

Definition of terms common to quality control and quality assurance standards are contained in AASHTO R-10.

Definition of many common terms relating to HMA are contained in ASTM D 8.

Definitions of terms used in reference to other standards are as defined therein.

Definition of terms in mathematical expressions are as generally used in standard practice. Unique terms are defined in the section containing the first presentation of such terms.

1.5.2 Definitions

AAP	AASHTO Accreditation Program
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
HMA	Hot-Mix Asphalt
JMF	Job mix formula
JMF1	Job mix formula number 1 obtained from laboratory mixture design process
JMF2	Job mix formula number 2 obtained from trial field section placed at start of field production of the HMA
JMF3	Job mix formula number 3 defined by contractor and approved by engineer to represent acceptable mixture during production of HMA
JMFi	Job mix formula i defined by contractor and approved by engineer to represent acceptable mixture during production of HMA
PG	Performance-Graded Asphalt Binder
QA	Quality Assurance
QC	Quality Control
QCP	Quality Control Plan
SHA	State Highway Agency

1.6 LABORATORY, EQUIPMENT, AND PERSONNEL

1.6.1 Laboratory

Laboratories (contractor, engineer, and referee) performing tests in accordance with these standards shall be currently accredited by the AASHTO Accreditation Program (AAP) for the materials being tested (asphalt binders, aggregates, or HMA). Testing laboratories conducting inspections or tests not covered by the AAP shall comply with the applicable requirements of AASHTO R 18 and ASTM D 3666.

1.6.2 Equipment

All laboratory equipment (contractor, engineer, and referee laboratory) used to perform tests in accordance with these standards shall be calibrated and calibration verified at established intervals by the State Highway Agency according to relevant State Highway Agency standards.

1.6.3 Personnel

All personnel (contractor, State Highway Agency, and referee) shall be appropriately qualified through certification procedures established by the State Highway Agency. Certification shall ensure that minimum standards are in place for personnel to obtain samples; process samples; operate necessary equipment; verify equipment accuracy and calibration; interpret test results; and inspect the construction operation. Tables C.2 through C.5 show the requirements for certification for individual test methods.

1.7 SAMPLING AND TESTING

1.7.1 Quality Control Sampling and Testing

The contractor shall perform the sampling, testing, inspection, and calibrations as shown in Table C.1 and as defined in the Quality Control Plan.

1.7.2 Quality Assurance Sampling and Testing

The engineer will perform the sampling, testing, and inspection as shown in Table C.1 and as defined in the specification as quality assurance sampling and testing.

1.7.3 Referee Sampling and Testing

This sampling and testing will be performed by a third party mutually agreed upon at the start of the project. The third party sampling and testing group can be the State Highway Agency central laboratory or a commercial laboratory which has the proper certifications and accreditations.

1.7.4 Independent Assurance Program

An unbiased and independent evaluation of all sampling and testing used in the acceptance program shall be performed by the State Highway Agency in conformance with Federal Highway Administration policies.

1.8 QUALITY CONTROL PLAN

1.8.1 General

The contractor shall maintain a quality control system that is based on an established Quality Control Plan. The Quality Control Plan shall be prepared by the contractor and will be approved by the engineer. The Quality Control Plan shall be a written document covering all personnel, equipment, supplies, and facilities necessary to obtain samples, perform and document tests, and otherwise ensure the quality of the product.

A draft, written Quality Control Plan for this specification item shall be presented at the Preconstruction Conference for the project. The contractor proposed Quality Control Plan for this specification item shall be submitted to the engineer for approval at least 10 working days before the start of HMA production. The engineer will accept or reject the Quality Control Plan within 5 working days of submittal.

The Quality Control Plan shall reference the following standards for qualification, control, and guidelines (Table C.1):

AASHTO R 4	Statistical Procedures
AASHTO R 9	Acceptance Sampling Plans for Highway Construction
AASHTO R 10	Definition of Terms for Specifications and Procedures
AASHTO R 11	Using Significant Digits in Test Data to Determine Conformance with Specifications
AASHTO R 18	Establishing and Implementing a Quality System for Construction Materials Testing Laboratories
ASTM D 3666	Minimum Requirements for Agencies Testing and Inspecting Bituminous Paving Materials

The Quality Control Plan's administration, including compliance with the plan and its modifications, is the responsibility of the contractor. The Quality Control Plan can be wholly performed by the contractor or, wholly or in part, by an independent organization under contract to the contractor.

The engineer will make no partial payments for materials that are subject to specific quality control requirements without an approved Quality Control Plan.

1.8.2 Requirements of Quality Control Plan

The Quality Control Plan shall, as a minimum, address the items contained in AASHTO R 18 and ASTM D 3666 as briefly summarized below.

1.8.2.1 Personnel

The Quality Control Plan shall provide an organization chart defining the area of responsibility and authority of each individual. This organization chart shall show all quality control personnel by name, function, and experience, and how these individuals integrate with other management, production and construction functions, and workforce.

The names and qualifications of personnel shall be provided in the Quality Control Plan. The plan will indicate the total staff required to implement all elements of the quality control programs, including inspection, testing, and reporting functions.

If an outside organization or an independent laboratory is used for implementation of all or part of the Quality Control Plan, the personnel assigned are subject to the qualification requirements of this section. The Quality Control Plan shall indicate on the organization chart which personnel are contractor employees and which are provided by an outside organization and define the lines of reporting.

The Quality Control Manager shall be defined in the Quality Control Plan and is responsible for the successful operation of the plan to ensure compliance with the specifications. The Quality Control Manager reports directly to a responsible officer in the contractor's organization.

Quality Control Technicians shall be certified as defined in Section 1.6.3 and as shown in Tables C.2 through C.5 of this specification and shall perform the following functions:

- Inspect all plant equipment used in proportioning and mixing to ensure proper calibration and operating condition.
- Perform quality control tests necessary to adjust and control mix proportioning in accordance with the job mix formula.
- Inspect all equipment used in placing, finishing, and compacting material to ensure proper operating condition.
- Inspect during construction to ensure placing, joint construction, and compaction are in conformance with the specifications.
- Perform all quality control testing as required in this specification.
- Detail the criteria to be used to correct unsatisfactory production processes and construction practices.
- Perform all reporting required in the specification.

The Quality Control Manager and Quality Control Technicians shall by their actions address all elements that affect the quality of the HMA including the following:

- HMA mixture design.
- Aggregate gradation.
- Quality of materials.
- Aggregate stockpile management.
- Proportioning of HMA.
- Mixing of materials.
- Storage of materials.
- Transportation of materials.
- Placing, compaction, and finishing (joints, in-place density, smoothness, segregation, etc.).
- Sampling and testing.

1.8.2.2 Quality Control Testing Laboratory

The Quality Control Testing Laboratory shall be accredited by the AAP as defined in Section 1.6.1 of this specification. Testing laboratories conducting inspections or tests not covered by the AAP shall comply with the applicable requirements of AASHTO R 18 and ASTM D 3666.

1.8.2.3 Sampling

Random sampling techniques as defined in ASTM D 3665 and Attachment A to this specification shall be used.

1.8.2.4 Records and Control Charts

The contractor shall record all sampling, testing, and inspection data on forms as defined in the Quality Control Plan. The contractor shall maintain complete testing and inspection records and post all test data in the Quality Control Laboratory within 2 hours of completion of the daily testing as defined in the approved Quality Control Plan.

Mean and range control charts shall be prepared by the contractor and shall be posted on a daily basis. Attachment B contains methods for preparing mean and range control charts.

1.8.2.5 Sample Management

The Quality Control Plan shall contain a description of procedures for sample identification, storage, retention, and disposal.

1.8.2.6 Internal Quality Assurance System

The Quality Control Plan shall contain a description of the contractor internal quality assurance system. This system shall contain, as a minimum, inspections, testing of proficiency samples, and other activities as defined in AASHTO R 18.

CHAPTER 2—MATERIALS

2.1 ASPHALT BINDER

The asphalt binder shall meet the requirements of AASHTO MP 1 (Specification for Performance-Graded Asphalt Binder as shown in Table C.6).

2.2 AGGREGATE

The coarse and fine aggregate shall meet the requirements for aggregates as defined in AASHTO MP 2 Specification for Superpave Volumetric Design as shown in Tables C.7 through C.10.

2.2.1 Coarse Aggregate

Coarse aggregate shall be retained on the 4.75-mm (No. 4) sieve and shall consist of clean, hard, durable particles and shall be free from frozen lumps, deleterious matter, and harmful coatings.

2.2.2 Fine Aggregate

Fine aggregate shall be passing the 4.75-mm (No. 4) sieve and shall consist of clean, hard, durable particles and shall be free from frozen lumps, deleterious matter, and harmful coating.

2.3 MINERAL FILLER

Mineral filler shall meet the requirements of AASHTO M 17 or ASTM D 242 (Mineral Filler for Bituminous Paving Mixtures).

2.4 LIME

Hydrated lime shall meet the requirements of AASHTO M 303 or ASTM C 1097 (Lime for Asphalt Mixtures).

2.5 BAGHOUSE FINES

The addition of fines collected by a baghouse in the HMA plant is permitted when a metering system is provided. The baghouse fines shall be considered as part of the aggregate for gradation purposes and gradation compliance.

2.6 RECYCLED ASPHALT PAVEMENT

Recycled asphalt pavement (RAP) is defined as salvaged, milled, pulverized, broken, or crushed asphalt-bound pavement. RAP is allowed in HMA provided all requirements contained in this specification are met.

CHAPTER 3—CONSTRUCTION

3.1 GENERAL

The contractor shall maintain all equipment for the handling of materials, mixing, hauling, placing, and compaction of the mixture in good repair and operating condition to produce a quality product.

3.2 STOCKPILING, STORING, FEEDING, AND DRYING MATERIALS

3.2.1 Stockpiling of Aggregate(s)

Prior to stockpiling aggregates the contractor shall clear the area of trash, weeds, and grass. The area shall be relatively smooth and well drained.

The contractor shall perform stockpiling in a manner that will minimize aggregate degradation, segregation, and/or mixing of one stockpile with another.

The contractor shall not allow foreign material to contaminate the stockpile(s).

The contractor shall separate aggregates proportioned prior to the heating and drying process into a minimum of two sizes, one coarse and one fine.

The contractor shall keep all stockpiles separate.

3.2.2 Feeding and Drying of Aggregate

If applicable, the contractor shall feed the various sizes of aggregate through the cold aggregate bins and a proportioning device that will provide a uniform and constant flow of material in the required proportions.

3.2.3 RAP Feed System

If RAP is used, the contractor shall introduce the RAP through a separate cold feed bin that has adequate controls to provide a uniform and consistent flow of material in the required proportion.

3.2.4 Storing and Heating Asphalt Binders

The contractor shall equip tanks for storing asphalt binders and for heating and holding the asphalt binder at the required temperatures.

The contractor shall keep all equipment in a clean condition at all times and operate it in such a manner that there shall be no contamination with foreign material.

The contractor shall store asphalt binder in tanks separate and apart from the dryer burner fuel. Tanks with separate compartments that (1) may be used to store two or more products in a single enclosure or casing and (2) that share any common components (bulkheads, heating coils, valves, etc.) that would allow either product to contaminate the other will not be permitted for asphalt cement storage unless all of the compartments are filled with the same product.

The contractor shall equip the heating apparatus with a continuous recording thermometer with a 24-hour chart that shall record the temperature of the asphalt binder at the location of the highest temperature. This thermometer shall be in operation during production of the HMA.

The temperature of the asphalt binder just prior to mixing shall be that defined by AASHTO PP 28 for laboratory preparation of HMA samples.

When modified asphalt binders are used, the material shall be stored and maintained in storage tanks as recommended by the supplier.

The contractor shall equip tanks to allow for measuring the quantities of asphalt binder remaining in the tank.

The circulating system shall provide proper and continuous circulation during the operating period.

The contractor shall provide a sampling port in the feed line between the asphalt binder plant storage tank and the mixing chamber.

3.2.5 Feeding Mineral Filler and Baghouse Fines

Mineral filler and/or baghouse fines shall be drawn from a storage facility in which the mineral filler is agitated by air or other means to keep it in a uniform free flowing condition. The mineral filler and/or baghouse fines shall be delivered to the mixer from a vane type metering device which is interlocked (electric-driven feeders shall be actuated from the same circuit) to the flow of each aggregate feeder. The drive shaft on the vane feeder for the mineral filler, baghouse fines, or both shall be equipped with a revolution counter reading to one-tenth of a revolution and a means for varying the rate.

3.2.6 Modifiers/Additives

All storage and feeding systems for modifiers and additives must be approved by the engineer.

3.3 MIXING PLANTS

The contractor shall use mixing plants of either the weigh-batch, continuous mixing (proportioning after drying), continuous mixing (proportioning before drying), or drum mixing type.

3.3.1 Burner Fuel and Burner

The contractor shall limit fuel used for heating aggregates to the following types: natural gas, liquefied natural gas, fuel oil (ASTM D 396, grades No. 1 and No. 2), butane, propane, and diesel fuel oil (ASTM D975, grades No. 1-D and No. 2-D).

The contractor shall certify that burner fuels comply with the above requirements.

The contractor shall ensure that the burner used for heating the aggregate shall achieve complete combustion of the approved fuel and not leave any fuel residue that will adhere to the heated aggregate.

3.3.2 Requirements for All Plants

The contractor shall install and maintain adequate equipment and take necessary precautions to meet applicable federal, state, and local government air quality and water quality regulations.

The contractor shall install or have available platform scales to determine the weights of HMA and hauling vehicles.

The contractor shall provide adequate and safe access to the top of the truck bodies by a platform or other suitable access ports in the truck bodies for the engineer to obtain the mix temperature.

The contractor shall provide adequate and safe equipment at the plant to afford access for plant operation, maintenance, sampling, and calibration.

The contractor shall calibrate the plant in accordance with Quality Control Plan.

3.3.3 Batch Plant

The contractor shall use a fully automated and computerized batch plant.

The contractor shall provide a twin shaft pugmill-type batch mixer that is steam-jacketed or heated by other approved methods and operated to produce a uniform mixture.

The contractor shall equip the mixer with an accurate time lock to control the operations of a complete mixing cycle with an accuracy of 2 sec.

3.3.4 Continuous Mixing Plants (Proportioned After Drying, Proportioned Before Drying, Drum Mixer)

The contractor shall provide a fully automated and computerized plant with satisfactory means to provide a positive interlocking control between the flow of each feeder and the flow of the asphalt binder, mineral filler, and baghouse fines.

The contractor shall provide the plant with a continuous mixer, adequately heated and operated to produce a uniform mixture with a uniform coating of asphalt binder on the aggregate at the discharge.

3.4 SURGE-STORAGE SYSTEM

3.4.1 Capacity

The contractor shall equip all continuous and drum mixer plants with a surge-storage system having a capacity in excess of 18 Mg (20 tons).

3.4.2 Segregation

The contractor shall equip the surge-storage system with an approved surge batcher or other approved method that will prevent segregation of the HMA as it is being stored or discharged into the hauling vehicle.

3.4.3 Storage

Temporary storage or holding of HMA during the production day will be allowed. Storage for periods exceeding 24 hours will not be permitted unless authorized.

3.5 HAULING EQUIPMENT

3.5.1 Trucks

The contractor shall use trucks for hauling HMA with tight, clean, smooth metal beds which have been thinly coated with a minimum amount of lime solution or other approved release agent to prevent the mixture from adhering to the truck beds. The contractor shall not use diesel or kerosene as a release agent.

3.5.2 Discharge into Hauling Vehicles

The contractor shall discharge the HMA from the surge-storage system directly into the hauling vehicle.

3.6 SURFACE PREPARATION

3.6.1 Utilities and Drainage Structure

The contractor shall locate, reference, and protect all utility covers, monuments, curbs and gutters, and other items affected by the paving operations.

3.6.2 Objectionable Material

The contractor shall remove all dirt, sand, leaves, and other objectionable material from the prepared surface before placing the HMA.

3.7 PLACING

3.7.1 Equipment

When the HMA is being produced by more than one HMA mixing plant, the contractor shall place the material produced by each plant with separate spreading and compacting equipment.

The contractor shall dump and spread the HMA on the prepared surface with the spreading and finishing machine.

If window pick-up equipment is used, the contractor will provide equipment capable of removing and loading substantially all of the mixture deposited on the roadbed into the spreading and finishing machine.

The contractor shall use bituminous pavers that are self-contained, power-propelled units, provided with an activated screed or strike-off assembly, heated if necessary, and capable of spreading and finishing courses of HMA in lane and shoulder widths applicable to the specified typical section and thicknesses shown on the plans.

The contractor shall equip pavers with a receiving hopper having sufficient capacity for a uniform spreading operation and a distribution system to place the mixture uniformly in front of the screed.

The contractor shall equip the screed with automatic controls which will make adjustments in both transverse and longitudinal directions.

3.7.2 Operation

The contractor shall provide a placing operation to provide a smooth, uniform textured surface without tearing, shoving, gouging, segregation, or streaks.

3.8 JOINTS

3.8.1 Longitudinal

The contractor shall offset longitudinal construction joints of successive courses of HMA at least 150 mm (6 in.).

The contractor shall place the HMA so that any longitudinal joints constructed are within 300 mm (12 in.) of the final traffic lane lines.

3.8.2 Transverse Joints

The contractor shall place additional HMA to provide a temporary 1 : 50 transition at the end of placement.

The contractor shall form transverse joints by removing the temporary transition material, exposing the full depth of the previous layer, and forming a clean, vertical edge.

The contractor shall place a brush coat of asphalt emulsion on the contact surface of the joint before any additional mixture is placed.

3.9 COMPACTION

3.9.1 General

The contractor shall compact the pavement thoroughly and uniformly, with the necessary equipment, to obtain the density and cross-section of the finished paving mixture, meeting the requirements of the plans and specifications.

The contractor shall thoroughly compact the edges of pavement not accessible to conventional rollers with suitable types of tampers, plates, trench rollers, etc.

The contractor shall commence initial rolling at the lower edge and progress towards the highest portion of the roadbed.

The contractor shall perform rolling in a manner that cracking, shoving, or displacement are avoided.

The contractor shall use rollers in good condition and capable of rolling and changing direction without adversely affecting the mat.

The contractor shall properly moisten the wheels of the rollers to prevent adhesion of the HMA. The contractor shall not use diesel or kerosene for this purpose.

3.9.2 Specified Air Void Requirements

The contractor shall compact the pavement to conform to the specified in-place air void requirements.

3.10 RESTRICTIONS

3.10.1 Frozen Base

The contractor shall not place the HMA when frozen materials are present in the base.

3.10.2 Adverse Weather

The contractor shall not place the HMA during rain or snowfall or when the roadway is wet.

CHAPTER 4—MIXTURE DESIGN

4.1 GENERAL

4.1.1 Description

HMA shall be a uniform mixture of asphalt binder, aggregate and/or mineral filler, recycled asphalt pavement, additives, modifiers, and baghouse fines.

4.1.2 Responsibility

The mixture design shall be the responsibility of the contractor.

4.1.3 Required Information

The information describing the mixture design shall be submitted by the contractor and contain all information obtained during the mixture design process as defined in AASHTO PP 28 (Superpave Volumetric Design for Hot-Mix Asphalt).

4.2 REQUIREMENTS

4.2.1 Asphalt Binder

The asphalt binder grade shall meet the requirements of AASHTO MP 1 (Performance-Graded Asphalt Binder) (Table C.6) and shall conform to the grade as shown on the plans. The asphalt binder grade will be selected with consideration given to AASHTO MP 2 (Superpave Volumetric Mix Design) requirements.

The asphalt binder samples for mixture design purposes and for approval of the mixture design by the engineer shall be obtained by the contractor. The asphalt binder shall be sampled from the refinery or contractor's storage plant in accordance with AASHTO T 40 and shall be representative of the asphalt binder that will be used during construction of the project. The asphalt binder will be tested by the engineer according to Table C.11 and AASHTO PP 6.

4.2.2 Aggregate

The aggregates shall meet the requirements of AASHTO MP 2 (Superpave Volumetric Mix Design (Tables C.7 through C.9) and the source properties shown in Table C.10, and they shall conform to the gradation as shown on the plans.

Aggregate samples for mixture design purposes and for approval of the mixture design by the engineer shall be obtained by the contractor. The aggregate samples shall not be obtained until a minimum of 4,550 Mg (5,000 tons) or 25 percent of the required contract quantity of aggregate has been proportionately produced (whichever is less) and placed in stockpiles. The sampled aggregate shall be representative of the aggregate to be used during construction of the project. The aggregate will be tested by the engineer according to Table C.12.

4.2.3 Mixture Design

The HMA mixture design shall be performed in accordance with AASHTO PP 28 (Superpave Volumetric Design for Hot-Mix Asphalt) with the compactive effort defined by use of Table C.13.

4.2.4 Mixture Properties

The designed HMA shall meet the mixture properties contained in AASHTO PP 28 (Superpave Volumetric Design for HMA) and shown in Table C.14.

4.3 SUBMITTALS

4.3.1 Materials

Split samples of the asphalt binder used by the contractor for mixture design purposes shall be submitted to the engineer a minimum of 30 working days prior to the start of the HMA production. The asphalt binder shall be supplied in four, 0.95-L (1-qt) containers of the "paint can type."

Split samples of aggregates used by the contractor for mixture design purposes shall be submitted to the engineer a minimum of 30 working days prior to the start of the HMA production.

The contractor shall submit the HMA mixture design 15 working days prior to the start of the HMA production. This submittal shall contain asphalt binder, aggregate, and HMA mixture properties as defined in Section 4.2 of this specification.

4.4 APPROVAL

4.4.1 Responsibility

The approval of the laboratory mixture design will be the responsibility of the engineer.

4.4.2 Basis of Approval

Approval of the mixture design will be based on the engineer's test results for the asphalt binder and the aggregate and a "paper" review of the submitted mixture design. The asphalt binder and the aggregate must meet the requirements of AASHTO PP 1 (Performance-Graded Asphalt) (Table C.6) and AASHTO MP 2 (Superpave Volumetric Mix Design) (Tables C.7 through C.10).

4.4.3 Time Required for Approval

The laboratory HMA mixture design will be approved or disapproved within 5 working days of submittal.

4.4.4 Mixtures Not Approved by the Engineer

If the furnished materials or mixture design are not approved by the engineer, the contractor shall resubmit new materials or a new mixture design after amending, correcting, or developing a new mixture design or obtaining new materials. The approval process will start over with the submittal of the new mixture design.

4.5 NEW LABORATORY MIX DESIGN

4.5.1 Asphalt Binder Source

If the asphalt binder source or grade changes, the contractor shall submit a new mixture design.

4.5.2 Aggregate Source

If the aggregate source changes or the aggregate material characteristics change significantly within the source, the contractor shall submit a new mixture design.

4.5.3 Number of New Mix Designs

New mixture designs shall be submitted as frequently as necessary to produce a quality HMA pavement.

4.5.4 Approval

All new mixture designs are subject to the approval process defined in Section 4.4 of this specification.

CHAPTER 5—JOB MIX FORMULA

5.1 DESCRIPTION

The job mix formula shall include single values for the following:

- Percentage by weight of aggregate passing each specified sieve size (whole number percent).
- Bin percentage of each aggregate used (whole number percent).
- Percentage of asphalt binder by total weight of HMA (0.1 percent).
- Percentage of baghouse fines by dry weight of aggregate (0.1 percent).
- Percentage of mineral filler by dry weight of aggregate (0.1 percent).
- Gyratory compacted weight-volume values at design asphalt binder content.
  - Air voids (0.1 percent).
  - Voids in mineral aggregate (0.1 percent).
  - Voids filled with asphalt (0.1 percent).
  - Dust-to-binder ratio (0.1 percent).
  - Bulk specific gravity of compacted mixture (0.001).
  - Theoretical maximum specific gravity of HMA (0.001).
  - Temperature for mixing and compaction for laboratory and field operations (whole number °C).

5.2 JOB MIX FORMULA 1 (LABORATORY MIXTURE DESIGN)

The approved laboratory mixture design (Section 4 of these specifications) shall be job mix formula 1 (JMF 1). JMF 1 shall be the targets for aggregate gradation and asphalt binder content for the “Field Trial Section.”

5.3 FIELD TRIAL SECTION(S)

5.3.1 Period of Production

On the first day of HMA production, three trial mixtures of a minimum of 455 Mg (500 tons) each shall be produced. The trial mixtures shall be produced at the medium hot-mix plant speed used during the calibration of the plant. The trial mixtures may be placed on the shoulder of the roadway, in a passing lane on a multilane highway, or in another location approved by the engineer. The field trial sections shall become a section of the completed roadway. Production of HMA shall be suspended for a maximum of 3 working days or until the production job mix formula has been approved. Working days will not be charged during the work suspension period.

5.3.2 Composition of Field Trial Mixtures

JMF 1 shall be the target for the Field Trial Mixture No. 1. Field Trial Mixture No. 2 shall conform to JMF 1, except the asphalt binder content shall be 0.4 percent above the target asphalt binder content of JMF 1. Field Trial Mixture No. 3 shall conform to JMF 1, except the asphalt binder content shall be 0.4 percent below the target asphalt binder content of JMF 1. The contractor may request the placement of additional trial mixtures. The placement of additional trial mixtures must be approved by the engineer.

5.3.3 Approval of Field Trial Mixtures

A loose sample of HMA shall be obtained by the contractor from behind the laydown machine prior to compaction from each 91 Mg (100 tons) of HMA placed. The sample will be split by the contractor into three, approximately equal portions suitable for mixture testing as outlined below. The engineer will receive and test one portion of the sampled mixture. The contractor shall test the second portion of the sampled mixture. The third portion of the sampled mixture will be retained for referee testing if required.

The loose mixture will be tested by the engineer and shall be tested by the contractor to determine conformance to JMF 1 as determined by Tables C.14, C.15, and C.16. For each of the three Field Trial Mixtures, a minimum of 5 gradations (AASHTO T 27), asphalt binder contents (ASTM D 6307), theoretical maximum specific gravity (AASHTO T 209), and mixture volumetric (AASHTO TP 4 and PP 28) values will be reported. The gyratory-compacted weight-volume properties reported (air voids, voids in mineral aggregate, voids filled with asphalt, and bulk specific gravity of compacted samples) will be the average of three compacted samples for each of the 91 Mg (100 tons) of HMA sampled.

5.3.4 Comparison of Contractor's and Engineer's Test Results

For each of the three Field Trial Mixtures, a comparison shall be made between the contractor's and engineer's test results using the t-test (alpha = 0.01) described in Attachment D. Field Trial Mixture No. 1 shall be used initially to perform the comparison. If the contractor's and engineer's test results are not determined to be statistically similar for Field Trial Mixture No. 1, an investigation will be performed to determine the reason for the difference(s). After appropriate changes are made, Field Trial Mixture No. 2 will be tested and evaluated. If the contractor's and engineer's test results are not determined to be statistically similar for Field Trial Mixture No. 2, an investigation will be performed to determine the reason for the difference(s). After appropriate changes are made, Field Trial Mixture No. 3 will be tested and evaluated. If the contractor's and engineer's test results are not determined to be statistically similar for Field Trial Mixture No. 3, an investigation will be performed to determine the reason for the difference(s). The contractor and engineer will perform an agreed upon test program on laboratory prepared samples to resolve differences between test results. If this laboratory test program does not resolve the differences, a referee testing program will be initiated. At the conclusion of this effort, the contractor's and engineer's test results are to be statistically similar in value as determined by Attachment D.

5.3.5 Acceptance of Field Trial Section

Acceptance of the Field Trial Section will be based on the engineer's test results. The acceptable mixture shall meet the requirements of Table C.14 and shall have an acceptable quality level (percent within limits [PWL]) of 90 percent within the JMF 1 limits as determined by Attachment C with the tolerances shown in Tables C.15 and C.16 for aggregate gradation and weight-volume properties. The asphalt binder content may vary from the limits shown in Table C.15 to provide the desired weight-volume properties.

5.3.6 Pay Adjustment Factor

The pay adjustment factor for the field trial sections will be 1.00.

5.4 JOB MIX FORMULA 2 (FIELD TRIAL SECTION)

JMF 2 shall be determined based on the results of the Field Trial Sections. If none of the three Field Trial Sections placed (as described in Section 5.3 of the specification) meet the acceptance requirements of Section 5.3.5 of this specification, additional field trial sections shall be placed until the requirements of Section 5.3 are met or a new mixture design is prepared by the contractor and approved by the engineer (Section 4 of the specification).

JMF 2 is described in Section 5.1 of this specification.

5.5 SECOND DAY PRODUCTION

The second day of production shall be based on JMF 2. Pay adjustment factors will be calculated based on JMF 2. Pay adjustment factors will be applied to the second day of production and thereafter.

5.6 JOB MIX FORMULA ADJUSTMENT

5.6.1 Changes Allowed

If adjustments to the JMF 2 are needed during production to meet the specification requirements or to maximize the quality of the mixture, the JMF shall be adjusted prior to the start of the production of the lot. This new JMF shall be identified as JMF 3. The contractor shall notify the engineer of these changes. A lot may be terminated at the end of a sublot if requested by the contractor and approved by the engineer. The pay factor for the early terminated lot will be based on the tests obtained for the sublot(s) produced.

5.6.2 Acceptance of Change

Changes in asphalt binder content and aggregate gradation "single values" will be allowed provided the changes do not exceed the specification limits as defined in Tables C.7 and C.8 and the tolerances shown in Table C.17 which are based on JMF 1.

The contractor shall obtain five samples of loose mixture behind the paver during the production of the first lot produced with JMF 3 as target values (Field Trial Section). The gyratory-compacted weight-volume requirements shown in Table C.14 shall be met. The mixture shall meet the acceptable quality level of 90 percent within JMF 2 limits as determined by Attachment C with the tolerances shown in Table C.16 for air voids, voids in mineral aggregate, voids filled with asphalt, and dust-to-binder ratio.

5.6.3 Job Mix Formula 3

JMF 3 shall be based on the results of the Field Trial Section placed as described in Section 5.6.2 of the specification. JMF 3 is described in Section 5.1 of this specification.

5.6.4 New Job Mix Formulae

Additional job mix formulae will be allowed provided they meet the requirements contained in Section 5.6 of this specification.

5.7 COMPACTION ROLLING PATTERN

During placement of the Field Trial Sections, the contractor shall establish a roller pattern(s) to ensure that the produced HMA meets the in-place air void requirements of 99.7 percent within the limits of 2 to 7 percent air voids as determined by AASHTO T 269. An acceptable roller pattern must be established prior to the start of “day two” production.

CHAPTER 6—LOT AND SUBLOT

6.1 GENERAL

Acceptance of the HMA will be based on the acceptance of lot quantities. Random sampling of asphalt binder, aggregate, and the HMA shall be performed on a lot and sublot basis according to Attachment A. Lots and sublots shall be established for asphalt binder, aggregate, HMA production and placement, and ride quality as defined below. Lots can be terminated at any time with the approval of the engineer.

6.2 ASPHALT LOT/SUBLOT

6.2.1 Definition

An asphalt binder lot is equal to the quantity of asphalt binder in 2,000 tons of HMA.

6.2.2 Incomplete Production Lots

If a lot is begun and cannot be completed due to the end of the project, an incomplete lot is created. The test results from this incomplete lot will be combined with the previous lot.

6.2.3 Small Production Quantities

When the anticipated daily production is less than 455 Mg (500 tons) of HMA, the engineer may elect either to waive all sampling and testing for that day or to follow Section 6.2.2 of this specification. If the engineer elects to waive sampling and testing, the pay adjustment factor for the asphalt binder will be 1.00.

6.3 AGGREGATE LOT/SUBLOT

6.3.1 Definition

An aggregate lot is equal to the quantity of aggregate in 2,000 tons of HMA.

6.3.2 Incomplete Production Lots

If a lot is begun and cannot be completed due to the end of the project, an incomplete lot is created. The test results from this incomplete lot will be combined with the previous lot.

6.3.3 Small Production Quantities

When the anticipated daily production is less than 455 Mg (500 tons) of HMA, the engineer may elect either to waive all sampling and testing for that day or to follow Section 6.3.2 of this specification.

6.4 HOT-MIX ASPHALT PRODUCTION LOT/SUBLOT

6.4.1 Definition

A production lot shall consist of four equal sublots. The production/placement sublot shall be 455 Mg (500 tons).

6.4.2 Incomplete Production Lots

If a lot is begun and cannot be completed due to weather, equipment breakdown, end of the project, or other circumstances, an incomplete lot is created. The sublot test results from this incomplete lot will be combined with the previous production lot or the next lot produced. If two or fewer sublot test results are available from the incomplete lot, these test results will be combined with the previous production lot. If three or more sublot test results are available from the incomplete lot, these test results will be combined with the next lot produced.

6.4.3 Small Production Quantities

When the anticipated daily production is less than 455 Mg (500 tons), the engineer may elect either to waive all sampling and testing requirements or to follow Section 6.4.2 of this specification. If the engineer elects to waive sampling and testing, the pay adjustment factor will be 1.00.

6.5 HOT-MIX ASPHALT PLACEMENT LOT/SUBLOT

6.5.1 Definition

A placement lot shall consist of the area of HMA placed in a production lot, excluding miscellaneous areas. A placement sublot shall consist of one-fifth of the area of the placement lot.

6.5.2 Incomplete Placement Lots

An incomplete placement lot shall consist of the area placed in an incomplete production lot as described in Section 6.4.2 of this specification, excluding miscellaneous areas. For these incomplete placement lots, one placement sample location shall be selected for each production sublot placed and the test results combined with the previous lot or the next lot produced as defined in Section 6.4.2 of this specification.

6.5.3 Miscellaneous Areas

Areas that are not generally subject to primary traffic such as driveways, mailbox turnouts, crossovers, gores, and other similar areas are considered to be miscellaneous areas. Shoulders and ramps are not considered miscellaneous areas. Miscellaneous areas are the only areas that are not eligible for random placement locations and will be assigned a pay factor of 1.00.

6.5.4 Shoulders and Ramps

Shoulders and ramps are not subject to in-place air voids determination, unless otherwise shown on the plans. When shoulders and ramps are not subject to in-place air voids determination, the compaction shall be in accordance with roller patterns established in Section 5.7 of this specification. The contractor may declare the shoulders and/or ramps as eligible for in-place air void testing and pay adjustments; however, the contractor must notify the engineer in writing prior to beginning of the mix production. The engineer must approve this request.

6.5.5 Level-Ups and Thin Overlays

For the purpose of calculating placement pay adjustment factors, level-ups and thin overlays will be considered as miscellaneous areas and will be assigned a placement pay factor of 1.00. The placement pay adjustment factor will be 1.00 for any layer thickness designated on the plans less than 38 mm (1.5 in.) or for level-up areas. The contractor shall establish a rolling pattern that shall achieve in-place air voids in accordance with the roller patterns established in Section 5.7 of this specification.

6.6 RIDE QUALITY LOT

A ride quality lot is equal to 1.6 lane-km (1 lane-mi). A lane-km is defined as a kilometer length of a mainline travel lane shown on the permanent striping plan. Pay factors for ride quality are based on a lot; therefore, the definition of a sublot is not needed.

CHAPTER 7—QUALITY CONTROL

7.1 QUALITY CONTROL SAMPLING AND TESTING FOR ASPHALT BINDERS

7.1.1 Sampling

Stratified random sampling procedures shall be used as described in Attachment A. For every 2,000 tons of HMA produced, a single asphalt binder sample shall be taken at randomly determined times from the feed line located between the contractor's storage tank and plant mixing chamber according to AASHTO T 40. The samples shall be split into three individual 0.95 L (1-qt) cans. The samples shall be obtained by the contractor and witnessed by the engineer. One of the split samples will be tested by the engineer for quality assurance purposes as shown in Table C.18. The second split sample shall be held by the contractor and can be used for testing. The third sample will be retained by the engineer for referee testing.

7.1.2 Lots and Sublots

Every 2,000 tons of HMA produced shall constitute an asphalt binder lot (represented by one random sample). The quantity of binder represented by a lot may vary.

7.1.3 Quality Control Testing (optional)

The requirements contained in AASHTO PP 26 shall be the quality control testing requirements for the asphalt binder. Additional quality control testing by the contractor is at the contractor's discretion. The contractor is encouraged to perform the high temperature Dynamic Shear Rheometer (AASHTO TP 5) tests on original and rolling thin film oven (AASHTO T 240) aged samples obtained as described in Section 7.1.1.

7.1.4 Comparison of Quality Control and Quality Assurance Tests

Split samples of asphalt binder tested by the contractor for quality control and the engineer for quality assurance will be within the allowable differences shown in Table C.19. If the split sample data are not within these allowable differences, an immediate investigation shall be conducted to determine the cause(s) of the differences. Unless available facts indicate otherwise, the investigation shall include a review of sampling and testing procedures used by both the contractor and engineer.

7.2 QUALITY CONTROL SAMPLING AND TESTING FOR AGGREGATES

7.2.1 Sampling

Stratified random sampling procedures shall be used as described in Attachment A. For every 2,000 tons of HMA produced, a single aggregate sample shall be taken at randomly determined times from the combined cold feed according to AASHTO T 2 for each lot. The sample shall be split into three individual samples of sufficient size for performing the tests shown in Table C.20. The samples shall be obtained by the contractor and witnessed by the engineer.

One of the split samples shall be tested by the contractor for quality control purposes and one sample will be tested by the engineer for quality assurance purposes as described in Table C.20. The third sample will be retained by the engineer for referee testing.

7.2.2 Lots and Sublots

Every 2,000 tons of HMA produced shall be constituted an aggregate sublot (represented by one random sample). The quantity of aggregate represented by a lot may vary.

7.2.3 Quality Control Testing

The requirements shown in Table C.20 shall be the quality control testing requirements for the aggregate. The Coarse Aggregate Angularity (ASTM D 5821), Fine Aggregate Angularity (AASHTO T 304), Flat and Elongated Particles (ASTM D 4791), Sand Equivalent (AASHTO T 176), and Deleterious Materials (AASHTO T 112) tests shall be performed on each sublot sample. Los Angeles Abrasion (AASTO T 96) and soundness tests (AASHTO T 104) shall be performed on each lot from a randomly selected sample.

7.2.4 Comparison of Quality Control and Quality Assurance Tests

Split samples of aggregate tested by the contractor for quality control purposes and by the engineer for quality assurance purposes shall be within the allowable differences shown in Table C.21. If the split sample data are not within these allowable differences, an immediate investigation shall be conducted to determine the cause(s) of the differences. Unless available facts indicate otherwise, the investigation shall include a review of sampling and testing procedures used by both the contractor and the engineer.

7.2.5 Quality Control Requirements

The aggregate quality control process shall be considered in control if two general criteria are met: (1) that the aggregate meet the requirements of Tables C.7 through C.10 for each sample tested and (2) that the test results obtained by the contractor for quality control and the engineer for quality assurance are within the differences shown in Table C.21 as described in Section 7.2.4.

7.3 QUALITY CONTROL SAMPLING AND TESTING FOR HMA PRODUCTION

7.3.1 Sampling

Stratified random sampling procedures shall be used as described in Attachment A. A sample of HMA shall be obtained at randomly determined points from behind the paver prior to compaction for each sublot according to AASHTO T 168. The sample shall be split into three individual samples of sufficient size for performing the tests shown in Tables C.22 and C.23. The samples shall be obtained by the contractor and witnessed by the engineer. One of the samples shall be tested by the contractor for quality control purposes and one sample will be tested by the engineer for quality assurance purposes as described in Tables C.22 and C.23. The third sample will be retained by the engineer for referee testing.

7.3.2 Lots and Sublots

A production lot shall consist of four equal sublots. A production sublot shall be 455 Mg (500 tons) (Section 6.4).

7.3.3 Quality Control Testing

The requirements shown in Tables C.22 and C.23 shall be the quality control testing requirements for HMA production. The asphalt binder content shall be determined by the Ignition Method (ASTM D 6307) and the Aggregate Gradation by AASHTO T 27.

The loose HMA shall be compacted by the Superpave Gyratory Compactor (AASHTO TP 4) and the following weight-volume parameters determined at N_{design} number of gyrations: Bulk Specific Gravity (AASHTO T 166), Theoretical Maximum Specific Gravity (AASHTO T 209), Air Voids (AASHTO T 269), Voids in Mineral Aggregate (AASHTO PP 28), Voids Filled with Asphalt (AASHTO PP 28), and Dust-to-Binder Ratio (AASHTO PP28).

The aggregate gradation shall be determined with those sieves used to specify the gradation for mixture design purposes (Tables C.7, C.8, and C.23). The average of three Superpave gyratory-compacted samples shall be reported as the mixture volumetrics parameters from each sublot sample of HMA.

7.3.4 Quality Control Requirements

The contractor shall determine and record the following in a daily summary: quantities of asphalt binder, aggregate, mineral filler, and fibers (if required) used; the quantities of HMA produced; the HMA production and compaction temperatures (hourly basis as a minimum); and the results of the testing shown in Tables C.22 and C.23.

The HMA production process shall be considered in control if two general criteria are met. The first general criterion requires that the asphalt binder content, aggregate gradation, and the field-mixed, laboratory-compacted weight-volume parameters identified in Table C.22 be within the limits shown in Tables C.15 and C.16 for each sublot. The target values used to determine compliance are those for the last approved JMF (JMF 2, JMF 3, etc.).

The second general criterion requires that the contractor meet the requirements associated with statistical control charts as outlined below. The contractor shall use statistical control charts to determine if the variability in HMA properties and/or variability is due to random causes or assignable causes.

Statistical control charts shall be prepared for aggregate gradation on the 2.36-mm (No. 8) and 0.075-mm (No. 200) sieves, asphalt binder content and gyratory-compacted air void contents. Target values and upper and lower control limits for the control charts are determined from the Field Trial Sections (Section 5.0) and the first few days of production. The initial production test results that are used to develop these statistical control charts shall agree with the production results obtained during the Field Trial Sections (Section 5.0).

The grand mean and average range of these test data shall be used to develop x-bar (mean) and R (range) control charts for each property: gradation on the 2.36-mm (No. 8) and 0.075-mm (No. 200) sieves, asphalt binder content, and gyratory-compacted air void content according to Attachment B. The upper and lower control limits shall be set at plus or minus 2 times the standard deviation and plus or minus 3 times the standard deviation, defined as the warning and action control limits, respectively. If the warning and action control limits are not within the allowable JMF tolerances (Tables C.15 and C.16) for the approved JMF, the contractor shall modify the HMA production process to reduce the variability and bring the control limits within the tolerances.

Eight consecutive plotted points on one side or the other of the target value, or one point outside the warning or action limit, indicates an HMA mixture compositional change. If any one or more of these conditions occurs, the next sublot shall be immediately tested. If the next sublot test result indicates noncompliance with the above-stated criteria, the contractor shall adjust the asphalt binder content, aggregate gradation, or both to provide mixture compliance. The mixture produced after these changes have been made shall be subject to the evaluation process identified in Section 5.6.

7.3.5 Comparison of Quality Control and Quality Assurance Tests

Split production samples of HMA tested by the contractor for quality control and by the engineer for quality assurance and pay factor shall be compared by the Student t-test, as described in Attachment D, with alpha equal to 0.10. Comparisons are possible for gradation on the 2.36-mm (No. 8) and 0.075-mm (No. 200) sieves, asphalt binder content, gyratory-compacted air voids and bulk specific gravity, and theoretical maximum specific gravity (Table C.22). If the results do not compare favorably for a given lot, the reason(s) should be immediately determined and resolved. Unless available facts indicate otherwise, the investigation shall include a review of sampling and testing procedures used by both the contractor and the engineer.

7.4 QUALITY CONTROL SAMPLING AND TESTING FOR HMA PLACEMENT

7.4.1 Sampling

Stratified random sampling procedures shall be used as described in Attachment A. Nondestructive or destructive sampling and testing procedures may be used by the contractor to determine in-place air void content. When nondestructive measurements (ASTM D 2950) are used, a minimum of four locations shall be used in each sublot. When pavement core samples (ASTM D 5361) are used, a minimum of two locations shall be used in each sublot. When nondestructive testing is used, the device shall be calibrated with core samples to determine air void content. The field trial sections shall be used to determine roller patterns to obtain the desired in-place air voids (Section 5.7). In-place air void determinations shall not be made within 0.45 m (18 in.) of a longitudinal joint or within 1.5 m (60 in.) of a transverse joint.

7.4.2 Lots and Sublots

A placement lot shall consist of the area of HMA placed in a production lot, excluding miscellaneous areas as described in Section 6.5.3. A placement sublot shall consist of one-fifth of the area of the placement lot (Section 6.5).

7.4.3 Quality Control Testing

The requirements shown in Table C.22 shall be the quality control testing requirements for HMA placement (in-place air voids). The in-place air voids shall be determined from core samples according to AASHTO T 269 based on Bulk Specific Gravity determinations of core samples by AASHTO T 166 or T 275 and on Theoretical Maximum Specific Gravity according to AASHTO T 209. Theoretical specific gravity values determined by sublot for the quality control production testing (Section 7.3) shall be used. If in-place air voids are determined by use of the Nuclear Gage (ASTM D 2950), correlation between the gage and core samples shall be required.

7.4.4 Quality Control Requirements

The contractor shall determine and record the following in a daily summary: the amount of HMA delivered to the paver (truck loads and Mg [or tons] per truck), temperature of the HMA in the truck at the plant (hourly basis as a minimum), and temperature of the HMA in the mat prior to initial compaction (hourly basis as a minimum).

The HMA production process shall be considered in control if two general criteria are met. The first general criterion requires that the in-place air voids be within the limits shown in Table C.16 (2 to 7 percent) for each sublot (average of two samples if cores are used and average of four samples if nondestructive methods are used).

The second general criterion requires that the contractor meet the requirements associated with statistical control charts as outlined below. The contractor shall use statistical control charts to determine if the HMA properties, their variability, or both are due to random causes or assignable causes.

Statistical control charts shall be prepared for in-place air voids according to Attachment B. Target values and control limits for in-place air voids shall be based on information obtained during the placement of the Field Trial Sections (Section 5.0) and the first day's pavement construction. The first day's pavement construction test results that are used to develop these statistical control charts shall agree with the production results obtained during the Field Trial Sections (Section 5.0).

The grand mean and average range of these test data shall be used to develop x-bar (mean) and R (range) control charts for in-place air voids according to Attachment B. The upper and lower control limits shall be set at plus or minus 2 times the standard deviation and plus or minus 3 times the standard deviation, defined as the warning and action control limits, respectively. If the warning and action control limits are not within the allowable range of 2 to 7 percent (Table C.16), the contractor shall modify the HMA placement and perhaps the production process to reduce the variability and bring the control limits within the tolerances.

One test point outside the upper or lower warning control limit shall be considered an indication that the control of the laydown and compaction process may be unsatisfactory and shall require the contractor to confirm that the process parameters are within acceptable bounds by obtaining additional core samples.

One test point outside the upper or lower action control limits or eight consecutive test points on one side or the other of the target value shall be judged as a lack of control in the laydown and compaction process. The contractor shall stop production of the HMA until the assignable cause(s) for the lack of control is identified and remedied. The contractor shall report within 24 hours to the engineer the assignable cause(s) for the stop in production and the action taken to remedy the assignable cause.

7.4.5 Comparison of Quality Control and Quality Assurance Test

In-place air void determinations performed by the contractor for quality control testing and by the engineer for quality assurance (Table C.22) shall be compared by the Student t-test, as described in Attachment D, with alpha equal to 0.10. If the test results do not compare favorably for a given lot, the reason(s) should be immediately determined and resolved. Unless available facts indicate otherwise, the investigation shall include a review of sampling and testing procedures used by both the contractor and the engineer.

7.5 QUALITY CONTROL SAMPLING AND TESTING FOR RIDE QUALITY

Quality control testing for ride quality is not required. The contractor is encouraged to determine a ride quality evaluation according to ASTM E 1274 or other suitable method (Table C.5).

CHAPTER 8—QUALITY ASSURANCE

8.1 QUALITY ASSURANCE SAMPLING AND TESTING FOR ASPHALT BINDERS

8.1.1 Sampling

Stratified random sampling procedures shall be used as described in Attachment A. For every 2,000 tons of HMA produced, a single asphalt binder sample shall be taken at randomly determined times from the feed line located between the contractor's storage tank and the plant mixing chamber according to AASHTO T 40. The sample shall be split into three individual 0.95-L (1-qt) cans. The samples shall be obtained by the contractor and witnessed by the engineer. One of the split samples will be tested by the engineer for quality assurance and pay factor determination. The second sample shall be held by the contractor and can be used for testing. The third sample will be retained by the engineer for referee testing.

8.1.2 Lots and Sublots

An asphalt binder lot is equal to the quantity of asphalt binder in 1,800 Mg (2,000 tons) of HMA.

8.1.3 Quality Assurance Testing

The testing shown in Table C.18 will be the minimum quality assurance testing requirements for the asphalt binder.

8.1.4 Quality Assurance Requirements

The quality assurance tests will be used to determine compliance to the specification for performance-graded asphalt binder shown in Table C.6. If the sample tested does not meet the specification for the grade specified for the project, the engineer will immediately contact the contractor and testing will be initiated by the engineer on the other four sublot samples of asphalt binder. An investigation will be conducted to determine the cause(s) for not meeting the specification. The asphalt binder test results obtained by the contractor will be included in this investigation, as well as the results from Section 7.1.4 of the specification.

If differences between contractor’s and engineer’s test results cannot be resolved, referee testing will be conducted by the engineer’s central laboratory or a third party laboratory. The referee testing will be performed on the split samples retained by the engineer and the results will be used for final acceptance and pay adjustments.

8.2 QUALITY ASSURANCE SAMPLING AND TESTING FOR AGGREGATES

8.2.1 Sampling

Stratified random sampling procedures shall be used as described in Attachment A. For every 2,000 tons of HMA produced, a single aggregate sample shall be taken at randomly determined times from the combined cold feed according to AASHTO T 2. The sample shall be split into three individual samples of sufficient size for performing the tests shown in Table C.20. The samples shall be obtained by the contractor and witnessed by the engineer. One of the split samples shall be tested by the contractor for quality control purposes as described in Section 7.2 of this specification. One sample will be tested by the engineer for quality assurance purposes as described in Table C.20. The third sample will be retained by the engineer for referee testing.

8.2.2 Lots and Sublots

An aggregate lot is equal to the quantity of aggregate in 1,800 Mg (2,000 tons) of HMA.

8.2.3 Quality Assurance Testing

The testing shown in Table C.20 will be the minimum quality assurance testing requirements for the aggregate.

8.2.4 Quality Assurance Requirements

The quality assurance test results will be used to determine compliance to the specification for aggregates as shown in Tables C.9 and C.10. If the sample tested does not meet the specification for the aggregates, the engineer will immediately contact the contractor and testing will be initiated by the engineer on the other four sublot samples of aggregate. An investigation will be conducted to determine the cause(s) for not meeting the specification. The aggregate test results obtained by the contractor will be included in this investigation, as well as the results from Section 7.2.4 of this specification.

If differences between the contractor’s and engineer’s test results cannot be resolved, referee testing will be conducted by the engineer’s central laboratory or a third party laboratory. The referee testing will be performed on the split samples retained by the engineer and the results will be used for final acceptance.

8.3 QUALITY ASSURANCE SAMPLING AND TESTING FOR HMA PRODUCTION

8.3.1 Sampling

Stratified random sampling procedures shall be used as described in Attachment A. A sample of HMA shall be obtained at randomly determined points from behind the paver prior to compaction for each sublot according to AASHTO T 168. The sample shall be split into three individual samples of sufficient size for performing the tests shown in Tables C.22 and C.23. The samples shall be obtained by the contractor and witnessed by the engineer. One of the samples shall be tested by the contractor for quality control purposes and one sample will be tested by the engineer for quality assurance purposes and pay factor determination. The third sample will be retained by the engineer for referee testing.

8.3.2 Lots and Sublots

A production lot shall consist of four equal sublots. A production sublot shall be 455 Mg (500 tons).

8.3.3 Quality Assurance Testing

The requirements shown in Tables C.22 and C.23 will be the minimum quality assurance testing requirements for HMA production.

8.3.4 Quality Assurance Requirements

The quality assurance test results will be used to determine compliance to the specification for HMA production as described below.

PWL will be determined according to Attachment C for asphalt binder content (ASTM D 6307), gradation 0.075 mm (No. 200) (AASHTO T 27), and gyratory-compacted air voids (AASHTO TP 4 and AASHTO T 269) on a lot basis. The last JMF approved according to Section 5.0 of this specification will provide the target values for all specified properties. The upper and lower specification limits will be determined from tolerances shown in Tables C.16 and C.17 for asphalt binder content, gradation, and gyratory-compacted air voids. The lot tested must be 100 percent within limits for asphalt binder content, gradation, and gyratory-compacted air voids to be accepted. If the lot tested does not meet this requirement, the engineer will immediately contact the contractor. An investigation will be conducted to determine the cause(s) for not meeting the specification. The HMA production sample testing conducted by the contractor will be included in this investigation, as well as the results from Section 7.3.4 of this specification.

If differences between the contractor's and engineer's test results cannot be resolved, referee testing will be conducted by the engineer's central laboratory or a third party laboratory. The referee testing will be performed on the split samples retained by the engineer and the results will be used for final acceptance.

8.4 QUALITY ASSURANCE SAMPLING AND TESTING FOR HMA PLACEMENT

8.4.1 Sampling

Stratified random sampling procedures shall be used as described in Attachment A. Core samples shall be used by the engineer to determine in-place air void contents. A minimum of two locations will be used for each sublot. In-place air void determinations shall not be made within 0.45 m (18 in.) of a longitudinal joint or within 1.5 m (60 in.) of a transverse joint.

8.4.2 Lots and Sublots

A placement lot shall consist of the area of HMA placed in a production lot, excluding miscellaneous areas as described in Section 6.5.3. A placement sublot shall consist of one-fifth of the area of the placement lot (Section 6.5).

8.4.3 Quality Assurance Testing

The requirements shown in Table C.22 will be the minimum quality assurance testing requirements for HMA placement. The in-place air voids shall be determined from core samples according to AASHTO T 269 based on Bulk Specific Gravity determination of core samples by AASHTO T 166 or AASHTO T 275, and on the Theoretical Maximum Specific Gravity according to AASHTO T 209.

8.4.4 Quality Assurance Requirements

The quality assurance test results will be used to determine compliance to the specification for HMA placement as described below.

PWL will be determined according to Attachment C for the core sample test results. The upper and lower specification limits will be 7 and 2 percent air voids, respectively. The lot tested must be 99.7 percent within limits to be accepted. If the lot tested does not meet this requirement, the engineer will immediately contact the contractor. An investigation will be conducted to determine the cause(s) for not meeting the specification. The HMA placement sampling and testing performed by the contractor will be included in this investigation, as well as the results from Section 7.4.3 of this specification.

If differences between the contractor's and engineer's test results cannot be resolved, referee testing will be conducted by the engineer's central laboratory or a third party laboratory. The referee testing will be performed on the cores obtained by the engineer and the results will be used for final acceptance.

8.5 RIDE QUALITY

8.5.1 Sampling

Ride quality is determined by continuous measurement along a pavement and hence sampling is not required.

8.5.2 Lot and Sublot

A ride quality lot is equal to 1.6 lane-km (1 lane-mile). A lane-km is defined as a kilometer length of a mainline travel lane shown on the permanent striping plan. Pay factors for ride quality are based on a lot; therefore, the definition of a sublot is not needed.

8.5.3 Quality Assurance Testing

Quality assurance testing for acceptance will be performed by one of the methods identified in Table C.5.

8.5.4 Quality Assurance Requirements

The quality assurance tests will be used to determine compliance to the specification. The method of acceptance is contained in the standard specification.

CHAPTER 9—ACCEPTANCE

9.1 GENERAL

The acceptance of the HMA is based on the acceptance plan described below for the asphalt binder, aggregate, HMA production, HMA placement, ride quality, and other requirements as defined below.

9.2 ASPHALT BINDER

The acceptance of the asphalt binder is based on compliance to (a) the quality control sampling and testing plan and (b) the quality assurance sampling and testing plan contained in Sections 7.1 and 8.1, respectively, of this specification.

9.3 AGGREGATE

The acceptance of the aggregate is based on compliance to (a) the quality control sampling and testing plan and (b) the quality assurance sampling and testing plan contained in Sections 7.2 and 8.2, respectively, of this specification.

9.4 HOT-MIX ASPHALT PRODUCTION

The acceptance of the HMA production is based on compliance to (a) the quality control sampling and testing plan and (b) the quality assurance sampling and testing plan contained in Sections 7.3 and 8.3, respectively, of this specification.

9.5 HOT-MIX ASPHALT PLACEMENT

The acceptance of the HMA placement is based on compliance to (a) the quality control sampling and testing plan and (b) the quality assurance sampling and testing plan contained in Sections 7.4 and 8.4, respectively, of this specification.

9.6 RIDE QUALITY

The acceptance of the ride quality is based on compliance to the quality assurance plan contained in Section 8.5 of this specification.

9.7 IRREGULARITIES AND SEGREGATION

If a pattern of surface irregularities, including but not limited to color, texture, roller marks, tears, uncoated aggregate particles, or segregation is detected by either the contractor or engineer, the contractor shall make an investigation into the cause(s) and immediately take the appropriate corrective action. With approval of the engineer, placement may continue for no more than 1 day of production from the time the engineer first notified the contractor if corrective actions are being taken. If no appropriate action is taken or if the problem exists after 1 day, paving shall cease until the contractor further investigates the cause(s) and the engineer approves further production to determine the effectiveness of corrective action.

Segregated areas shall be corrected at the contractor's expense as directed by the engineer. Correction may include removal and replacement. Disputes will be resolved at the district or regional level.

9.8 INDIVIDUAL LOADS

Individual loads of HMA in the truck can be rejected by the engineer. Except for rejection based on temperature, uncoated aggregate or nonuniformity, each rejected load will be tested by the engineer if requested by the contractor. The request for testing by the contractor must be made to the engineer within 4 hours of rejection. Tests shall be conducted by the engineer according to the quality assurance testing plan described in Tables C.22 and C.23 and Section 8.3 of this specification for HMA production. If tests are within limits as described in Section 8.3.4, payment will be made for the load at a pay factor of 1.0. If test results are not within limits described in Section 8.3.4, no payment will be made for the load. The engineer will perform the sampling and testing on the disputed loads.

CHAPTER 10—MEASUREMENT

10.1 GENERAL

The quantity of HMA will be measured by mass Mg (or ton) of the type actually used in the completed and accepted work in accordance with the plans and specification for the project. The composite HMA is defined as the asphalt binder, aggregate, recycled asphalt pavement, additives, and modifiers as noted on the plans, approved by the engineer, or both.

If mixing is performed by a drum-mix plant, measurement will be made on scales as described in the standard specifications. If mixing is performed by a weigh-batch or modified weight-batch plant, measurement will be determined on the batch scales unless surge storage is used. Records of the number of batches, batch design, and the mass of the completed HMA shall be supplied by the contractor. Where surge storage is used, measurement of the material will be made on truck scales or suspended hopper scales.

CHAPTER 11—PAY ADJUSTMENT FACTOR AND PAYMENT

11.1 GENERAL

Separate pay adjustment factors and payments will be calculated for asphalt binders, HMA, and ride quality. Pay adjustment factors and payment methods are described below.

11.2 ASPHALT BINDER

11.2.1 Pay Adjustment Factor

Asphalt binder pay adjustment will be determined by an adjustment to the price of the HMA contained in an asphalt binder lot. The pay adjustment factor will be determined by use of Table C.24. If the value of the measured properties meet the compliance limit of Table C.24, the price adjustment factor is 1.00. The price adjustment factor will be 0.75 at the rejection limit. If any measured property is outside the rejection limit (less than 0.75), the HMA will be rejected.

For each property whose value lies between the compliance limit and the rejection limit, the pay adjustment factor will be calculated assuming a linear variation between 1.00 and 0.75. For a lot having more than one parameter out of specification, a composite pay adjustment will be calculated by summing the reduction of each individual property calculation.

The HMA will be accepted with reduced composite price reduction if none of the properties are outside the rejection limit and the composite pay adjustment factor is above 0.75. The HMA will be rejected if one or more of the properties of the asphalt binder fall outside of the rejection limits or if the composite pay factor is below 0.75.

11.2.2 Payment

The amount of price reduction will be based on the HMA price quoted in the contractor's bid. If the price per Mg (or ton) of HMA is unbalanced, the previous year's average bid price per Mg (or ton) will be used. The pay for HMA considering the asphalt binder pay adjustment factor is equal to the HMA price (per Mg or ton) times the asphalt binder pay adjustment factor.

11.3 AGGREGATE

Aggregates do not have pay adjustment factors.

11.4 HOT-MIX ASPHALT

11.4.1 General

HMA pay adjustment will be based on a lot basis. The pay adjustment factors will be based on Method "A" or Method "B" as described below.

11.4.2 Method "A" Pay Adjustment Factors

The engineer will determine pay adjustment factors using the October 2000 release of the software for the "Performance-Related Specification for Hot-Mix Asphalt" for each pavement lot.

The engineer will perform the testing for pay factor determination as described in Tables C.22 and C.23. Pay factor testing shall be performed as part of the quality assurance testing program for HMA production and placement described in Section 8.0 of this specification.

11.4.3 Method "B" Pay Adjustment Factors

The engineer will develop a matrix of input values to cover a range of functional classifications, pavement structures, traffic levels, and decision tree criteria. The engineer will use the software for the "Performance-Related Specification for Hot-Mix Asphalt" to generate a pay factor for each set of inputs and fill the cells within the matrix with the pay factors generated by the software. To determine the pay factor, the engineer will use the values within the table in lieu of the software.

11.4.4 Payment

The amount of payment will be based on the HMA price quoted in the contractor's bid. If the price per Mg (or ton) of HMA is unbalanced, the previous year's average bid price per Mg (or ton) will be used. The payment for HMA considering the HMA pay adjustment factor is equal to the HMA price (per Mg or ton) times the HMA pay adjustment factor.

11.5 RIDE QUALITY

The ride quality payment will be based on the standard specification.

11.6 CONTRACTOR ACCEPTANCE OF PAY ADJUSTMENT FACTORS

If the pay adjustment or pay factor for any lot of HMA is below 1.00, the contractor has the option (a) to remove and replace the lot or (b) to agree to accept the lot at an adjusted unit price determined by Section 11.4 of this specification. If the pay adjustment factor for any lot is less than 0.70, the HMA shall be removed at the expense of the contractor. Replacement material shall meet the requirements of this specification. The contractor and engineer will sign for acceptance of payment on a lot-by-lot basis.

11.7 PAY ADJUSTMENT FACTORS BELOW 1.00

The contractor shall take corrective action if any one of the asphalt binder, HMA, or ride quality pay adjustment factors is below 1.00 for a lot. If three consecutive pay adjustment factors for any single item (asphalt binder, HMA, or ride quality) are below 1.00 (for example, three consecutive pay factors for HMA), construction shall terminate. An investigation will be conducted by the contractor and engineer to determine the cause(s) for not receiving a 1.00 pay adjustment factor. The HMA sampling and testing performed by the contractor will be included in this investigation.

If differences between the contractor's and engineer's test results cannot be resolved, referee testing will be conducted by the engineer's central laboratory or a third party laboratory. The referee testing will be performed on split samples retained by the engineer and the results will be used for determination of the final pay factors and payment.

1.0 INTRODUCTION	2.0 MATERIALS	3.0 CONSTRUCTION	4.0 MIXTURE DESIGN	5.0 JOB MIX FORMULA
1.1 DESCRIPTION 1.2 USE 1.3 SCOPE 1.4 DOCUMENTS 1.5 TERMINOLOGY 1.6 LAB, EQUIPMENT & PERSONNEL 1.7 SAMPLING & TESTING 1.8 QC PLAN	2.1 ASPHALT BINDER 2.2 AGGREGATE 2.3 MINERAL FILLER 2.4 LIME 2.5 BAGHOUSE FINES 2.6 RAP	3.1 GENERAL 3.2 STOCKPILING, STORAGE, FEEDING & DRYING 3.3 MIXING PLANTS 3.4 SURGE-STORAGE SYSTEMS 3.5 HAULING EQUIPMENT 3.6 SURFACE PREPARATION 3.7 PLACING 3.8 JOINTS 3.9 COMPACTION 3.10 RESTRICTIONS	4.1 GENERAL 4.2 REQUIREMENTS 4.3 SUBMITTALS 4.4 APPROVAL 4.5 NEW LAB DESIGN	5.1 DESCRIPTION 5.2 JOB MIX FORMULA 1 5.3 FIELD TRIAL SECTIONS 5.4 JOB MIX FORMULA 2 5.5 SECOND DAY PRODUCTION 5.6 JOB MIX FORMULA ADJUSTMENTS 5.7 COMPACTION ROLLING PATTERN

6.0 LOT AND SUBLOT	7.0 QUALITY CONTROL	8.0 QUALITY ASSURANCE	9.0 ACCEPTANCE	10.0 MEASUREMENT
6.1 GENERAL 6.2 ASPHALT BINDER 6.3 AGGREGATE 6.4 HMA PRODUCTION 6.5 HMA PLACEMENT 6.6 RIDE QUALITY	7.1 ASPHALT BINDERS 7.2 AGGREGATES 7.3 HMA PRODUCTION 7.4 HMA PLACEMENT 7.5 RIDE QUALITY	8.1 ASPHALT BINDERS 8.2 AGGREGATES 8.3 HMA PRODUCTION 8.4 HMA PLACEMENT 8.5 RIDE QUALITY	9.1 GENERAL 9.2 ASPHALT BINDER 9.3 AGGREGATE 9.4 HMA PRODUCTION 9.5 HMA PLACEMENT 9.6 RIDE QUALITY 9.7 IRREGULARITIES & SEGREGATION 9.8 INDIVIDUAL LOADS	10.1 GENERAL **11.0 PAY ADJUSTMENT FACTOR & PAYMENT** 11.1 GENERAL 11.2 ASPHALT BINDER 11.3 AGGREGATE 11.4 HMA 11.5 RIDE QUALITY 11.6 PAY FACTOR ACCEPTANCE 11.7 PAY FACTORS BELOW 1.00

Figure C.1. Hot-mix asphalt quality control/quality assurance guide specification section designations.

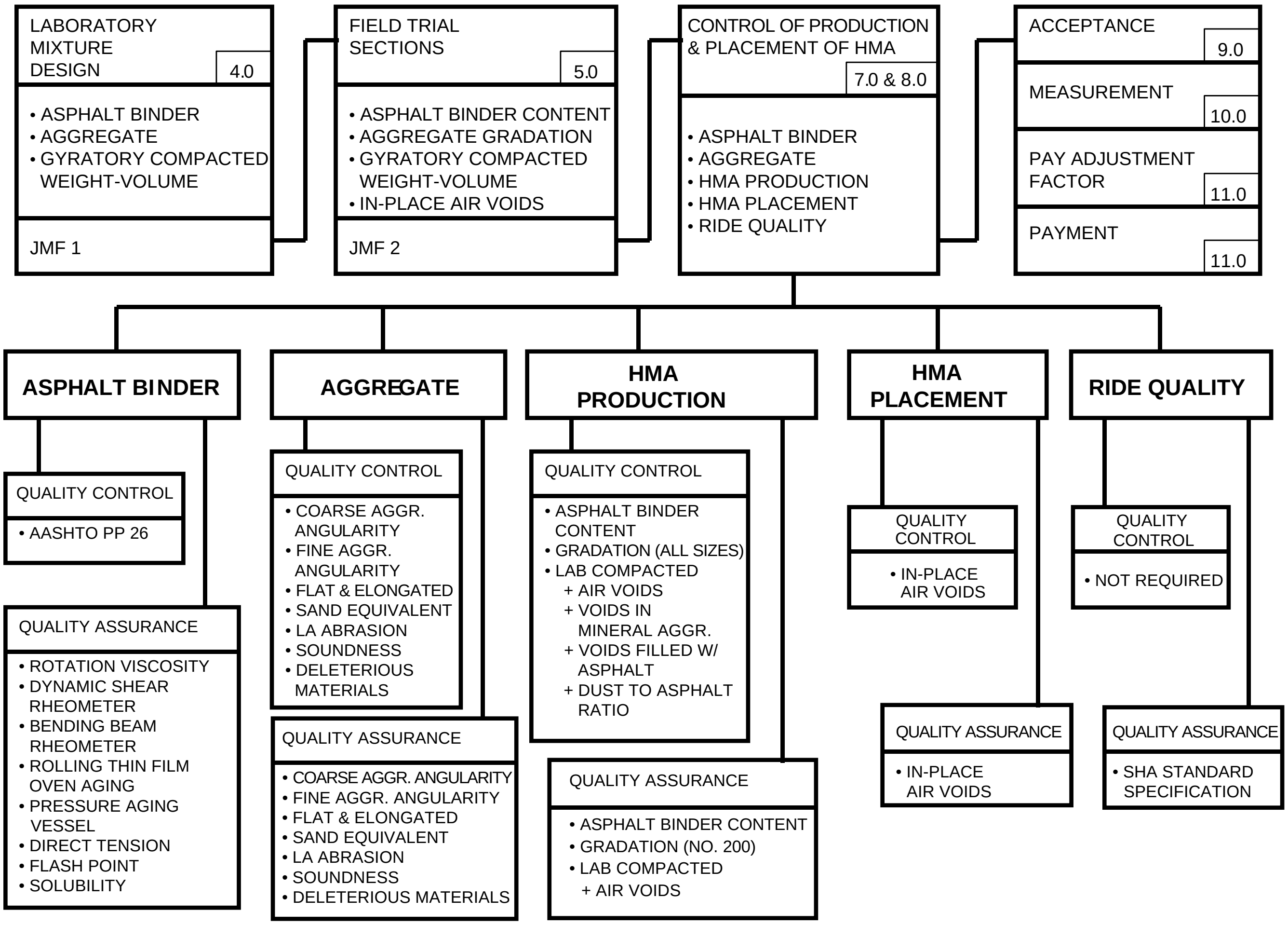


Figure C.2. Hot-mix asphalt quality control/quality assurance guide specification tests.

TABLE C.1 General quality control/quality assurance specification references

Test Designation	Test Method Number	
	AASHTO	ASTM
Statistical procedures	R4	
Acceptance sampling plans for highway construction	R9	
Definition of terms for specifications and procedures	R10	
Indicating which places are to be considered significant in specified limiting values	R11	
Establishing and implementing a quality system for construction materials testing laboratories	R18	
Terminology relating to materials for roads and pavements		D8
Random sampling of construction materials		D3665
Minimum requirements for agencies testing and inspecting bituminous paving materials		D3666
Probability sampling of materials		E105
Choice of sample size to estimate the average quality of a lot or process		E122
Acceptance of evidence based on the results of probability sampling		E141
Quality control systems for organizations producing and applying bituminous paving materials		D4561
Organizations engaged in the certification of personnel testing and inspecting bituminous paving materials		D5506

TABLE C.2 Asphalt binder tests

Test Designation	Test Method Number		Certification Level
	AASHTO	ASTM	
Rotational Viscometer (RV)	TP48	D4402	
Dynamic Shear Rheometer (DSR)	TP5		
Bending Beam Rheometer (BBR)	TP1		
Direct Tension Test (DTT)	TP3		
Rolling Thin Film Oven (RTFO)	T240	D2872	
Pressure Aging Vessel (PAV)	PP1		
Flash point	T48	D92	
Sampling bituminous materials	T40	D140	
Solubility of bituminous materials	T44	D2042	
Performance-graded binders	MP1		
Specific gravity of bituminous materials	T228	D70	
Certifying suppliers of performance-graded asphalt binders	PP26		
Grading or verifying the performance grade of an asphalt binder	PP6		

TABLE C.3 Aggregate tests

Test Designation	Test Method Number		Certification Level
	AASHTO	ASTM	
Sieve analysis (gradation)	T27	C136	
Sieve analysis of extracted aggregate (gradation) Minus 0.075 mm (No. 200) by washing	T30 T11	 C117	
Specific gravity and absorption of fine aggregate	T84	C128	
Specific gravity and absorption of coarse aggregate	T85	C127	
Coarse aggregate (angularity) fractured particles (CAA)		D5821	
Uncompacted void content of fine aggregate (fine aggregate angularity) (FAA)	T304		
Flat or elongated particles in coarse aggregates		D4791	
Sand equivalent	T176	D2419	
Los Angeles abrasion	T96	C131, C535	
Soundness	T104	C88	
Deleterious materials	T112	C142	
Sampling aggregates	T2	D75	
Reducing samples of aggregate to testing size	T248	C702	
Bulk specific gravity of mineral filler	T100	D854	
Sieve analysis of mineral filler	T37	D546	
Liquid limit, plastic limit, and plastic index	T89, T90	D4318	
Shrinkage limit	T92		
Moisture content of aggregate	T255	C566	

TABLE C.4 Hot-mix asphalt tests

Test Designation	Test Method Number		Certification Level
	AASHTO	ASTM	
Bulk specific gravity of compacted HMA-saturated surface dry (SSD) method	T166	D2726	
Bulk specific gravity of compacted HMA-paraffin	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 for HMA	PP28		
Mixture conditioning of HMA	PP2		
SHRP gyratory compactor	TP4		
Sampling HMA	T168	D979	
Sampling compacted HMA		D5361	
Resistance of HMA to moisture damage	T283	D4867	
Thickness of compacted HMA		D3549	
Nuclear density		D2950	
Asphalt content by nuclear method	T287	D4125	
Asphalt content by solvent extraction	T164	D2172	
Asphalt content by ignition method	T308	D6307	
Marshall and Hveem mixture design	R12		
Marshall stability	T245	D1559	
Hveem stability	T246	D1560	
California kneading compactor	T247	D1561	

TABLE C.5 Pavement roughness measurement

Test Designation	Test Method Number		Certification Level
	AASHTO	ASTM	
Measurement of vehicular response to traveled surface roughness	T286	E1082	
Trailers used for measuring vehicle response to road roughness		E1215	
Longitudinal profile of traveled surface with an accelerometer established inertial profiling reference		E950	
Pavement roughness using a profilograph		E1274	
Road roughness by static level method		E1364	

TABLE C.6 Performance-graded asphalt binder specification

PERFORMANCE GRADE	PG 46			PG 52							PG 58					PG 64					
	34	40	46	10	16	22	28	34	40	46	16	22	28	34	40	10	16	22	28	34	40
Average 7-day Maximum Pavement Design Temperature, °C[a]	<46			<52							<58					<64					
Minimum Pavement Design Temperature, °C[a]	>-34	>-40	>-46	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-40
ORIGINAL BINDER																					
Flash Point Temp, T48: Minimum °C	230																				
Viscosity, ASTM D4402:[b] Maximum, 3Pa•s, Test Temp, °C	135																				
Dynamic Shear, TP5:[c] $G^*/\sin\delta$[g], Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
ROLLING THIN FILM OVEN RESIDUE (T240)																					
Mass Loss, Maximum, percent	1.00																				
Dynamic Shear, TP5: $G^*/\sin\delta$[g], Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	46			52							58					64					
PRESSURE AGING VESSEL RESIDUE (PP1)																					
PAV Aging Temperature, °C[d]	90			90							100					100					
Dynamic Shear, TP5: $G^*/\sin\delta$[g], Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	10	7	4	25	22	19	16	13	10	7	25	22	19	16	13	31	28	25	22	19	16
Physical Hardening[e]	Report																				
Creep Stiffness, TP1:[f] S, Maximum, 300.0 MPa, *m*-value, Minimum, 0.300 Test Temp @ 60s, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	-24	-30	-36	0	-6	-12	-18	-24	-30	-36	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-30

°F = 1.8°C + 32
1 psi = 6.9 kPa
1 in. = 25.4 mm

TABLE C.6 (Continued)

PERFORMANCE GRADE	PG 70						PG 76					PG 82				
	10	16	22	28	34	40	10	16	22	28	34	10	16	22	28	34
Average 7-day Maximum Pavement Design Temperature, °C [b]	<70						<76					<82				
Minimum Pavement Design Temperature, °C [b]	>-10	>-16	>-22	>-28	>-34	>-40	>-10	>-16	>-22	>-28	>-34	>-10	>-16	>-22	>-28	>-34
ORIGINAL BINDER																
Flash Point Temp, T48: Minimum °C	230															
Viscosity, ASTM D4402:[b] Maximum, 3Pa•s, Test Temp, °C	135															
Dynamic Shear, TP5:[c] G*/sinδ, Minimum, 1.00 kPa Test Temp @ 10 rad/s, °C	70						76					82				
ROLLING THIN FILM OVEN RESIDUE (T240)																
Mass Loss, Maximum, percent	1.00															
Dynamic shear, TP5: G*/sinδ, Minimum, 2.20 kPa Test Temp @ 10 rad/s, °C	70						76					82				
PRESSURE AGING VESSEL RESIDUE (PP1)																
PAV Aging Temperature, °C [d]	100(110)						100(110)					100(110)				
Dynamic Shear, TP5: G*/sinδ, Maximum, 5000 kPa Test Temp @ 10 rad/s, °C	34	31	28	25	22	19	37	34	31	28	25	40	37	34	31	28
Physical Hardening[e]	Report															
Creep Stiffness, TP1:[f] S, Maximum, 300.0 MPa, *m*-value, Minimum, 0.300 Test Temp @ 60s, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	0	-6	-12	-18	-24
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, °C	0	-6	-12	-18	-24	-30	0	-6	-12	-18	-24	-0	-6	-12	-18	-24

°F = 1.8°C + 32
1 psi = 6.9 kPa
1 in. = 25.4 mm

[a] Pavement temperatures are estimated from air temperatures using an algorithm contained in the Long-Term Pavement Performance (LTPP) Bind program, may be provided by the specifying agency, or by following the procedures as outlined in MP2 and PP28.

[b] This requirement may be waived at the discretion of the specifying agency if the supplier warrants that the asphalt binder can be adequately pumped and mixed at temperatures that meet all applicable safety standards.

[c] For quality control of unmodified asphalt cement production, measurement of the viscosity of the original asphalt cement may be used to supplement dynamic shear measurements of G*/sinδ at test temperatures where the asphalt is a Newtonian fluid.

[d] The PAV aging temperature is based on simulated climatic conditions and is one of three temperatures 90°C (194°F), 100°C (212°F), or 110°C (230°F). The PAV aging temperature is 100°C (212°F) for PG 58- and above, except in desert climates, where it is 110°C.

[e] Physical Hardening - TP1 is performed on a set of asphalt beams according to Section 13.1, except the conditioning time is extended to 24 hours ± 10 minutes at 10°C above the minimum performance temperature. The 24-hour stiffness and *m*-value are reported for information purposes only.

[f] If the creep stiffness is below 300 MPa (43.5 ksi), the direct tension test is not required. If the creep stiffness is between 300 and 600 MPa (43.5 and 87.0 ksi), the direct tension failure strain requirement can be used in lieu of the creep stiffness requirement. The *m*-value requirement must be satisfied in both cases.

[g] G*/sinδ = high temperature stiffness and G*sinδ = intermediate temperature stiffness.

TABLE C.7 Aggregate gradation control points

	Nominal Maximum Aggregate Size - Control Point (Percent Passing)									
	37.5 mm		25.0 mm		19.0 mm		12.5 mm		9.5 mm	
Sieve Size	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
50.0 mm	100	-	-	-	-	-	-	-	-	-
37.5 mm	90	100	100	-	-	-	-	-	-	-
25.0 mm	-	90	90	100	100	-	-	-	-	-
19.0 mm	-	-	-	90	90	100	100	—	—	—
12.5 mm	-	-	-	-	-	90	90	100	100	-
9.5 mm	-	-	-	-	-	-	-	90	90	100
4.75 mm	-	-	-	-	-	-	-	-	-	90
2.36 mm	15	41	19	45	23	49	28	58	32	67
0.075 mm	0	6	1	7	2	8	2	10	2	10

1 in. = 25.4 mm

TABLE C.8 Boundaries of aggregate-restricted zone

Sieve Size within Restricted Zone	Minimum and Maximum Boundaries of Sieve Size for Nominal Maximum Aggregate Size (Minimum and Maximum Percent Passing)									
	37.5 mm		25.0 mm		19.0 mm		12.5 mm		9.5 mm	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
0.300 mm	10.0	10.0	11.4	11.4	13.7	13.7	15.5	15.5	18.7	18.7
0.600 mm	11.7	15.7	13.6	17.6	16.7	20.7	19.1	23.1	23.5	27.5
1.18 mm	15.5	21.5	18.1	24.1	22.3	28.3	25.6	31.6	31.6	37.6
2.36 mm	23.3	27.3	26.8	30.8	34.6	34.6	39.1	39.1	47.2	47.2
4.75 mm	34.7	34.7	39.5	39.5	–	–	–	–	–	–

1 in. = 25.4 mm

TABLE C.9 Superpave aggregate consensus property requirements

Design ESALs[1] (million)	Coarse Aggregate Angularity (Percent), minimum		Uncompacted Void Content of Fine Aggregate (Percent), minimum		Sand Equivalent (Percent), minimum	Flat and Elongated[3] (Percent), maximum
	≤ 100 mm	> 100 mm	≤ 100 mm	> 100 mm		
< 0.3	55/-	-/-	-	-	40	-
0.3 to < 3	75/-	50/-	40	40	40	10
3 to < 10	85/80[2]	60/-	45	40	45	
10 to < 30	95/90	80/75	45	40	45	
≥ 30	100/100	100/100	45	45	50	

1 Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years, and choose the appropriate N_{design} level.
2 85/80 denotes that 85% of the coarse aggregate has one fractured face and 80% has two or more fractured faces.
3 Criterion based upon a 5:1 maximum-to-minimum ratio.

Note: If less than 25% of a layer is within 100 mm (4 in.) of the surface, the layer may be considered to be below 100 mm (4 in.) for mixture design purposes.

TABLE C.10 Source aggregate property requirements

Test Method			Requirement
Description	Number		
	AASHTO	ASTM	
Los Angeles abrasion (500 rev.)	T96	C131, C535	45 percent maximum
Soundness (Mg $S0_4$) (5 cycles)	T104	C88	18 percent maximum
Deleterious materials	T112	C142	1 percent maximum

TABLE C.11 Asphalt binder sampling and testing for mixture design acceptance

Test Method			Location of Sampling	Frequency
Description	Number			
	AASHTO	ASTM		
Rotational viscometer (original asphalt)		D4402		1
Dynamic shear rheometer (original asphalt)	TP5			1
Dynamic shear rheometer (RTFO-aged)	TP5			1
Dynamic shear rheometer (PAV-aged)	TP5			1
Bending beam rheometer (PAV-aged)	TP1			1
Direct tension (PAV-aged)	TP3			1
Flash point (original asphalt)	T48	D92		1
Solubility (original asphalt)	T44	D2042		1
Specific gravity (original asphalt)	T228	D70		1

TABLE C.12 Aggregate sampling and testing for mixture design acceptance

Test Method			Location of Sampling	Frequency
Description	Number			
	AASHTO	ASTM		
Coarse aggregate angularity		D5821	Stockpiles	1
Fine aggregate angularity	T304		Stockpiles	1
Flat and elongated particles		D4791	Stockpiles	1
Sand equivalent	T176	D2419	Stockpiles	1
Los Angeles abrasion	T96	C131, C535	Stockpiles	1
Soundness	T104	C88	Stockpiles	1
Deleterious materials	T112	C142	Stockpiles	1

TABLE C.13 Superpave gyratory-compaction effort

Design ESALs[1] (million)	Compaction Parameters			Typical Roadway Application[2]
	$N_{initial}$	N_{design}	N_{max}	
< 0.3	6	50	75	Applications include roadways with very light traffic volumes such as local roads, county roads, and city streets where truck traffic is prohibited or at a very minimal level. Traffic on these roadways would be considered local in nature, not regional, intrastate, or interstate. Special purpose roadways serving recreational sites or areas may also be applicable to this level.
0.3 to < 3	7	75	115	Applications include many collector roads or access streets. Medium-trafficked city streets and the majority of county roadways may be applicable to this level.
3 to < 30	8	100	160	Applications include many two-lane, multilane, divided, and partially or completely controlled access roadways. Among these are medium to highly trafficked city streets, many state routes, U.S. highways, and some rural interstates.
≥ 30	9	125	205	Applications include the vast majority of the U.S. Interstate system, both rural and urban in nature. Special applications such as truck-weighing stations or truck-climbing lanes on two-lane roadways may also be applicable to this level.

[1] Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years, and choose the appropriate N_{design} level.

[2] Typical Roadway Applications as defined by *A Policy on Geometric Design of Highway and Streets*, 1994, AASHTO.

Note 1: When specified by the agency and the top of the design layer is ≥ 100 mm (4 in.) from the pavement surface and the estimated design traffic level ≥ 0.3 million ESALs, decrease the estimated design traffic level by one, unless the mixture will be exposed to significant main line and construction traffic prior to being overlaid. If less than 25 percent of the layer is within 100 mm (4 in.) of the surface, the layer may be considered to be below 100 mm (4 in.) for mixture design purposes.

Note 2: When the design ESALs are between 3 to < 10 million ESALs the agency may, at its discretion, specify $N_{initial}$ at 7, N_{design} at 75, and N_{max} at 115, based on local experience.

TABLE C.14 Superpave volumetric mixture design requirements

<table>
<tr><th rowspan="3">Design ESALs[1] (million)</th><th colspan="3">Required Density (% of Theoretical Maximum Specific Gravity)</th><th colspan="5">Voids in the Mineral Aggregate (Percent), minimum</th><th rowspan="3">Voids Filled with Asphalt, (Percent) minimum</th><th rowspan="3">Dust-to-Binder Ratio Range</th></tr>
<tr><th rowspan="2">$N_{initial}$</th><th rowspan="2">N_{design}</th><th rowspan="2">N_{max}</th><th colspan="5">Nominal Maximum Aggregate Size (mm)</th></tr>
<tr><th>37.5</th><th>25.0</th><th>19.0</th><th>12.5</th><th>9.5</th></tr>
<tr><td>< 0.3</td><td>≤ 91.5</td><td rowspan="5">96.0</td><td rowspan="5">≤ 98.0</td><td rowspan="5">11.0</td><td rowspan="5">12.0</td><td rowspan="5">13.0</td><td rowspan="5">14.0</td><td rowspan="5">15.0</td><td>70 - 80[3,4]</td><td rowspan="5">0.6 - 1.2</td></tr>
<tr><td>0.3 to < 3</td><td>≤ 90.5</td><td>65 - 78[4]</td></tr>
<tr><td>3 to < 10</td><td rowspan="3">≤ 89.0</td><td rowspan="3">65 - 75[2,4]</td></tr>
<tr><td>10 to < 30</td></tr>
<tr><td>≥ 30</td></tr>
</table>

1 in. = 25.4 mm

[1] Design ESALs are the anticipated project traffic level expected on the design lane over a 20-year period. Regardless of the actual design life of the roadway, determine the design ESALs for 20 years and choose the appropriate N_{design} level.

[2] For 9.5-mm (0.375-in.) nominal maximum size mixtures, the specified VFA range shall be 73% to 76% for design traffic levels ≥3 million ESALs.

[3] For 25.0-mm (1.0-in.) nominal maximum size mixtures, the specified lower limit of the VFA shall be 67% for design traffic levels < 0.3 million ESALs.

[4] For 37.5-mm (1.5-in.) nominal maximum size mixtures, the specified lower limit of the VFA shall be 64% for all design traffic levels.

Note: If the aggregate gradation passes beneath the boundaries of the aggregate restricted zone specified in Table C.8, consideration should be given to increasing the dust-to-binder ratio criteria from 0.6–1.2 to 0.8–1.6.

TABLE C.15 Aggregate gradation and asphalt binder content tolerances for field trial section acceptance

Sieve Size*		Tolerance, Mass
Metric, mm	U.S. Customary	
50	2.0-in.	± 3.0
37.5	1.5-in.	± 3.0
25	1-in.	± 3.0
19	3/4-in.	± 3.0
12.5	1/2-in.	± 3.0
9.5	3/8-in.	± 3.0
4.75	No. 4	± 3.0
2.36	No. 8	± 2.0
1.18	No. 16	± 2.0
0.600	No. 30	± 2.0
0.300	No. 50	± 2.0
0.150	No. 100	± 2.0
0.075	No. 200	± 0.7
Asphalt binder content,** percent by mass		± 0.13

*The gradation (AASHTO T27) shall be determined after the asphalt content is determined by the Ignition Test (AASHTO T308).
**Asphalt content determined by AASHTO T308 (Ignition Test).

Note: Tolerances based on JMF 1.

TABLE C.16 Volumetric tolerances for field trial section acceptance*

Test Method			Tolerances
Description	Number		
	AASHTO	ASTM	
Gyratory-compacted sample properties at N_{design}	TP4		
• Air voids (V_{air})	T269	D3203	± 1
• Voids in mineral aggregate (VMA)	PP28		± 1
• Voids filled with asphalt (VFA)	PP28		± 5
• Bulk specific gravity (Gmb)	T166	D2726	± 0.022
• Dust-to-binder ratio	PP28		0.6 to 1.6
• Theoretical maximum specific gravity (Gmm)	T209	D2041	± 0.015
• In-place air voids	T269	D3203	2 to 7

Note: Tolerances based on JMF 1.

TABLE C.17 Aggregate gradation and asphalt binder content tolerances for acceptance of job mix formula 3

Sieve Size*		Tolerance, Mass
Metric, mm	U.S. Customary	
50	2.0-in.	± 5
37.5	1.5-in.	± 5
25	1-in.	± 5
19	3/4-in.	± 5
12.5	1/2-in.	± 5
9.5	3/8-in.	± 5
4.75	No. 4	± 5
2.36	No. 8	± 5
1.18	No. 16	± 3
0.600	No. 30	± 3
0.300	No. 50	± 3
0.150	No. 100	± 3
0.075	No. 200	± 1.6
Asphalt binder content,** percent by mass		± 0.4

*The gradation (AASHTO T27) shall be determined after the asphalt content is determined by the Ignition Test (AASHTO T308).
**Asphalt content determined by AASHTO T308 (Ignition Test).

Note: Tolerances based on JMF 1.

TABLE C.18 Asphalt binder sampling and testing

Test Method			Contractor's Quality Control Testing*		Engineer's Verification Testing		Engineer's Quality Assurance Testing		Engineer's Pay Factor Testing	
Description	Number		Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency
	AASHTO	ASTM								
Rotational viscometer (original asphalt)		D4402					Feed line**	1 per lot***		
Dynamic shear rheometer (original asphalt)	TP5						Feed line	1 per lot		
Dynamic shear rheometer (RTFO-aged)	TP5 T240						Feed line	1 per lot		
Dynamic shear rheometer (PAV-aged)	TP5 PP1						Feed line	1 per lot		
Bending beam rheometer (PAV-aged)	TP1 PP1						Feed line	1 per lot		
Direct tension (PAV-aged)	TP3 PP1						Feed line	1 per lot		
Flash point****	T48	D92					Feed line	1 per lot		
Solubility****	T44	D2042					Feed line	1 per lot		
Specific gravity****	T228	D70								

* Meet requirements of PP26.
** Asphalt binder feed line between contractor's storage tank and plant mixing chamber.
*** One lot equals 2,000 tons of HMA. For quality assurance purposes, a minimum of one lot should be tested for every five lots of HMA produced.
**** Optional tests.

TABLE C.19 Acceptable difference between contractor's and engineer's test results for asphalt binders

Test Method			
Designation	Number		Acceptable Difference*
	AASHTO	ASTM	
Dynamic shear rheometer on original asphalt, G*/sin δ	TP5		20 percent**
Dynamic shear rheometer on RTFOT-aged binder, G*/sin δ	TP5 TP240	D2872	20 percent**
Dynamic shear rheometer on PAV-aged binder, G* sin δ	TP5 PP1		20 percent**
Bending beam rheometer on PAV-aged binder, S-value	TP1 PP1		10 percent**
Bending beam rheometer on PAV-aged binder, *m*-value	TP1 PP1		0.015
Failure strain on PAV-aged binder, failure strain	TP3 PP1		1.0 percent

*Based on UDOT specification.
**Percent of average value of two test results.

TABLE C.20 Aggregate sampling and testing

Test Method			Contractor's Quality Control Testing		Engineer's Verification Testing		Engineer's Quality Assurance Testing		Engineer's Pay Factor Testing	
Description	Number		Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency
	AASHTO	ASTM								
Coarse aggregate angularity**		D5821	Combined cold feed	1 sample per lot*			Combined cold feed	1 sample per lot		
Fine aggregate angularity**	T304		Combined cold feed	1 sample per lot			Combined cold feed	1 sample per lot		
Flat and elongated particles**		D4791	Combined cold feed	1 sample per lot			Combined cold feed	1 sample per lot		
Sand equivalent**	T176	D2419	Combined cold feed	1 sample per lot			Combined cold feed	1 sample per lot		
Los Angeles abrasion***	T96	C131, C535	Combined cold feed	1 sample per lot			Combined cold feed	1 sample per 3 lots		
Soundness***	T104	C88	Combined cold feed	1 sample per lot			Combined cold feed	1 sample per 3 lots		
Deleterious materials***	T112	C142	Combined cold feed	1 sample per lot			Combined cold feed	1 sample per lot		

* One lot equals 2,000 tons of HMA.
** For quality control purposes, a minimum of one lot should be tested for every five lots sampled and produced.
*** On approved sources, quality assurance testing should be performed on a quarterly basis. On new sources, quality assurance testing should be performed at the start of a project and quarterly thereafter.

TABLE C.21 Acceptable difference between contractor's and engineer's test results for aggregates

Test Method			
Designation	Number		Acceptable Difference*
	AASHTO	ASTM	
Coarse aggregate angularity		D5821	28
Fine aggregate angularity	T304		1
Flat and elongated particles		D4791	3**
Sand equivalent	T176	D2419	9
Los Angeles abrasion	T96	C131, C535	13
Soundness	T104	C88	70 percent***
Deleterious materials	T112	C142	1.7

*Represent multi-laboratory precision for AASHTO or ASTM test methods.
**Estimated.
***Magnesium sulfate, percent of average value of test results.

TABLE C.22 Quality control, quality assurance, and pay factor tests or HMA production and placement

Test Method			Contractor's Quality Control Testing		Engineer's Verification Testing		Engineer's Quality Assurance Testing**		Engineer's Pay Factor Testing	
Description	Number		Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency	Location of Sampling	Frequency
	AASHTO	ASTM								
A. Gradation	T27	C136	Behind paver	1 per sublot*			Behind paver	1 per sublot	Behind paver	1 per sublot
B. Asphalt binder content		D6307	Behind paver	1 per sublot			Behind paver	1 per sublot	Behind paver	1 per sublot
C. Gyratory-compacted sample properties at N_{design}	TP4		Behind paver	1 per sublot			Behind paver	1 per sublot		
1. Air voids (V_{air})	T269	D3203		1 per sublot				1 per sublot		
2. Voids in mineral aggregates (VMA)	PP28			1 per sublot						
3. Voids filled with asphalt (VFA)	PP28			1 per sublot						
4. Bulk specific gravity (Gmb)	T166	D2726		1 per sublot				1 per sublot		
5. Dust-to-binder ratio	PP28			1 per sublot						
6. Theoretical maximum specific gravity (Gmm)	T209	D2041		1 per sublot				1 per sublot		1 per sublot
D. In-place air voids	T269	D3203 D2950	Cores from pavement	2 per sublot			Cores from pavement	2 per sublot	Cores from pavement	2 per sublot

* Four sublots per lot. One lot equals 2,000 tons of HMA.
** Engineer's quality assurance testing will be a minimum of one lot for every four lots of quality control testing.

TABLE C.23 Aggregate gradation determination requirements for quality control, quality assurance, and pay factor testing

Sieve Size		Contractor Quality Control Testing	Engineer's Quality Assurance Testing	Engineer's Pay Factor Testing
Metric, mm	U.S. Customary			
50	2.0-in.			
37.5	1.5-in.			
25	1-in.	X*		
19	3/4-in.	X		
12.5	1/2-in.	X		
9.5	3/8-in.			
4.75	No. 4			
2.36	No. 8	X	X	X
1.18	No. 16	X		
0.600	No. 30	X		
0.300	No. 50	X		
0.150	No. 100			
0.075	No. 200	X	X	X

*Use gradation control sieves for nominal maximum aggregate size specified (19-mm [3/4-in.]) nominal maximum size aggregate requirements shown.

TABLE C.24 Pay factor adjustment for asphalt binder

Property	Compliance Limit for Price Adjustment of 1.00	Rejection Limit for Price Adjustment of 0.75
$G^*/\sin\delta$ of the original performance-graded asphalt binder at high grade temperature, kPa	0.84 Min.	0.70 Min.
$G^*/\sin\delta$ of RTFO residue at high temperature, kPa	1.74 Min.	1.40 Min.
Stiffness of the PAV residue at low grade temperature + 10°C, MPa	311 Max.	355 Max.
Slope (*m*-value) of the creep curve at low grade temperature +10°C	0.294 Min.	0.265 Min.
Failure strain of PAV residue in direct tension at low grade temperature + 10°C[1]	1.04 Min.	0.78 Min.
[1]Use only for binders for which the test temperature for the low temperature properties is −18°C or colder.		

1 psi = 6.9 kPa
°F = 1.8°C + 32

Source: UDOT

ATTACHMENT A

STRATIFIED RANDOM SAMPLING

1.1 SCOPE

1.2

This method outlines the procedures for selecting sampling sites in accordance with appropriate random sampling techniques. Random sampling is the selection of a sample in such a manner that every portion of the material or construction to be sampled has an equal chance of being selected as the sample. It is intended that all samples, regardless of size, type, or purpose, shall be selected in an unbiased manner, based entirely on chance.

2.1 SECURING SAMPLES

2.2

Samples shall be taken as directed by the contractor's Quality Control Representative for quality control purposes and the SHA representative for acceptance purposes.

2.3

Sample location and sampling procedure are as important as testing. It is essential that the sample location be chosen in an unbiased manner.

3.1 RANDOM NUMBER TABLE

3.2

For test results or measurements to be meaningful, it is necessary that the material to be sampled or measured, be selected at random, which means using a table of random numbers. Table 1 consists of random numbers for this purpose. To use the table in selecting sample locations, proceed as follows.

3.3

A random number table is a collection of random digits. The random numbers that are presented in this attachment are shown in a two-place decimal format. Note the two columns in Table 1 labeled X and Y. The numbers in either column can be used to locate a random sample when only a single dimension is required to locate the sample (e.g., time, tonnage, and units). When two dimensions are required to locate the sample, the number in the X column is used to calculate the longitudinal location, and the number in the Y column is used to calculate the transverse location. In the Y column, each number is preceded by an L or R, designating that the sample increment is to be located transversely from the left or right edge of the pavement. Figure 1 illustrates the procedure for locating samples in two dimensions.

3.4

Determine the lot size (continuous production for quality control at HMA plant) and stratify the lot into a number of sublots per lot for the material being sampled.

3.5

For each lot, use consecutive two-digit random numbers from Table 1. For example, if the specification specifies five sublots per lot and the number 15 is randomly selected as the starting point for the first lot, numbers 15 through 19 would be the five consecutive two-digit random numbers. For the second lot, another random starting point, number 91, for example, is selected and the numbers 91 through 95 are used for the five consecutive two-digit random numbers. The same procedure is used for additional lots.

3.6

For samples taken from the roadway, use the decimal values in column X and column Y to determine the coordinates of the sample locations.

3.7

In situations where coordinate locations do not apply (e.g., plant or stockpile samples), use those decimal values from column X *or* column Y.

4.1 THE RANDOM SAMPLE

4.2

Examples demonstrating the use of the random sampling technique under various conditions are given in Sections 4.2.1 through 4.2.3.

4.2.1 Sampling by Time Sequence

Assume that HMA for use in paving is to be sampled to determine the percentage of asphalt. It will be sampled at the place of manufacture. The task is to select a random sampling plan in order to distribute the sampling over the half-day or the full-day, whichever is more applicable. Assume that the lot size is a day's production and that five samples are required from each lot. The plant is assumed to operate continuously for 9 hours (beginning at 7:00 a.m. and continuing until 4:00 p.m., with no break for lunch).

4.2.1.2

Lot Size. The lot size is a day's production. The plant begins operation at 7:00 a.m. and stops at 4:00 p.m. Hence, the lot size is 9 hours of production.

4.2.1.3

Sublot Size. Stratify the lot into five equal sublots, because five samples are required. To accomplish this, select five equal time intervals during the 9 hours that the plant is operating as follows:

$$\text{Sublot Time Interval} = \frac{(9\ \text{h/lot})(60\ \text{min/h})}{5\ \text{sublots/lot}}$$

Sublot Time Interval = 108 min/sublot

4.2.1.4

Sublot Samples. Next, choose five random numbers from the random number table. The first block randomly selected is reproduced below.

Sequence Number	X	Y
12	0.57	R 0.49
13	0.35	R 0.90
14	0.69	L 0.63
15	0.59	R 0.68
16	0.06	L 0.03

4.2.1.5

The selected random numbers taken from the X column are 0.57, 0.35, 0.69, 0.59, and 0.06. To randomize the sampling times within each sublot, the time interval (108 minutes) computed in Section 4.2.1.3 is used. This time interval is multiplied by each of the five random numbers previously selected:

Sublot 1: $0.57 \times 108 = 62$ min.
Sublot 2: $0.35 \times 108 = 38$ min.
Sublot 3: $0.69 \times 108 = 75$ min.
Sublot 4: $0.59 \times 108 = 64$ min.
Sublot 5: $0.06 \times 108 = 6$ min.

4.2.1.6

The times calculated in Section 4.2.1.5 are added to the starting times for each sublot. This results in the randomized times at which the samples are to be obtained. The sampling sequence is as follows:

Sublot Number	Sampling Time
1	7:00 a.m. + 62 min = 8:02 a.m.
2	8:48 a.m. + 38 min = 9:26 a.m.
3	10:36 a.m. + 75 min = 11:51 a.m.
4	12:24 p.m. + 64 min = 1:28 p.m.
5	2:12 p.m. + 6 min = 2:18 p.m.

4.2.1.7

The random sampling times from Section 4.2.1.6 are shown in Figure 2. If production is not available at the indicated time, a sample should be obtained at the first opportunity following the indicated time. Sampling on a time-basis is practical only when the process is continuous. Intermittent processes obviously present many difficulties.

4.2.2 Sampling by Material Mass

4.2.2.1

HMA for use in paving must be sampled to determine the asphalt content. In this example, the specification defines the lot size as 4,500 Mg (5,000 tons) and states that five samples must be obtained from the lot. The sampling is to be done from the hauling units at the manufacturing source. The total tonnage for the project is 18,000 Mg (20,000 tons).

4.2.2.2

Lot Size and Number of Lots. The lot size is 4,500 Mg (5,000 tons). Because there are 18,000 Mg (20,000 tons) of bituminous mix required for the project, the total number of lots is as follows:

$$\text{Number of Lots} = \frac{18{,}000\ \text{Mg}\ (20{,}000\ \text{tons})}{4{,}500\ \text{Mg}\ (5{,}000\ \text{tons})/\text{lot}} = 4\ \text{lots}$$

4.2.2.3

Sublot Size. Stratify each lot into five equal sublots. The sublot size is as follows:

$$\text{Sublot Size} = \frac{4,500 \text{ Mg } (5,000 \text{ tons})/\text{lot}}{5 \text{ sublots}/\text{lot}} = 4 \text{ lots}$$

Sublot Size = 900 Mg (1,000 tons)/sublot

The relationship between lot and sublot size is shown in Figure 3.

4.2.2.4

Sublot Samples. The number of samples per lot is five (one per sublot). Therefore, five random numbers are selected from the table of random numbers. Again, the first block of numbers from the random number table is reproduced below. Note that a different set of numbers than that used in Section 4.2.1.4 is selected:

Sequence Number	X	Y
67	0.93	R 0.17
68	0.40	R 0.50
69	0.44	R 0.15
70	0.03	L 0.60
71	0.19	L 0.37

4.2.2.5

Select random numbers this time from the Y column, disregarding the L or R: 0.17, 0.50, 0.15, 0.60, and 0.37. Multiply the numbers by the size of each of the five sublots as follows:

Sublot Number	Sublot Random Number	Size Mg (tons)	Sample from Mg (ton) No.
1	0.17	900 (1,000)	150 (170)
2	0.50	900 (1,000)	450 (500)
3	0.15	900 (1,000)	140 (150)
4	0.60	900 (1000)	540 (600)
5	0.37	900 (1,000)	330 (370)

4.2.2.5.1

The technician must obtain the first sample at approximately the 150th Mg (170th ton) of the first sublot. The technician must then wait until the first sublot is completed, 900 Mg (1,000 tons), before selecting the second sample at the 450th Mg (500th ton) of the second sublot. The same sequence is followed for obtaining the remaining three samples.

4.2.2.5.2

The sampling sequence for the lot of 4,500 Mg (5,000 tons) should be as follows:

Sublot 1: 150th Mg (170th ton)
Sublot 2: 900 + 450 (1,000 + 500) = 1350th Mg (1500th ton)
Sublot 3: 1800 + 140 (2,000 + 150) = 1940th Mg (2150th ton)
Sublot 4: 2700 + 540 (3,000 + 600) = 3240th Mg (3600th ton)
Sublot 5: 3600 + 330 (4000 + 370) = 3930th Mg (4370th ton)

4.2.2.5.3

Different random numbers are selected for the other four lots.

4.2.2.6

Sampling by material mass is a simple means of obtaining a random sample. Interruptions in the process do not affect randomization, and the relationship between the number of samples and the lot remains unchanged. Sublot sampling based on mass is illustrated in Figure 4.

4.2.3 Sampling an Area

4.2.3.1

Suppose that HMA from the roadway is to be sampled to determine the density for quality control or acceptance purposes. In this example, the specification states the lot size is 1,524 linear meters (5,000 ft), and five samples per lot are required. In addition, assume that the paving width is 3.66 m (12 ft) and that the project begins at Station 100 + 00 and ends at Station 160 + 96 (300 + 00 ft).

4.2.3.2

The specifications require a lot size of 1,524 linear meters (5,000 ft). The distance from Station 100 + 00 to Station 160 + 96 (300 + 00 ft) is 6,096 m (20,000 ft). The number of lots is calculated as follows:

$$\text{Number of Lots} = \frac{6096 \text{ m } (20,000 \text{ ft})}{1524 \text{ m } (5,000 \text{ ft})/\text{lot}} = 4 \text{ lots}$$

4.2.3.3

The beginning station for the first lot is 100 + 00. This lot ends at Station 115 + 24 (150 + 00) as shown in Figure 5. This is equal to 1,524 m (5,000 ft). The 1,524 m (5,000 ft) of paving must be stratified into five equal sublots, since five samples per lot are required. The sublot size is calculated as follows:

$$\text{Sublot Size} = \frac{1524\text{ m }(5{,}000\text{ ft})/\text{lot}}{5\text{ sublots}/\text{lot}}$$

$$= 304.8\text{ m }(1{,}000\text{ ft})/\text{sublot}$$

4.2.3.4

Figure 5 shows how this lot is divided.

4.2.3.5

The location at which each sample will be obtained must be randomized in the longitudinal and the transverse directions. This was previously illustrated in Figure 1.

4.2.3.6

The random number selection procedure is the same as used for the previous examples, except that two sets (columns and rows) of random numbers are selected: one for the transverse position and one for the longitudinal position.

4.2.3.7

A set of five random numbers for the longitudinal (X) and transverse (Y) positions of the sample is chosen by using the first and second blocks of random numbers from the random number table. These are reproduced as follows:

Sequence Number	X	Y
37	0.41	L 0.10
38	0.28	R 0.23
39	0.22	L 0.18
40	0.21	L 0.94
41	0.27	L 0.52

4.2.3.8

The X and Y random numbers in Section 4.2.3.7 are multiplied by the sublot length and paving width respectively, as shown below:

Sublot 1 (starting Station 100 + 00)
Coordinate X = 0.41 × 304.8 m (1,000 ft) = 125 m (410 ft)
Coordinate Y = 0.10 × 3.66 m (12 ft) = 0.4 m (1.2 ft)

Sublot 2 (starting Station 103 + 04.8 [110 + 00 ft])
Coordinate X = 0.28 × 304.8 m (1,000 ft) = 85.3 m (280 ft)
Coordinate Y = 0.23 × 3.66 m (12 ft) = 0.8 m (2.8 ft)

Sublot 3 (starting Station 106 + 09.6 [120 + 00 ft])
Coordinate X = 0.22 × 304.8 (1,000 ft) = 67 m (220 ft)
Coordinate Y = 0.18 × 3.66 m (12 ft) = 0.7 m (2.2 ft)

Sublot 4 (starting Station 109 + 14.4 [130 + 00 ft])
Coordinate X = 0.21 × 304.8 m (1,000 ft) = 64 m (210 ft)
Coordinate Y = 0.94 × 3.66 m (12 ft) = 3.4 m (11.3 ft)

Sublot 5 (starting Station 112 + 19.2 [140 + 00 ft])
Coordinate X = 0.27 × 304.8 m (1,000 ft) = 82.3 m (270 ft)
Coordinate Y = 0.52 × 3.66 m (12 ft) = 1.9 m (6.2 ft)

4.2.3.9

The longitudinal distance (X) is added to the beginning station of the sublot and the companion transverse distance (Y) is measured from the selected edge of paving. The L values of Y will be measured from the left edge of paving (looking ahead) and the R values of Y will be measured from the right edge of paving.

<u>Sample No.</u>

1. Station 100 + 00 + 125 m (410 ft) = 101 + 25 (104 + 10 ft); @ 0.4 m (1.2 ft) from left edge
2. Station 103 + 04.8 (110 + 00 ft) + 85.3 m (280 ft) = 103 + 90.1 (112 + 80 ft); @ 0.8 m (2.8 ft) from right edge
3. Station 106 + 09.6 (120 + 00 ft) + 67 m (220 ft) = 106 + 76.6 (122 + 20 ft); @ 0.7 m (2.2 ft) from left edge
4. Station 109 + 14.4 (130 + 00 ft) + 64 m (210 ft) = 109 + 78.4 (132 + 10 ft); @ 3.4 m (11.3 ft) from left edge
5. Station 112 + 19.2 (140 + 00 ft) + 82.3 m (270 ft) = 113 + 01.5 (142 + 70 ft); @ 1.9 m (6.2 ft) from left edge

4.2.3.10

Figure 6 illustrates the sampling locations based on these calculations.

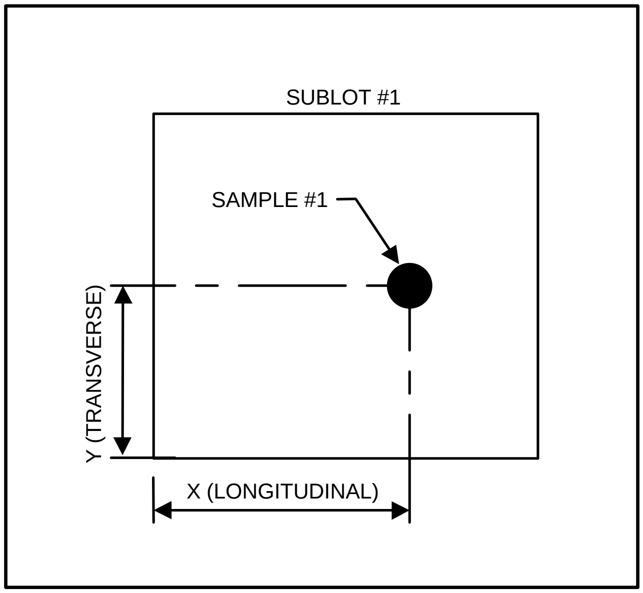


Figure 1. Determination of sample location using random numbers.

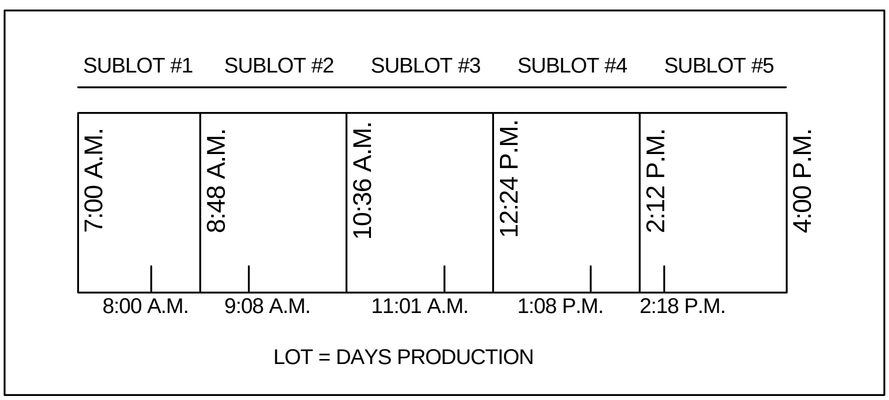


Figure 2. Sublot sample times based on time sequence.

sublot #1	sublot #2	sublot #3 900 Mg (typical)	sublot #4	sublot #5

<-------------------4,500 Mg (5,000 ton) lot------------------->

Figure 3. Relationship between lot and sublots based on mass.

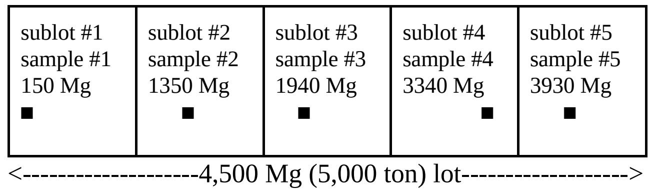


Figure 4. Sublot sample based on mass.

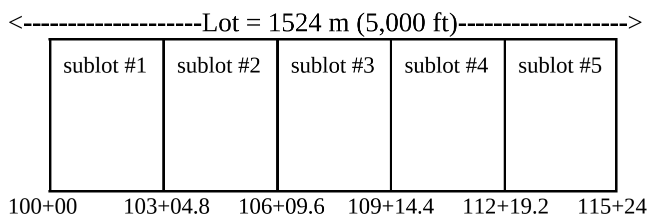


Figure 5. Relationship between lot and sublots based on area.

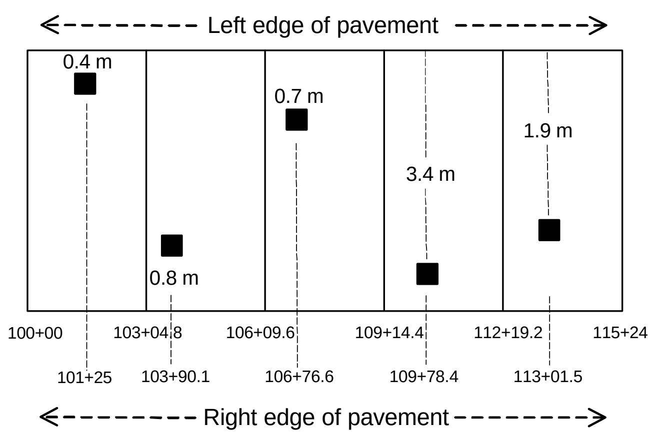


Figure 6. Sublot sample location based on area.

TABLE 1 Random positions in decimal fractions (two places)

Sequence No.	X	Y	Sequence No.	X	Y
1.	0.29	R 0.66	51.	0.87	L 0.36
2.	0.74	R 0.49	52.	0.34	L 0.19
3.	0.89	L 0.79	53.	0.37	R 0.33
4.	0.60	R 0.39	54.	0.97	L 0.79
5.	0.88	R 0.31	55.	0.13	R 0.56
6.	0.72	L 0.54	56.	0.85	R 0.64
7.	0.12	R 0.08	57.	0.14	L 0.04
8.	0.09	L 0.94	58.	0.99	R 0.74
9.	0.62	L 0.11	59.	0.40	L 0.76
10.	0.71	R 0.59	60.	0.37	L 0.09
11.	0.36	L 0.38	61.	0.90	R 0.74
12.	0.57	R 0.49	62.	0.09	L 0.70
13.	0.35	R 0.90	63.	0.66	L 0.97
14.	0.69	L 0.63	64.	0.89	L 0.55
15.	0.59	R 0.68	65.	0.67	L 0.44
16.	0.06	L 0.03	66.	0.02	R 0.65
17.	0.08	L 0.70	67.	0.93	R 0.17
18.	0.67	L 0.68	68.	0.40	R 0.50
19.	0.83	R 0.97	69.	0.44	R 0.15
20.	0.54	R 0.58	70.	0.03	L 0.60
21.	0.82	R 0.50	71.	0.19	L 0.37
22.	0.66	R 0.73	72.	0.92	L 0.45
23.	0.06	L 0.27	73.	0.20	L 0.85
24.	0.03	L 0.13	74.	0.05	R 0.56
25.	0.55	L 0.29	75.	0.46	R 0.58
26.	0.64	L 0.77	76.	0.43	R 0.91
27.	0.30	R 0.57	77.	0.97	L 0.55
28.	0.51	R 0.67	78.	0.06	R 0.51
29.	0.29	R 0.09	79.	0.72	L 0.78
30.	0.63	R 0.82	80.	0.95	L 0.36
31.	0.53	L 0.86	81.	0.16	L 0.61
32.	0.99	R 0.22	82.	0.29	R 0.47
33.	0.02	R 0.89	83.	0.48	R 0.15
34.	0.61	L 0.87	84.	0.73	R 0.64
35.	0.76	R 0.16	85.	0.05	L 0.94
36.	0.87	L 0.77	86.	0.43	L 0.05
37.	0.41	L 0.10	87.	0.87	R 0.98
38.	0.28	R 0.23	88.	0.37	L 0.71
39.	0.22	L 0.18	89.	0.94	L 0.26
40.	0.21	L 0.94	90.	0.57	L 0.63
41.	0.27	L 0.52	91.	0.26	R 0.80
42.	0.39	R 0.91	92.	0.01	L 0.79
43.	0.57	L 0.10	93.	0.83	R 0.59
44.	0.82	L 0.12	94.	0.71	L 0.21
45.	0.14	L 0.94	95.	0.65	L 0.63
46.	0.50	R 0.58	96.	0.65	L 0.87
47.	0.93	L 0.03	97.	0.72	R 0.92
48.	0.43	L 0.29	98.	0.85	L 0.78
49.	0.99	L 0.36	99.	0.04	L 0.46
50.	0.61	R 0.25	100.	0.29	L 0.95

X = Decimal fraction of total length measured along the road from starting point.
Y = Decimal fraction measured across the road from either outside edge towards center line of the paved lane.

ATTACHMENT B

STATISTICAL CONTROL CHARTS

1.1 PROCESS CONTROL

1.2

The process control procedure recommended is the use of control charts, particularly statistical control charts. Control charts provide a means of verifying that a process is in control. It is important to understand that statistical control charts do not get or keep a process under control. The process must still be controlled by the plant or construction personnel. Control charts simply provide a visual warning mechanism to identify when the contractor or material supplier should look for possible problems with the process.

1.3

Variation of construction materials is inevitable and unavoidable. The purpose of control charts, then, is not to eliminate variability, but to distinguish between the inherent or chance causes of variability and assignable causes. Chance causes (sometimes known as common causes) are a part of every process and can be reduced but generally not eliminated. Assignable causes (sometimes known as special causes) are factors that can be eliminated, thereby reducing variability. Chance causes are something that a contractor or material supplier must learn to live with. They cannot be eliminated, but it may be possible to reduce their effects. Assignable causes, however, can be eliminated if they can be identified. Examples of assignable causes include gradation for an aggregate blend being out of specification because of a hole in one of the sieves or the cold feed conveyor setting being incorrectly adjusted.

1.4

The statistical control chart enables the contractor to distinguish between chance and assignable causes. Based on statistical theory, construction materials under production control exhibit a "bell-shaped" or normal distribution curve.

1.5

Statistical control charts for average or means rely on the fact that, for a normal distribution, essentially all of the values fall within ±3 standard deviations from the mean. The normal distribution can be used because the distribution of sample means is normally distributed. The data, therefore, can be assumed to be within $\pm 3\sigma$ of the mean or target when the process is in control and only chance causes are acting on the system.

1.6

A statistical control chart can be viewed as a normal distribution curve on its side (Figure 1). For a normal curve, only about 0.27 percent (1 out of 370) of the measurements should fall outside ±3 standard deviations from the mean. Therefore, control limits (indicating that an investigation for an assignable cause should be conducted) are set at $+3\sigma\bar{x}$ and $-3\sigma\bar{x}$.

1.7

A statistical control chart includes a target value, upper and lower control limits, and a series of data points that are plotted. The target is based on the population or production mean and the control limits are established from the population or production standard deviation as shown in Figure 2.

2.1 FORMS OF STATISTICAL CONTROL CHARTS

2.2

Of the many forms of statistical control charts, two are most practical and useful for construction materials and processes. These are the control chart for means or averages (referred to commonly as x-bar chart or $\bar{x}$ chart) and the control chart for ranges (referred to commonly as an R-chart). The x-bar chart is typically used to control the production process about the average or target value. The R-chart considers the variability of the material and prevents extremely large positive and negative results from canceling out and not being detectable on the control chart for means or averages. The range, which is the easiest measure of spread to use in the field, is usually used in place of the standard deviation.

2.3

Population or production parameters (i.e., averages and ranges) are either known (or specified) or estimated from the early stages of the production process. In most cases, they are estimated. It is not a good idea for a producer to use the mean, range, or standard deviation specified or used by the highway agency when it developed the specification limits. The mean, range, or standard deviation of a producer's process is independent of the specification limits; they are established by the process capability.

2.4

When the mean and standard deviation are not known, they are estimated by the *grand average or mean* ($\bar{\bar{x}}$) and the *average range* (R-bar). The grand average or mean is defined as the average value of a group of averages. The average range is defined as the average of individual range values. The grand mean becomes the target value for the $\bar{x}$ chart and the average range becomes the target value for the R-bar chart.

2.5

The following formulae are used to construct the two control charts:

$\bar{X}$ Chart

Upper Control Limit (UCL) = $\bar{X} + (A_2 \times \bar{R})$

Lower Control Chart (LCL) = $\bar{X} - (A_2 \times \bar{R})$

R Chart

Upper Control Limit (UCL) = $D_4 \times \bar{R}$

Lower Control Limit (LCL) = $D_3 \times \bar{R}$

The factors A_2, D_3, and D_4 are obtained from Table 1 for the appropriate sample size, n. Note that the sample size is always greater than one. For each quality control test, the samples are grouped to form a subgroup two or more.

3.1 CONTROL CHARTS WHEN MEAN AND STANDARD DEVIATION ARE UNKNOWN

3.2

The data shown in Table 2 will be used to illustrate the calculation for a control chart when the population parameters are unknown and are estimated from the early production process. The table contains the gradation results for percent passing the 4.75-mm (No. 4) sieve for 40 production days (4 tests per day). The average and range of the first 20 subgroups are used to estimate the mean and standard deviation of the population. When this is done:

$$\bar{X} = \frac{18.4 + 18.0 + \ldots + 18.4 + 16.4}{20} = \frac{365.9}{20} = 18.3$$

$$\bar{R} = \frac{2.1 + 4.1 + \ldots + 4.7 + 3.9}{20} = \frac{83.6}{20} = 4.2$$

3.3

Subsequent to estimating x-bar and R-bar in accordance with Section 3.2, the upper and lower control limits can be calculated from the formulae in Section 2.5. Note that the values for A_2, D_3, and D_4 are for a sample subgroup of n = 4 because four samples are used to establish each average, $\bar{x}$, and range, R.

$\bar{X}$ Chart

UCL = $\bar{X} + (A_2 \times \bar{R}) = 18.3 + (0.73 \times 4.2) = 21.4$

LCL = $\bar{X} - (A_2 \times \bar{R}) = 18.3 - (0.73 \times 4.2) = 15.2$

Target Value = $\bar{X} = 18.3$

$\bar{R}$ Chart

UCL = $D_4 \times \bar{R}) = 2.28 \times 4.2 = 9.6$

LCL = $D_3 \times \bar{R}) = 0.0 \times 4.2 = 0.0$

Target Value = $\bar{R} = 4.2$

3.4

After establishing the target value and control limits, construct the control charts using the data in Table 2. Figures 3 and 4 illustrate the x-bar and R-bar charts for the data.

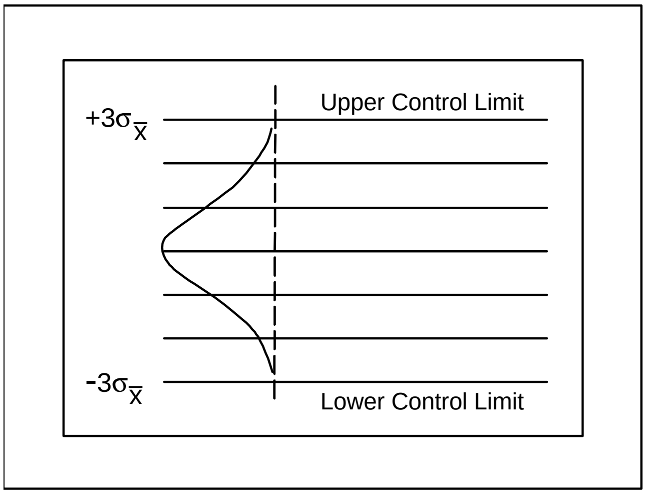


Figure 1. Example of statistical control chart.

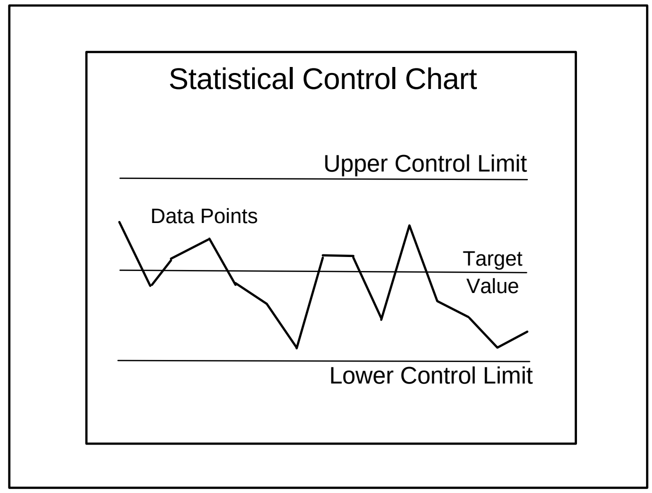


Figure 2. Elements of statistical control chart.

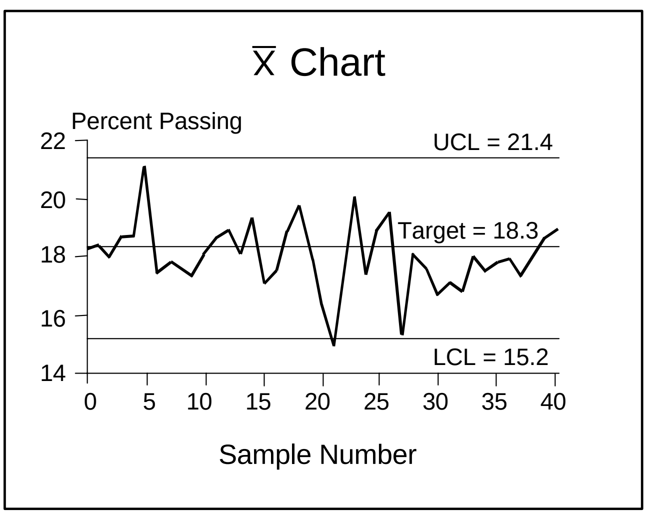


Figure 3. X-bar chart for percent passing 4.75-mm (#4) sieve.

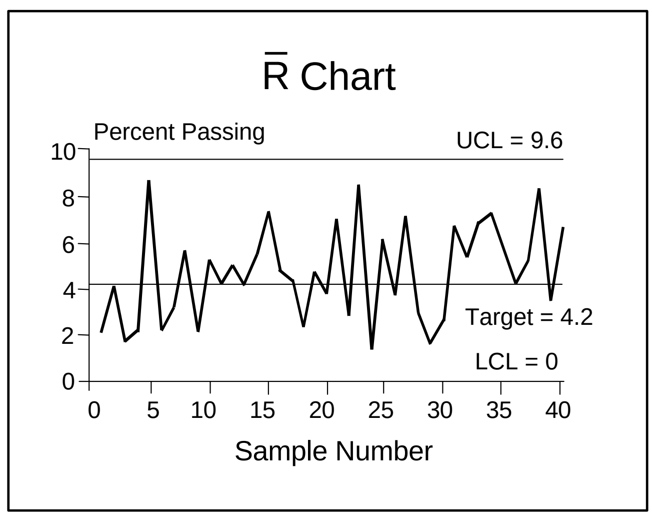


Figure 4. R-bar chart for percent passing 4.75-mm (#4) sieve.

TABLE 1 Factors for statistical control charts

n	A_2	D_3	D_4
2	1.88	0	3.27
3	1.02	0	2.58
4	0.73	0	2.28
5	0.58	0	2.12
6	0.48	0	2.00
7	0.42	0.08	1.92

TABLE 2 Data for demonstration example

Percent Passing 4.75-mm (No. 4) Sieve						
No.	X_1	X_2	X_3	X_4	$\bar{\bar{x}}$	**R**
1	18.9	18.2	19.3	17.2	18.4	2.1
2	18.2	16.3	17.2	20.4	18.0	4.1
3	18.5	19.5	17.8	19.1	18.7	1.7
4	19.7	17.6	18.3	19.2	18.7	2.1
5	23.5	22.5	14.9	23.6	21.1	8.7
6	16.6	16.9	17.4	18.8	17.4	2.2
7	19.0	17.9	15.8	18.4	17.8	3.2
8	14.5	17.7	18.0	20.1	17.6	5.6
9	18.5	16.3	17.3	17.2	17.3	2.2
10	15.2	20.4	20.4	16.4	18.1	5.2
11	19.5	16.4	20.7	17.7	18.6	4.3
12	17.4	17.9	17.7	22.4	18.9	5.0
13	15.6	18.1	18.7	19.8	18.1	4.2
14	22.2	17.1	16.7	21.2	19.3	5.5
15	20.1	12.8	18.5	17.0	17.1	7.3
16	19.6	18.0	17.4	14.8	17.5	4.8
17	19.5	19.9	20.1	15.7	18.8	4.4
18	19.9	20.7	19.8	18.3	19.7	2.4
19	20.9	19.9	16.5	16.3	18.4	4.7
20	14.2	18.1	17.1	16.2	16.4	3.9
21	16.7	13.6	11.4	18.4	15.0	7.0
22	18.7	17.3	16.1	15.8	17.0	2.9
23	22.7	18.3	23.8	15.3	20.0	8.5
24	17.3	18.2	16.8	17.2	17.4	1.4
25	20.8	16.7	16.0	22.1	18.9	6.1
26	17.5	21.3	19.1	20.2	19.5	3.8
27	13.6	16.8	19.2	12.1	15.4	7.1
28	19.5	18.3	16.5	18.1	18.1	3.0
29	17.7	18.5	17.4	16.9	17.6	1.6
30	16.4	17.5	15.2	17.8	16.7	2.6
31	15.3	14.5	17.3	21.2	17.1	6.7
32	13.7	18.4	16.1	19.1	16.8	5.4
33	13.4	20.3	18.8	19.5	18.0	6.9
34	14.6	21.9	18.5	14.9	17.5	7.3
35	16.0	20.4	14.7	20.0	17.8	5.7
36	17.2	18.5	15.8	20.0	17.9	4.2
37	18.8	15.0	20.2	15.2	17.3	5.2
38	21.4	17.7	13.1	19.6	18.0	8.3
39	16.5	18.8	20.0	19.2	18.6	3.5
40	19.4	18.6	15.4	22.0	18.9	6.6

ATTACHMENT C

PERCENT WITHIN LIMITS FOR SUPERPAVE-DESIGNED HMA

1.1 SCOPE

1.2

This attachment provides the procedure for determining the percentage of material or construction within the specification limits (PWL) established for the Superpave-designed HMA.

2.1 SIGNIFICANCE AND USE

2.2

The PWL is the principal calculation used in the Acceptance Plan to determine the acceptability of material or construction on the project.

3.1 PWL CALCULATION

3.2

Estimate the PWL in accordance with Sections 3.2.1 through 3.2.9.

3.2.1

Locate n sampling positions in the lot by use of Table 1 or other appropriate random number tables.

3.2.2

Make a measurement at each location or take a test portion and make the measurement on the test portion, as appropriate.

3.2.3

Average the lot measurements to find $\bar{x}$.

$$\bar{x} = \sum_{i=1}^{n} \frac{x_i}{n}$$

3.2.4

Determine the sample standard deviation, "s", of the lot measurements.

$$s = \sqrt{\sum_{i=1}^{n} \frac{(x_i - \bar{x})^2}{n-1}}$$

3.2.5

Find the Quality Index, Q_U, by subtracting the average ($\bar{x}$) of the measurements from the upper specification limit (U) and dividing the results by "s".

$$Q_U = \frac{U - \bar{x}}{s}$$

3.2.6

Find the Quality Index Q_L by subtracting the lower specification limit (L) from the average ($\bar{x}$) and dividing the results by "s".

$$Q_L = \frac{\bar{x} - L}{s}$$

3.2.7

Estimate the percentage of material or construction that will fall within the upper tolerance limit (UTL) by locating the entry in Table 1 nearest the calculated Q_U in the column appropriate to the total number (n) of measurements.

3.2.8

Estimate that percentage of material or construction that will fall within the lower tolerance limit (LTL) by locating the entry in Table 1 nearest the calculated Q_L in the column appropriate to the total number (n) of measurements.

3.2.9

In cases where both upper (UTL) and lower (LTL) tolerance limits are required, find the percentage of material that will fall within tolerances by adding P_U, the percent within the upper tolerance limit (UTL), to P_L, the percent within the lower tolerance limit (LTL), and subtract 100 from the sum.

Total PWL = $(P_U + P_L) - 100$

TABLE 1 Quality index values for estimating percent within limits

PWL	n = 3	n = 4	n = 5	n = 7	n = 10	n = 15
99	1.16	1.47	1.68	1.89	2.04	2.14
98	1.15	1.44	1.61	1.77	1.86	1.93
97	1.15	1.41	1.55	1.67	1.74	1.80
96	1.15	1.38	1.49	1.59	1.64	1.69
95	1.14	1.35	1.45	1.52	1.56	1.59
94	1.13	1.32	1.40	1.46	1.49	1.51
93	1.12	1.29	1.36	1.40	1.43	1.44
92	1.11	1.26	1.31	1.35	1.37	1.38
91	1.10	1.23	1.27	1.30	1.32	1.32
90	1.09	1.20	1.23	1.25	1.26	1.27
89	1.08	1.17	1.20	1.21	1.21	1.22
88	1.07	1.14	1.16	1.17	1.17	1.17
87	1.06	1.11	1.12	1.12	1.13	1.13
86	1.05	1.08	1.08	1.08	1.08	1.08
85	1.03	1.05	1.05	1.05	1.04	1.04
84	1.02	1.02	1.02	1.01	1.00	1.00
83	1.00	0.99	0.98	0.97	0.96	0.96
82	0.98	0.96	0.95	0.94	0.93	0.92
81	0.96	0.93	0.92	0.90	0.89	0.89
80	0.94	0.90	0.88	0.87	0.85	0.85
79	0.92	0.87	0.85	0.83	0.82	0.82
78	0.89	0.84	0.82	0.80	0.79	0.78
77	0.87	0.81	0.79	0.77	0.76	0.75
76	0.84	0.78	0.76	0.74	0.72	0.72
75	0.82	0.75	0.73	0.71	0.69	0.69
74	0.79	0.72	0.70	0.67	0.66	0.66
73	0.77	0.69	0.67	0.64	0.63	0.62
72	0.74	0.66	0.64	0.61	0.60	0.59
71	0.71	0.63	0.60	0.58	0.57	0.56
70	0.68	0.60	0.58	0.55	0.54	0.54
69	0.65	0.57	0.55	0.53	0.51	0.51
68	0.62	0.54	0.52	0.50	0.48	0.48
67	0.59	0.51	0.49	0.47	0.46	0.45
66	0.56	0.48	0.46	0.44	0.43	0.42
65	0.53	0.45	0.43	0.41	0.40	0.40
64	0.49	0.42	0.40	0.38	0.37	0.37
63	0.46	0.39	0.37	0.35	0.35	0.34
62	0.43	0.36	0.34	0.33	0.32	0.31
61	0.39	0.33	0.31	0.30	0.30	0.29
60	0.36	0.30	0.28	0.25	0.25	0.25

Note 1: For negative values of Q_U or Q_L, P_U or P_L is equal to 100 minus the tabular P_U or P_L.
Note 2: If the value of Q_U or Q_L does not correspond exactly to a value in the table, use the next higher value.

ATTACHMENT D

STATISTICAL VERIFICATION TEST

1.1 SCOPE

1.2

The engineer will determine the acceptability of the contractor's quality control test data for material acceptance purposes using the *t*-test for sample means.

2.1 SIGNIFICANCE AND USE

2.2

The contractor's quality control test data will be considered verified if it agrees with the engineer's quality assurance data at a level of significance, $\alpha = 0.01$.

3.1 STATISTICAL VERIFICATION

3.2

The *t*-value, t, of the group of test data to be verified is computed as follows:

$$t = \frac{|\bar{x}_c - \bar{x}_v|}{s_p \sqrt{\dfrac{1}{n_c} + \dfrac{1}{n_v}}}$$

and

$$s_p^2 = \frac{s_c^2(n_c - 1) + s_v^2(n_v - 1)}{n_c + n_v - 2}$$

where

n_c = number of contractor's quality control tests (minimum of two required)
n_v = number of verification tests (minimum of one required)
$\bar{x}_c$ = mean of the contractor's quality control tests
$\bar{x}_v$ = mean of the verification tests
s_p = pooled standard deviation (when $n_v = 1$, $s_p = s_c$)
s_c = standard deviation of the contractor's quality control tests
s_v = standard deviation of the verification tests or, when $n_v = 1$, S_c).

3.3

Compute t using the equation above and compare it with the critical *t*-value, t_{crit}, from the following table:

Critical *t*-value for verification testing

degrees of freedom ($n_c + n_v - 2$)	t_{crit} for $\alpha = 0.01$
1	63.657
2	9.925
3	5.841
4	4.604
5	4.032
6	3.707
7	3.499
8	3.355
9	3.250
10	3.169
11	3.106
12	3.055
13	3.012
14	2.977
15	2.947
16	2.921
17	2.898
18	2.878
19	2.861
20	2.845
21	2.831
22	2.819
23	2.807
24	2.797
25	2.787
26	2.779
27	2.771
28	2.763
29	2.756
30	2.750
31–40	2.704
41–60	2.660
61–120	2.617
greater than 120	2.576

3.4

When the *t*-values of the test data from the engineer's verification tests and the contractor's quality control tests are compared with t_{crit} from the previous table, if t is less than or equal to t_{crit} ($t \le t_{crit}$), the difference between the contractor's quality control test data and the corresponding engineer's verification test data is not significant, and the contractor's test data are verified. When t is greater than t_{crit} ($t > t_{crit}$), the difference between the contractor's quality control test data and the corresponding engineer's verification test data is significant, and the contractor's test data are not verified.